JARAMILLO -TONNESON & ASSOCIATES CONSULTING STRUCTURAL ENGINEERS 4411 E. KNOX RD. PHOENIX, AZ 85044	Project: Quick N' Clean			Job Ref.: 2022152
	Section: CALCULATION REPORT			Sheet no./rev.:
	Calc. by: JMB	Contact Phone #:	EMAIL:	Date: 1/18/2023

CLIENT:

CSHQA
2696 S Colorado Blvd. Suite 525
Denver, Colorado 80222

PROJECT:

Quick N' Clean Car Wash

AT :

Parker Road & Pine Lane Avenue
Parker, Colorado

CODE:

2021 International Building Code w/City of Parker Amendments

Structural Calculations

"PRACTICAL DESIGN AND CONSTRUCTION EXPERIENCE LIKE NO OTHER"

JARAMILLO -TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project		Job Ref.
Quick N' Clean		2022152
Section		Sheet no./rev.
CALCULATION REPORT		

Calc. by	Contact Phone #	EMAIL:	Date
JMB			1/19/2023

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LOADS:

ROOFS: (HIGH & LOW)

ROOF LIVE LOAD = 20 PSF (REDUCIBLE).
GROUND SNOW LOAD, Pg = 45 PSF.
ROOF SNOW LOAD, Pf = 31.5 PSF (NON-REDUCIBLE).
SNOW EXPOSURE FACTOR, Ce = 1.0
SNOW THERMAL FACTOR, Ct = 1.0
SNOW IMPORTANCE FACTOR, I = 1.0
ROOF DEAD LOAD = 20PSF.

LATERAL:

WIND:

ULTIMATE DESIGN WIND SPEED (3-SECOND GUST), V(ult) = 107 MPH.
EXPOSURE C.
RISK CATEGORY, II.

SEISMIC:

SEISMIC IMPORTANCE FACTOR, I = 1.0.
RISK CATEGORY, II.
MAPPED SHORT PERIOD SPECTRAL ACCELERATION, Ss = 0.200
MAPPED ONE SECOND SPECTRAL ACCELERATION, S1 = 0.056
SOIL SITE CLASS, C.
DESIGN SHORT PERIOD SPECTRAL ACCELERATION, Sds = 0.174
DESIGN ONE SECOND SPECTRAL ACCELERATION, Sd1 = 0.056
SEISMIC DESIGN CATEGORY, B.

SNOW LOAD = 0.7(1.0)(1.0)(1.0)(45PSF) = 31.5 PSF ROOF SNOW FLAT ROOF LOAD

"ASCE710S.xls" Program
Version 1.1

SNOW LOADING ANALYSIS

Per ASCE 7-16 Code for Buildings with Flat or Low Slope Roofs (<= 5 deg. or 1 in./ft.) for Balanced Snow, Drift, and Rain-on-Snow Surcharge Loadings

Job Name:		Subject:			
Job No:		Originator:		Checker:	

Input Data:

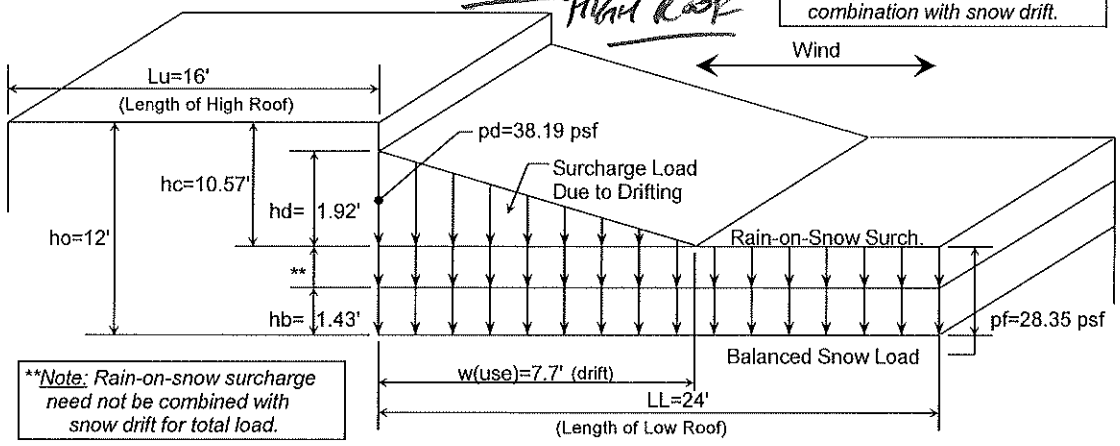
Building Risk Category =	II		Table 1.5-1, page 2
Ground Snow Load, p_g =	45.00	psf	Figure 7-1, pages 34-35 and Table 7-1, page 30
Length of High Roof, L_u =	16.00	ft.	Length of Roof Upwind of the Snow Drift
Length of Low Roof, L_L =	24.00	ft.	Length of Roof Downwind of the Snow Drift
Dist. from Eave to Ridge, W =	8.00	ft.	Horizontal Distance from Eave to Ridge
Type of Roof =	Gable		Type of Roof = Monoslope, Gable, or Hip
Obstruction Height, h_o =	12.00	ft.	High Roof - Low Roof Elevations
Roof Slope, S =	0.25	in./ft.	S = Rise per foot of Run
Exposure Factor, Ce =	0.90		Table 7-2, page 30
Thermal Factor, Ct =	1.00		Table 7-3, page 30

Results:

Roof Angle, θ =	1.1935	deg.	θ = ATAN(S/12)
Importance Factor, I_s =	1.00		Table 1.5-2, page 5
Snow Density, γ =	19.85	pcf	$\gamma = 0.13*p_g+14 <= 30$ (Eqn. 7.7-1, page 33)
Flat Roof Snow Load, p_f =	28.35	psf	$p_f = 0.7*C_e*C_t*I_s*p_g$ (Eqn. 7.3-1, page 29)
*Min. Roof Snow Load, p_m =	20.00	psf	$p_m = p_g*I_s$ for $p_g <= 20$, $p_m = 20*I_s$ for $p_g > 20$
Balanced Snow Load Ht., h_b =	1.43	ft.	$h_b = p_{f(use)}/\gamma$ (Section 7.1, page 29)
Clear Height, h_c =	10.57	ft.	$h_c = h_o-h_b >= 0$ (Section 7.1, page 29)
Leeward Drift Height, h_{dL} =	1.92	ft.	h_{dL} = 0.43*Lu^1/3*(pg+10)^1/4-1.5, with Lu>=25' (Figure 7-9)
Windward Drift Height, h_{dw} =	1.44	ft.	h_{dw} = 0.75*(0.43*LL^1/3*(pg+10)^1/4-1.5), with LL>=25'
Max. Drift Height, $h_{d(max)}$ =	1.92	ft.	$h_{d(max)}$ = maximum of: (h_{dL} or h_{dw})
Ratio, h_c/h_b =	7.40		If h_c/h_b >= 0.2 , then snow drifts are required to be applied
Drift Length, w =	7.70	ft.	If $h_{d(max)}<=h_c$: $w = 4*h_{d(max)}$, if $h_{d(max)}>h_c$: $w = 4*h_{d(max)}^2/h_c$
Design Drift Height, h_d =	1.92	ft.	If $h_{d(max)} <= h_c$: $h_d = h_{d(max)}$, if $h_{d(max)} > h_c$: $h_d = h_c$
Drift Length, $w_{(max)}$ =	84.57	ft.	$w_{(max)} <= 8*h_c$
Drift Length, $w_{(use)}$ =	7.70	ft.	$w_{(use)}$ = minimum of: w or $w_{(max)}$
Wt. of Drift at High End, p_d =	38.19	psf	$p_d = h_d*\gamma$ (maximum value)
Wt. of Drift at Low End, p_{de} =	0.00	psf	pde = 0, as Low Roof Length (LL) >= w(max)
Rain-on-Snow Surch., p_{rs} =	0.00	psf	prs = 5.0 psf when 0 < pg <=20 and θ < W/50) (Sect. 7.10)
Balanced Snow Load, $p_{f(bal)}$ =	28.35	psf	$p_{f(bal)} = p_f+p_{rs}$
**Total Snow Load, $p_{(total)}$ =	66.54	psf	$p_{(total)} = p_{f(bal)}+p_d$

USE 38 PSF ALONG HIGH ROOF

*Note: Minimum flat roof snow load, pm, need not be used in combination with snow drift.



**Note: Rain-on-snow surcharge need not be combined with snow drift for total load.

Configuration of Snow Drift on Lower Roof

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

JOB TITLE QNC Car Wash

JOB NO. 2022152 SHEET NO.
CALCULATED BY JMB DATE
CHECKED BY SJJ DATE

Snow Loads - from adjacent building or roof:

ASCE 7- 16

Nominal Snow Forces

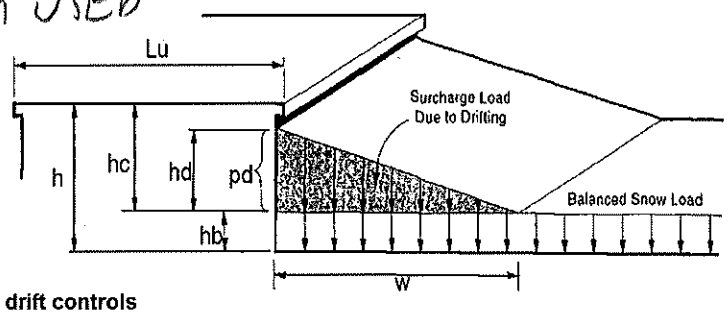
		Higher Roof	Lower Roof
Roof slope	=	1.2 deg	0.25 / 12 = 1.2 deg
Horiz. eave to ridge dist (W)	=	18.0 ft	8.0 ft
Roof length parallel to ridge (L)	=	150.0 ft	24.0 ft
Projection height (roof step) h	=		10.0 ft
Building separation s	=		0.0 ft
Type of Roof		Hip and gable w/ trussed systems	Monoslope
Ground Snow Load	Pg =	45.0 psf	45.0 psf
Risk Category	=	II	II
Importance Factor	I =	1.0	1.0
Thermal Factor	Ct =	1.00	1.00
Exposure Factor	Ce =	1.0	1.0
Pf = 0.7*Ce*Ct*I*Pg	=	31.5 psf	31.5 psf
Unobstructed Slippery Surface		no	no
Sloped-roof Factor	Cs =	1.00	1.00
Balanced Snow Load	**Ps =**	**31.5 psf**	**31.5 psf**
Rain on Snow Surcharge Angle		0.36 deg	0.16 deg
Code Maximum Rain Surcharge		5.0 psf	5.0 psf
Rain on Snow Surcharge	=	0.0 psf	0.0 psf
Ps plus rain surcharge	=	31.5 psf	31.5 psf
Minimum Snow Load	Pm =	20.0 psf	20.0 psf
Uniform Roof Design Snow Load	=	**31.5 psf**	**31.5 psf**
Building Official Minimum	=		

NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

NOT USED

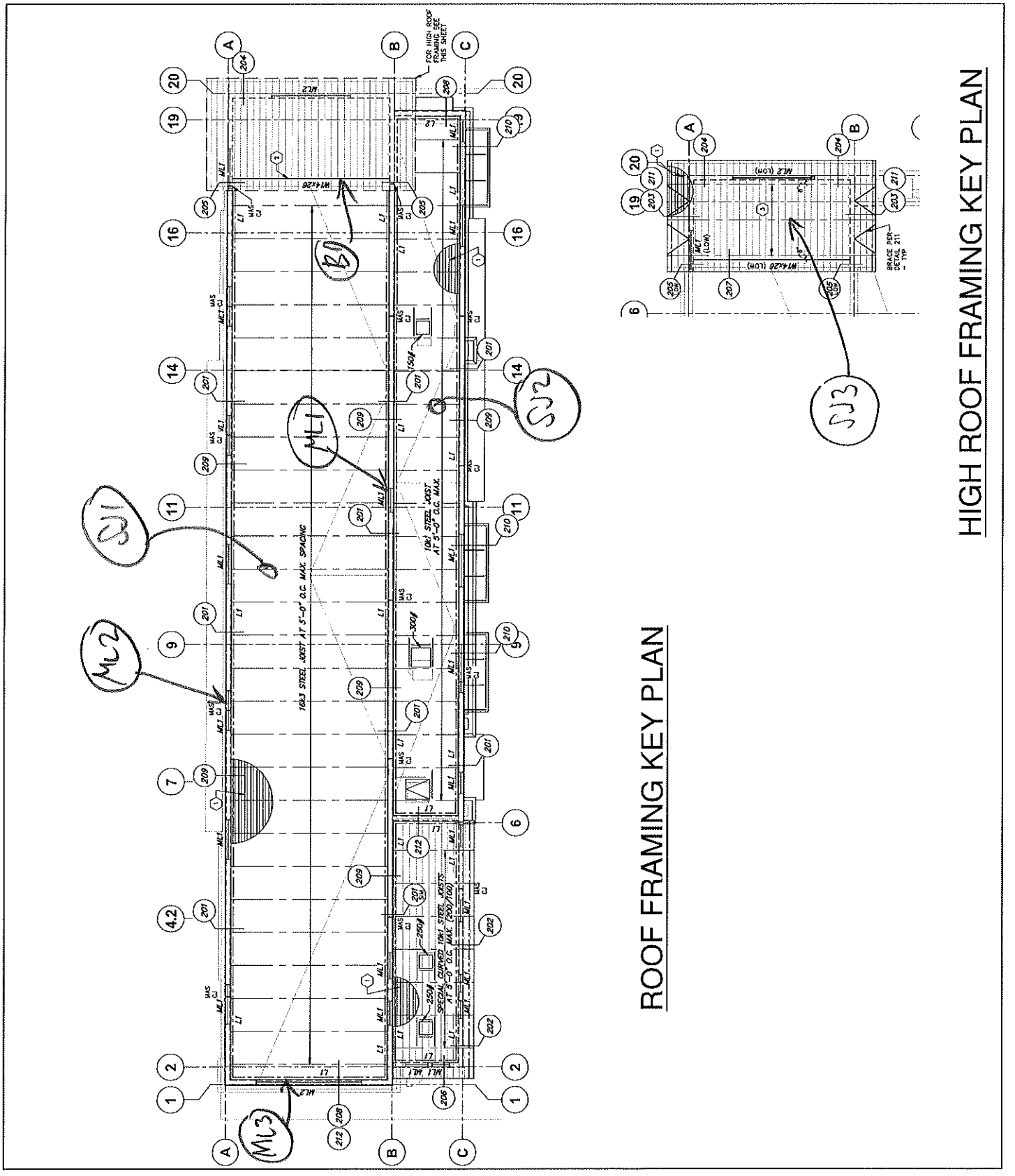
Upper roof length	lu =	48.0 ft
Snow density	γ =	19.9 pcf
Balanced snow height	hb =	1.58 ft
	hc =	6.41 ft
hc/hb >0.2 = 5.3	**Therefore, design for drift**	
Adj structure factor	=	1.00
Drift height (hd)	=	2.76 ft
Drift width	w =	11.02 ft
Surcharge load:	pd = γ*hd =	**54.7 psf**
Balanced Snow load:	=	31.5 psf
		86.2 psf **Leeward drift controls**



Windward Snow Drifts - from low roof against high roof

Lower roof length	lu =	24.0 ft
Adj structure factor	=	1.00
Drift height	hd =	1.41 ft
Drift width	w =	5.63 ft
Surcharge load:	pd = γ*hd =	**28.0 psf**

USE AT PARAPETS

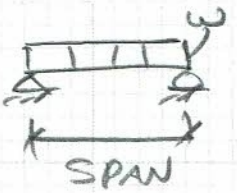

HIGH ROOF FRAMING KEY PLAN
ROOF FRAMING KEY PLAN
FOR HIGH ROOF FRAMING SEE THIS SHEET
10K1 STEEL JOIST AT 5'-0" O.C. MAX. SPACING
SPECIAL CURVED 10K1 STEEL JOISTS AT 5'-0" O.C. MAX. (200/100)
BRACE PER DETAIL 211 - TYP
ML1
ML2
ML3
SJ1
SJ2
JJ3

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #.
QNC - LAS VEGAS	2020081
Section	Sheet no./rev.
Calc. by JJB / Contact Phone # / EMAIL / Date	

SJ1 (STEEL JOISTS) 25'-0" SPAN

$w = (20_{DL} + 34.7_{SL})(5') = 100\,PLF_{DL} + 175\,PLF_{SL}$



USE 16K4 STEEL JOISTS AT 5'-0" O.C.

STEEL JOISTS WILL BE DESIGNED AND SPECIFIED BY STEEL JOIST MANUFACTURER - TYP.

SJ2 (STEEL JOISTS) 10'-8" SPAN

$w = (20_{DL} + 34.7_{SL})(5') = 100\,PLF_{DL} + 174\,PLF_{SL}$

USE 10K1 STEEL JOISTS AT 5'-0" O.C.

SJ3 (STEEL JOISTS) 25' SPAN (HORIZONTAL SPAN)

$w = (20_{DL} + 31.5_{SL})(5') = 100\,PLF_{DL} + 174\,PLF_{SL}$

SIM. AS SJ1 EXCEPT SPECIAL CURVED JOISTS

USE SPECIAL CURVED STEEL JOISTS AT 5'-0" O.C. MAX.

ML1 4' SPAN (MASONRY LINTEL)

$w = (20_{DL} + 31.5_{SL})(24'/2 + 10.67'/2) + 78_{DL}(1.33') + 55_{DL}(8.33')$
$+ 28_{SL}(1.5)x2 = 909\,PLF_{DL} + 630\,PLF_{SL}$

Snow Drift at parapet

USE 16" DEEP MASONRY LINTEL W/ 2 #5 CONT. BOTTOM

ML2 6' SPAN (MASONRY LINTEL)

$w = (20_{DL} + 31.5_{SL})(24.67'/2) + 78_{DL}(1.33') + 55_{DL}(16')$
$+ 28_{SL}(1.5) = 681\,PLF_{DL} + 431\,PLF_{SL}$

Snow Drift at parapet

USE 16" DEEP MASONRY LINTEL W/ 2 #5 CONT. BOTTOM

"JOIST.xls" Program
Version 1.6

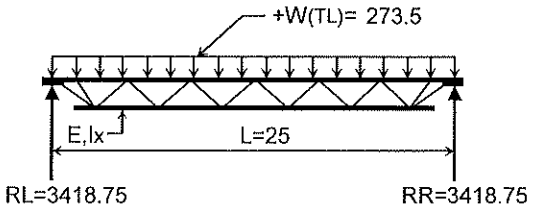
K-SERIES JOIST ANALYSIS

For Uniformly Loaded, Open-Web Steel Joists
Using Steel Joist Institute (SJI) Standard Load Tables

Job Name:	QNC	Subject:	SJ1		
Job Number:	2022152	Originator:		Checker:	

Input Data:

Joist Designation (Size) =	16K4	
Joist Span, L =	25.0	ft.
Joist Spacing, S =	5.0000	ft.
Unif. Dead Load, w(DL) =	20.00	psf
Unif. Live Load, w(LL) =	34.70	psf
Deflect. Factor, DF(LL) =	240	



Results:

Required Loads:

Dead Load, W(DL) =	100.0	plf	W(DL) = w(DL)*S
Live Load, W(LL) =	173.5	plf	W(LL) = w(LL)*S
Total Load, W(TL) =	273.5	plf	W(TL) = W(DL)+W(LL)
Reactions, RL & RR =	3418.8	lbs.	RL = RR = W(TL)*L/2

Nomenclature

For Joist Selected							
Joist Size	Joist Weight (plf)	Joist Inertia, Ix (in.^4)	Allowable Loads and Stress Ratios W(TL) (plf)	Flexure Ratio	W(LL) (plf)	Deflect. Ratio	# Rows of Bridging
16K4	7.0	78.4	313	0.874	293	0.593	2

	For 15 Lightest Acceptable K-series Joist Systems							
	Joist Sizes	Joist Weight (plf)	Joist Inertia, Ix (in.^4)	Allowable Loads and Stress Ratios W(TL) (plf)	Flexure Ratio	W(LL) (plf)	Deflect. Ratio	# Rows of Bridging
1	18K3	6.6	86.0	294	0.930	321	0.540	2
2	20K3	6.7	106.9	329	0.831	399	0.435	2
3	**16K4**	7.0	78.4	313	0.874	293	0.593	2
4	18K4	7.2	100.5	355	0.770	375	0.463	2
5	16K5	7.5	88.0	353	0.775	329	0.528	2
6	20K4	7.6	125.4	396	0.691	468	0.371	2
7	14K6	7.7	70.3	334	0.819	263	0.661	2
8	18K5	7.7	112.9	400	0.684	422	0.412	2
9	22K4	8.0	153.1	438	0.624	572	0.304	2
10	16K6	8.1	95.6	384	0.712	357	0.486	2
11	20K5	8.2	140.7	446	0.613	525	0.330	2
12	24K4	8.4	183.3	479	0.571	684	0.254	2
13	18K6	8.5	122.6	435	0.629	458	0.379	2
14	16K7	8.6	105.7	428	0.639	395	0.440	2
15	22K5	8.8	171.6	493	0.555	641	0.271	2

**(1-5) Denotes row of bridging nearest mid-span required to be diagonal bridging.

Notes: 1. OSHA is interpreting Section 29CFR-1926.751(c)2 to mean all joists whose length >= 40' require bolted diagonal bridging in place before slackening of hoisting lines.
2. For point loads on K-series joists, the magnitude and location of load should be indicated on the design drawings and ***"SP"*** must be added to the joist designation.

"JOIST.xls" Program
Version 1.6

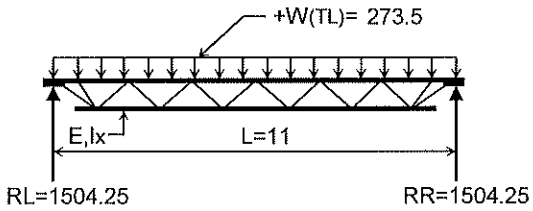
K-SERIES JOIST ANALYSIS

For Uniformly Loaded, Open-Web Steel Joists
Using Steel Joist Institute (SJI) Standard Load Tables

Job Name:	QNC	Subject:	SJ2		
Job Number:	2022152	Originator:		Checker:	

Input Data:

Joist Designation (Size) =	10K1	
Joist Span, L =	11.0	ft.
Joist Spacing, S =	5.0000	ft.
Unif. Dead Load, w(DL) =	20.00	psf
Unif. Live Load, w(LL) =	34.70	psf
Deflect. Factor, DF(LL) =	360	



Results:

Required Loads:

Dead Load, W(DL) =	100.0	plf	W(DL) = w(DL)*S
Live Load, W(LL) =	173.5	plf	W(LL) = w(LL)*S
Total Load, W(TL) =	273.5	plf	W(TL) = W(DL)+W(LL)
Reactions, RL & RR =	1504.3	lbs.	RL = RR = W(TL)*L/2

Nomenclature

For Joist Selected							
Joist Size	Joist Weight (plf)	Joist Inertia, Ix (in.^4)	Allowable Loads and Stress Ratios W(TL) (plf)	Flexure Ratio	W(LL) (plf)	Deflect. Ratio	# Rows of Bridging
10K1	5.0	17.6	550	0.497	542	0.320	1

	For 2 Lightest Acceptable K-series Joist Systems							
	Joist Sizes	Joist Weight (plf)	Joist Inertia, Ix (in.^4)	Allowable Loads and Stress Ratios W(TL) (plf)	Flexure Ratio	W(LL) (plf)	Deflect. Ratio	# Rows of Bridging
1	10K1	5.0	17.6	550	0.497	542	0.320	1
2	8K1	5.1	12.3	532	0.514	377	0.460	1

**(1-5) Denotes row of bridging nearest mid-span required to be diagonal bridging.

Notes: 1. OSHA is interpreting Section 29CFR-1926.751(c)2 to mean all joists whose length >= 40' require bolted diagonal bridging in place before slackening of hoisting lines.

2. For point loads on K-series joists, the magnitude and location of load should be indicated on the design drawings and ***"SP"*** must be added to the joist designation.

