New Automated Car Wash Facility

Almaville Road & General Forrest, Smyrna, TN

Structural Design Calculations

Architect: Neri Architects

March 24, 2023
Permit

DG STRUCTURAL ENGINEERING LLC
WEB: www.dgstructural.com

Design Criteria:
International Building Code 2018 (village website)
ASCE7-16

Ground Snow load = 10 psf
Frost depth = 12"

Wind speed = 105 mph
Exposure B

Site class = D (assumed)
Risk Category = II
S_S = 0.255
S_1 = 0.129
S_{DS} = 0.272
S_{D1} = 0.201
Seismic Design category D

Carwash Roof loads:
Low roof:
Superimposed Dead load = 10 psf (+precast)
8" Precast = 60 psf
Live load = 20 psf

2nd floor: Storage
Live load = 125 psf
Dead load = 60 psf (2" VLI20+3" Normal wt conc. deck)

High roof:
Dead load = 20 psf (metal deck)
Live load = 20 psf

1st floor loads: Slab on grade
Live load = 100 psf (Retail)

Footing:
Net Soil bearing capacity = 3000 psf (assumed)

Wind Speed, V = 115 mph

Exterior Load bearing wall:
Roof = 110*20'/2 = 1100 plf
Masonry wall = 120*20' = 2400 plf
Total = 3500 plf
Width of footing req'd = 3500/3000 = 1.2'
2'-0" wide wall footing

Interior Load bearing wall:
Roof = 110*20'/2 = 1100 plf
2nd floor = 185*8' = 1480 plf
High roof = 40*8' = 320 plf
8" masonry wall = 60*40' = 2400 plf
Total = 5300 plf
Width of footing req'd = 5300/3000 = 1.8'
2'-0" wide wall footing

Roof framing:
8" precast planks (60 psf self weight)
Span = 20'-0" (interior bearing wall/beams)
Allowable load =134 psf

High Roof loads:
Live load = 20 psf
Dead load = 20 psf (metal deck/steel)

Steel beam: Span = 16', Spacing = 6'-0" max
Load = 40*6' = 240 plf
M = 8.3 k-ft
W8x18 (see spreadsheet)

2nd floor loads:
LL = 125 psf
DL = 60 psf

Steel beam: Span = 16', Spacing = 6'-6" max
Lad = 185*7' = 1295 plf
W8x28

Roof beam design:
Steel beam 1: Span = 17'-0"
roof load = 110*10' = 1100 plf
masonry wall = 60*12' = 720 plf
Total load = 1820 plf
W16x40
M = 68.8 k-ft
R = 16.2 k
Wind force = 20*12' = 240 plf
R = 2.4 k
Combined = 0.2+0.86 = 1.06~1

Column: Ht = 20' HSS6x6x1/2
P = 16.2+16.2 = 32.4 k
H = 4.8 k @ 14'
Eq H1-3 = 0.97
Def = 0.79"
Footing:
Area req'd = 34/3 = 11.33 sq. ft
4'-0" sq. footing

Roof beam design:
Steel beam 2: Span = 17'-0"
roof load = 110*16' = 1760 plf
Total load = 1760 plf
W12x40
M = 53.5 k-ft
R = 13 k

Column: Ht = 20'
P = 13+13 = 26 k
HSS6x6x1/2, Pall = 131 k

DG Structural Engineering LLC	Neri Charleston, Il Car Wash	BY:	AP	
MASONRY WALL DESIGN		Project #:		page:

ACI 530

* HOLLOW BLOCK MASONRY   Thickness= 8 in
* FACE SHELL MORTAR BEDDING
* BUILDING EXTERIOR WALL

f'm= 2500 psi

An=	46	in2/ft	Rebar #	5	
Ix=	343.7	in4/ft	Spacing	32	in
Sx=	90.1	in3/ft	Fs=	24000	ksi
r=	2.73	in	As=	0.12	in2/ft

h= 21 ft = 252.0 in
h/r= 92.2 < 99

Pa= 17.3 k/ft for axial only

Floor Load					**Roof Load**				
Trib Width=	0	ft			Trib width=	10	ft		
DL=	0	psf			DL=	70	psf		
LL=	0	psf			LL=	40	psf		
Total=	0	psf	= 0	ksf	Total=	110	psf	= 0.11	klf
Per foot: DL=			0.000	klf	Per foot: DL=			0.700	klf
LL=			0.000	klf	LL=			0.400	klf
Total=			0.000	klf	Total=			1.100	klf
Eccentricity=			0	in	Eccentricity=			2	in
Mdl=			0.000	k.in/ft	Mdl=			1.400	k.in/ft
Mll=			0.000	k.in/ft	Mll=			0.800	k.in/ft
Mtotal=			0.000	k.in/ft	Mtotal=			2.200	k.in/ft

No.of floors: 0
Total height of wall 25 ft.
Openings % 0 %
Wall self weight 0.725 k/ft

Total DL= 1.425 k/ft
Total LL= 0.400 k/ft
Total total load= 1.825 k/ft

fa1= 39 psi   2.3.3.2.2
Fa (unreinforced)= 354 psi OK//

Wind Load w= 20 psf

Mw= 13.230 k.in/ft
Mw + Mtotal/2= 14.330 k.in/ft

Working Stress Analysis

d=	3.813	in	Es=	29000000	psi
b=	12	in	Em=	2250000	psi
As=	0.12	in2/ft			
p=	0.002515		m=	13	
n=	0.220911		nd=	0.842223	in
j=	0.926363			<1.25in OK//	

fa2= 802.9 psi
fs= 35267.36 psi >24000x1.33psi NG!!

fa1 + fa2 = 841.9 psi 1/3fmx1.33= 1110.83 psi OK//

Shear

V = M/h + wh/2 = 218.7 lb/ft

Shear stress = 4.8 psi

Fv =√ f'm = 50.0 psi OK//
(max 50psi)

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PROJECT	Smyrna, IL Car Wash		DATE	3/14/2023
SUBJECT			BY	AP

AISC 14th Ed ASD Beam design

Location: High roof beam

Span L=	**16.000**	**ft**				**Fy=**	**50 ksi**
Lb=	2	ft	= 24 in	(unbraced length)		E=	29000.0 ksi
Beam Size Provided:	W8X18		Lp=	4.34 ft		MR=	42.4 k.ft
			Lr=	13.48 ft		VR=	37.4 kips

Uniformly Distributed Loads

				(psf)		
Self weight					18	plf
Floor 1 trib	6.000	ft	DL @	20	120	plf
			LL @	20	120	plf
Wall ht	0	ft		0	0	plf
Floor 2 trib	0	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 3 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		90	0	plf
				w dead=	138	plf
				w live=	120	plf
				w total=	258	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 0.138 | 0.120 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment=	4.42	k.ft
Maximum LL Moment=	3.84	k.ft
Total Moment Mmax=	**8.26**	**k.ft**

	left	Right	
DL Reaction=	1.10	1.10	kips
LL Reaction=	0.96	0.96	kips
Total	2.06	2.06	kips

Max Shear Vmax= 2.06 kips

MR=	42.42	k.ft	**>Mmax**	**OK**
VR=	37.444	kips	**>Vmax**	**OK**

Live load deflection=	0.10	in	=L/	1948	**OK**
Total deflection=	0.21	in	=L/	906	**OK**

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AISC 14th Ed ASD Beam design

Location: 2nd floor beam

Span L=	16.000 ft				**Fy=**	**50**	**ksi**
Lb=	2 ft	= 24 in	(unbraced length)		E=	29000.0	ksi
Beam Size Provided:	W8X28	Lp=	5.72 ft		MR=	67.9	k.ft
		Lr=	21.00 ft		VR=	45.9	kips

Uniformly Distributed Loads

				(psf)		
Self weight					28	plf
Floor 1 trib	7.000	ft	DL @	60	420	plf
			LL @	125	875	plf
Wall ht	0	ft		0	0	plf
Floor 2 trib	0	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 3 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		90	0	plf
				w dead=	448	plf
				w live=	875	plf
				w total=	1323	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 0.448 | 0.875 k/ft

Other loads

	Dead loads	Live loads	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment= 14.34 k.ft
Maximum LL Moment= 28.00 k.ft
Total Moment Mmax= 42.34 k.ft

	left	Right	
DL Reaction=	3.58	3.58	kips
LL Reaction=	7.00	7.00	kips
Total	10.58	10.58	kips

Max Shear Vmax= 10.58 kips

MR= 67.86 k.ft **>Mmax OK**
VR= 45.942 kips **>Vmax OK**

Live load deflection=	0.45	in	=L/	423	**OK**
Total deflection=	0.69	in	=L/	280	**OK**

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AISC 14th Ed ASD Beam design Location: 2nd floor beam stair

Span L=	16.000	ft					**Fy=**	**50**	**ksi**
Lb=	2	ft	=	24	in	(unbraced length)	E=	29000.0	ksi
Beam Size Provided:	W10X39		Lp=	6.99	ft		MR=	116.8	k.ft
			Lr=	24.24	ft		VR=	62.5	kips