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 18 JAN 2023, 3:33PM

Masonry Beam

File: 2022152 - QNC Car Wash.ec6

Lic. # : KW-06012033

JTA Engineering, LLC

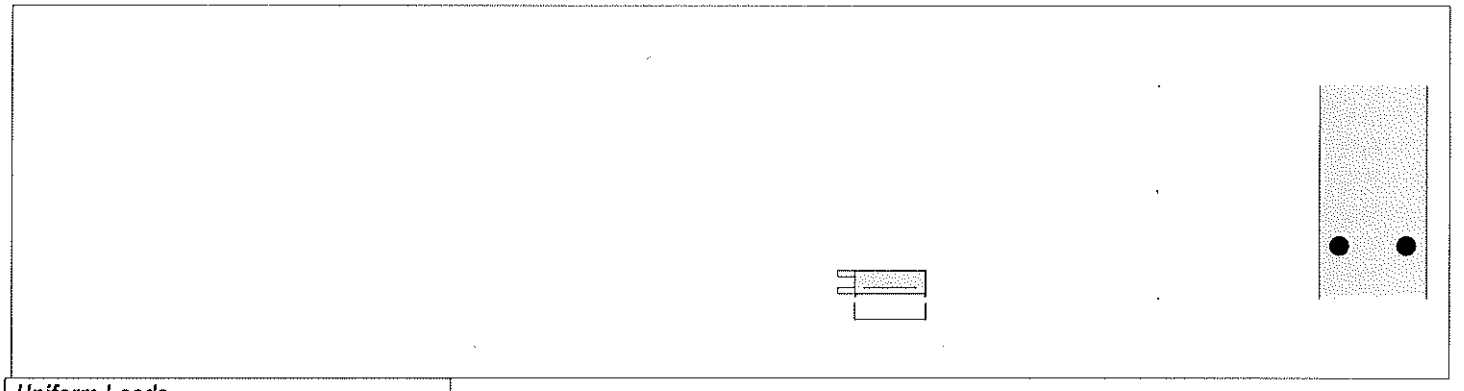
DESCRIPTION: ML1 (Masonry Lintel)

Code References

Calculations per TMS 402-16, IBC ~~2018~~ 2021, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

f'm	1,500.0 psi	Clear Span	4.0 ft	Rebar Size	5
Fs	24,000.0 psi	Beam Depth	1.330 ft	# Bars E/F	1
Em = f'm *	750.0	Thickness	8 in	Top Clear	4.0 in
Wall Wt Mult	1.0	End Fixity	Pin-Pin	Btm Clear	4.0 in
Block Type	Medium Wt	Equiv. Solid Thick	7.60 in	# Bar Sets	1
Lateral Wind Load	32.0 psf	Wall Weight	78.0 psf	Bar Spacing	5.0 in
Beam is Fully Braced ?	No	E	1,125.0 ksi		
Lateral Wall Weight Seismic Factor	0.1740	n	25.778		
Calculate vertical beam weight ?	No				



Uniform Loads

	Start X	End X	Dead Load	L : Floor Live	Lr : Roof Live	S : Snow	W : Wind	E : Earthquake
#1	ft	ft	0.9090			0.630		k/ft
#2	ft	ft						k/ft
#3	ft	ft						k/ft
#4	ft	ft						k/ft

DESIGN SUMMARY

Design OK

Maximum Stress Ratios...	Vertical	Lateral	SRSS Combination
fb/Fb	0.2661	0.02173	**0.2670** : 1.00
fv/Fv	0.7746	0.01943	**0.7749** : 1.00

Minimum Mn = 1.3 * Fcr * S = 3.507 k-ft

Maximum Moment	Actual	Allowable
Vertical Loads for Load Combination : +D+S	3.078 k-ft	11.565 k-ft
Lateral Loads for Load Combination : +1.024D+2.10E	0.07578 k-ft	3.487

Maximum Shear	Actual	Allowable
Vertical Loads for Load Combination : +D+S	33.752 psi	43.571 psi
Lateral Loads for Load Combination : +1.024D+2.10E	0.7525 psi	38.730 psi

Vertical Strength

As	0.620 in^2
rho	0.006799
np	0.1753
k : ((np)^2+2np)^.5-np	0.4422
j = 1 - k/3	0.8526
M:mas=Fb k j b d^2/2	11.565 k-ft
M:Stl = Fs As j d	12.645 k-ft

Lateral Strength

(Checking lateral bending for span)

As	0.310 in^2
rho	0.003077
np	0.07932
k : (np^2+2np)^.5-np	0.3268
j = 1 - k/3	0.8911
M:mas=Fb k j b d^2/2	5.209 k-ft
M:Stl = Fs As j d	3.487 k-ft

DESCRIPTION: ML1 (Masonry Lintel)

Detailed Load Combination Results

Load Combination	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
D Only	1.82	11.56	19.94	43.57	0.00	3.49	0.00	38.73
+D+S	3.08	11.56	33.75	43.57	0.00	3.49	0.00	38.73
+D+0.750S	2.76	11.56	30.30	43.57	0.00	3.49	0.00	38.73
+D+0.60W	1.82	11.56	19.94	43.57	0.05	3.49	0.51	38.73
+D+0.450W	1.82	11.56	19.94	43.57	0.04	3.49	0.38	38.73
+D+0.750S+0.450W	2.76	11.56	30.30	43.57	0.04	3.49	0.38	38.73
+0.60D+0.60W	1.09	11.56	11.96	43.57	0.05	3.49	0.51	38.73
+1.024D+2.10E	1.86	11.56	20.42	43.57	0.08	3.49	0.75	38.73
+1.018D+0.750S+1.575E	2.80	11.56	30.66	43.57	0.06	3.49	0.56	38.73
+0.5756D+2.10E	1.05	11.56	11.48	43.57	0.08	3.49	0.75	38.73

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 2:42PM
File: 2022152 - QNC Car Wash.ec6

Masonry Beam

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: ML2 (Masonry Lintel)

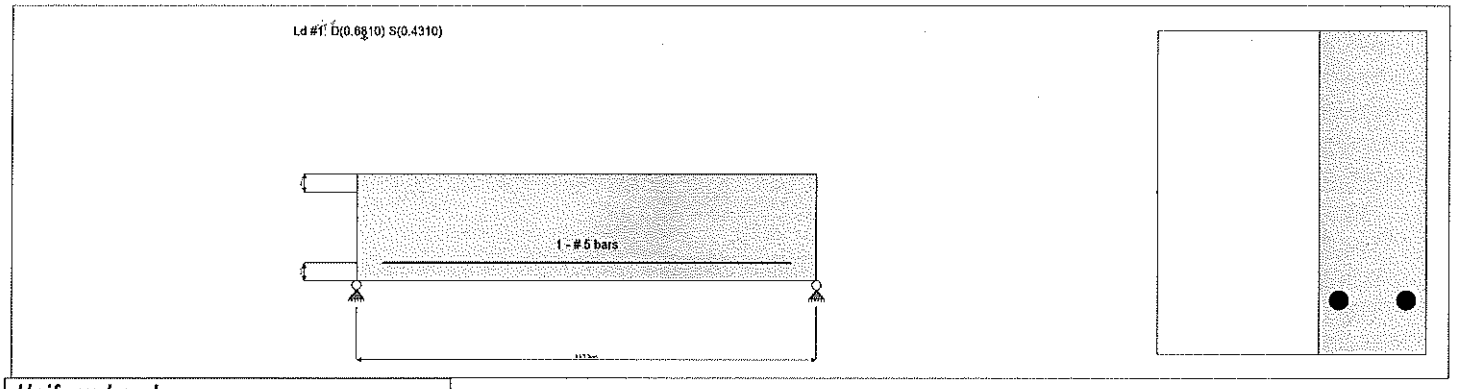
2021 OK

Code References

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

f'm	1,500.0 psi	Clear Span	8.670 ft	Rebar Size	5
Fs	24,000.0 psi	Beam Depth	2.0 ft	# Bars E/F	1
Em = f'm *	750.0	Thickness	8 in	Top Clear	4.0 in
Wall Wt Mult	1.0	End Fixity	Pin-Pin	Btm Clear	4.0 in
Block Type	Medium Wt	Equiv. Solid Thick	7.60 in	# Bar Sets	1
Lateral Wind Load	32.0 psf	Wall Weight	78.0 psf	Bar Spacing	5.0 in
Beam is Fully Braced ?	No	E	1,125.0 ksi		
Lateral Wall Weight Seismic Factor	0.1740	n	25.778		
Calculate vertical beam weight ?	No				



Uniform Loads

	Start X	End X	Dead Load	L : Floor Live	Lr : Roof Live	S : Snow	W : Wind	E : Earthquake
#1	ft	ft	0.6810			0.4310		k/ft
#2	ft	ft						k/ft
#3	ft	ft						k/ft
#4	ft	ft						k/ft

DESIGN SUMMARY

Design OK

Maximum Stress Ratios...	Vertical	Lateral	SRSS Combination
fb/Fb	0.4796	0.1507	**0.5028** : 1.00
fv/Fv	0.7255	0.04211	**0.7267** : 1.00

Minimum Mn = 1.3 * Fcr * S = 7.930 k-ft

Maximum Moment	Actual	Allowable
Vertical Loads for Load Combination : +D+S	10.448 k-ft	21.784 k-ft
Lateral Loads for Load Combination : +1.027D+2.10E	0.5354 k-ft	3.553

Maximum Shear	Actual	Allowable
Vertical Loads for Load Combination : +D+S	31.610 psi	43.571 psi
Lateral Loads for Load Combination : +1.027D+2.10E	1.631 psi	38.730 psi

Vertical Strength	
As	0.620 in^2
rho	0.004066
np	0.1048
k : ((np)^2+2np)^.5-np	0.3649
j = 1 - k/3	0.8784
M:mas=Fb k j b d^2/2	27.492 k-ft
M:Stl = Fs As j d	21.784 k-ft

Lateral Strength (Checking lateral bending for span)	
As	0.310 in^2
rho	0.002046
np	0.05275
k : (np^2+2np)^.5-np	0.2763
j = 1 - k/3	0.9079
M:mas=Fb k j b d^2/2	6.747 k-ft
M:Stl = Fs As j d	3.553 k-ft

DESCRIPTION: ML2 (Masonry Lintel)

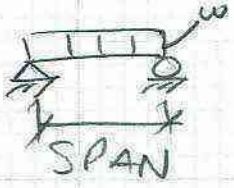
Detailed Load Combination Results

Load Combination	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
D Only	6.40	21.78	19.36	43.57	0.00	3.55	0.00	38.73
+D+S	10.45	21.78	31.61	43.57	0.00	3.55	0.00	38.73
+D+0.750S	9.44	21.78	28.55	43.57	0.00	3.55	0.00	38.73
+D+0.60W	6.40	21.78	19.36	43.57	0.36	3.55	1.10	38.73
+1.027D+2.10E	6.57	21.78	19.87	43.57	0.54	3.55	1.63	38.73
+D+0.450W	6.40	21.78	19.36	43.57	0.27	3.55	0.82	38.73
+D+0.750S+0.450W	9.44	21.78	28.55	43.57	0.27	3.55	0.82	38.73
+1.020D+0.750S+1.575E	9.56	21.78	28.93	43.57	0.40	3.55	1.22	38.73
+0.60D+0.60W	3.84	21.78	11.61	43.57	0.36	3.55	1.10	38.73
+0.5734D+2.10E	3.67	21.78	11.10	43.57	0.54	3.55	1.63	38.73

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #
QNC - LAS VEGAS	2020081
Section	Sheet no./rev.
Calc. by JB / Contact Phone # / EMAIL / Date	

ML3 16' SPAN (MASONRY LINTEL)



$w = (20_{DL} + 31.5_{SL})(5'/2) + 78_{DL}(3.33') + 63_{DL}(6.5') + 28_{SL}(1.5') = 720\ PLF_{DL} + 121\ PLF_{SL}$

Snow Drift at parapet

USE 40" DEEP MASONRY LINTEL W/ 2#5 TOP AND 2#6 BOTTOM

B1 24' SPAN (STEEL BEAM)

$w = (20_{DL} + 34.7_{SL})(5'+2') + 38_{SL}(2') + 15_{DL}(10') = 290\ PLF_{DL} + 319\ PLF_{SL}$

USE W14x26

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 18 JAN 2023, 4:02PM

Masonry Beam

Lic. # : KW-06012033 — JTA Engineering, LLC

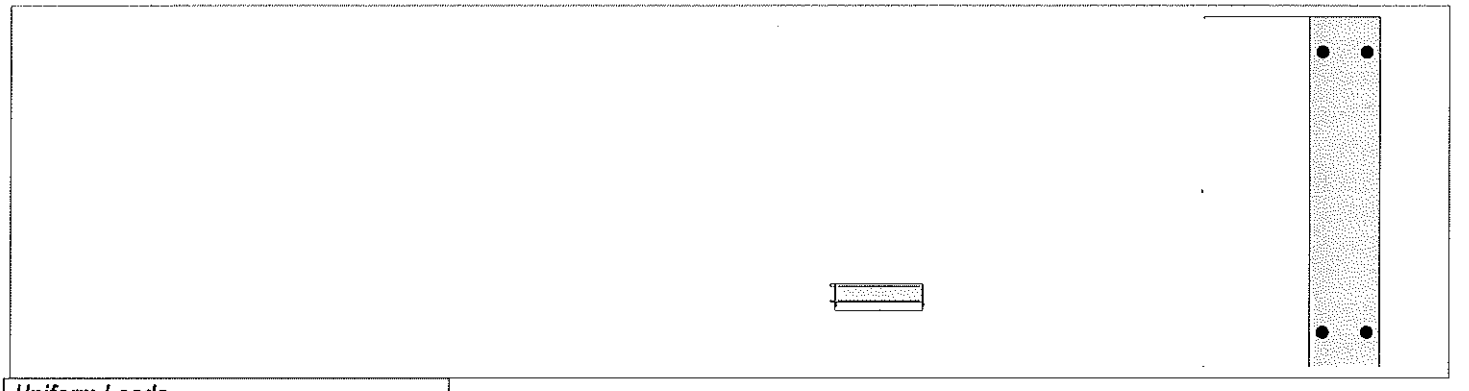
DESCRIPTION: ML3 (Masonry Lintel)

Code References

Calculations per TMS 402-16, IBC ~~2018~~ 2021, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

f'm	1,500.0 psi	Clear Span	16.0 ft	Rebar Size	5.0
Fs	24,000.0 psi	Beam Depth	3.330 ft	# Bars E/F	1
Em = f'm *	750.0	Thickness	8 in	Top Clear	4.0 in
Wall Wt Mult	1.0	End Fixity	Pin-Pin	Btm Clear	4.0 in
Block Type	Medium Wt	Equiv. Solid Thick	7.60 in	# Bar Sets	2
Lateral Wind Load	32.0 psf	Wall Weight	78.0 psf	Bar Spacing	5.0 in
Beam is Fully Braced ?	Yes	E	1,125.0 ksi		
Lateral Wall Weight Seismic Factor	0.1740	n	25.778		
Calculate vertical beam weight ?	No				



Uniform Loads

	Start X	End X	Dead Load	L : Floor Live	Lr : Roof Live	S : Snow	W : Wind	E : Earthquake
#1	ft	ft	0.720			0.1210		k/ft
#2	ft	ft						k/ft
#3	ft	ft						k/ft
#4	ft	ft						k/ft

DESIGN SUMMARY

Design OK

Maximum Stress Ratios...	Vertical	Lateral	SRSS Combination
fb/Fb	0.6676	0.4306	**0.7945** : 1.00
fv/Fv	0.5632	0.07772	**0.5685** : 1.00

Minimum Mn = 1.3 * Fcr * S = 21.984 k-ft

Maximum Moment	Actual	Allowable
Vertical Loads for Load Combination : +D+S	26.912 k-ft	40.308 k-ft
Lateral Loads for Load Combination : +1.027D+2.10E	3.036 k-ft	7.049

Maximum Shear	Actual	Allowable
Vertical Loads for Load Combination : +D+S	24.537 psi	43.571 psi
Lateral Loads for Load Combination : +1.027D+2.10E	3.010 psi	38.730 psi

Vertical Strength	
As	0.620 in^2
rho	0.002261
np	0.05829
k : ((np)^2+2np)^.5-np	0.2881
j = 1 - k/3	0.9040
M:mas=Fb k j b d^2/2	72.218 k-ft
M:Stl = Fs As j d	40.308 k-ft

Lateral Strength (Checking lateral bending for span)	
As	0.620 in^2
rho	0.002458
np	0.06336
k : (np^2+2np)^.5-np	0.2982
j = 1 - k/3	0.9006
M:mas=Fb k j b d^2/2	12.027 k-ft
M:Stl = Fs As j d	7.049 k-ft

DESCRIPTION: ML3 (Masonry Lintel)

Detailed Load Combination Results

Load Combination	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
D Only	23.04	40.31	21.01	43.57	0.00	7.05	0.00	38.73
+D+S	26.91	40.31	24.54	43.57	0.00	7.05	0.00	38.73
+D+0.750S	25.94	40.31	23.65	43.57	0.00	7.05	0.00	38.73
+D+0.60W	23.04	40.31	21.01	43.57	2.05	7.05	2.03	38.73
+1.027D+2.10E	23.65	40.31	21.57	43.57	3.04	7.05	3.01	38.73
+D+0.450W	23.04	40.31	21.01	43.57	1.53	7.05	1.52	38.73
+D+0.750S+0.450W	25.94	40.31	23.65	43.57	1.53	7.05	1.52	38.73
+1.020D+0.750S+1.575E	26.40	40.31	24.07	43.57	2.28	7.05	2.26	38.73
+0.60D+0.60W	13.82	40.31	12.60	43.57	2.05	7.05	2.03	38.73
+0.5734D+2.10E	13.21	40.31	12.05	43.57	3.04	7.05	3.01	38.73

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 18 JAN 2023, 4:29PM
File: 2022152 - QNC Car Wash.ec6

Lic. # : KW-06012033 JTA Engineering, LLC

Steel Beam

DESCRIPTION: B1 (Steel Lintel)

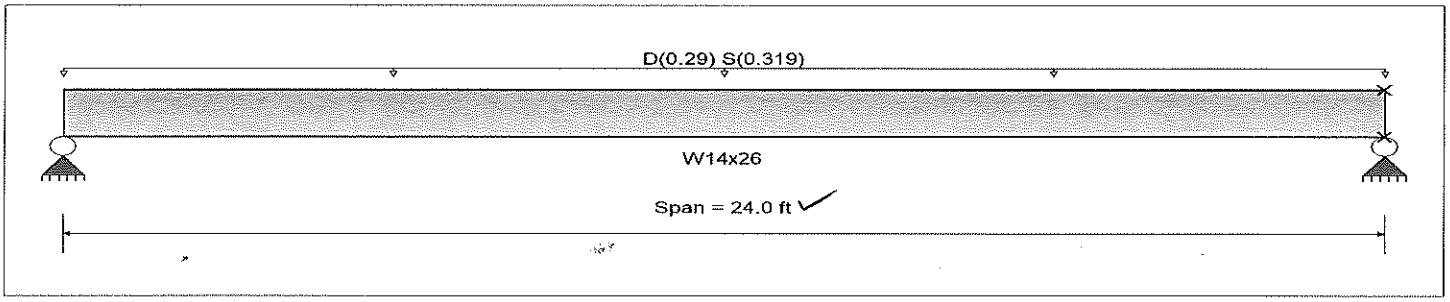
2021 ok

CODE REFERENCES

Calculations per AISC 360-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.290, S = 0.3190 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.456** : 1	Maximum Shear Stress Ratio =	**0.107** : 1
Section used for this span	**W14x26**	Section used for this span	**W14x26**
Ma : Applied	45.720 k-ft	Va : Applied	7.620 k
Mn / Omega : Allowable	100.299 k-ft	Vn/Omega : Allowable	70.890 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	12.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.337 in	Ratio =	855 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.670 in	Ratio =	430 >=180.
Max Upward Total Deflection	0.000 in	Ratio =	0 <180.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 24.00 ft	1	0.227	0.053	22.75		22.75	167.50	100.30	1.00	1.00	3.79	106.34	70.89
+D+S													
Dsgn. L = 24.00 ft	1	0.456	0.107	45.72		45.72	167.50	100.30	1.00	1.00	7.62	106.34	70.89
+D+0.750S													
Dsgn. L = 24.00 ft	1	0.399	0.094	39.98		39.98	167.50	100.30	1.00	1.00	6.66	106.34	70.89
+0.60D													
Dsgn. L = 24.00 ft	1	0.136	0.032	13.65		13.65	167.50	100.30	1.00	1.00	2.28	106.34	70.89

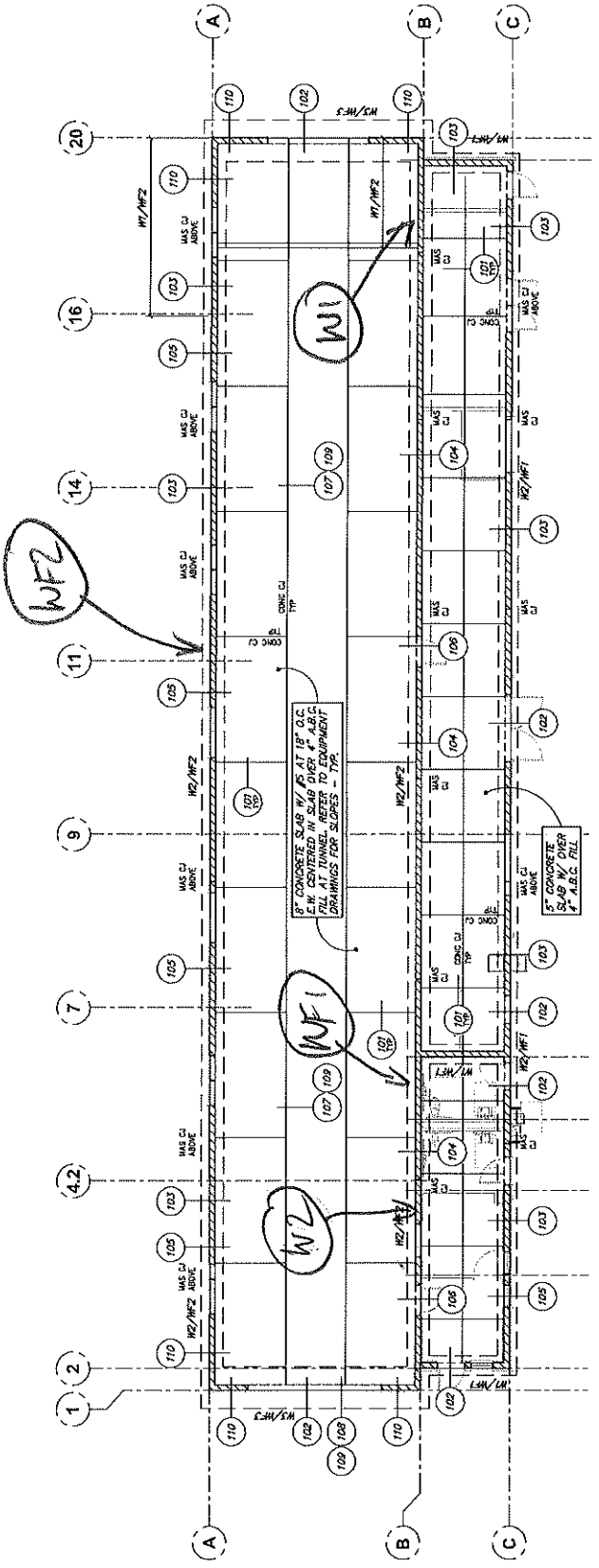
Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6702	12.069		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	7.620	7.620
Overall MINimum	2.275	2.275
D Only	3.792	3.792
+D+S	7.620	7.620
+D+0.750S	6.663	6.663
+0.60D	2.275	2.275
S Only	3.828	3.828



FOUNDATION KEY PLAN

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

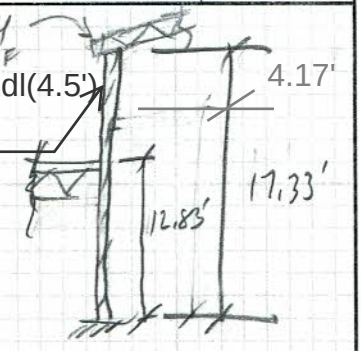
Project	Job #.
QNC	2020022
Section	Sheet no./rev.
Calc. by JMB / Contact Phone # / EMAIL / Date	

W1 17.33' HIGH BEARING WALL

$W = (20_{DL} + 34.7_{SL})(25.67'/2 + 10.67'/2) + 47dl(4.17') + 63dl(4.5') + 38sl(2') = 844\ PLF_{DL} + 707\ PLF_{SL}$

W3 - 6" MASONRY WALL OK BY INSPECTION

WIND = 25.4 PSF (C+C) (STRENGTH)

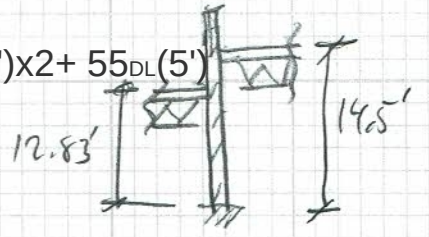


USE 8" MASONRY W/ 1#5 VERT. AT 16" O.C.