Uniformly Distributed Loads

				(psf)		
Self weight					39	plf
Floor 1 trib	7.000	ft	DL @	60	420	plf
			LL @	125	875	plf
Wall ht	0	ft		0	0	plf
Floor 2 trib	0	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 3 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
				w dead=	459	plf
				w live=	875	plf
				w total=	1334	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 0.459 | 0.875 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	1.50	2.40	kips
a=	8.00	8.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment=	20.69	k.ft
Maximum LL Moment=	37.60	k.ft
Total Moment Mmax=	**58.29**	**k.ft**

	left	Right	
DL Reaction=	4.42	4.42	kips
LL Reaction=	8.20	8.20	kips
Total	12.62	12.62	kips

Max Shear Vmax= **12.62 kips**

MR=	116.77	k.ft	**>Mmax OK**
VR=	62.496	kips	**>Vmax OK**

Live load deflection=	0.27	in	=L/	708	**OK**
Total deflection=	0.42	in	=L/	458	**OK**

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AISC 14th Ed ASD Beam design Location: Roof beam 1

Span L=	17.000	**ft**					**Fy=**	**50 ksi**
Lb=	17	ft	= 204 in	(unbraced length)			E=	29000.0 ksi
Beam Size Provided:	W16X40		Lp=	5.55 ft			MR=	79.6 k.ft
			Lr=	15.89 ft			VR=	97.6 kips

Uniformly Distributed Loads

				(psf)		
Self weight					40	plf
Floor 1 trib	10.000	ft	DL @	70	700	plf
			LL @	20	200	plf
Wall ht	0	ft		0	0	plf
Floor 2 trib	0	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 3 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	12	ft		80	960	plf
				w dead=	1700	plf
				w live=	200	plf
				w total=	1900	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 1.700 | 0.200 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment=	61.41	k.ft
Maximum LL Moment=	7.23	k.ft
Total Moment Mmax=	**68.64**	**k.ft**

	left	Right	
DL Reaction=	14.45	14.45	kips
LL Reaction=	1.70	1.70	kips
Total	16.15	16.15	kips

Max Shear Vmax= 16.15 kips

MR=	79.62	k.ft	**>Mmax**	**OK**
VR=	97.6	kips	**>Vmax**	**OK**

Live load deflection=	0.03	in	=L/	8154	**OK**
Total deflection=	0.24	in	=L/	858	**OK**

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AISC 14th Ed ASD Beam design Location: Roof beam 2

Span L=	17.000	**ft**				**Fy=**	**50**	**ksi**
Lb=	17	ft	= 204 in	(unbraced length)		E=	29000.0	ksi
Beam Size Provided:	W12X40		Lp=	6.85 ft		MR=	105.1	k.ft
			Lr=	21.12 ft		VR=	70.2	kips

Uniformly Distributed Loads

				(psf)		
Self weight					40	plf
Floor 1 trib	16.000	ft	DL @	70	1120	plf
			LL @	20	320	plf
Wall ht	0	ft		0	0	plf
Floor 2 trib	0	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 3 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
				w dead=	1160	plf
				w live=	320	plf
				w total=	1480	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 1.160 | 0.320 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment=	41.91	k.ft
Maximum LL Moment=	11.56	k.ft
Total Moment Mmax=	**53.47**	**k.ft**

	left	Right	
DL Reaction=	9.86	9.86	kips
LL Reaction=	2.72	2.72	kips
Total	12.58	12.58	kips

Max Shear Vmax= **12.58 kips**

MR=	105.05	k.ft	**>Mmax**	**OK**
VR=	70.21	kips	**>Vmax**	**OK**

Live load deflection=	0.07	in	=L/	3020	**OK**
Total deflection=	0.31	in	=L/	653	**OK**

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AISC 14th Ed ASD Beam design

Location: L1

Span L=	12.000 ft				**Fy=**	**50 ksi**
Lb=	12 ft	= 144 in	(unbraced length)		E=	29000.0 ksi
Beam Size Provided:	W16X26	Lp=	3.96 ft		MR=	51.3 k.ft
		Lr=	11.17 ft		VR=	78.5 kips

Uniformly Distributed Loads

				(psf)	
Self weight					26 plf
Floor 1 trib	2.000 ft	DL @		70	140 plf
		LL @		20	40 plf
Wall ht	0 ft			0	0 plf
Floor 2 trib	0 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	0 ft			0	0 plf
Floor 3 trib	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	0 ft			0	0 plf
Floor 4 trib	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	0 ft			0	0 plf
Floor 5 trib	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	12 ft			80	960 plf
				w dead=	1126 plf
				w live=	40 plf
				w total=	1166 plf

Alternative uniform loads

w dead=	*0 plf*
w live=	*0 plf*
w total alt=	*0 plf*

Note:include self weight in dead load

Design uniform load w= 1.126 | 0.040 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment= 20.27 k.ft
Maximum LL Moment= 0.72 k.ft
Total Moment Mmax= 20.99 k.ft

	left	Right	
DL Reaction=	6.76	6.76	kips
LL Reaction=	0.24	0.24	kips
Total	7.00	7.00	kips

Max Shear Vmax= 7.00 kips

MR= 51.31 k.ft **>Mmax OK**
VR= 78.5 kips **>Vmax OK**

Live load deflection=	0.00 in	=L/	67353	**OK**
Total deflection=	0.06 in	=L/	2311	**OK**

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AISC 14th Ed ASD Beam design

Location: L3

Span L=	**18.000**	**ft**				**Fy=**	**50 ksi**
Lb=	9	ft	= 108 in	(unbraced length)		E=	29000.0 ksi

Beam Size Provided: W16X45

Lp=	5.55 ft	MR=	180.5 k.ft
Lr=	16.46 ft	VR=	111.1 kips

Uniformly Distributed Loads

				(psf)	
Self weight					45 plf
roof	10.000 ft	DL @		70	700 plf
		LL @		20	200 plf
Wall ht	0 ft			0	0 plf
High roof	8 ft	DL @		20	160 plf
		LL @		20	160 plf
Wall ht	0 ft			0	0 plf
roof	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	0 ft			0	0 plf
Floor 4 trib	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	0 ft			0	0 plf
Floor 5 trib	0.000 ft	DL @		0	0 plf
		LL @		0	0 plf
Wall ht	15 ft			80	1200 plf
				w dead=	2105 plf
				w live=	360 plf
				w total=	2465 plf

Alternative uniform loads

w dead=	*0 plf*
w live=	*0 plf*
w total alt=	*0 plf*

Note:include self weight in dead load

Design uniform load w= 2.105 | 0.360 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment=	85.25	k.ft
Maximum LL Moment=	14.58	k.ft
Total Moment Mmax=	**99.83**	**k.ft**

	left	Right	
DL Reaction=	18.95	18.95	kips
LL Reaction=	3.24	3.24	kips
Total	22.19	22.19	kips

Max Shear Vmax= 22.19 kips

MR=	180.54 k.ft	**>Mmax**	**OK**
VR=	111.09 kips	**>Vmax**	**OK**

Live load deflection=	0.05 in	=L/	4317	**OK**
Total deflection=	0.34 in	=L/	630	**OK**

DG Structural Engineering LLC	CLIENT		SHEET NO.	
	PROJECT	Smyrna, IL Car Wash	DATE	3/14/2023
	SUBJECT		BY	AP

AISC 14th Ed ASD Beam design Location: L4

Span L=	12.000	ft				**Fy=**	**50 ksi**
Lb=	9	ft	= 108 in	(unbraced length)		E=	29000.0 ksi
Beam Size Provided:	W16X26		Lp=	3.96 ft		MR=	80.1 k.ft
			Lr=	11.17 ft		VR=	78.5 kips

Uniformly Distributed Loads

				(psf)		
Self weight					26	plf
roof	10.000	ft	DL @	70	700	plf
			LL @	20	200	plf
Wall ht	0	ft		0	0	plf
High roof	8	ft	DL @	20	160	plf
			LL @	20	160	plf
Wall ht	0	ft		0	0	plf
2ND FLOOR	8.000	ft	DL @	60	480	plf
			LL @	125	1000	plf
Wall ht	0	ft		0	0	plf
Floor 4 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	0	ft		0	0	plf
Floor 5 trib	0.000	ft	DL @	0	0	plf
			LL @	0	0	plf
Wall ht	15	ft		80	1200	plf
				w dead=	2566	plf
				w live=	1360	plf
				w total=	3926	plf

Alternative uniform loads

w dead=	*0*	*plf*
w live=	*0*	*plf*
w total alt=	*0*	plf

Note:include self weight in dead load

Design uniform load w= 2.566 | 1.360 k/ft

Other loads

	Dead loads	**Live loads**	
P50x1=	0.00	0.00	kips
Pax1=	0.00	0.00	kips
a=	0.00	0.00	ft *
P33x2=	0.00	0.00	kips
P25x3=	0.00	0.00	kips
Pax2=	0.00	0.00	k/ft
a=	0.00		ft *