W2 14.5' HIGH BEARING WALL

$W = (20_{DL} + 34.7_{SL})(25.67'/2 + 10.67'/2) + 28sl(1.5')x2 + 55_{DL}(5') = 639\ PLF_{DL} + 715\ PLF_{SL}$

WIND = 25.4 PSF



USE 8" MASONRY W/ 1#5 VERT. AT 32" O.C.

WF1

$W = (20_{DL} + 20_{WL})(25.67'/2 + 10.67'/2) + 55_{DL}(18') = 1827\ PLF_{TL}$

REQ'D FOOTING WIDTH = 1827 PLF$_{TL}$ / 2000 PSF = 0.9' < 2.67' ⇒ OK

USE 14" THICK x 2'-8" WIDE CONT. CONCRETE FOOTING W/ 3#5 CONT. AND #5 AT 18" O.C. TRANSVERSE

WF2

$W = (20_{DL} + 20_{WL})(25.67'/2) + 55_{DL}(18') = 1581\ PLF_{TL}$

REQ'D FOOTING WIDTH = 1581 PLF$_{TL}$ / 2000 PSF = 0.8' < 2.67' ⇒ OK

USE 14" THICK x 2'-8" WIDE CONT. CONCRETE FOOTING W/ 4#5 CONT. AND #5 AT 18" O.C. TRANSVERSE

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

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Masonry Slender Wall

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: W1 (Masonry Wall)

2021 ok

Code References

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

2021 OK

General Information

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16

Construction Type : Grouted Hollow Concrete Masonry

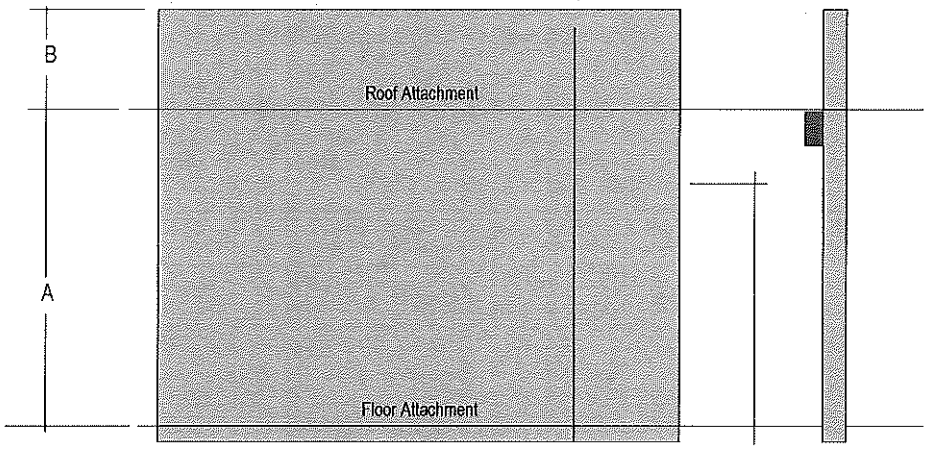
F'm	=	1.50 ksi	Nom. Wall Thickness		8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness		7.625 in	Min Allow Out-of-plane Defl Ratio	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance		3.750 in			
Em = f'm *	=	900.0	Lower Level Rebar . . .			Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.1461	Bar Size	#	5			
Grout Density	=	140 pcf	Bar Spacing		16 in			
Block Weight		Medium Weight						
Wall Weight	=	63.0 psf						

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height = 17.33 ft ✓
B Parapet height = ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . *(Applied per foot of Strip Width)*		DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind
Ledger Load	Eccentricity 6.750 in					k/ft
Concentric Load		.844 ✓			.707 ✓	k/ft

Lateral Loads

Wind Loads :
Full area WIND load 32.0 psf

Seismic Loads :
Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.174
Fp = Wall Wt. * 0.06960 = 4.385 psf

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
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JTA Engineering, LLC

Masonry Slender Wall

Lic. # : KW-06012033

DESCRIPTION: W1 (Masonry Wall)

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+W	Maximum Bending Stress Ratio =	0.3623 ✓		
		Max Mu	1.282 k-ft	Phi * Mn	3.539 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/	263	Allowable Defl. Ratio	150.0
		Max. Deflection	0.7916 in		
PASS	Axial Load Check +1.20D+0.50S+W	Max Pu / Ag	29.670 psi	Max. Allow. Defl.	1.386 in
		Location	8.376 ft	0.2 * f'm	300.0 psi
PASS	Reinforcing Limit Check				
		Actual As/bd	0.0050	Max Allow As/bd	0.1461

Maximum Reactions . . .	*for Load Combination....*	
Top Horizontal	W Only	0.2773 k
Base Horizontal	W Only	0.2773 k
Vertical Reaction	+D+S	2.643 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	0.2*f'm*b*t k	Moment Values Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+0.50W at 8.09 to 8.67	1.712	20.880	0.50	0.62	0.90	3.64	0.233	0.0050	0.1458
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60S+0.50W at 8.09 to 8.67	2.843	20.880	0.50	0.63	0.90	3.88	0.233	0.0050	0.1454
+1.20D+W at 8.09 to 8.67	1.712	20.880	0.50	1.31	0.90	3.64	0.233	0.0050	0.1458
+1.20D+0.50S+W at 8.09 to 8.67	2.065	20.880	0.50	1.34	0.90	3.72	0.233	0.0050	0.1456
+0.90D+W at 8.09 to 8.67	1.284	20.880	0.50	1.28	0.90	3.55	0.233	0.0050	0.1459
+1.235D+0.20S+3.0E at 8.09 to 8.67	1.903	20.880	0.50	0.50	0.90	3.68	0.233	0.0050	0.1457
+0.8652D+3.0E at 8.09 to 8.67	1.234	20.880	0.50	0.50	0.90	3.54	0.233	0.0050	0.1459

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Mactual k-ft	Stiffness I gross in^4	I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 8.09 to 8.67	1.426	0.50	0.75	376.00	44.54	48.940	0.277	750.3
+D+0.450W at 8.09 to 8.67	1.426	0.50	0.55	376.00	44.54	70.072	0.082	2,546.0
+D+0.750S+0.450W at 8.09 to 8.67	1.957	0.50	0.55	376.00	45.62	70.012	0.084	2,474.1
+0.60D+0.60W at 8.09 to 8.67	0.856	0.50	0.74	376.00	43.36	48.005	0.269	772.1
+D+0.70E at 8.09 to 8.67	1.426	0.50	0.12	376.00	44.54	376.000	0.012	16,779.6
+D+0.750S+0.5250E at 8.09 to 8.67	1.957	0.50	0.09	376.00	45.62	376.000	0.009	22,269.0
+0.60D+0.70E at 8.09 to 8.67	0.856	0.50	0.12	376.00	43.36	376.000	0.012	16,861.4
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 8.67 to 9.24	0.000	0.50	1.20	376.00	41.55	42.286	0.792	262.7
E Only at 8.67 to 9.24	0.000	0.50	0.16	376.00	41.55	376.000	0.017	11,888.9

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.936 k

DESCRIPTION: W1 (Masonry Wall)

+D+S	0.0 k	0.00 k	2.643 k
+D+0.750S	0.0 k	0.00 k	2.466 k
+D+0.60W	0.2 k	0.17 k	1.936 k

DESCRIPTION: W1 (Masonry Wall)

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.450W	0.1 k	0.13 k	1.936 k
+D+0.750S+0.450W	0.1 k	0.13 k	2.466 k
+0.60D+0.60W	0.2 k	0.17 k	1.161 k
+D+0.70E	0.0 k	0.03 k	1.936 k
+D+0.750S+0.5250E	0.0 k	0.02 k	2.466 k
+0.60D+0.70E	0.0 k	0.03 k	1.161 k
S Only	0.0 k	0.00 k	0.707 k
W Only	0.3 k	0.28 k	0.000 k
E Only	0.0 k	0.04 k	0.000 k

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

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JTA Engineering, LLC

Masonry Slender Wall

Lic. # : KW-06012033

DESCRIPTION: W2 (Masonry Wall)

2021 OK

Code References

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

2021 OK

General Information

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16

Construction Type : Grouted Hollow Concrete Masonry

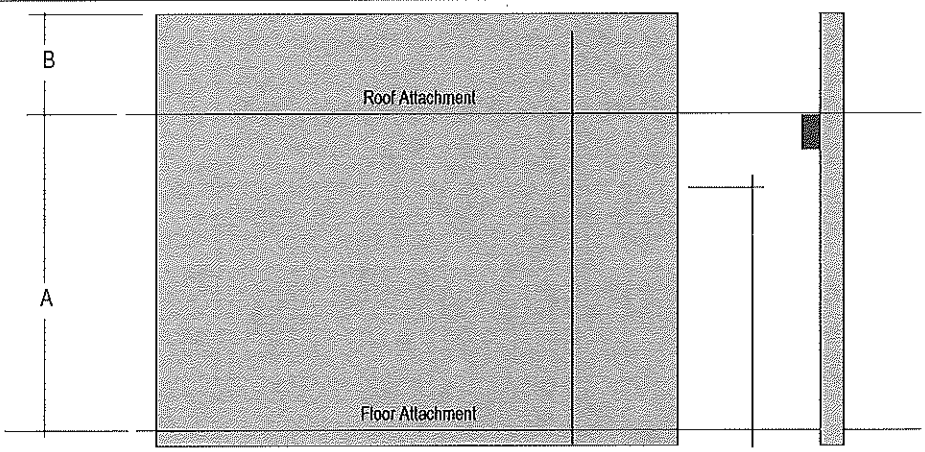
F'm	=	1.50 ksi	Nom. Wall Thickness	8	in	Temp Diff across thickness =	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625	in	Min Allow Out-of-plane Defl Ratio =	0.0
Fr - Rupture	=	267.0 psi	Rebar "d" distance	3.810	in		
Em = f'm *	=	900.0	Lower Level Rebar . . .			Minimum Vertical Steel % =	0.0020
Max % of ρ bal.	=	0.1114	Bar Size #	5			
Grout Density	=	140 pcf	Bar Spacing	32	in		
Block Weight		Medium Weight					
Wall Weight	=	55.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height = 14.5 ft ✓
B Parapet height = ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . *(Applied per foot of Strip Width)*		DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind
Ledger Load	Eccentricity 6.750 in					k/ft
Concentric Load		.639 ✓			.715 ✓	k/ft

Lateral Loads

Wind Loads :
Full area WIND load 32 psf

Seismic Loads :
Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.174
Fp = Wall Wt. * 0.06960 = 3.828 psf

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JTA Engineering, LLC

Masonry Slender Wall

Lic. # : KW-06012033

DESCRIPTION: W2 (Masonry Wall)

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+W	Maximum Bending Stress Ratio =	**0.4363** ✓		
		Max Mu	0.8445 k-ft	Phi * Mn	1.936 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/	2,533	Allowable Defl. Ratio	150.0
		Max. Deflection	0.06870 in		
PASS	Axial Load Check +1.20D+0.50S+W	Max Pu / Ag	27.801 psi	Max. Allow. Defl.	1.160 in
		Location	7.008 ft	0.2 * f'm	300.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.002539	Max Allow As/bd	0.1114

Maximum Reactions . . . *for Load Combination....*

Top Horizontal	W Only	0.2320 k
Base Horizontal	W Only	0.2320 k
Vertical Reaction	+D+S	2.152 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	0.2*f'm*b*t k	Moment Values Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+0.50W at 6.77 to 7.25	1.277	17.640	2.00	0.42	0.90	2.02	0.116	0.0025	0.1111
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60S+0.50W at 6.77 to 7.25	2.421	17.640	2.00	0.43	0.90	2.30	0.116	0.0025	0.1107
+1.20D+W at 6.77 to 7.25	1.277	17.640	2.00	0.85	0.90	2.02	0.116	0.0025	0.1111
+1.20D+0.50S+W at 6.77 to 7.25	1.635	17.640	2.00	0.85	0.90	2.11	0.116	0.0025	0.1110
+0.90D+W at 6.77 to 7.25	0.958	17.640	2.00	0.84	0.90	1.94	0.116	0.0025	0.1113
+1.235D+0.20S+3.0E at 6.77 to 7.25	1.457	17.640	2.00	0.30	0.90	2.07	0.116	0.0025	0.1111
+0.8652D+3.0E at 6.77 to 7.25	0.921	17.640	2.00	0.30	0.90	1.93	0.116	0.0025	0.1113

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Mactual k-ft	Stiffness I gross in^4	I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 6.77 to 7.25	1.064	2.00	0.51	342.40	26.52	342.400	0.042	4,192.0
+D+0.450W at 6.77 to 7.25	1.064	2.00	0.38	342.40	26.52	342.400	0.031	5,589.3
+D+0.750S+0.450W at 6.77 to 7.25	1.601	2.00	0.38	342.40	27.89	342.400	0.031	5,569.2
+0.60D+0.60W at 6.77 to 7.25	0.639	2.00	0.51	342.40	25.42	342.400	0.041	4,203.6
+D+0.70E at 6.77 to 7.25	1.064	2.00	0.07	342.40	26.52	342.400	0.006	30,036.4
+D+0.750S+0.5250E at 6.77 to 7.25	1.601	2.00	0.05	342.40	27.89	342.400	0.004	39,905.0
+0.60D+0.70E at 6.77 to 7.25	0.639	2.00	0.07	342.40	25.42	342.400	0.006	30,120.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 7.25 to 7.73	0.000	2.00	0.84	342.40	23.72	342.400	0.069	2,532.7
E Only at 7.25 to 7.73	0.000	2.00	0.10	342.40	23.72	342.400	0.008	21,171.7

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.436 k

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
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Masonry Slender Wall

Lic. # : KW-06012033 JTA Engineering, LLC

DESCRIPTION: W2 (Masonry Wall)

+D+S	0.0 k	0.00 k	2.151 k
+D+0.750S	0.0 k	0.00 k	1.973 k
+D+0.60W	0.1 k	0.14 k	1.436 k

JARAMILLO-TONNESON & ASSOCIATES
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JTA Engineering, LLC

Masonry Slender Wall

Lic. # : KW-06012033

DESCRIPTION: W2 (Masonry Wall)

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.450W	0.1 k	0.10 k	1.436 k
+D+0.750S+0.450W	0.1 k	0.10 k	1.973 k
+0.60D+0.60W	0.1 k	0.14 k	0.862 k
+D+0.70E	0.0 k	0.02 k	1.437 k
+D+0.750S+0.5250E	0.0 k	0.01 k	1.973 k
+0.60D+0.70E	0.0 k	0.02 k	0.862 k
S Only	0.0 k	0.00 k	0.715 k
W Only	0.2 k	0.23 k	0.000 k
E Only	0.0 k	0.03 k	0.000 k

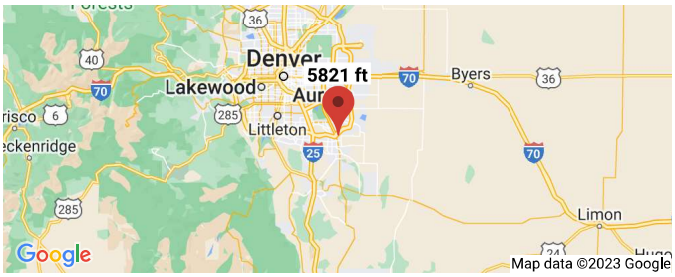
This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

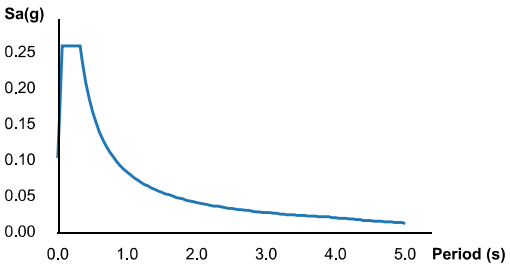
Hazards by Location

Search Information

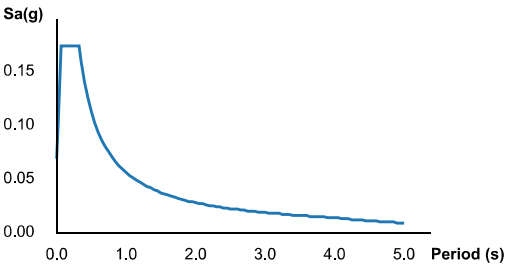
Address:	Parker Road and Pine Land Avenue, Parker, CO
Coordinates:	39.5441697, -104.7719669
Elevation:	5821 ft
Timestamp:	2023-01-18T19:10:07.811Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	C



MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.2	MCE_R ground motion (period=0.2s)
S_1	0.056	MCE_R ground motion (period=1.0s)
S_{MS}	0.26	Site-modified spectral acceleration value
S_{M1}	0.084	Site-modified spectral acceleration value
S_{DS}	0.174	Numeric seismic design value at 0.2s SA
S_{D1}	0.056	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.3	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s
CR_S	0.95	Coefficient of risk (0.2s)
CR_1	0.926	Coefficient of risk (1.0s)
PGA	0.109	MCE_G peak ground acceleration
F_{PGA}	1.291	Site amplification factor at PGA
PGA_M	0.141	Site modified peak ground acceleration
T_L	4	Long-period transition period (s)
SsRT	0.2	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.211	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.056	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.061	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Code Search

Code: ASCE 7 - 16

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length (L)	150.0 ft	
Least width (B)	36.0 ft	
Mean Roof Ht (h)	14.1 ft	
Parapet ht above grd	18.6 ft	
Minimum parapet ht	3.5 ft	

Live Loads:

Roof
0 to 200 sf: 20 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12 psf

Floor:

Typical Floor		N/A
Partitions		N/A
Partitions	N/A	
Partitions		N/A
Partitions	N/A	

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

JOB NO. 2022152 SHEET NO.
CALCULATED BY JMB DATE
CHECKED BY SJJ DATE

Wind Loads : ASCE 7- 16

Ultimate Wind Speed	107 mph
Nominal Wind Speed	82.9 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.849
Kh case 2	0.849
Type of roof	Monoslope

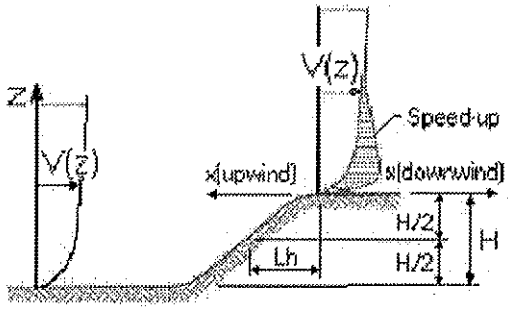
Topographic Factor (Kzt)

Topography		Flat
Hill Height	(H)	0.0 ft
Half Hill Length (Lh)		0.0 ft
Actual H/Lh	=	0.00
Use H/Lh	=	0.00
Modified Lh	=	0.0 ft
From top of crest: x =		50.0 ft
Bldg up/down wind?		downwind

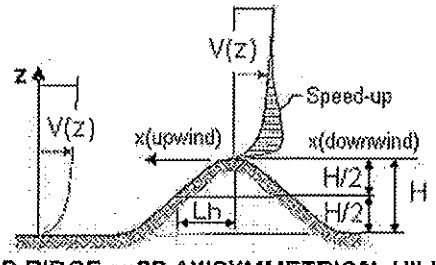
H/Lh= 0.00 K_1 = 0.000
x/Lh = 0.00 K_2 = 0.000
z/Lh = 0.00 K_3 = 1.000

At Mean Roof Ht:
Kzt = $(1+K_1K_2K_3)^2$ = 1.00

H< 15ft;exp C
∴ Kzt=1.0



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	14.1 ft
B =	36.0 ft
/z (0.6h) =	15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.39 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

$\bar{e}$ =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	427.1 ft
Q =	0.93
I_z =	0.23
G =	0.89 use G = 0.85

Flexible or Dynamically Sensitive Structure

34 ıcy (η_1) =	0.0 Hz			
Damping ratio (β) =	0			
/b =	0.65			
/α =	0.15			
Vz =	90.4			
N_1 =	0.00			
R_n =	0.000			
R_h =	28.282	η =	0.000	h = 14.1 ft
R_B =	28.282	η =	0.000	
R_L =	28.282	η =	0.000	
g_R =	0.000			
R =	0.000			
Gf =	0.000			

JTA Engineering
15825 S 46th St, Suite 129
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623.237.9349

Enclosure Classification

Test for Enclosed Building: Ao < 0.01Ag or 4 sf, whichever is smaller

Test for Open Building: All walls are at least 80% open.
Ao ≥ 0.8Ag

Test for Partially Enclosed Building: Predominately open on one side only

	Input			Test	
Ao	500.0	sf	Ao ≥ 1.1Aoi	NO	
Ag	600.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	1000.0	sf	Aoi / Agi ≤ 0.20	YES	Building is NOT Partially Enclosed
Agi	10000.0	sf			

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
Ao ≥ 1.1Aoi
Ao > smaller of 4' or 0.01 Ag
Aoi / Agi ≤ 0.20
Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Test for Partially Open Building: A building that does not qualify as open, enclosed or partially enclosed. (This type building will have same wind pressures as an enclosed building.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog): 0 sf
Unpartitioned internal volume (Vi) : 0 cf
Ri = 1.00

Ground Elevation Factor (Ke)

Grd level above sea level = 0.0 ft
Constant = 0.00256 Adj Constant = 0.00256
Ke = 1.0000

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

Wind Loads - MWFRS all h (Except for Open Buildings)

Kh (case 2) =	0.85	h =	14.1 ft	GCpi =	+/-0.18
Base pressure (q_h) =	**21.1 psf**	ridge ht =	23.0 ft	G =	0.85
Roof Angle (θ) =	1.2 deg	L =	150.0 ft	qi = qh	
Roof tributary area - (h/2)*L:	1058 sf	B =	36.0 ft		
(h/2)*B:	254 sf				

Ultimate Wind Surface Pressures (psf)

Surface	Wind Normal to Ridge B/L = 0.24 Cp	q_hGC_p	h/L = 0.39 w/+q_iGC_{pi}	w/-q_hGC_{pi}	Wind Parallel to Ridge L/B = 4.17 Dist.*	Cp	q_hGC_p	h/L = 0.09 w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	14.4	see table below			0.80	14.4	see table below	
Leeward Wall (LW)	-0.50	-9.0	-12.8	-5.2		-0.20	-3.6	-7.4	0.2
Side Wall (SW)	-0.70	-12.6	-16.4	-8.8		-0.70	-12.6	-16.4	-8.8
Leeward Roof (LR)		**				Included in windward roof			
Neg Windward Roof: 0 to h/2*	-0.90	-16.2	-20.0	-12.4	0 to h/2*	-0.90	-16.2	-20.0	-12.4
h/2 to h*	-0.90	-16.2	-20.0	-12.4	h/2 to h*	-0.90	-16.2	-20.0	-12.4
h to 2h*	-0.50	-8.99	-12.79	-5.18	h to 2h*	-0.50	-9.0	-12.8	-5.2
> 2h*	-0.30	-5.39	-9.20	-1.59	> 2h*	-0.30	-5.4	-9.2	-1.6
Pos/min windward roof press.	-0.18	-3.2	-7.0	0.6	Min press.	-0.18	-3.2	-7.0	0.6

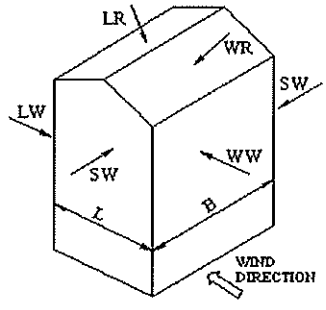
**Roof angle < 10 degrees. Therefore, leeward roof is included in windward roof pressure zones.

*Horizontal distance from windward edge

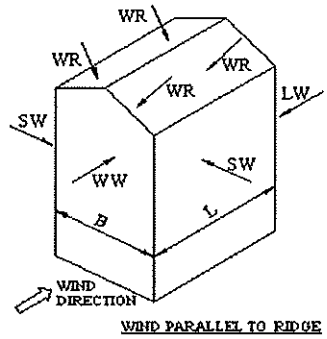
For monoslope roofs, entire roof surface is either windward or leeward surface.