* Note: a< L/2

Analysis Results

Maximum DL Moment= 46.19 k.ft
Maximum LL Moment= 24.48 k.ft
Total Moment Mmax= 70.67 k.ft

	left	Right	
DL Reaction=	15.40	15.40	kips
LL Reaction=	8.16	8.16	kips
Total	23.56	23.56	kips

Max Shear Vmax= 23.56 kips

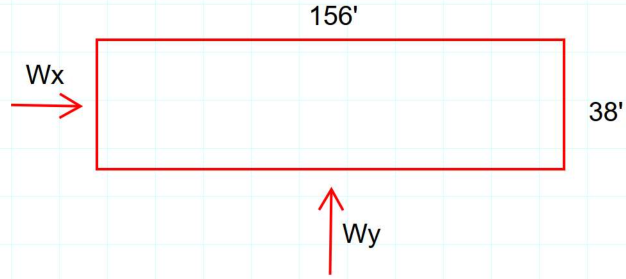
MR= 80.05 k.ft **>Mmax OK**
VR= 78.5 kips **>Vmax OK**

Live load deflection=	0.07	in	=L/	1981	**OK**
Total deflection=	0.21	in	=L/	686	**OK**

DG Structural Engineering LLC

CLIENT Neri Architects PROJECT NO. SHEET NO 12
PROJECT Car wash, Smyrna, TN PREP_BY AP DATE 3-14-2023
SUBJECT CHKD BY DATE

Wind speed, V= 115 MPH (ASCE 7-16)
qh = 20.16 psf
Net pressure, p = 15.93 psf Use 16 psf
Wx=16*9'*38' = 5.5 k
Wy=16*'9*156' = 22.5 k

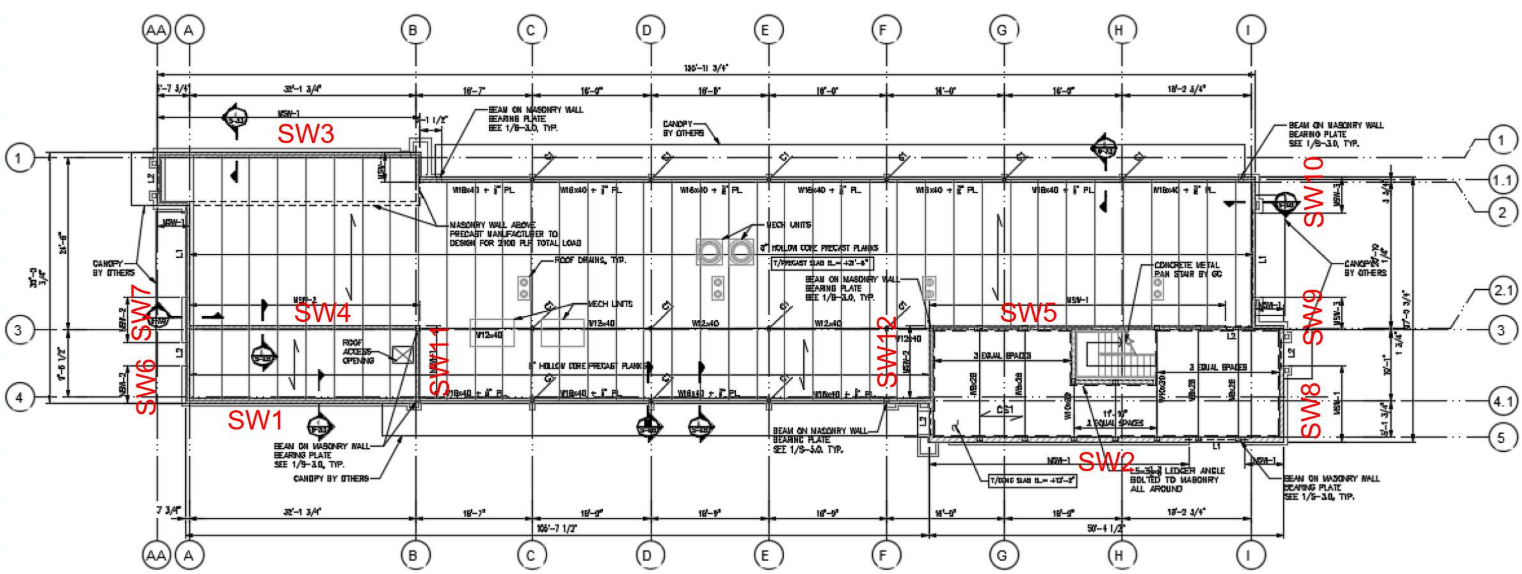


Seismic analysis
SS = 0.255
S1 = 0.129
SDS = 0.272
SD1 = 0.201
Site class D
Seismic design category D