Windward Wall Pressures at "z" (psf)

	z	Kz	Kzt	Windward Wall q_zGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Combined WW + LW Normal to Ridge	Parallel to Ridge
h=	0 to 15'	0.85	1.00	14.4	10.6	18.2	23.4	18.0
ridge =	23.0 ft	0.93	1.00	15.7	11.9	19.5	24.7	19.3



WIND NORMAL TO RIDGE



WIND PARALLEL TO RIDGE

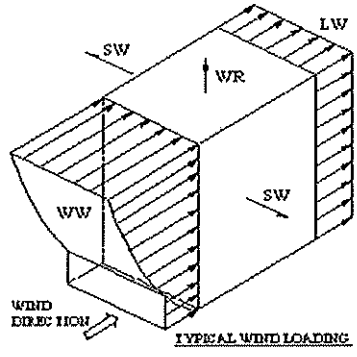
NOTE:
See figure in ASCE7 for the application of full and partial loading of the above wind pressures. There are 4 different loading cases.

Parapet

z	Kz	Kzt	qp (psf)
18.6 ft	0.89	1.00	22.1

Windward parapet: 33.2 psf (GCpn = +1.5)
Leeward parapet: -22.1 psf (GCpn = -1.0)

Windward roof overhangs (add to windward roof pressure) : 14.4 psf (upward)



TYPICAL WIND LOADING

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.85	h =	14.1 ft	0.2h = 2.8 ft
Base pressure (qh) =	**21.1 psf**	0.6h =	8.5 ft	
Minimum parapet ht =	3.5 ft	GCpi =	+/-0.18	
Roof Angle (θ) =	1.2 deg	qi = qh =	21.1 psf	
Type of roof =	Monoslope			

Roof

	Surface Pressure (psf)								User input	
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf	53 sf	120 sf
Negative Zone 1	-39.8	-37.1	-33.7	-31	-28.4	-26.3	-25.0	-25.0	-33.4	-30.4
Negative Zone 1'	-22.8	-22.8	-22.8	-22.8	-19.7	-17.1	-16.0	-16.0	-22.8	-22.0
Negative Zone 2	-52.4	-49.1	-44.6	-41.2	-37.9	-35.2	-33.4	-33.4	-44.3	-40.4
Negative Zone 3	-52.4	-49.1	-44.6	-41.2	-37.9	-35.2	-33.4	-33.4	-44.3	-40.4
Positive Zone 1 & 1'	16	16	16	16	16.0	16.0	16.0	16.0	16.0	16.0
Positive Zones 2 & 3	22.8	21.8	20.5	19.5	18.5	17.7	17.1	16.1	20.4	19.2
Overhang Zone 1&1'	-36	-35.3	-34.5	-33.8	-28.4	-24.0	-21.1	-21.1	-34.4	-32.4
Overhang Zone 2	-48.6	-44.1	-38.2	-33.7	-29.2	-25.6	-23.3	-23.3	-37.8	-32.5
Overhang Zone 3	-48.6	-44.1	-38.2	-33.7	-29.2	-25.6	-23.3	-23.3	-37.8	-32.5

Negative zone 3 = zone 2, since parapet >= 3ft.
Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 3.8 psf)

Parapet

qp = 22.1 psf

		Surface Pressure (psf)					User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	24 sf
CASE A:	Zone 2 :	70.8	66.2	60.2	55.6	51.0	44.9	65.0
	Zone 3 :	70.8	66.2	60.2	55.6	51.0	44.9	65.0
CASE B :	Interior zone :	-41.8	-39.7	-36.9	-34.8	-32.7	-29.9	-39.1
	Corner zone :	-47.8	-44.6	-40.4	-37.2	-34.1	-29.9	-43.8

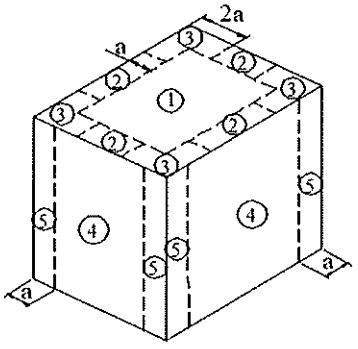
Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	29 sf	52 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-24.7	-21.4	-20.4	-19.0	-23.2	-22.3
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-30.5	-23.7	-21.7	-19.0	-27.3	-25.6
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	22.8	19.5	18.5	17.1	21.3	20.4

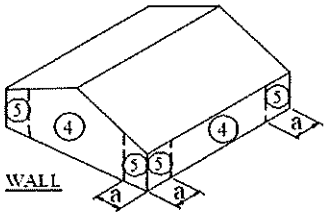
Note: GCp reduced by 10% due to roof angle <= 10 deg.

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

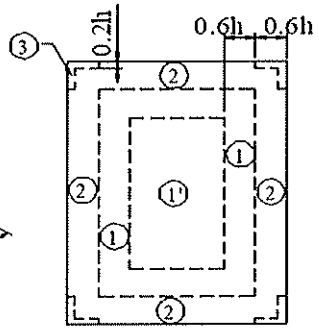
Location of C&C Wind Pressure Zones - ASCE 7-16



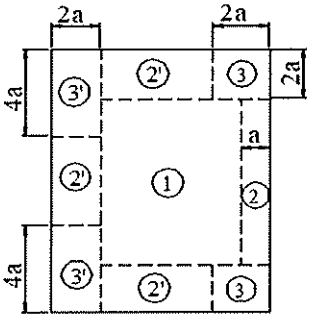
Roofs w/ $\theta \leq 10°$
and all walls
h > 60'



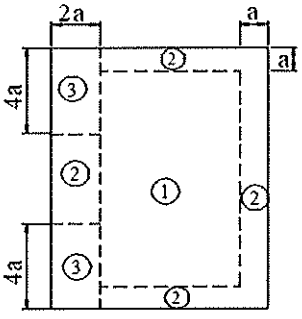
Walls h ≤ 60'
& alt design h<90'



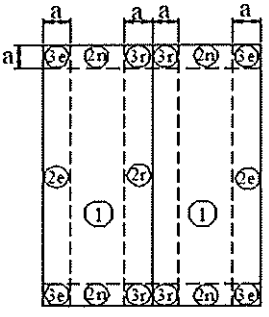
Gable, Sawtooth and
Multispan Gable $\theta \leq 7$ degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



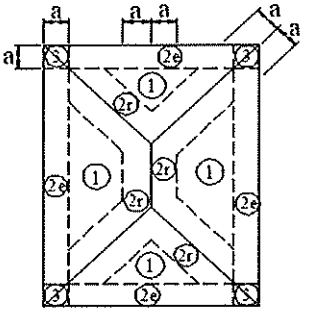
Monoslope roofs
$3° < \theta \leq 10°$
h ≤ 60' & alt design h<90'



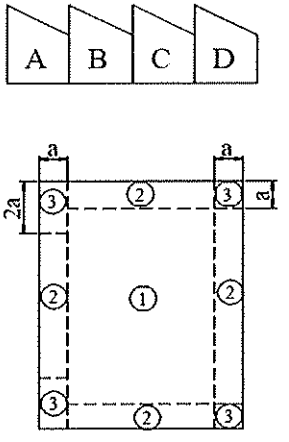
Monoslope roofs
$10° < \theta \leq 30°$
h ≤ 60' & alt design h<90'



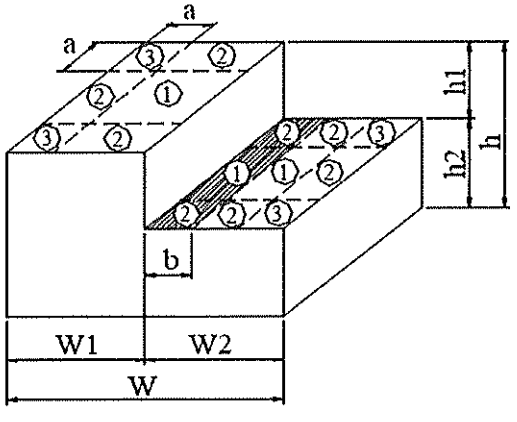
Multispan Gable &
Gable $7° < \theta \leq 45°$

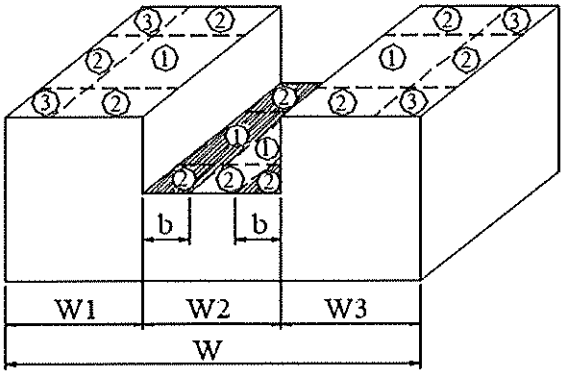


Hip $7° < \theta \leq 27°$



Sawtooth $10° < \theta \leq 45°$
h ≤ 60' & alt design h<90'





Stepped roofs $\theta \leq 3°$
h ≤ 60' & alt design h<90'

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #.
QNC	
Section	Sheet no./rev.
Calc. by JB	Contact Phone # / EMAIL / Date

LATERAL ANALYSIS

DIAPHRAGM 1

SEISMIC WEIGHT

ROOF WEIGHT = $20_{PSF} \times 3246\,ft^2 = 64.9^K$

WALLS = $63_{PLF}(15'/2 + 2.33') \times 8.2' + 55_{PLF}(15'/2 + 2.33') \times 265.5'$
$+ 78_{PLF}(14.75'/2 + 2.58') \times 9.33' + 78_{PLF}(4.67') \times 16'$
$= 161.7^K$

$\Sigma_{TOTAL} = 64.9^K + 161.7^K = 226.6^K$

$V_{BASE} =$ 11.22 ^{K}

DIAPHRAGM 2

SEISMIC WEIGHT

ROOF WEIGHT = $20_{PSF} \times 533\,ft^2 = 10.66^K$

WALLS = $63_{PLF}(17.33'/2) \times 26.33' + 78_{PLF}(17.33'/2) \times 13.33' + 78_{PLF}(6') \times 12'$
$= 29^K$

$\Sigma_{TOTAL} = 10.66^K + 29^K = 39.66^K$

$V_{BASE} =$ 1.96 ^{K}

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

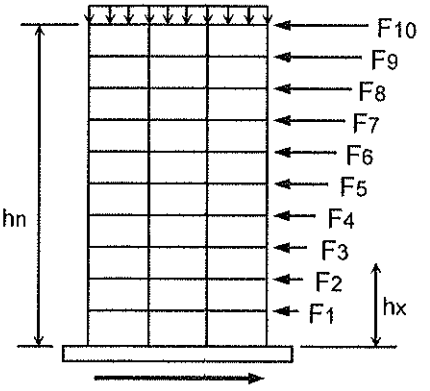
Per IBC 2018 and ASCE 7-16 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	QNC Car Wash	Subject:	Lateral		
Job Number:	2022152	Originator:	JMB	Checker:	SJJ

Input Data:

Risk Category =	II	IBC 2018, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2, page 5
Soil Site Class =	C	ASCE 7-16 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., Ss =	0.200	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S1 =	0.056	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, TL =	4.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, hn =	15.000	ft.
Actual Calc. Period, Tc =	0.100	sec. from independent analysis
Seismic Resist. System =	A8	Intermediate reinf. masonry shear walls (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 11.22 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	15.000	226.60

Total Weight, W = ΣWx = 226.60 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

Fa =	1.300	ASCE 7-16 Table 11.4-1, page 66
Fv =	1.500	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	0.260	SMS = Fa*Ss, ASCE Eqn. 11.4-1, page 65
SM1 =	0.084	SM1 = Fv*S1, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.173	SDS = 2*SMS/3, ASCE 7-16 Eqn. 11.4-3, page 65
SD1 =	0.056	SD1 = 2*SM1/3, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1, page 67
Category(for S_{D1}) =	A	ASCE 7-16 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2, page 90
Approx. Period, T_a =	0.152	sec., $T_a = C_T * h_n^{(x)}$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.700	ASCE 7-16 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.259	sec., $T_{(max)} = C_u * T_a$, ASCE 7-16 Section 12.8.2, page 90
Fundamental Period, T =	0.100	sec., T = Tc <= Cu*Ta, ASCE 7-16 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-16 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	2.5	ASCE 7-16 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.25	ASCE 7-16 Table 12.2-1, pages 73-75
C_S =	0.050	$C_S = S_{DS}/(R/I)$, ASCE 7-16 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.160	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-16 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-16 Eqn. 12.8-5
Use: C_S =	0.050	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 11.22 kips, $V = C_S * W$, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx} * V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11, page 91
Vertical Distribution Factor: $C_{vx} = W_x * h_x^k / (\Sigma W_i * h_i^k)$, ASCE 7-16 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	h_x^k (ft.)	$W_x * h_x^k$ (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears
1	226.60	15.000	3399.0	1.000	11.22	11.22
Σ =	226.60		3399.0	1.000	11.22	

Comments:

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

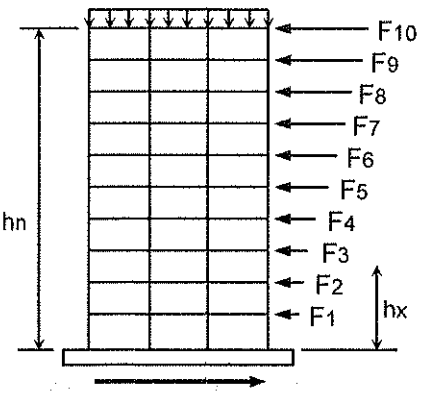
Per IBC 2018 and ASCE 7-16 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	QNC Car Wash	Subject:	Lateral - Diaphragm 2		
Job Number:	2022152	Originator:	JMB	Checker:	SJJ

Input Data:

Risk Category =	II	IBC 2018, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2, page 5
Soil Site Class =	C	ASCE 7-16 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., Ss =	0.200	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S1 =	0.056	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, TL =	4.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, hn =	17.330	ft.
Actual Calc. Period, Tc =	0.100	sec. from independent analysis
Seismic Resist. System =	A8	Intermediate reinf. masonry shear walls (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 1.96 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	17.330	39.66

Total Weight, W = ΣWx = 39.66 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

Fa =	1.300	ASCE 7-16 Table 11.4-1, page 66
Fv =	1.500	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	0.260	SMS = Fa*Ss, ASCE Eqn. 11.4-1, page 65
SM1 =	0.084	SM1 = Fv*S1, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.173	SDS = 2*SMS/3, ASCE 7-16 Eqn. 11.4-3, page 65
SD1 =	0.056	SD1 = 2*SM1/3, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1, page 67
Category(for S_{D1}) =	A	ASCE 7-16 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2, page 90
Approx. Period, T_a =	0.170	sec., $T_a = C_T * h_n^{(x)}$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.700	ASCE 7-16 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.289	sec., $T_{(max)} = C_u * T_a$, ASCE 7-16 Section 12.8.2, page 90
Fundamental Period, T =	0.100	sec., T = Tc <= Cu*Ta, ASCE 7-16 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-16 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	2.5	ASCE 7-16 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.25	ASCE 7-16 Table 12.2-1, pages 73-75
C_S =	0.050	$C_S = S_{DS}/(R/I)$, ASCE 7-16 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.160	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-16 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-16 Eqn. 12.8-5
Use: C_S =	0.050	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 1.96 kips, $V = C_S * W$, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx} * V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x * h_x^k / (\Sigma W_i * h_i^k)$, ASCE 7-16 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	h_x^k (ft.)	$W_x * h_x^k$ (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears
1	39.66	17.330	687.3	1.000	1.96	1.96
Σ =	39.66		687.3	1.000	1.96	

Comments:

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #
GNC - LAS VEGAS	20200081
Section	Sheet no./rev.
Calc. by JB / Contact Phone # / EMAIL / Date	

LATERAL ANALYSIS

DIAPHRAGM 3

SEISMIC WEIGHT

ROOF WEIGHT = $20_{DL} \times 460 ft^2 = 9.2^k$

WALLS = $55_{DL}(13.33'/2 + 2.83') \times 37.33' + 63_{DL}(11.5'/2 + 5.17') \times 10' + 63_{DL}(\frac{11.5'}{2}) \times 10'$
$+ 55_{DL}(9.8'/2) \times 36.67' = 39.88^k$

$\Sigma_{TOTAL} = 9.2^k + 39.88^k = 49^k$

$V_{BASE} =$ 2.4 ^{k}

DIAPHRAGM 4

SEISMIC WEIGHT

ROOF WEIGHT = $20_{DL} \times 1046.7 ft^2 = 20.9^k$

WALLS = $55_{DL}(13.2'/2 + 3.47') \times 96.2' + 55_{DL}(13'/2 + 3.67') \times 105.33'$
$+ 63_{DL}(13.1'/2 + 3.57') \times 10' = 118.5^k$

$\Sigma_{TOTAL} = 20.9^k + 118.5^k = 139.4^k$

$V_{BASE} =$ 6.9 ^{k}

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

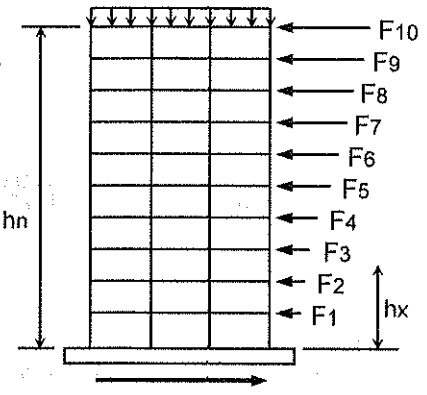
Per IBC 2018 and ASCE 7-16 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	QNC Car Wash	Subject:	Lateral - Diaphragm 3		
Job Number:	2022152	Originator:	JMB	Checker:	SJJ

Input Data:

Risk Category =	II	IBC 2018, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2, page 5
Soil Site Class =	C	ASCE 7-16 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., Ss =	0.200	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S1 =	0.056	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, TL =	4.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, hn =	11.500	ft.
Actual Calc. Period, Tc =	0.100	sec. from independent analysis
Seismic Resist. System =	A8	Intermediate reinf. masonry shear walls (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 2.43 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	11.500	49.00

Total Weight, W = ΣWx = 49.00 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

Fa =	1.300	ASCE 7-16 Table 11.4-1, page 66
Fv =	1.500	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	0.260	SMS = Fa*Ss, ASCE Eqn. 11.4-1, page 65
SM1 =	0.084	SM1 = Fv*S1, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.173	SDS = 2*SMS/3, ASCE 7-16 Eqn. 11.4-3, page 65
SD1 =	0.056	SD1 = 2*SM1/3, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1, page 67
Category(for S_{D1}) =	A	ASCE 7-16 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2, page 90
Approx. Period, T_a =	0.125	sec., $T_a = C_T * h_n^{(x)}$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.700	ASCE 7-16 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.212	sec., $T_{(max)} = C_u * T_a$, ASCE 7-16 Section 12.8.2, page 90
Fundamental Period, T =	0.100	sec., T = Tc <= Cu*Ta, ASCE 7-16 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-16 Table 12.2-1, pages 73-75
Overstrength Factor, Ωo =	2.5	ASCE 7-16 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.25	ASCE 7-16 Table 12.2-1, pages 73-75
C_S =	0.050	$C_S = S_{DS}/(R/I)$, ASCE 7-16 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.160	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-16 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-16 Eqn. 12.8-5
Use: C_S =	0.050	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 2.43 kips, $V = C_S * W$, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx} * V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x * h_x^k / (\Sigma W_i * h_i^k)$, ASCE 7-16 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	h_x^k (ft.)	$W_x * h^k$ (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears
1	49.00	11.500	563.5	1.000	2.43	2.43
Σ =	49.00		563.5	1.000	2.43	

Comments:

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

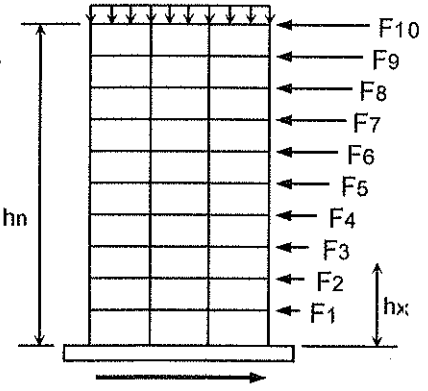
Per IBC 2018 and ASCE 7-16 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	QNC Car Wash	Subject:	Lateral - Diaphragm 4		
Job Number:	2022152	Originator:	JMB	Checker:	SJJ

Input Data:

Risk Category =	II	IBC 2018, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2, page 5
Soil Site Class =	C	ASCE 7-16 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., Ss =	0.200	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S1 =	0.056	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, TL =	4.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, hn =	13.000	ft.
Actual Calc. Period, Tc =	0.100	sec. from independent analysis
Seismic Resist. System =	A8	Intermediate reinf. masonry shear walls (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 6.9 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	13.000	139.40

Total Weight, W = ΣWx = 139.40 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

Fa =	1.300	ASCE 7-16 Table 11.4-1, page 66
Fv =	1.500	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	0.260	SMS = Fa*Ss, ASCE Eqn. 11.4-1, page 65
SM1 =	0.084	SM1 = Fv*S1, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.173	SDS = 2*SMS/3, ASCE 7-16 Eqn. 11.4-3, page 65
SD1 =	0.056	SD1 = 2*SM1/3, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1, page 67
Category(for S_{D1}) =	A	ASCE 7-16 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2, page 90
Approx. Period, T_a =	0.137	sec., $T_a = C_T * h_n^{(x)}$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.700	ASCE 7-16 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.233	sec., $T_{(max)} = C_u * T_a$, ASCE 7-16 Section 12.8.2, page 90
Fundamental Period, T =	0.100	sec., T = Tc <= $C_u * T_a$, ASCE 7-16 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-16 Table 12.2-1, pages 73-75
Overstrength Factor, Ωo =	2.5	ASCE 7-16 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.25	ASCE 7-16 Table 12.2-1, pages 73-75
C_S =	0.050	$C_S = S_{DS}/(R/I)$, ASCE 7-16 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.160	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-16 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-16 Eqn. 12.8-5
Use: C_S =	0.050	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	6.90	kips, $V = C_S * W$, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx} * V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x * h_x^k / (\Sigma W_i * h_i^k)$, ASCE 7-16 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	h_x^k (ft.)	$W_x * h_x^k$ (ft-kips)	C_{vx} (%)	Shear, Fx (kips)	Σ Story Shears
1	139.40	13.000	1812.2	1.000	6.90	6.90
Σ =	139.40		1812.2	1.000	6.90	

Comments:

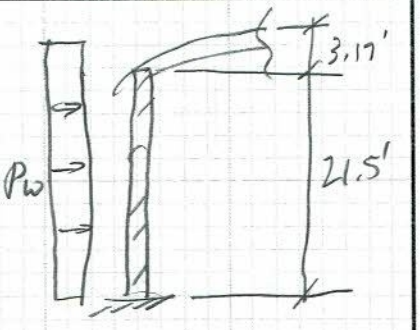
JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #
QNC - LAS VEGAS	2020081
Section	Sheet no./rev.
Calc. by JB / Contact Phone # / EMAIL / Date	

WIND

P_W = 24.7 PSF

W_{eq} = 24.7 PSF (3.17'/2 + 21.5'/2)
= 305 PLF (WIND ULTIMATE)