Intermediate reinforced masonry shear walls:
Cs = 0.078

Weight of structure:
8" masonry walls: 80*20'*(156*2+38*2) + 60*(156*14') = 752 k
8" Precast roof: 60*4388 = 263.3 k
2nd floor: 50*731 = 36.7
High roof DL = 20*(15'*16')+(8*36) = 10.6 k
Total dead load = 1062.6 k
Base shear, Vs=0.078*1062.6 = 82.9 k
ASD Comb: V = 58 k
Seismic governs

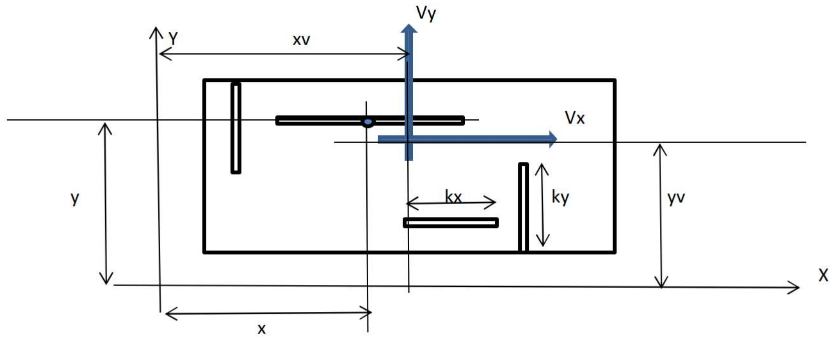
Masonry shear walls: Rigid diaphragm
8" Reinforced shear wall-typical
see below for shear wall location



DG Structural Engineering LLC PROJECT: Smyrna Car Wash
Architects: Neri 3/22/23 AP pg.

Rigid Diaphragm Stiffness Analysis

Units: Lb, Ft



Vx=	58	xv=	78	ex=	xv-xbar=	-16.1177	Vx.ey= 423.0265
Vy=	0	yv=	18	ey=	yv-ybar=	7.29356	Vy.ex= 0
							M= 423.0265
							Positive anti-clockwise

Direct forces

Wall /Frame #	x	y	kx	ky	x.ky	y.kx	Fxd	Fyd
1	16.67	0.5	15.54204	0	0	7.771018	9.200094	0
2	124	0.5	20.15151	0	0	10.07575	11.92867	0
3	14	35	17.23336	0	0	603.1675	10.20127	0
4	16.67	9.5	14.57586	0	0	138.4707	8.628166	0
5	127	9.5	30.47864	0	0	289.5471	18.04181	0
6	0.5	2.58	0	0.072277	0.036138	0	0	0
7	0.5	11.75	0	0.126492	0.063246	0	0	0
8	155.5	5.33	0	0.613845	95.45292	0	0	0
9	155.5	18	0	0.032013	4.977962	0	0	0
10	155.5	34.75	0	0.062455	9.711786	0	0	0
11	32.5	5.5	0	0.495061	16.08949	0	0	0
12	105.5	5.5	0	0.495061	52.22895	0	0	0
			97.9814	1.897204	178.5605	1049.032	58	0