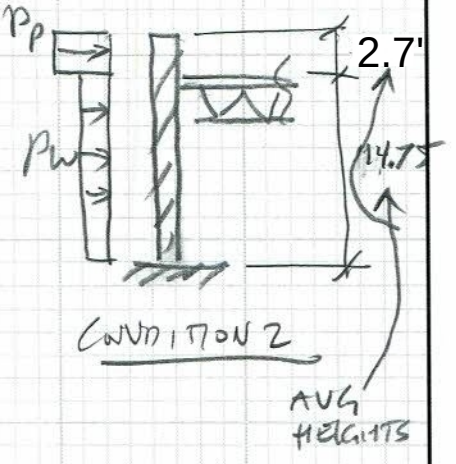


CONDITION 1 AT HIGH ROOF

CONDITION 2

P_P = 55 PSF

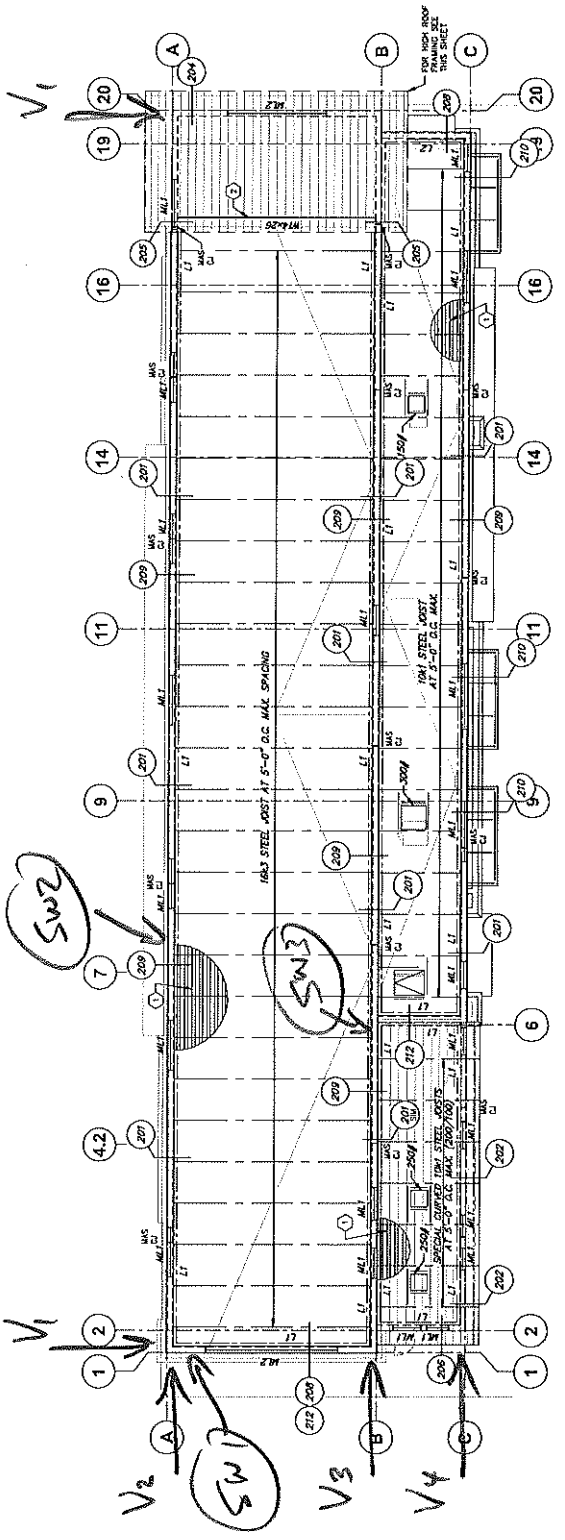
W_{eq} = 55 PSF(2.7') + 23 PSF(14.75'/2)
= 318 PLF (WIND ULTIMATE)

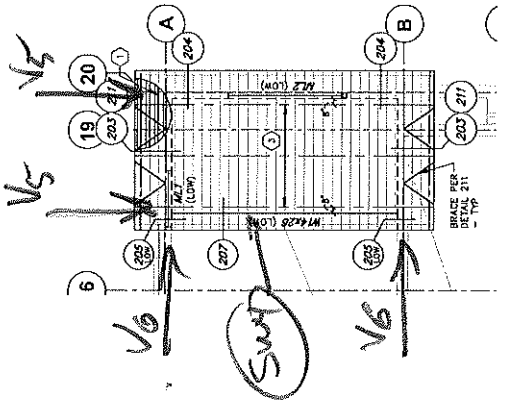


WIND GOVERNS E/W DIRECTION

SEISMIC GOVERNS N/S DIRECTION

LATERAL KEY PLAN





HIGH ROOF LATERAL KEY PLAN

$V_1 = 38 PLF (149.33'/2) = 23.7^k$ (WIND / STRENGTH) ⇒ GOVERNS

$V_{1,SEISMIC} = 31.5^k/2 = 15.7^k$ (SEISMIC/STRENGTH)

$V_2 = 38 PLF (24.67'/2) = 3.9^k$ (WIND/STRENGTH)

$V_{2,SEIS.} = 31.5^k/3 = 10.5^k$ (SEISMIC/STRENGTH) ⇒ GOVERNS

$V_3 = 38 PLF (\frac{35.33'}{2}) = 5.6^k$ (WIND/STRENGTH)

$V_{3,SEIS.} = 31.5^k/3 = 10.5^k$ (SEISMIC/STRENGTH) ⇒ GOVERNS

$V_4 = 38 PLF (10.67'/2) = 1.7^k$ (WIND/STRENGTH)

$V_{4,SEIS.} = 31.5^k/3 = 10.5^k$ (SEISMIC/STRENGTH)

$V_5 = 147 PLF (12.5'/2) = 11.92^k$ (WIND/STRENGTH) ⇒ GOVERNS

$V_{5,SEISMIC} = 1^k/2 = 0.5^k$ (SEISMIC/STRENGTH)

$V_6 = 147 PLF (24.67'/2) = 1.8^k$ (WIND / STRENGTH) ⇒ GOVERNS

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

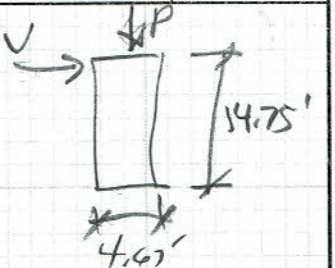
Project			Job #. 2020081
Section			Sheet no./rev.
Calc. by	Contact Phone #	EMAIL	Date

SW LINE 1 (SHEAR WALL DESIGN-WORST CASE)

$V = 23.7^{k}/2 = 11.85^{k}/\text{WALL}$

$P = [(20_{DL} + 31.5_{SL})(2.25'/2) + 78_{DL}(7.33')](16'/2)$

$= 4.75^{k}_{DL} + 0.284^{k}_{SL}$



USE 8" MASONRY W/ 1#6 VERT AT EACH FACE

SW LINE 1 FOOTING

$V = 11.85^{k} \times 0.6 = 7.1^{k}$ (ASD)

$P = 4.75^{k}_{DL} \times 0.6 = 2.85^{k}$

$M = 7.1^{k} \times 14.75' = 105$ k-ft (ASD)

USE 24" THICK x 5'-0" WIDE CONT. CONCRETE FOOTING W/ 6#5 CONT. TOP AND BOTTOM, #5 AT 12" O.C. TRANSVERSE T & B

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 12:35PM

Masonry Shear Wall

Lic. # : KW-06012033 JTA Engineering, LLC

DESCRIPTION: SW1 (Shear Wall)

2021 OK

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	14.750 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	4.670 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	3.0	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	4.670 ft
Effective Length 'd'	40.040 in
Nominal Block Thickness	8.0 in
Solid Grout?	Solid Grouted

Reinforcing in Field of Wall

Vertical Bar Size #	6
Vertical Bar Spacing	8 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	6
# Chord Cells @ Each End	4.0

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

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Engineer:
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Project Descr:

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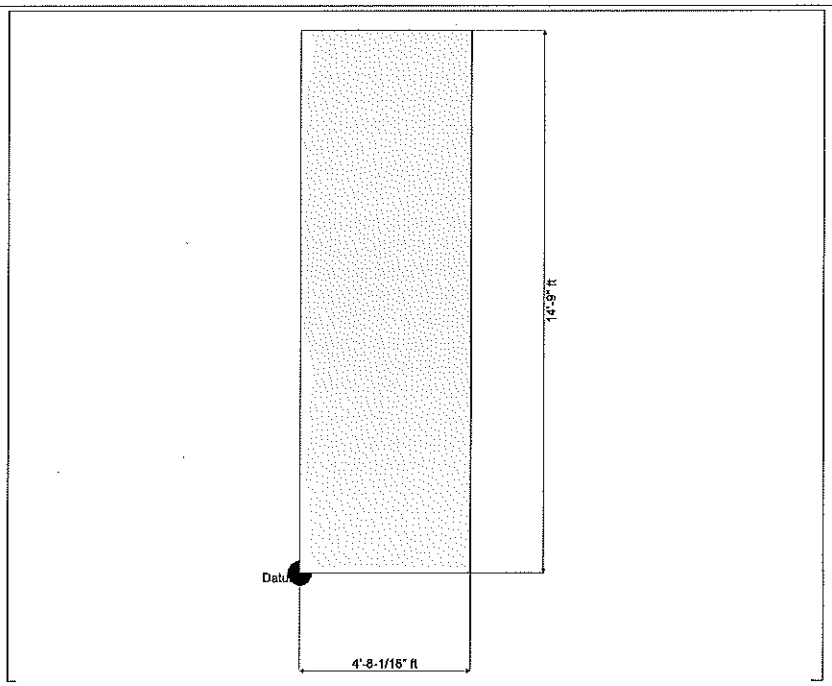
Masonry Shear Wall

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: SW1 (Shear Wall)

Wall Sketch



Applied Concentrated Vertical Loads

Load Location (ft)		Load Magnitude (kips)				
X Location	Y Location	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
4.0	11.0	4.750	0.0	0.0	0.2840	0.0

Applied Distributed Vertical Loads

Load Location (ft)			Load Magnitude (kips)				
Start Location	End Location	Height of Application	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
0.0	4.670	14.750	0.10	0.0	0.0	0.10	0.0

Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
14.750	0.0	0.0	0.0	11.850	0.0	0.0

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Masonry Shear Wall

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: SW1 (Shear Wall)

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	11.850 k
for Load Combination	+1.20D+W
Controlling Mu/(Vud)	1.00
Vn Masonry	40.858 k
Vn Steel	0.0 k
Vn Masonry + Vn Steel	40.858 k
Vn Max	65.981 k
Phi Vn	32.687 k
Ratio: Vu/PhiVn (controlling)	**0.3625** ✓
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	4.42
Pu	14.976 k
for Load Combination	+1.40D+0.20S+E
Phi Pn	+1.40D+0.20S+E k
Ratio: Pu/PhiPn (controlling)	**0.03077** ✓

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	11.54 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	40.040
As-flex < As-max ?	: 1.760 <= 2.705
Mu	184.514 k
for Load Combination	+1.20D+0.50S+W
Phi Mn	271.415 k
Ratio: Mu/PhiMn (controlling)	**0.6798** ✓

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1		11.072	14.826	0.747			
+1.20D							
Wall Level : 1		9.491	12.708	0.747			
+1.20D+0.50S							
Wall Level : 1		9.727	13.083	0.743			
+1.20D+0.50W							
Wall Level : 1	5.925	96.884	12.708	7.624	0.202	23.969	23.969

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Masonry Shear Wall

Lic. # : KW-06012033 JTA Engineering, LLC

DESCRIPTION: SW1 (Shear Wall)

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.20D+1.60S							
Wall Level : 1		10.247	13.909	0.737			
+1.20D+1.60S+0.50W							
Wall Level : 1	5.925	97.641	13.909	7.020	0.226	24.583	24.583
+1.20D+W							
Wall Level : 1	11.850	184.278	12.708	14.501	0.101	50.161	50.161
+1.20D+0.50S+W							
Wall Level : 1	11.850	184.514	13.083	14.103	0.105	50.353	50.353
+1.40D+0.20S+E							
Wall Level : 1	2.149	27.017	14.976	0.746			
+0.90D+W							
Wall Level : 1	11.850	181.905	9.531	19.086	0.094	49.776	49.776
+0.70D+E							
Wall Level : 1	2.149	21.386	7.413	0.747			

"FOOTINGS.xls" Program
Version 3.7

RECTANGULAR SPREAD FOOTING ANALYSIS

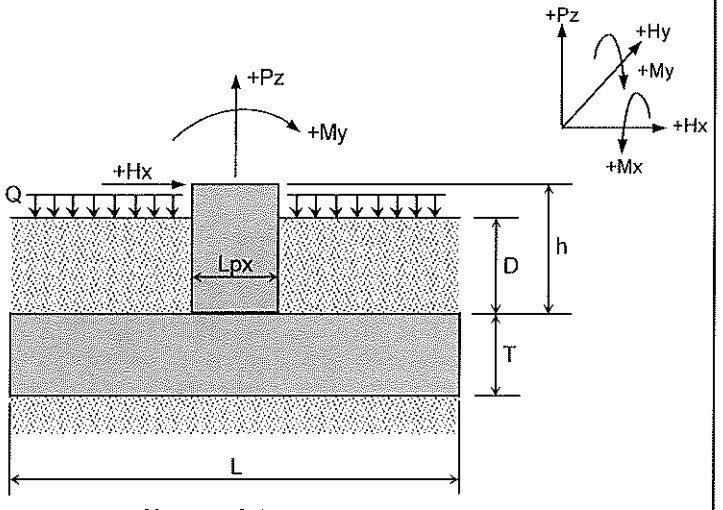
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	QNC Car Wash	Subject:	SW1 FOOTING		
Job Number:	2022152	Originator:	JMB	Checker:	

Input Data:

Footing Data:

Footing Length, L =	27.000	ft.
Footing Width, B =	5.000	ft.
Footing Thickness, T =	2.000	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	0.500	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

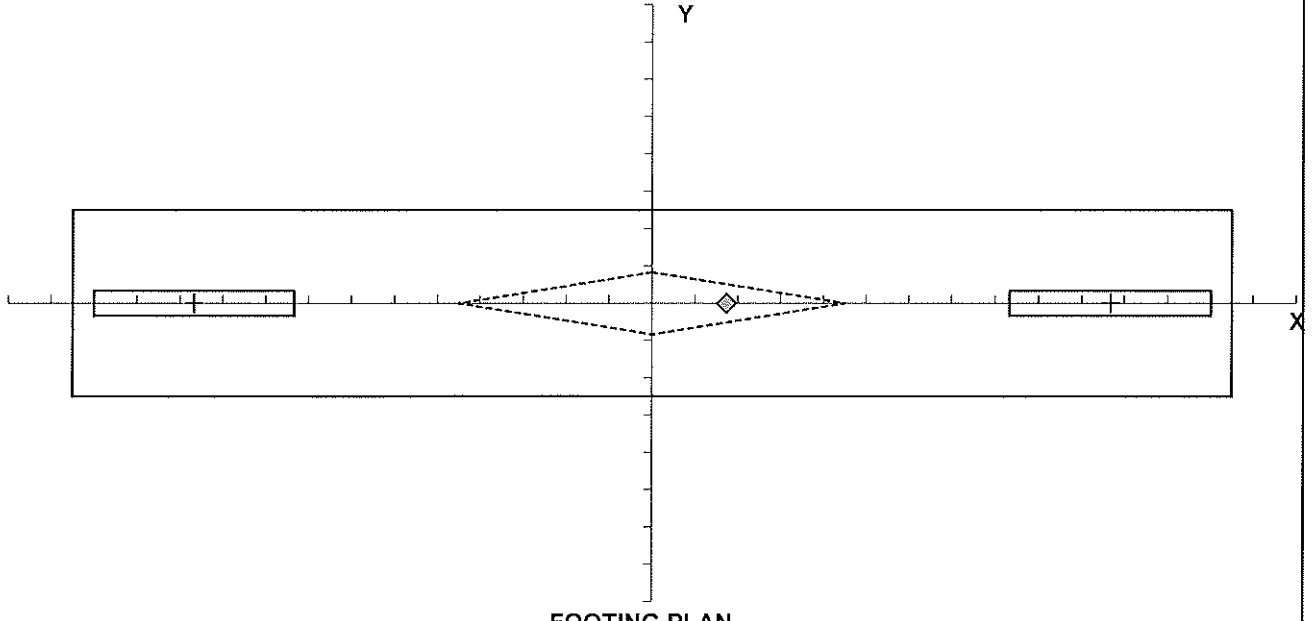


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1	
Xp (ft.) =	-10.670	10.670
Yp (ft.) =	0.000	0.000
Lpx (ft.) =	4.670	4.670
Lpy (ft.) =	0.670	0.670
h (ft.) =	0.000	0.000
Pz (k) =	-2.85	-2.85
Hx (k) =	7.10	7.10
Hy (k) =	0.00	0.00
Mx (ft-k) =	0.00	0.00
My (ft-k) =	105.00	105.00



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-51.45	kips
ex =	1.73	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	724.98	ft-kips
ΣMoy =	119.20	ft-kips
FS(ot)y =	6.082	>= 1.5

Sliding Check:

Pass(x) =	5.40	kips
Frict(x) =	20.58	kips
FS(slid)x =	1.830	>= 1.5
Pass(y) =	29.16	kips
Frict(y) =	20.58	kips
FS(slid)y =	N.A.	

Uplift Check:

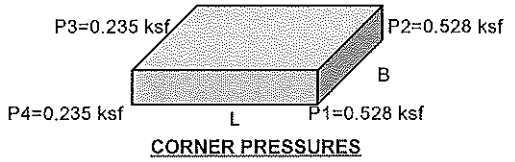
ΣPz(down) =	-51.45	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	27.000	ft.
Brg. Ly =	5.000	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P1 =	0.528	ksf
P2 =	0.528	ksf
P3 =	0.235	ksf
P4 =	0.235	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

Pmax(net) = Pmax(gross)-(D+T)*γs

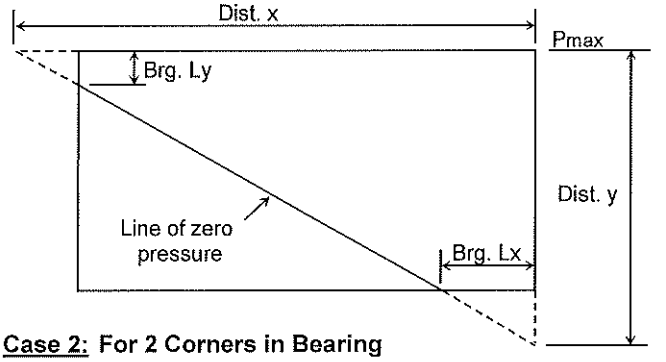
Pmax(net) =	0.228	ksf

2 000 psf x 0.33 = 26 000 psf

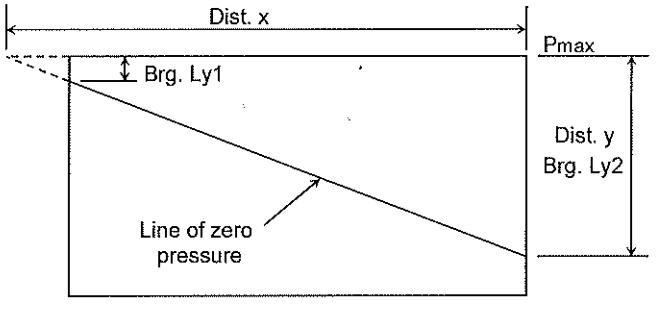
Comments:

Nomenclature for Biaxial Eccentricity:

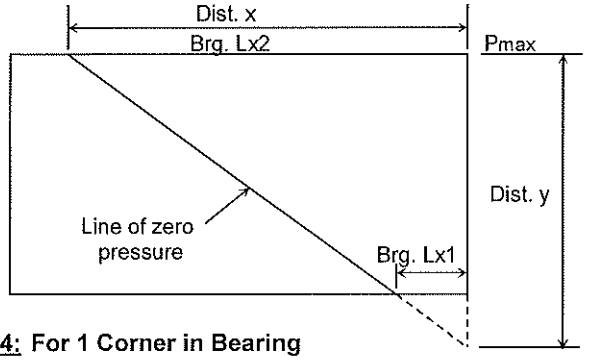
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



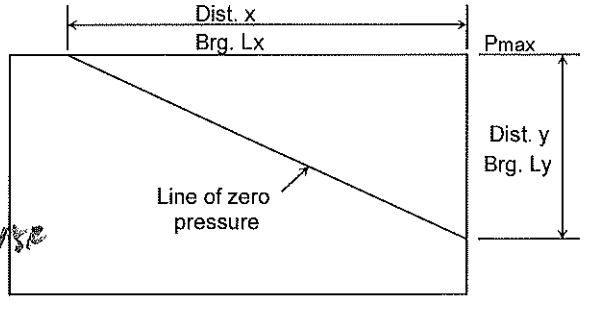
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



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)



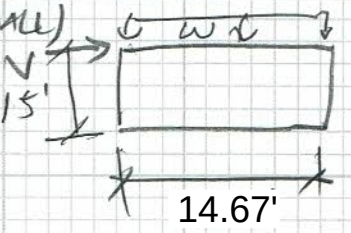
JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project		Job #.	
Section		Sheet no./rev.	
Calc. by	Contact Phone #	EMAIL	Date

SW LINE 2 (SHEAR WALL DESIGN-WORST CASE WALL)

$V = 10.5^k \left(\frac{14.67'}{94.83'} \right)$

$= 1.6^k$ (SEISMIC) STRENGTH



$W = (20_{DL} + 31.5_{SL})(24'/2) = 240 PLF_{DL} + 378 PLF_{SL}$

USE 8" MASONRY W/ 1 #5 VERT. AT 32" O.C. AND 1 #5 VERT. AT EACH END OF WALL

SW LINE 2 FOOTING

$V = 10.5^k$ (SEISMIC) $\times 0.7 = 7.4^k$

$P = 55_{DL} \times 15' \times 150' \times 0.9 = 111^k$

$M = 7.4^k \times 15' = 111$ k-ft

USE 14" THICK x 2'-8" WIDE CONT. CONCRETE FOOTING W/ 4 #5 CONT. AND #5 AT 18" O.C. TRANSVERSE

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 1:14PM
File: 2022152 - QNC Car Wash.ec6

JTA Engineering, LLC

Masonry Shear Wall

Lic. # : KW-06012033

DESCRIPTION: SW2 (Shear Wall)

2021 IBC

Code References

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	15.0 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	14.670 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	3.0	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	14.670 ft
Effective Length 'd'	172.040 in
Nominal Block Thickness	8.0 in
Solid Grout?	Partial Groute

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	32 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	1.0

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CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

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Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 1:14PM

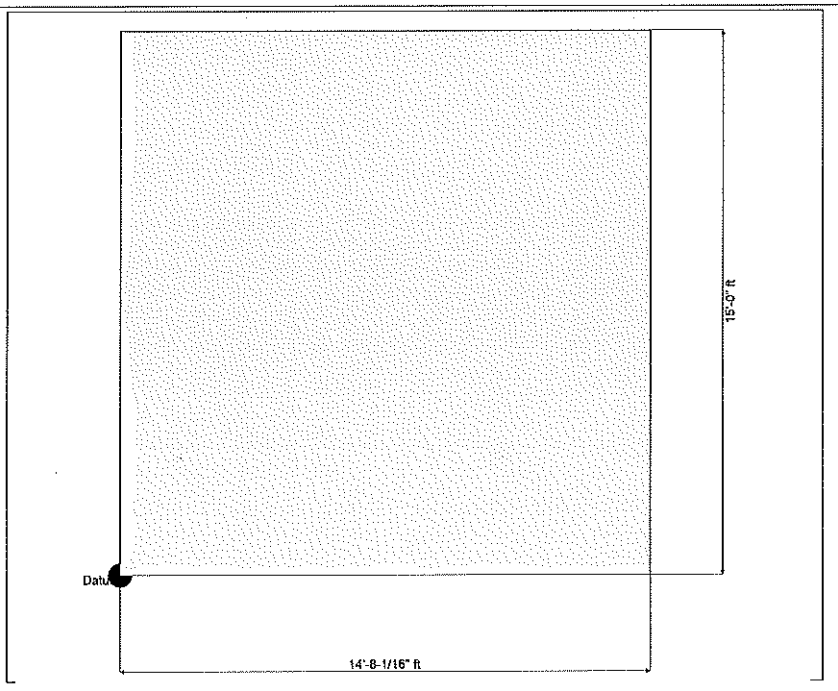
Masonry Shear Wall

Lic. # : KW-06012033

JTA Engineering, LLC

DESCRIPTION: SW2 (Shear Wall)

Wall Sketch



Applied Distributed Vertical Loads

Load Location (ft)			Load Magnitude (kips)				
Start Location	End Location	Height of Application	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
0.0	8.0	15.0	0.240	0.0	0.0	0.3780	0.0

Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
15.0	0.0	0.0	0.0	0.0	1.60	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	8.089 k
for Load Combination	+1.235D+0.20S+3.0E
Controlling Mu/(Vud)	0.77
Vn Masonry	98.254 k
Vn Steel	0.0 k
Vn Masonry + Vn Steel	98.254 k
Vn Max	121.526 k
Phi Vn	78.603 k
Ratio: Vu/PhiVn (controlling)	**0.1029**
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

DESCRIPTION: SW2 (Shear Wall)

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	1.05
Pu	20.297 k
for Load Combination	+1.20D+1.60S
Phi Pn	+1.20D+1.60S k
Ratio: Pu/PhiPn (controlling)	**0.02018** ✓

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	2.03 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	172.040
As-flex < As-max ?	0.310 <= 11.625
Mu	91.125 k
for Load Combination	+0.8652D+3.0E
Phi Mn	238.578 k
Ratio: Mu/PhiMn (controlling)	**0.3820** ✓

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1		8.964	18.035	0.497			
+1.20D							
Wall Level : 1		7.684	15.459	0.497			
+1.20D+0.50S							
Wall Level : 1		12.726	16.971	0.750			
+1.20D+1.60S							
Wall Level : 1		23.820	20.297	1.174			
+0.90D							
Wall Level : 1		5.763	11.594	0.497			
+1.235D+0.20S+3.0E							
Wall Level : 1	8.089	86.742	16.512	3.759	1.558	1.628	1.628
+0.8652D+3.0E							
Wall Level : 1	8.089	91.125	11.146	5.963	1.363	0.650	0.650

"FOOTINGS.xls" Program
Version 3.7

RECTANGULAR SPREAD FOOTING ANALYSIS

For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	QNC Car Wash	Subject:	SW2 LINE FOOTING		
Job Number:	2022152	Originator:	JMB	Checker:	

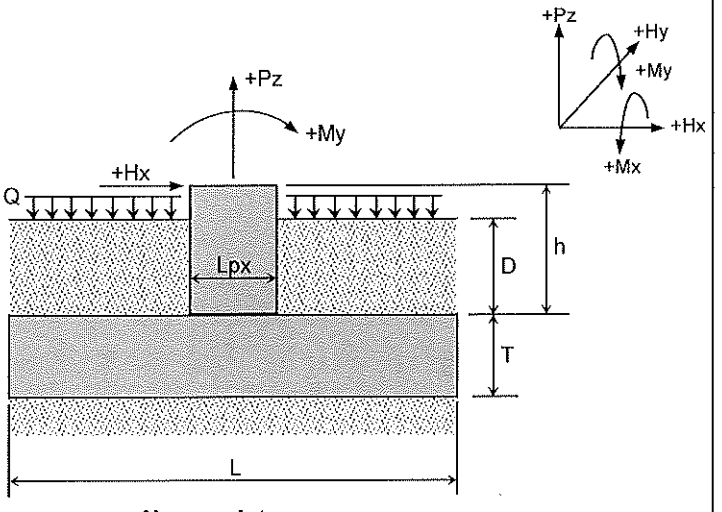
Input Data:

Footing Data:

Footing Length, L =	154.330	ft.
Footing Width, B =	2.670	ft.
Footing Thickness, T =	1.170	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	0.500	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

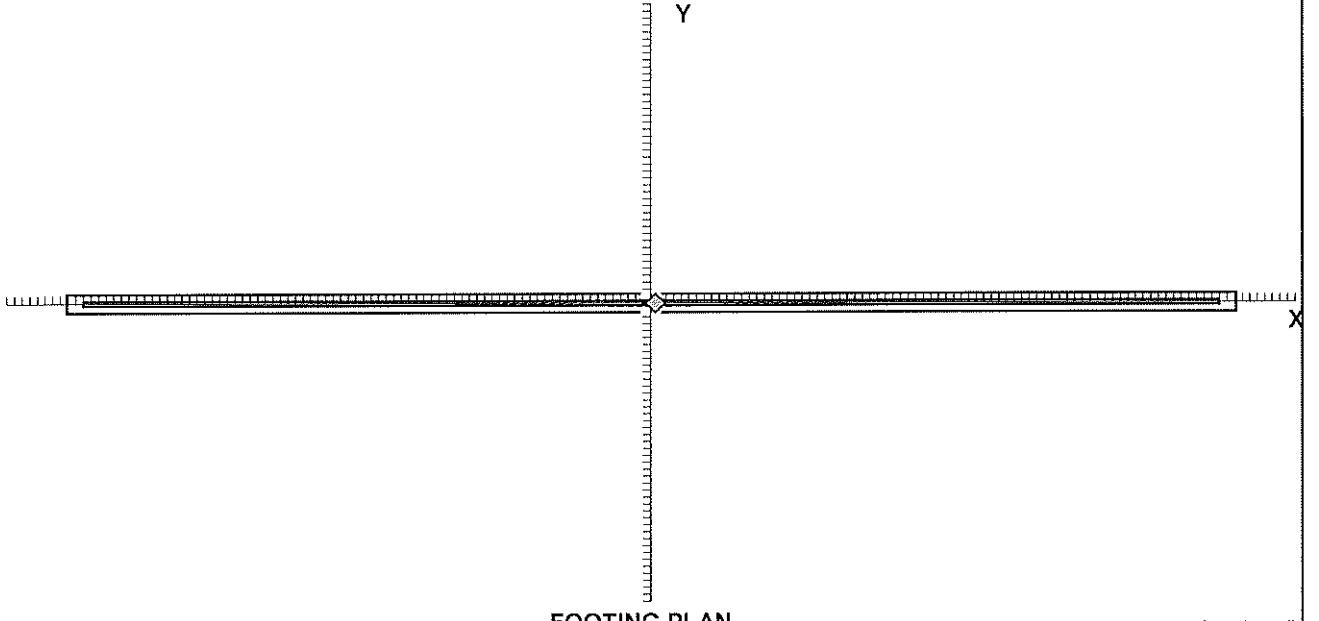
Pier/Loading Data:

Number of Piers = 1



Nomenclature

	Pier #1
Xp (ft.) =	0.000
Yp (ft.) =	0.000
Lpx (ft.) =	150.000
Lpy (ft.) =	0.670
h (ft.) =	0.000
Pz (k) =	-111.00
Hx (k) =	7.40
Hy (k) =	0.00
Mx (ft-k) =	0.00
My (ft-k) =	111.00



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-208.04	kips
ex =	0.58	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	16053.44	ft-kips
ΣMoy =	119.66	ft-kips
FS(ot)y =	134.161	>= 1.5

Sliding Check:

Pass(x) =	1.22	kips
Frict(x) =	83.22	kips
FS(slid)x =	11.410	>= 1.5
Pass(y) =	70.53	kips
Frict(y) =	83.22	kips
FS(slid)y =	N.A.	

Uplift Check:

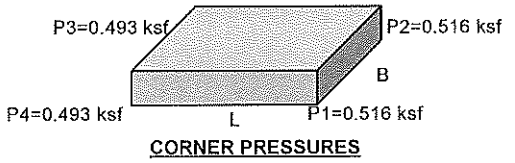
ΣPz(down) =	-208.04	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	154.330	ft.
Brg. Ly =	2.670	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P1 =	0.516	ksf
P2 =	0.516	ksf
P3 =	0.493	ksf
P4 =	0.493	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

Pmax(net) = Pmax(gross)-(D+T)*γs

Pmax(net) =	0.316	ksf

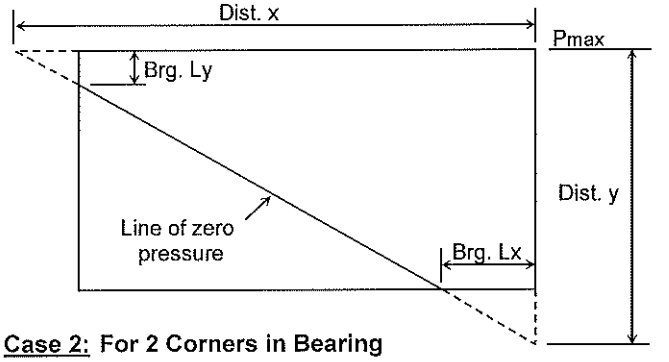
< 2000PSF x1.33 = 2660PSF

=> OK

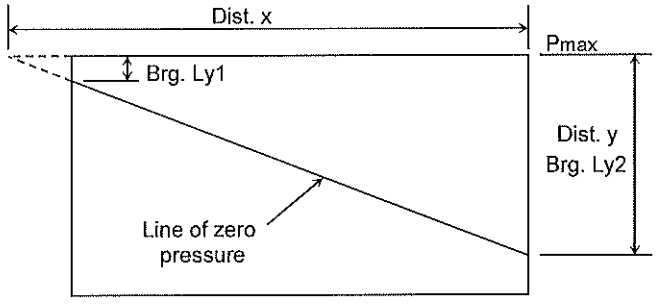
Comments:

Nomenclature for Biaxial Eccentricity:

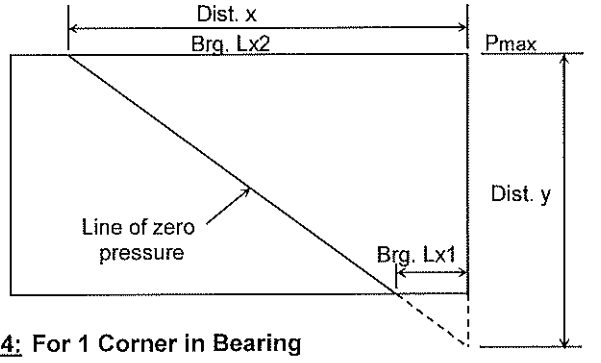
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



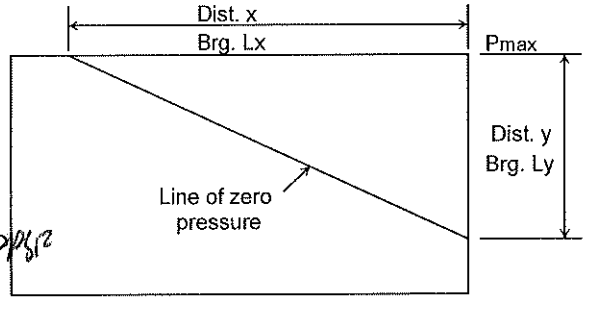
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



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



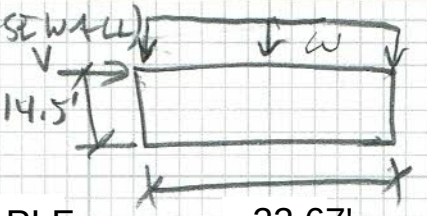
Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)



JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #.
QNC - LAS VEGAS	2020081
Section	Sheet no./rev.
Calc. by JB / Contact Phone # / EMAIL / Date	

SW LINE 3 (SHEAR WALL DESIGN - WORST CASE WALL)



$V = 10.5^{k} \left(\frac{22.67'}{94.83'} \right)$

$= 2.5^{k}$ (SEISMIC)

$W = (20_{DL} + 31.5_{SL})(24'/2 + \frac{10.67'}{2}) = 347\,PLF_{DL} + 546\,PLF_{SL}$

USE 8" MASONRY W/ 1#5 VERT. AT 32" O.C. AND 1#5 VERT. AT EACH END OF WALL

SW LINE 3 FOOTING

$V = 10.5^{k}$ (SEISMIC) $\times 0.7 = 7.4^{k}$ (ASD)

$P = 55_{DL}\,(14.5') \times 150' = 119^{k} \times 0.7 = 83^{k}_{DL}$

$M = 7.4^{k} \times 14.5' = 107$ k-ft

USE 14" THICK x 2'-8" WIDE CONT. CONCRETE FOOTING W/ 4#5 CONT. AND #5 AT 18" O.C. TRANSVERSE

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 1:34PM

Masonry Shear Wall

Lic. # : KW-06012033

JTA Engineering, LLC

DESCRIPTION: SW3 at Gridline B

2021 OK

Code References

Calculations per TMS 402-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	14.50 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	22.670 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	3.0	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	22.670 ft
Effective Length 'd'	268.040 in
Nominal Block Thickness	8 in
Solid Grout?	Partial Groute

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	32 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	1.0

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

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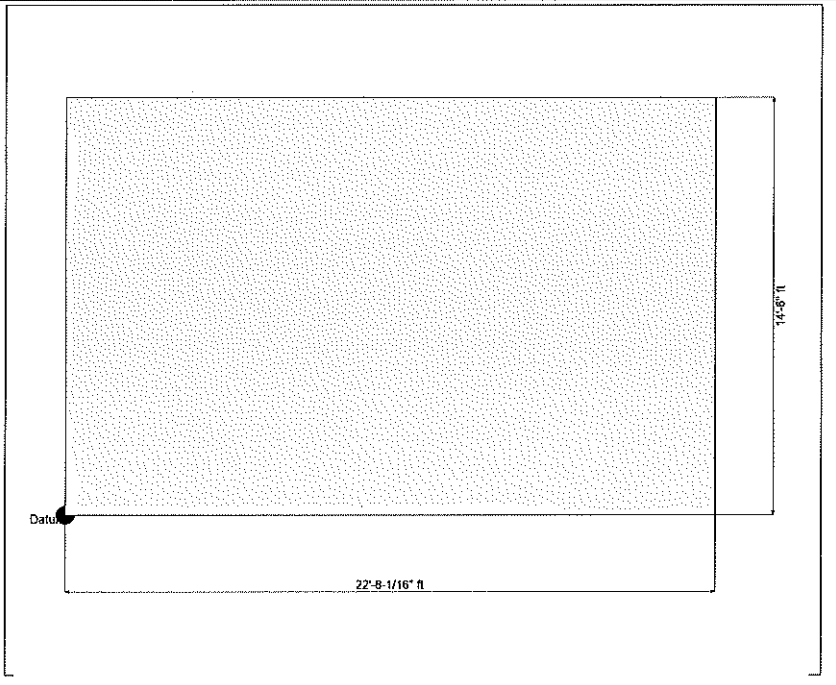
Masonry Shear Wall

Lic. # : KW-06012033

JTA Engineering, LLC

DESCRIPTION: SW3 at Gridline B

Wall Sketch



Applied Distributed Vertical Loads

Load Location (ft)			Load Magnitude (kips)				
Start Location	End Location	Height of Application	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
	22.670	14.50	0.3470			0.5460	

Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
14.750					2.50	

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	12.350 k
for Load Combination	+1.235D+0.20S+3.0E
Controlling Mu/(Vud)	0.52
Vn Masonry	176.789 k
Vn Steel	0.0 k
Vn Masonry + Vn Steel	176.789 k
Vn Max	210.990 k
Phi Vn	141.432 k
Ratio: Vu/PhiVn (controlling)	**0.08732**
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

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CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 1:34PM
File: 2022152 - QNC Car Wash.ec6

Masonry Shear Wall

Lic. # : KW-06012033
JTA Engineering, LLC

DESCRIPTION: SW3 at Gridline B

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	0.65
Pu	48.642 k
for Load Combination	+1.20D+1.60S
Phi Pn	+1.20D+1.60S k
Ratio: Pu/PhiPn (controlling)	**0.03183** ✓

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	2.03 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	268.040
As-flex < As-max ?	0.310 <= 18.111
Mu	145.784 k
for Load Combination	+1.235D+0.20S+3.0E
Phi Mn	372.498 k
Ratio: Mu/PhiMn (controlling)	**0.3914** ✓

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1			33.644				
+1.20D							
Wall Level : 1			28.838				
+1.20D+0.50S							
Wall Level : 1			35.027				
+1.20D+1.60S							
Wall Level : 1			48.642				
+0.90D							
Wall Level : 1			21.628				
+1.235D+0.20S+3.0E							
Wall Level : 1	12.349	145.784	32.150	3.441	2.905	2.935	2.935
+0.8652D+3.0E							
Wall Level : 1	12.349	145.784	20.792	5.321	2.354	0.203	0.203

RECTANGULAR SPREAD FOOTING ANALYSIS

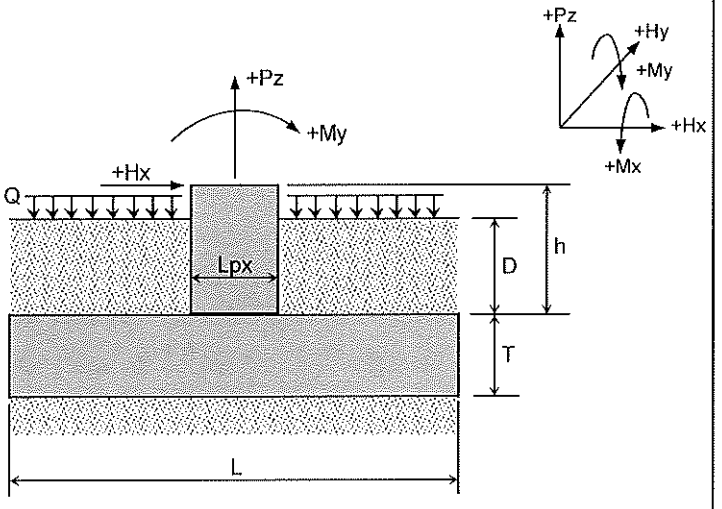
For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)
Subjected to Uniaxial or Biaxial Eccentricity

Job Name:	QNC Car Wash	Subject:	SW3 LINE FOOTING ALONG GRID B		
Job Number:	2022152	Originator:	JMB	Checker:	

Input Data:

Footing Data:

Footing Length, L =	154.330	ft.
Footing Width, B =	2.330	ft.
Footing Thickness, T =	1.170	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	0.500	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf

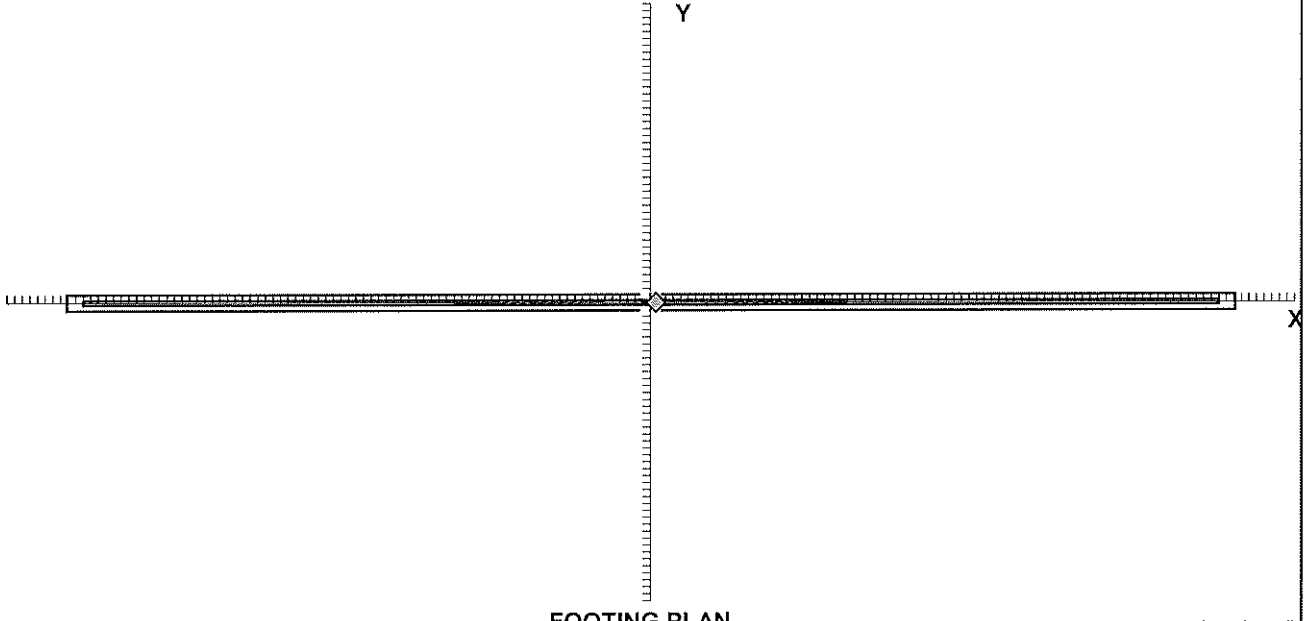


Nomenclature

Pier/Loading Data:

Number of Piers = 1

	Pier #1
Xp (ft.) =	0.000
Yp (ft.) =	0.000
Lpx (ft.) =	150.000
Lpy (ft.) =	0.670
h (ft.) =	0.000
Pz (k) =	-83.00
Hx (k) =	7.40 ✓
Hy (k) =	0.00
Mx (ft-k) =	0.00
My (ft-k) =	107.00 ✓



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-167.68	kips
ex =	0.69	ft. (<= L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	12939.27	ft-kips
ΣMoy =	115.66	ft-kips
FS(ot)y =	111.875	>= 1.5 ✓

Sliding Check:

Pass(x) =	1.06	kips
Frict(x) =	67.07	kips
FS(slid)x =	9.208	>= 1.5 ✓
Pass(y) =	70.53	kips
Frict(y) =	67.07	kips
FS(slid)y =	N.A.	

Uplift Check:

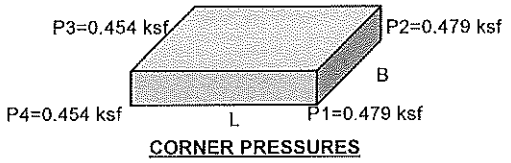
ΣPz(down) =	-167.68	kips
ΣPz(uplift) =	0.00	kips
FS(uplift) =	N.A.	

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	154.330	ft.
Brg. Ly =	2.330	ft.
%Brg. Area =	100.00	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P1 =	0.479	ksf
P2 =	0.479	ksf
P3 =	0.454	ksf
P4 =	0.454	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

Pmax(net) = Pmax(gross)-(D+T)*γs

Pmax(net) =	0.278	ksf

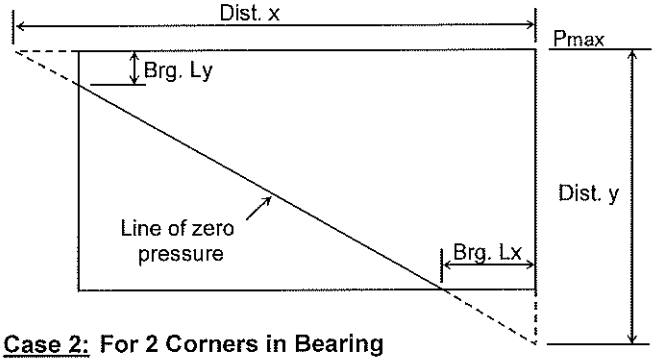
< 2000 PSF × 1.33 = 2660 PSF

OK

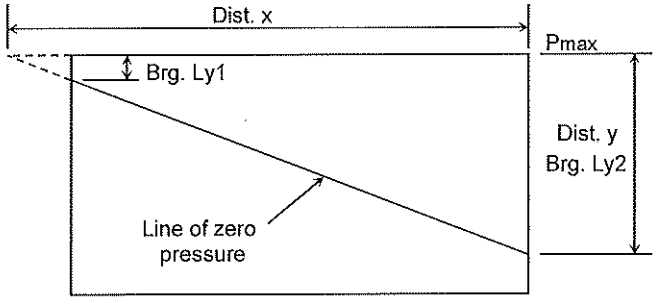
Comments:

Nomenclature for Biaxial Eccentricity:

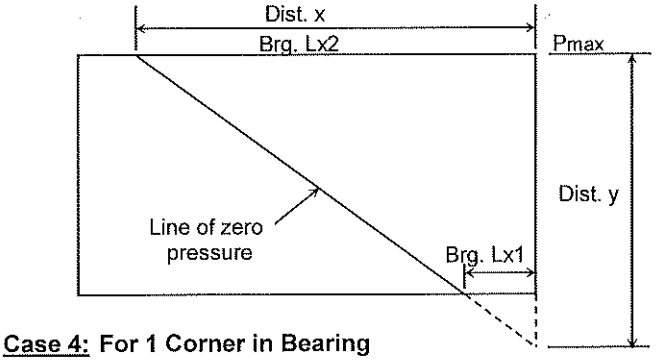
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



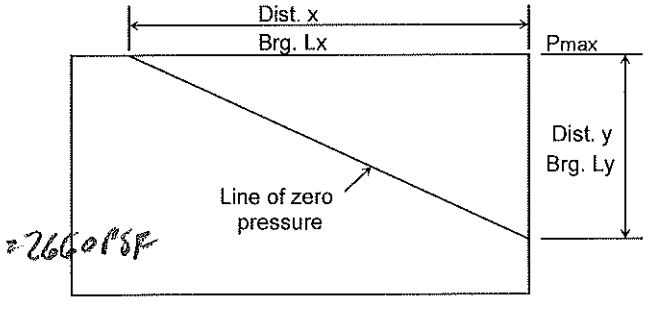
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



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)



JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project	Job #.
QNC.	
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LATERAL ANALYSIS

CHECK DIAPHRAGM CHORD (WORST CASE - OVERALL LENGTH)

$M = 318\ PLF\ (150')^2/8 = 894\ k\text{-}ft \times 0.6 = 536\ k\text{-}ft$

$T/C = M/d = \frac{536\ k\text{-}ft}{23.75'} = 22.6^k$

$\frac{P_n}{\Omega} = 0.6(1.93in^2)(36ksi) = 41.7^k > 22.6^k \Rightarrow OK$

(1.93 in² = AREA OF L4x4x1/4)

USE L4x4x1/4 AT CHORD/LEDGER

CHECK DECK

$w = 318\ PLF \times 0.6 = 191\ PLF\ (ASD) < \frac{1354\ PLF_{allow.}}{2.5} = 542\ PLF \Rightarrow OK$

(1½" STEEL DECK, 20 GA., 7 WELDS)

USE 1½" DEEP 20 GA STEEL DECK W/ 7-¼" ⌀ WELDS

AND 1½" TOP SEAM WELDS AT 12" O.C.