xbar= 94.1177
ybar= 10.70644

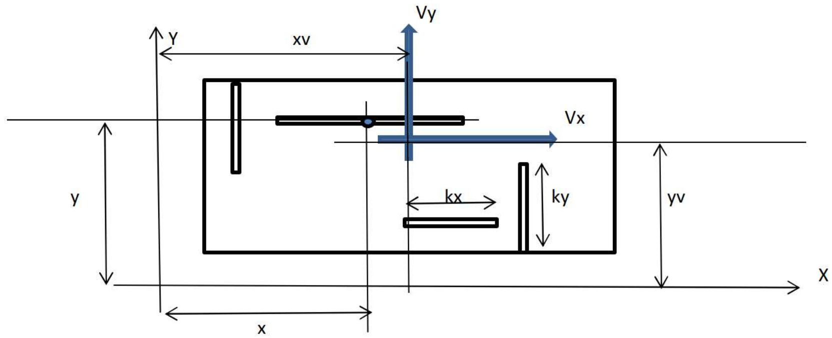
Eccentricity Forces — TOTAL FORCES

Wall/ Frame #	mx x-xbar	my y-ybar	ky.mx2	kx.my2	Fxm	Fym	Fx tot	Fy tot
1	-77.4477	-10.2064	0	1619.036	-3.30414	0	**5.895957**	**0**
2	29.8823	-10.2064	0	2099.211	-4.28408	0	**7.644585**	**0**
3	-80.1177	24.29356	0	10170.73	8.720408	0	**18.92168**	**0**
4	-77.4477	-1.20644	0	21.21514	-0.36628	0	**8.261884**	**0**
5	32.8823	-1.20644	0	44.36161	-0.76591	0	**17.2759**	**0**
6	-93.6177	-8.12644	633.4543	0	0	-0.14094	**0**	**-0.14094**
7	-93.6177	1.04356	1108.611	0	0	-0.24666	**0**	**-0.24666**
8	61.3823	-5.37644	2312.838	0	0	0.784834	**0**	**0.784834**
9	61.3823	7.29356	120.6167	0	0	0.04093	**0**	**0.04093**
10	61.3823	24.04356	235.3179	0	0	0.079852	**0**	**0.079852**
11	-61.6177	-5.20644	1879.619	0	0	-0.63539	**0**	**-0.63539**
12	11.3823	-5.20644	64.13852	0	0	0.117372	**0**	**0.117372**
			6354.595	13954.55	0.000	0.000	**58**	**0.000**
			S=	20309.15				

DG Structural Engineering LLC PROJECT: Smyrna Car Wash
Architects: Neri 3/22/23 AP pg.

Rigid Diaphragm Stiffness Analysis

Units: Lb, Ft



Vx=	0	xv=	78	ex=	xv-xbar=	-16.1177	Vx.ey= 0
Vy=	58	yv=	18	ey=	yv-ybar=	7.29356	Vy.ex= -934.827
							M= -934.827

Positive anti-clockwise

Wall /Frame #	x	y	kx	ky	x.ky	y.kx	Fxd (Direct forces)	Fyd (Direct forces)
1	16.67	0.5	15.54204	0	0	7.771018	0	0
2	124	0.5	20.15151	0	0	10.07575	0	0
3	14	35	17.23336	0	0	603.1675	0	0
4	16.67	9.5	14.57586	0	0	138.4707	0	0
5	127	9.5	30.47864	0	0	289.5471	0	0
6	0.5	2.58	0	0.072277	0.036138	0	0	2.209598
7	0.5	11.75	0	0.126492	0.063246	0	0	3.867027
8	155.5	5.33	0	0.613845	95.45292	0	0	18.76605
9	155.5	18	0	0.032013	4.977962	0	0	0.978667
10	155.5	34.75	0	0.062455	9.711786	0	0	1.909337
11	32.5	5.5	0	0.495061	16.08949	0	0	15.13466
12	105.5	5.5	0	0.495061	52.22895	0	0	15.13466
			97.9814	1.897204	178.5605	1049.032	0	58

xbar= 94.1177
ybar= 10.70644

Wall/ Frame #	mx x-xbar	my y-ybar	ky.mx2	kx.my2	Fxm (Eccentricity Forces)	Fym (Eccentricity Forces)	Fx tot (TOTAL FORCES)	Fy tot (TOTAL FORCES)
1	-77.4477	-10.2064	0	1619.036	7.301658	0	**7.301658**	**0**
2	29.8823	-10.2064	0	2099.211	9.46719	0	**9.46719**	**0**
3	-80.1177	24.29356	0	10170.73	-19.2708	0	**-19.2708**	**0**
4	-77.4477	-1.20644	0	21.21514	0.80943	0	**0.80943**	**0**
5	32.8823	-1.20644	0	44.36161	1.692547	0	**1.692547**	**0**
6	-93.6177	-8.12644	633.4543	0	0	0.311456	**0**	**2.521054**
7	-93.6177	1.04356	1108.611	0	0	0.54508	**0**	**4.412107**
8	61.3823	-5.37644	2312.838	0	0	-1.73437	**0**	**17.03168**
9	61.3823	7.29356	120.6167	0	0	-0.09045	**0**	**0.888218**
10	61.3823	24.04356	235.3179	0	0	-0.17646	**0**	**1.732875**
11	-61.6177	-5.20644	1879.619	0	0	1.404118	**0**	**16.53878**
12	11.3823	-5.20644	64.13852	0	0	-0.25938	**0**	**14.87529**
			6354.595	13954.55	0.000	0.000	**2.66E-15**	**58.000**

S= 20309.15

8" Masonry shear walls:
SW1, SW2, SW3, SW4 & SW5: L = 32'-0" (MSW1 on dwg)
Shear force, V = 20 k
Moment, M = 400 k-ft

SW6, SW7, SW9 & SW10: Length = 4'-0" (MSW2 on dwg)
Shear force, V = 5 k
Moment, M = 100 k-ft

SW8, SW11 & SW12: Length = 10'-0" (MSW3 on dwg)
Shear force, V = 17 k
Moment, M = 340 k-ft