Type HSB®-36

- 36/7 Weld Pattern at Supports
- Sidelaps connected with Button Punch or 1½" Top Seam Weld

Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^6)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	BP @ 24"	q	398	327	267	235	202	186	168		
		F	9.9+27R	11.9+20R	14+15R	15.7+12R	17.6+9R	19.1+7R	20.9+5R		
	BP @ 12"	q	434	355	303	266	238	218	204		
		F	9.5+27R	11.4+21R	13.1+16R	14.6+13R	16.1+10R	17.4+8R	18.7+6R		
	TSW @ 24"	q	770	777	661	682	601	627	566		
		F	5.1+30R	5.3+24R	6.1+20R	6.1+17R	6.6+15R	6.5+13R	7+12R		
	TSW @ 18"	q	937	911	781	785	787	710	720		
		F	4.4+30R	4.8+24R	5.4+20R	5.5+17R	5.6+15R	6+13R	6.1+12R		
	TSW @ 12"	q	1084	1033	998	972	952	936	924		
		F	4+30R	4.4+24R	4.7+20R	4.9+17R	5.1+15R	5.2+13R	5.3+12R		
	TSW @ 6"	q	1504	1476	1456	1442	1430	1236	1001		
		F	3.2+30R	3.6+24R	3.8+20R	4+17R	4.2+15R	4.3+14R	4.4+12R		
20	BP @ 24"	q	564	466	383	338	292	267	240	228	209
		F	9.3+16R	11+12R	12.8+8R	14.3+6R	16+4R	17.3+2R	19+1R	20.1+0R	21.7-1R
	BP @ 12"	q	616	507	434	382	343	313	292	274	260
		F	8.8+16R	10.5+12R	12+9R	13.3+7R	14.6+5R	15.8+3R	16.9+2R	18+1R	19+0R
	TSW @ 24"	q	1024	1026	873	896	793	821	742	772	711
		F	4.8+19R	4.9+15R	5.6+12R	5.5+11R	6+9R	5.8+8R	6.2+7R	6.1+7R	6.4+6R
	TSW @ 18"	q	1236	1197	1026	1027	1028	926	937	946	875
		F	4.2+19R	4.4+15R	5+13R	5+11R	5.1+9R	5.4+8R	5.4+7R	5.4+7R	5.7+6R
→	TSW @ 12"	q	1425	1354	1304	1267	1239	1217	1200	1085	912
		F	3.8+19R	4.1+15R	4.3+13R	4.5+11R	4.6+10R	4.7+8R	4.7+8R	4.8+7R	4.8+6R
	TSW @ 6"	q	1970	1930	1901	1880	1864	1621	1313	1085	912
		F	3.1+19R	3.4+15R	3.5+13R	3.7+11R	3.7+10R	3.8+9R	3.9+8R	3.9+7R	4+6R
18	BP @ 24"	q	979	812	670	595	516	475	423	400	366
		F	8.1+7R	9.5+4R	11+2R	12.2+1R	13.7+0R	14.8-1R	16.2-2R	17.2-3R	18.6-4R
	BP @ 12"	q	1070	885	761	673	607	556	515	483	458
		F	7.7+7R	9+5R	10.3+3R	11.4+2R	12.5+1R	13.5+0R	14.4-1R	15.3-2R	16.2-2R
	TSW @ 24"	q	1617	1598	1359	1383	1224	1258	1141	1177	1085
		F	4.3+9R	4.2+7R	4.7+6R	4.6+5R	5+4R	4.9+4R	5.2+3R	5+3R	5.3+3R
	TSW @ 18"	q	1928	1851	1586	1577	1570	1414	1425	1434	1326
		F	3.7+9R	3.8+7R	4.2+6R	4.2+5R	4.2+4R	4.5+4R	4.5+4R	4.4+3R	4.7+3R
	TSW @ 12"	q	2208	2084	1998	1935	1886	1848	1817	1659	1394
		F	3.4+9R	3.5+7R	3.6+6R	3.7+5R	3.8+5R	3.8+4R	3.9+4R	3.9+3R	3.9+3R
	TSW @ 6"	q	3036	2962	2910	2872	2842	2478	2007	1659	1394
		F	2.7+9R	2.9+8R	3+6R	3+5R	3.1+5R	3.1+4R	3.1+4R	3.2+3R	3.2+3R
16	BP @ 24"	q	1255	1052	869	780	677	629	561	532	485
		F	7.2+3R	8.4+1R	9.7+0R	10.8-1R	12.1-2R	13.1-2R	14.3-3R	15.2-4R	16.4-4R
	BP @ 12"	q	1395	1167	1013	902	820	756	704	662	628
		F	6.8+3R	8+2R	9.1+1R	10.1+0R	11-1R	11.9-2R	12.7-2R	13.5-3R	14.3-3R
	TSW @ 24"	q	2083	2073	1766	1805	1599	1649	1497	1548	1428
		F	3.7+5R	3.7+4R	4.1+3R	4+3R	4.3+2R	4.2+2R	4.5+2R	4.3+2R	4.6+1R
	TSW @ 18"	q	2496	2408	2067	2062	2058	1856	1874	1889	1748
		F	3.2+5R	3.3+4R	3.7+3R	3.6+3R	3.6+2R	3.9+2R	3.8+2R	3.8+2R	4+2R
	TSW @ 12"	q	2862	2713	2609	2532	2473	2427	2389	2310	1941
		F	2.9+5R	3.1+4R	3.1+3R	3.2+3R	3.2+3R	3.3+2R	3.3+2R	3.3+2R	3.4+2R
	TSW @ 6"	q	3918	3833	3773	3729	3695	3451	2795	2310	1941
		F	2.4+5R	2.5+4R	2.5+4R	2.6+3R	2.6+3R	2.6+2R	2.6+2R	2.7+2R	2.7+2R

See footnotes on page 28.

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

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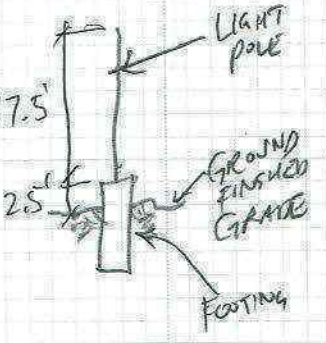
LIGHT POLE FOOTING CALC (LP1)

$P_W = 36 PSF \times 1/2' = 18 PLF$ ← CONSERVATIVE LOAD

$V = 18 PLF \times 26' = 360\#$

$0.6V = 216\#$



SEE CALC

USE 24" DIA. x 6'-0" CONCRETE PIER W/ 8 #5 VERTICAL AND #3 TIES AT 12" O.C.

ANCHORS BY OTHERS AND LIGHT POLE BY OTHERS.

"POLEFDN.xls" Program
Version 2.4

POLE FOUNDATION ANALYSIS

For Free-Top (Unconstrained) Rigid Round Piers Using Czerniak / PCA Method
Subjected Vertical Load, Horizontal Load, and/or Moment

Job Name:		Subject:	LP1 Calc		
Job Number:		Originator:	JMB	Checker:	

Input Data:

Pier Data:

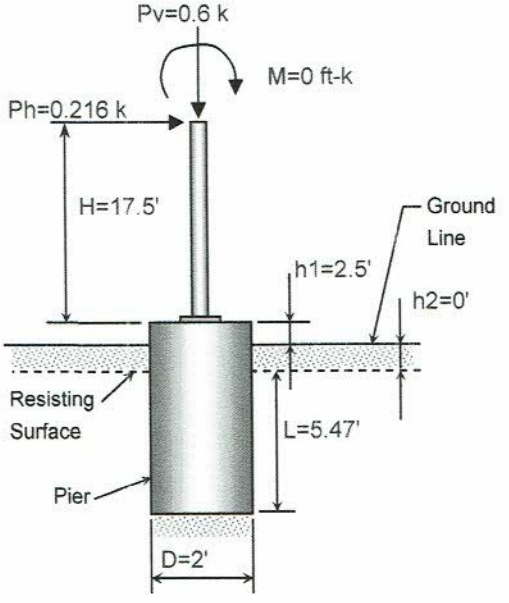
Pier Foundation Diameter, D =	2.000	ft.
Pier Height Above Soil, h1 =	2.500	ft.
Concrete Strength, f'c =	4.500	ksi

Soil Data:

Unit Weight of Soil, γ =	0.100	kcf
Angle of Internal Friction, ϕ =	30.00	deg.
Depth to Resisting Surface, h2 =	0.000	ft.
Allow. Soil Bearing Pressure, Pa =	1.500	ksf

Pier Loadings:

Axial Load, Pv =	0.600	kips
Horizontal Load, Ph =	0.216	kips
Distance from Ph to Top/Pier, H =	17.500	ft.
Externally Applied Moment, M =	0.000	ft-kips



Results:

Pier Embedment and Total Length:

Ho =	0.11	kips/ft.	Ho = Ph/D
Mo =	2.16	ft-kips/ft.	Mo = (M+Ph*(H+h1+h2))/D
E =	20.00	ft.	E = Mo/Ho
Kp =	3.000		Kp = TAN^2(45+ϕ/2) (passive soil pressure coefficient)
R =	0.300	ksf/ft.	R = Kp*γ (passive soil resistance/ft. depth)
L =	5.47	ft.	L = solution of cubic equation: L^3-14.14*Ho*L/R-18.85*Mo/R=0
L/D =	2.73		L/D <= 10 for valid short, rigid pier analysis L/D<=10, O.K.
Lt =	7.97	ft.	Lt = h1+h2+L (total length)

Nomenclature

Pier Side Soil Pressures:

a =	3.716	ft.	a = L*(4*E/L+3)/(6*E/L+4) ("pivot" point from top of resisting surface)
Pc =	0.557	ksf	Pc = 1.178*(4*Mo+3*Ho*L)^2/(L^2*(3*Mo+2*Ho*L))
Pc(allow) =	0.557	ksf	Pc(allow) = R*(a/2) Pc(allow)<Pc
Pt =	1.548	ksf	Pt = 9.425*(2*Mo+Ho*L)/L^2
Pt(allow) =	1.640	ksf	Pt(allow) = R*L Pt(allow)>=Pt, O.K.

Pier End Bearing Pressure:

Af =	3.14	ft.^2	Af = π*D^2/4 (pier base area)
Wf =	3.75	kips	Wf = (Af*Lt)*0.150 (pier weight)
ΣPv =	4.35	kips	ΣPv = Pv+Wf (total vertical load)
P(bot) =	1.386	ksf	P(bot) = ΣPv/Af Pa>=P(bot), O.K.

(continued)

Pier Shear and Moment:

Maximum Shear:
V(max) = 1.54 kips — (located at distance = a, from top of resisting surface)
V(max) = ABS(Ho*D*(1-3*(4*E/L+3)*(a/L)^2+4*(3*E/L+2)*(a/L)^3)

Maximum Moment:
M(max) = 4.11 ft-kips — (located at distance = a/2, from top of resisting surface)
M(max) = Ho*D*L*(E/L+a/2/L-(4*E/L+3)*(a/2/L)^3+(3*E/L+2)*(a/2/L)^4)

Pier Plain Concrete Stresses: (Plain concrete allowable stresses from ACI 318-11, Chapter 22)

Axial Compressive Stress:
fa = 5.87 psi — fa = (Pv+π*D^2/4*(h1+h2+a/2)*0.15)/(π*(D*12)^2/4)*1000

Flexural Tension/Compression Stress:
fb = 36.33 psi — fb = M(max)*12/(π*(D*12)^3/32)*1000

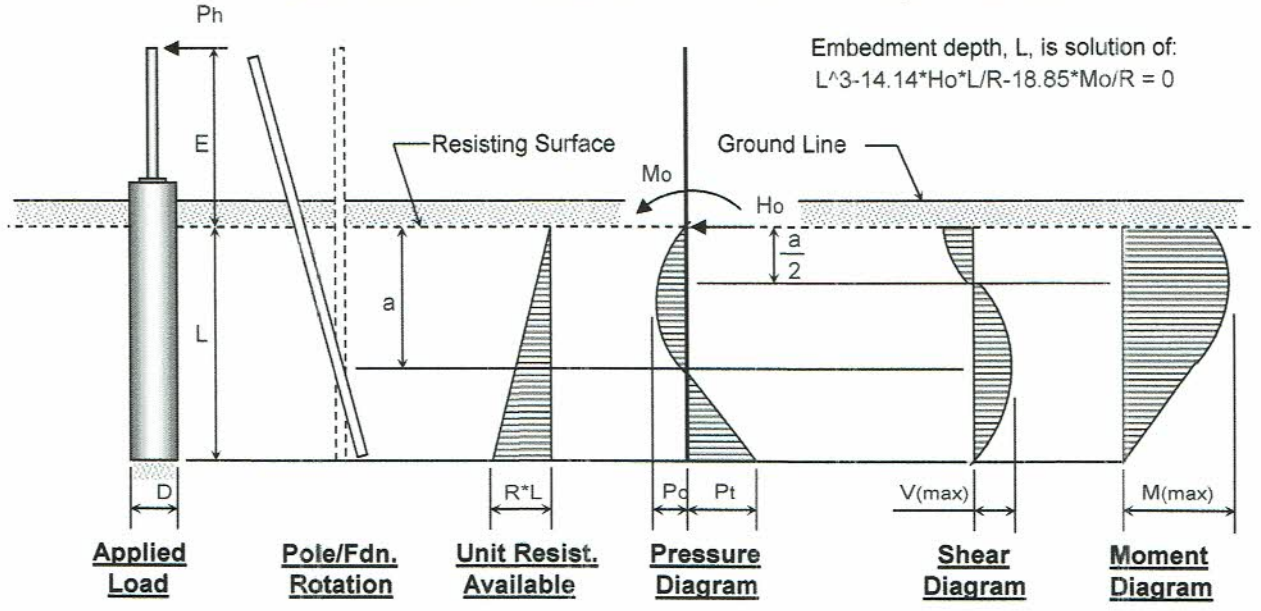
Combined Compression Stress: (ϕ = 0.60 per Sect. 9.3.5, and divide USD allowable by 1.6 for ASD)
fc = 42.20 psi — fc = fb+fa (compression)
Fc(allow) = 1434.38 psi — Fc(allow) = 0.85*ϕ*(f'c*1000)/1.6 — Fc(allow)>=fc, O.K.

Combined Tension Stress: (ϕ = 0.60 per Sect. 9.3.5, and divide USD allowable by 1.6 for ASD)
ft = 30.46 psi — ft = fb-fa (tension)
Ft(allow) = 125.78 psi — Ft(allow) = 5*ϕ*SQRT(f'c*1000)/1.6 — Ft(allow)>=ft, O.K.

Shear Stress: (ϕ = 0.60 per Sect. 9.3.5, and divide USD allowable by 1.6 for ASD)
fv = 4.55 psi — fv = 4/3*V(max)*1000/(π*(D*12)^2/4)
Fv(allow) = 50.31 psi — Fv(allow) = 2*ϕ*SQRT(f'c*1000)/1.6 — Fv(allow)>=fv, O.K.

Applied Lateral Load and Resistance of Pole/Foundation

Reference: "Resistance to Overturning of Single, Short Piles" - by Eli Czerniak
ASCE Journal of the Struct. Div., Vol. 83, No. ST2, Paper 1188, Mar. 1957



JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

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HIGH ROOF DIAPHRAGM : LATERAL ANALYSIS

SEISMIC WEIGHT

ROOF WEIGHT = $20_{DL} \times 533 ft^2 = 10.66^K$

WALLS = $47_{DL}(4.2'/2) \times 2^{+13'} + 47_{DL}(8'/2)^{+25'} + 15_{DL}(9.5'/2) \times 25'$

$= 9^K{}_{DL}$

$E_{TOTAL} = 10.66^K + 9^K = 19.66^K$

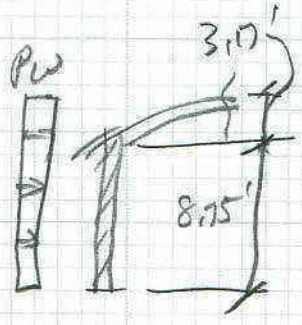
$V_{BASE} = 1^K$ - SEE CALC

WIND

$P_W = 24.7\ PSF$

$w_{eq} = 24.7 PSF (3.17'/2 + 8.75'/2) = 147\ PLF$ (WIND ULTIMATE)

← AVG. WALL HEIGHT



SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

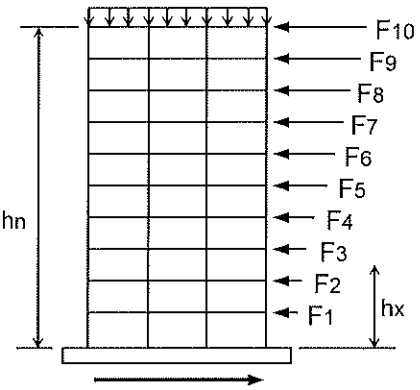
Per IBC 2018 and ASCE 7-16 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	QNC Car Wash	Subject:	Lateral - High Roof Diaphragm		
Job Number:	2022152	Originator:	JMB	Checker:	SJJ

Input Data:

Risk Category =	II	IBC 2018, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2, page 5
Soil Site Class =	C	ASCE 7-16 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., Ss =	0.200	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S1 =	0.056	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, TL =	4.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, hn =	7.200	ft.
Actual Calc. Period, Tc =	0.100	sec. from independent analysis
Seismic Resist. System =	A8	Intermediate reinf. masonry shear walls (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 0.97 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	7.200	19.66

Total Weight, W = ΣWx = 19.66 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

Fa =	1.300	ASCE 7-16 Table 11.4-1, page 66
Fv =	1.500	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	0.260	SMS = Fa*Ss, ASCE Eqn. 11.4-1, page 65
SM1 =	0.084	SM1 = Fv*S1, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.173	SDS = 2*SMS/3, ASCE 7-16 Eqn. 11.4-3, page 65
SD1 =	0.056	SD1 = 2*SM1/3, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1, page 67
Category(for S_{D1}) =	A	ASCE 7-16 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2, page 90
Approx. Period, T_a =	0.088	sec., $T_a = C_T*h_n\text{^}(x)$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.700	ASCE 7-16 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.149	sec., $T_{(max)} = C_u*T_a$, ASCE 7-16 Section 12.8.2, page 90
Fundamental Period, T =	0.100	sec., T = Tc <= Cu*Ta, ASCE 7-16 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-16 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	2.5	ASCE 7-16 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.25	ASCE 7-16 Table 12.2-1, pages 73-75
C_S =	0.050	$C_S = S_{DS}/(R/I)$, ASCE 7-16 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.160	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-16 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-16 Eqn. 12.8-5
Use: C_S =	0.050	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	0.97	kips, V = CS*W, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx}*V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x*h_x\text{^}k/(\Sigma W_i*h_i\text{^}k)$, ASCE 7-16 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	$h_x\text{^}k$ (ft.)	$W_x*h_x\text{^}k$ (ft-kips)	C_{vx} (%)	Shear, Fx (kips)	Σ Story Shears
1	19.66	7.200	141.6	1.000	0.97	0.97
Σ =	19.66		141.6	1.000	0.97	

Comments:

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

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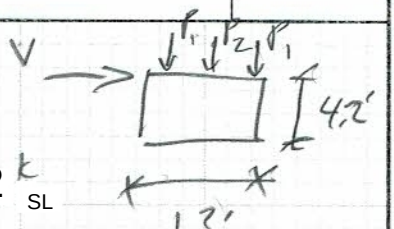
Calc. by	Contact Phone #	EMAIL	Date

SW LINE A AT HIGH ROOF



V = 1.8^{k} (WIND STRENGTH)

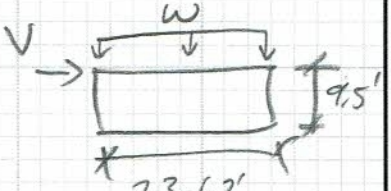
$P_1 = (20_{DL} + 31.5_{SL})(5.44'/2 + 2.45')(24.7'/2) = 1.27^{k}_{DL} + 2^{k}_{SL}$

$P_2 = (20_{DL} + 31.5_{SL})(5.44')(24.7'/2) = 1.34^{k}_{DL} + 2.1^{k}_{SL}$

USE 6" MASONRY W/ 1#5 VERT. AT 16" O.C. AND 1#5 VERT. AT EACH END OF WALL

SW LINE 4 AT HIGH ROOF WORKS BY INSPECTIONS AND ENGINEERING JUDGEMENT WITH SAME MASONRY AS SW LINE A AT HIGH ROOF WITH 25' OF WALL

SW LINE 3 AT HIGH ROOF



V = .92^{k} (WIND STRENGTH) x 0.6 = 0.55^{k} (ASD)

$w = (20_{DL} + 31.5_{SL})(1.8' + 0.67'/2) = 43^{PLF}_{DL} + 67^{PLF}_{SL}$

SHEAR = v = 550#/23.67' = 23 PLF < 760 PLF ⇒ OK

SHEAR CAPACITY FOR 1/2" THICK PLYWOOD SHEATH. W/ #8 SCREWS AT 4" O.C. AT EDGE AND 12" O.C. AT FIELD

USE 1/2" THICK OSB SHEATHING W/ #8 SCREWS AT 4" O.C. AT EDGE AND #8 SCREWS AT 12" O.C. AT FIELD

ACCEPTABLE →

CHECK OVERTURNING: $M_{OT} = 840^{\#} \times 9.5' = 7.98$ k-ft (ASD)

T = 7.98 k-ft / 23.67' = 0.34K < 2690# ⇒ OK

SIMPSON FC32 AT EACH END

USE SIMPSON FC32 AT EACH END OF WALL W/ 6 #10 SCREWS

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 2:29PM

Masonry Shear Wall

File: 2022152 - QNC Car Wash.ec6

Lic. # : KW-06012033
JTA Engineering, LLC

DESCRIPTION: SW LINE A AT HIGH ROOF - (Shear Wall)

Code References

2021 OK

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	4.20 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	13.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	3.0	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	13.0 ft
Effective Length 'd'	152.0 in
Nominal Block Thickness	6.0 in
Solid Grout?	Partial Groute

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	16 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	1.0

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
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Lic. # : KW-06012033

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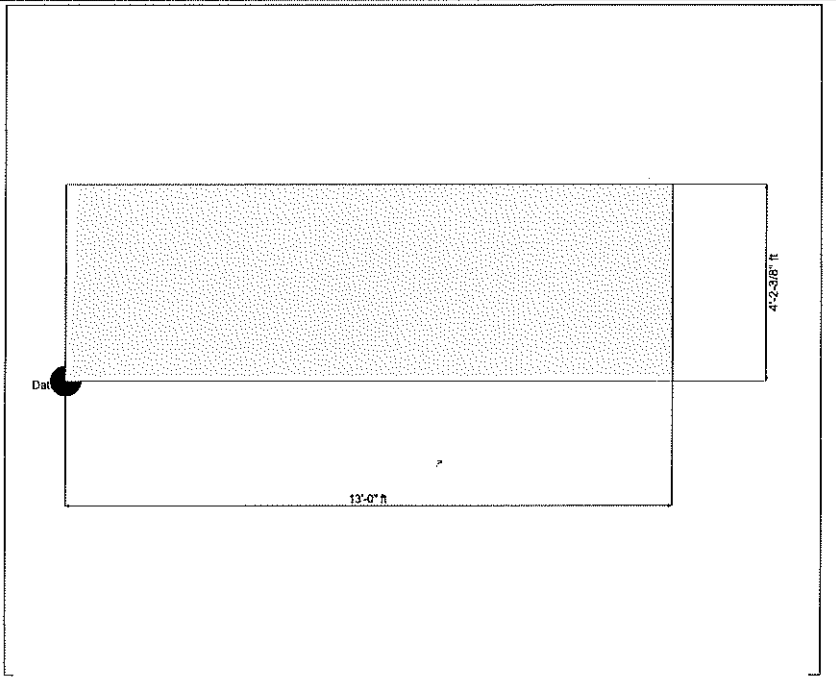
DESCRIPTION: SW LINE A AT HIGH ROOF - (Shear Wall)

Wall Sketch



Applied Concentrated Vertical Loads

Load Location (ft)		Load Magnitude (kips)				
X Location	Y Location	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
1.120	4.20	1.30	0.0	0.0	2.0	0.0
6.560	4.20	1.340	0.0	0.0	2.10	0.0
12.0	4.20	1.30	0.0	0.0	2.0	0.0

Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
4.20	0.0	0.0	0.0	1.80	0.0	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	1.80 k
for Load Combination	+1.20D+W
Controlling Mu/(Vud)	0.34
Vn Masonry	99.132 k
Vn Steel	0.0 k
Vn Masonry + Vn Steel	99.132 k
Vn Max	120.283 k
Phi Vn	79.306 k
Ratio: Vu/PhiVn (controlling)	**0.02270**
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

Lic. # : KW-06012033

DESCRIPTION: SW LINE A AT HIGH ROOF - (Shear Wall)

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	0.33
Pu	17.359 k
for Load Combination	+1.20D+1.60S
Phi Pn	+1.20D+1.60S k
Ratio: Pu/PhiPn (controlling)	**0.01953**

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	2.76 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	152.0
As-flex < As-max ?	: 0.310 <= 7.577
Mu	8.027 k
for Load Combination	+1.20D+0.50S+W
Phi Mn	210.118 k
Ratio: Mu/PhiMn (controlling)	**0.03820**

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1		0.331	8.866	0.037			
+1.20D							
Wall Level : 1		0.284	7.599	0.037			
+1.20D+0.50S							
Wall Level : 1		0.467	10.649	0.044			
+1.20D+0.50W							
Wall Level : 1	0.900	4.064	7.599	0.535	12.169		
+1.20D+1.60S							
Wall Level : 1		0.869	17.359	0.050			
+1.20D+1.60S+0.50W							
Wall Level : 1	0.900	4.649	17.359	0.268	28.798		
+1.20D+W							
Wall Level : 1	1.800	7.844	7.599	1.032	6.085		
+1.20D+0.50S+W							
Wall Level : 1	1.800	8.027	10.649	0.754	8.683		
+0.90D+W							
Wall Level : 1	1.800	7.773	5.699	1.364	5.078	0.022	0.022
+1.40D+0.20S+E							
Wall Level : 1	0.957	2.414	10.086	0.040			
+0.70D+E							
Wall Level : 1	0.957	2.175	4.433	0.037			

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

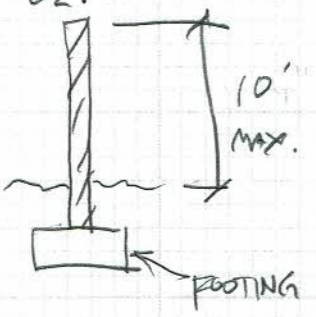
Project	Job #.
QNC - LAS VEGAS	220081
Section	Sheet no./rev.
Calc by JB / Contact Phone # / EMAIL / Date	

MASONRY SCREEN WALL DESIGN (DETAIL 02)

AT EQUIPMENT/VACCUUM AREA 10' HIGH MAX.

WIND = 26 PSF - SEE NEXT PAGE FOR CALC

*ONLY USING 6' MAX. AND 10' MAX IN DETAIL 02.



USE 8" MASONRY W/ 1#5 VERT. AT EACH FACE AT 8" O.C.

SEE CALC

USE 12" THICK MIN. x 6'-0" WIDE CONT. CONCRETE FOOTING W/ 6#6 CONT. TOP & BOTTOM

6' HIGH MAX. MASONRY SITE WALL AT TRASH ENCLOSURE

WIND ≤ 26 PSF

USE 8" MASONRY W/ 1#5 VERT. AT 32" O.C. - CENTERED

SEE CALC

USE 12" THICK x 3'-6" WIDE CONT. CONCRETE FOOTING W/ 2#5 CONT.

JTA Engineering
15825 S 46th St, Suite 129
Phoenix, AZ 85048
623.237.9349

JOB NO. 2022152 SHEET NO.
CALCULATED BY JMB DATE
CHECKED BY SJJ DATE

Wind Loads - Other Structures: ASCE 7- 16 Ultimate Wind Pressures

Wind Factor =	1.00		
Gust Effect Factor (G) =	0.85	Ultimate Wind Speed =	107 mph
Kzt =	1.00	Exposure =	C

USE

A. Solid Freestanding Walls & Solid Signs (& open signs with less than 30% open)

		s/h =	1.00	**Case A & B**	
Dist to sign top (h)	10.0 ft	B/s =	1.20	C_f =	1.44
Height (s)	10.0 ft	Lr/s =	0.00	F = qz G Cf As =	**25.9 As**
Width (B)	12.0 ft	Kz =	0.849	As =	0.0 sf
Wall Return (Lr) =		qz =	21.1 psf	F =	0 lbs
Directionality (Kd)	0.85				
Percent of open area to gross area	0.0%	Open reduction factor =	1.00	**CaseC**	

Case C reduction factors		Horiz dist from windward edge	Cf	F=qzGCfAs (psf)
Factor if s/h>0.8 =	0.80	0 to s	1.80	32.4 **As**
Wall return factor for Cf at 0 to s =	1.00	s to 2s	1.20	21.6 **As**

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JTA Engineering, LLC

Cantilevered Retaining Wall

Lic. # : KW-06012033

DESCRIPTION: 10FT HIGH MASONRY SITE WALL

Calculations per ACI 318-14, TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Criteria

Retained Height	=	0.50 ft
Wall height above soil	=	10.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft
Vertical component of active Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data

Allow Soil Bearing	=	1,500.0 psf ✓
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
Toe Active Pressure	=	40.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	120.00 pcf
Soil Density, Toe	=	120.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.450
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	26.0 psf ✓

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	2.47 OK ✓
Sliding	=	3.61 OK ✓
Total Bearing Load	=	1,672 lbs
...resultant ecc.	=	12.13 in
Soil Pressure @ Toe	=	748 psf OK ✓
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	898 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	8.6 psi OK
Footing Shear @ Heel	=	5.4 psi OK
Allowable	=	82.2 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	260.0 lbs
less 100% Passive Force	= -	187.5 lbs
less 100% Friction Force	= -	752.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

			Stem OK
Design Height Above Ftg	ft =		0.00
Wall Material Above "Ht"	=		Masonry
Thickness	in =		8.00
Rebar Size	=		# 5
Rebar Spacing	in =		16.00
Rebar Placed at	=		Edge
Design Data			
fb/FB + fa/Fa	=		0.789 ✓
Total Force @ Section	lbs =		260.0
Moment....Actual	ft-l =		1,430.0
Moment.....Allowable	ft-l =		1,812.8
Shear.....Actual	psi =		4.1
Shear.....Allowable	psi =		38.7
Wall Weight	psf =		63.0
Rebar Depth 'd'	in =		5.25
Lap splice if above	in =		30.00
Lap splice if below	in =		6.39
Hook embed into footing	in =		6.39
Masonry Data			
f'm	psi =		1,500
Fy	psi =		20,000
Solid Grouting	=		No
Modular Ratio 'n'	=		21.48
Short Term Factor	=		1.000
Equiv. Solid Thick.	in =		5.80
Masonry Block Type	=	2	
Masonry Design Method	=	ASD	

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 2:00PM
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JTA Engineering, LLC

Cantilevered Retaining Wall

Lic. # : KW-06012033

DESCRIPTION: 10FT HIGH MASONRY SITE WALL

Footing Dimensions & Strengths

Toe Width	=	2.17 ft
Heel Width	=	2.83
Total Footing Width	=	5.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 3.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	898	0 psf
Mu' : Upward	=	2,019	0 ft-lb
Mu' : Downward	=	686	686 ft-lb
Mu: Design	=	1,333	686 ft-lb
Actual 1-Way Shear	=	8.59	5.35 psi
Allow 1-Way Shear	=	82.16	82.16 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb			.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	45.0	0.50	22.5	Soil Over Heel	=	130.0	3.92	509.2
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=	-45.0	0.50	-22.5	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=	260.0	6.50	1,690.0	Soil Over Toe	=	130.0	1.08	140.8
					Surcharge Over Toe	=			
					Stem Weight(s)	=	661.5	2.50	1,653.8
					Earth @ Stem Transitions	=			
Total	=	260.0	**O.T.M.** =	1,690.0	Footing Weight	=	750.0	2.50	1,875.0
					Key Weight	=			
					Vert. Component	=			
					Total =		1,671.5 lbs	**R.M. =**	4,178.8

Resisting/Overturning Ratio = **2.47**
Vertical Loads used for Soil Pressure = 1,671.5 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

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Cantilevered Retaining Wall

Lic. # : KW-06012033
JTA Engineering, LLC

DESCRIPTION: 6FT HIGH MASONRY SITE WALL

Calculations per ACI 318-14, TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Criteria

Retained Height	=	0.50 ft
Wall height above soil	=	6.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf ✓
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
Toe Active Pressure	=	40.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	120.00 pcf
Soil Density, Toe	=	120.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.450
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	26.0 psf ✓

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	2.62 OK ✓
Sliding	=	4.24 OK ✓
Total Bearing Load	=	1,053 lbs
...resultant ecc.	=	8.00 in
Soil Pressure @ Toe	=	648 psf OK ✓
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	777 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	4.6 psi OK
Footing Shear @ Heel	=	3.5 psi OK
Allowable	=	82.2 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	156.0 lbs
less 100% Passive Force	= -	187.5 lbs
less 100% Friction Force	= -	478.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Masonry
Thickness	in =	8.00
Rebar Size	=	# 5
Rebar Spacing	in =	32.00
Rebar Placed at	=	Center
Design Data		
fb/FB + fa/Fa	=	0.830 ✓
Total Force @ Section	lbs =	156.0
Moment....Actual	ft-l =	546.0
Moment.....Allowable	ft-l =	658.2
Shear.....Actual	psi =	3.5
Shear.....Allowable	psi =	38.7
Wall Weight	psf =	55.0
Rebar Depth 'd'	in =	3.75
Lap splice if above	in =	45.00
Lap splice if below	in =	6.39
Hook embed into footing	in =	6.39
Masonry Data		
f'm	psi =	1,500
Fy	psi =	20,000
Solid Grouting	=	No
Modular Ratio 'n'	=	21.48
Short Term Factor	=	1.000
Equiv. Solid Thick.	in =	4.90
Masonry Block Type	=	2
Masonry Design Method	=	ASD

JARAMILLO-TONNESON & ASSOCIATES
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JTA Engineering, LLC

Cantilevered Retaining Wall

Lic. # : KW-06012033

DESCRIPTION: 6FT HIGH MASONRY SITE WALL

Footing Dimensions & Strengths

Toe Width	=	1.42 ft
Heel Width	=	2.08
Total Footing Width	=	3.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 3.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	777	0 psf
Mu' : Upward	=	816	0 ft-lb
Mu' : Downward	=	316	316 ft-lb
Mu: Design	=	500	316 ft-lb
Actual 1-Way Shear	=	4.61	3.50 psi
Allow 1-Way Shear	=	82.16	82.16 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

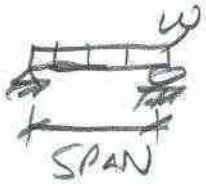
		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb			.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	45.0	0.50	22.5	Soil Over Heel	=	85.0	2.79	237.3
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=	-45.0	0.50	-22.5	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=	156.0	4.50	702.0	Soil Over Toe	=	85.0	0.71	60.2
					Surcharge Over Toe	=			
					Stem Weight(s)	=	357.5	1.75	625.6
					Earth @ Stem Transitions	=			
Total	=	156.0	**O.T.M.** =	702.0	Footing Weight	=	525.0	1.75	918.8
					Key Weight	=			
					Vert. Component	=			
					Total =		1,052.5 lbs	**R.M. =**	1,841.9

Resisting/Overturning Ratio = 2.62
Vertical Loads used for Soil Pressure = 1,052.5 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DETAIL 210

M1 5' SPAN



$w = (10_{DL} + 31.5_{SL})(4')$

$= 40 PLF_{DL} + 126 PLF_{SL}$

+ SELF-WEIGHT

WIND = 18 PSF (4') = 72 PLF (STRENGTH)

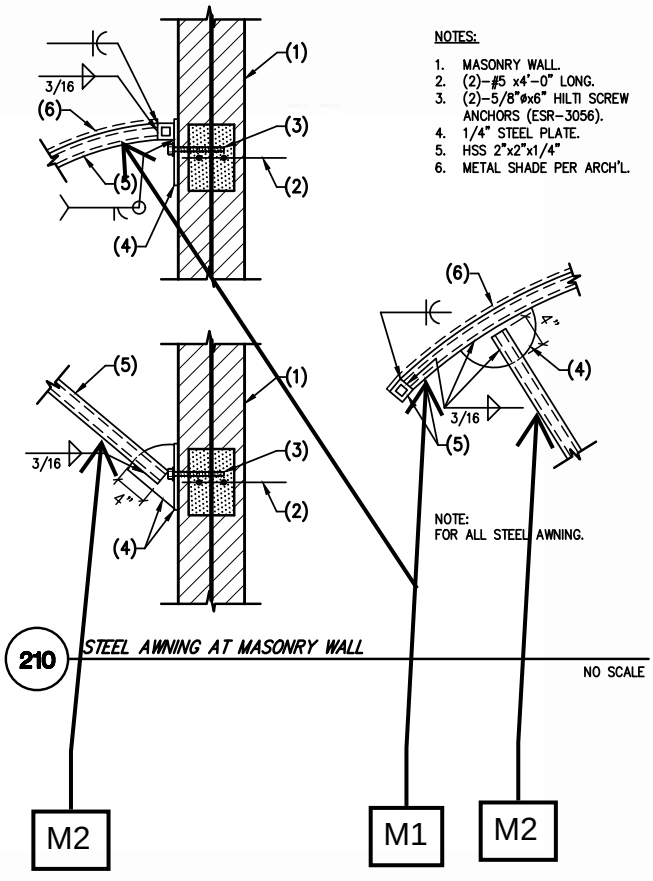
USE HSS 2x2x1/4 -CURVED PER ARCHITECT

M2 3' SPAN

$P = 0.024^{K}_{DL} + 0.315^{K}_{SL} + 0.180^{K}_{WIND}$

↑ REACTION FROM M1

USE HSS 2x2x1/4



JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
PHOENIX, AZ 85044

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 19 JAN 2023, 3:07PM

Steel Beam

File: 2022152 - QNC Car Wash.ec6

Lic. # : KW-06012033

JTA Engineering, LLC

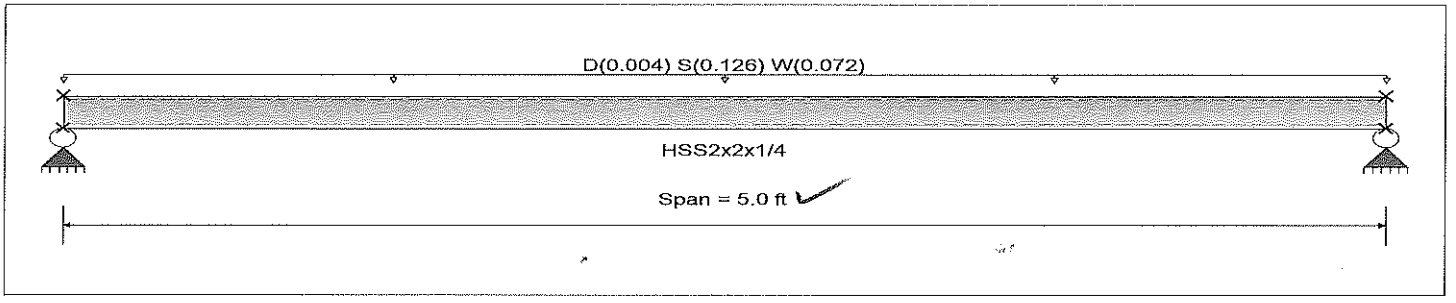
DESCRIPTION: M1 (Steel Awning Member)

CODE REFERENCES

Calculations per AISC 360-16, IBC ~~2018~~ 2021, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.0040, S = 0.1260, W = 0.0720 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.193 : 1**	Maximum Shear Stress Ratio =	**0.034 : 1**
Section used for this span	**HSS2x2x1/4**	Section used for this span	**HSS2x2x1/4**
Ma : Applied	0.426 k-ft	Va : Applied	0.3408 k
Mn / Omega : Allowable	2.213 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
Location of maximum on span	2.500 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.047 in	Ratio =	1,277 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.034 in	Ratio =	1749 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+S													
Dsgn. L = 5.00 ft	1	0.191	0.034	0.42		0.42	3.70	2.21	1.14	1.00	0.34	16.73	10.02
D Only													
Dsgn. L = 5.00 ft	1	0.013	0.002	0.03		0.03	3.70	2.21	1.14	1.00	0.02	16.73	10.02
+D+0.60W													
Dsgn. L = 5.00 ft	1	0.074	0.013	0.16		0.16	3.70	2.21	1.14	1.00	0.13	16.73	10.02
+D-0.60W													
Dsgn. L = 5.00 ft	1	0.048	0.008		-0.11	0.11	3.70	2.21	1.14	1.00	0.08	16.73	10.02
+D+0.750S+0.450W													
Dsgn. L = 5.00 ft	1	0.193	0.034	0.43		0.43	3.70	2.21	1.14	1.00	0.34	16.73	10.02
+D+0.750S-0.450W													
Dsgn. L = 5.00 ft	1	0.101	0.018	0.22		0.22	3.70	2.21	1.14	1.00	0.18	16.73	10.02
+0.60D+0.60W													
Dsgn. L = 5.00 ft	1	0.069	0.012	0.15		0.15	3.70	2.21	1.14	1.00	0.12	16.73	10.02
+0.60D-0.60W													
Dsgn. L = 5.00 ft	1	0.053	0.009		-0.12	0.12	3.70	2.21	1.14	1.00	0.09	16.73	10.02
+1.168D													
Dsgn. L = 5.00 ft	1	0.016	0.003	0.03		0.03	3.70	2.21	1.14	1.00	0.03	16.73	10.02
+1.126D+0.750S													
Dsgn. L = 5.00 ft	1	0.148	0.026	0.33		0.33	3.70	2.21	1.14	1.00	0.26	16.73	10.02
+0.4320D													
Dsgn. L = 5.00 ft	1	0.006	0.001	0.01		0.01	3.70	2.21	1.14	1.00	0.01	16.73	10.02

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
4411 E. KNOX RD.
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Steel Beam

Lic. # : KW-06012033
JTA Engineering, LLC

DESCRIPTION: M1 (Steel Awning Member)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.0470	2.514		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.315	0.315
Overall MINimum	0.024	0.024
D Only	0.024	0.024
S Only	0.315	0.315
W Only	0.180	0.180

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Steel Beam

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: M2

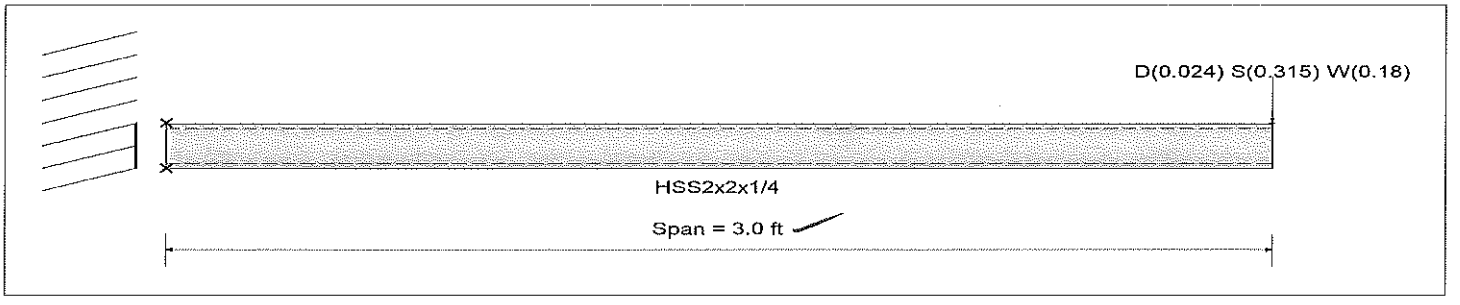
2021 OK

CODE REFERENCES

Calculations per AISC 360-16, IBC ~~2018~~, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 0.0240, S = 0.3150, W = 0.180 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.474 : 1	Maximum Shear Stress Ratio =	0.036 : 1
Section used for this span	HSS2x2x1/4	Section used for this span	HSS2x2x1/4
Ma : Applied	1.048 k-ft	Va : Applied	0.3575 k
Mn / Omega : Allowable	2.213 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+0.750S+0.450W	Load Combination	+D+0.750S+0.450W
Location of maximum on span	0.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.226 in	Ratio =	319 >=240.
Max Upward Transient Deflection	-0.129 in	Ratio =	558 >=240.
Max Downward Total Deflection	0.249 in	Ratio =	289 >=180
Max Upward Total Deflection	-0.064 in	Ratio =	1117 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 3.00 ft	1	0.044	0.004		-0.10	0.10	3.70	2.21	1.00	1.00	0.04	16.73	10.02
+D+S													
Dsgn. L = 3.00 ft	1	0.471	0.035		-1.04	1.04	3.70	2.21	1.00	1.00	0.36	16.73	10.02
+D+0.750S													
Dsgn. L = 3.00 ft	1	0.364	0.028		-0.81	0.81	3.70	2.21	1.00	1.00	0.28	16.73	10.02
+D+0.60W													
Dsgn. L = 3.00 ft	1	0.190	0.015		-0.42	0.42	3.70	2.21	1.00	1.00	0.15	16.73	10.02
+D-0.60W													
Dsgn. L = 3.00 ft	1	0.103	0.008	0.23		0.23	3.70	2.21	1.00	1.00	0.08	16.73	10.02
+D+0.450W													
Dsgn. L = 3.00 ft	1	0.153	0.012		-0.34	0.34	3.70	2.21	1.00	1.00	0.12	16.73	10.02
+D-0.450W													
Dsgn. L = 3.00 ft	1	0.066	0.006	0.15		0.15	3.70	2.21	1.00	1.00	0.06	16.73	10.02
+D+0.750S+0.450W													
Dsgn. L = 3.00 ft	1	0.474	0.036		-1.05	1.05	3.70	2.21	1.00	1.00	0.36	16.73	10.02
+D+0.750S-0.450W													
Dsgn. L = 3.00 ft	1	0.254	0.020		-0.56	0.56	3.70	2.21	1.00	1.00	0.20	16.73	10.02
+0.60D+0.60W													
Dsgn. L = 3.00 ft	1	0.173	0.013		-0.38	0.38	3.70	2.21	1.00	1.00	0.13	16.73	10.02
+0.60D-0.60W													
Dsgn. L = 3.00 ft	1	0.120	0.009	0.27		0.27	3.70	2.21	1.00	1.00	0.09	16.73	10.02
+0.60D													

JARAMILLO-TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
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Steel Beam

Lic. # : KW-06012033 — JTA Engineering, LLC

DESCRIPTION: M2

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 3.00 ft	1	0.026	0.002		-0.06	0.06	3.70	2.21	1.00	1.00	0.02	16.73	10.02

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750S+0.450W	1	0.2488	3.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.315	
Overall MINimum	0.040	
D Only	0.040	
S Only	0.315	
W Only	0.180	
-W	-0.180	

JARAMILLO TONNESON & ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
15825 S 46TH ST, SUITE 129
PHOENIX, AZ 85048

Project			Job #.
Section			Sheet no./rev.
Calc. by	Contact Phone #	EMAIL	Date

VACCUM CANOPY FOOTING

VACCUM WEIGHT = 400#

UPLIFT AT CANOPY = 10.5' x 7.2' x -16PSF = -1210# (STRENGTH WIND UPLIFT)

TRY 2.5'ø x 4'-8" DEEP

PIER WEIGHT = [π(1.25')²(4.67')(0.15 kcf)] x 0.6 = 2.063K > 1.21K ⇒ OK

USE 30" DIAMETER x 4'-8" DEEP CONCRETE PIER FOOTING W/ (8) #6 VERTICALS AND #3 TIES AT 12" O.C.

PAY CANOPY FOOTING

WEIGHT = 700#

UPLIFT AT CANOPY = 17' x 9.75' x -16PSF = -2652#

TRY 2.5'ø x 6' DEEP

PIER WEIGHT = π(1.25')²(6')(0.15kcf) = 4.42K x 0.6 = 2.652K ≥ 2.652K ⇒ OK

USE 30" DIAMETER x 6'-0" DEEP CONCRETE PIER FOOTING W/ (8) #6 VERTICALS AND #3 TIES AT 12" O.C.