DG STRUCTURAL ENGINEERING LLC
401 S LASALLE ST, STE 1305, CHICAGO, IL 60605

Project #	Neri	page	
	Smryna Car Wash	by	AP

MASONRY PIER / SHEAR WALL Location: SW1

fm'=	2500	psi					
Rebar	0.62	in2					
M=	400	k.ft		Note: Use ASCE combination			
P=	1	kips					
V=	20	kips					
b=	2.5	in		b for shear=	7.625	in	Use 2.5" for hollow
d full=	384	in	Full depth				Use b for fully grouted
d eff=	380	in	Effective depth				
ro=	0.000653						
m=	13						
m.ro=	0.008484			s=	0.004614	ksi	(s=P/b.d.fc)
m.ro-s=	0.003871						
n=	0.126			M+P.(d-df/2)=	415.67	k.ft	
j=	0.958			M-P.(df/2-nd/3)=	385.33	k.ft	

Concrete stress:

fc=	**0.228**	**ksi**	fm'/3=	0.833	ksi	**OK**

Steel stress:

fs=	**20.490**	**ksi**	fall=	24	ksi	**OK**

Shear check:

Allowable shear stress: Min of			Use	60.94	psi
a) 1.5 sqrt (fm')=	75.00	psi			
b) 120psi	120.00	psi			
e) 60psi + 0.45 Nv/An=	60.94	psi	Note: Partially grouted pier assumed		

Maximum shear stress = 1.5V/(3"xdeff)= 10.35 psi **OK**

Comments: Provide (2)-#5 at ends

DG STRUCTURAL ENGINEERING LLC
401 S LASALLE ST, STE 1305, CHICAGO, IL 60605

Project # Neri
Smryna Car Wash

page
by AP

MASONRY PIER / SHEAR WALL

Location: SW6

fm'=	2500	psi
Rebar	1.24	in2

M=	100	k.ft	Note: Use ASCE combination
P=	1	kips	
V=	5	kips	

b=	7.625	in		b for shear=	7.625	in	Use 2.5" for hollow
d full=	48	in	Full depth				Use b for fully grouted
d eff=	44	in	Effective depth				

ro=	0.003696				
m=	13				
m.ro=	0.048048	s=	0.004422	ksi	(s=P/b.d.fc)
m.ro-s=	0.043626				

n=	0.269	M+P.(d-df/2)=	101.67	k.ft
j=	0.910	M-P.(df/2-nd/3)=	98.33	k.ft

Concrete stress:

fc=	**0.674**	**ksi**	fm'/3=	0.833	ksi	**OK**

Steel stress:

fs=	**23.761**	**ksi**	fall=	24	ksi	**OK**

Shear check:

Allowable shear stress: Min of			Use 62.46 psi
a) 1.5 sqrt (fm')=	75.00	psi	
b) 120psi	120.00	psi	
e) 60psi + 0.45 Nv/An=	62.46	psi	Note: Partially grouted pier assumed

Maximum shear stress = 1.5V/(3"xdeff)= 22.35 psi **OK**

Comments: Provide (4)-#5 at ends

DG STRUCTURAL ENGINEERING LLC
401 S LASALLE ST, STE 1305, CHICAGO, IL 60605

Project # Neri — Smryna Car Wash — page — by AP

MASONRY PIER / SHEAR WALL Location: SW8

fm'=	2500	psi						
Rebar	1.76	in2						
M=	340	k.ft	Note: Use ASCE combination					
P=	1	kips						
V=	17	kips						
b=	7.625	in		b for shear=	7.625	in	Use 2.5" for hollow	
d full=	120	in	Full depth				Use b for fully grouted	
d eff=	116	in	Effective depth					
ro=	0.00199							
m=	13							
m.ro=	0.025868			s=	0.002684	ksi	(s=P/b.d.fc)	
m.ro-s=	0.023184							
n=	0.205			M+P.(d-df/2)=	344.67	k.ft		
j=	0.932			M-P.(df/2-nd/3)=	335.66	k.ft		

Concrete stress:

fc=	**0.421**	**ksi**	fm'/3=	0.833	ksi	**OK**

Steel stress:

fs=	**21.180**	**ksi**	fall=	24	ksi	**OK**

Shear check:

Allowable shear stress: Min of			Use 60.98	psi
a) 1.5 sqrt (fm')=	75.00	psi		
b) 120psi	120.00	psi		
e) 60psi + 0.45 Nv/An=	60.98	psi	Note: Partially grouted pier assumed	

Maximum shear stress = 1.5V/(3"xdeff)= 28.83 psi **OK**

Comments: Provide (4)-#6 at ends

DG Structural Engineering LLC

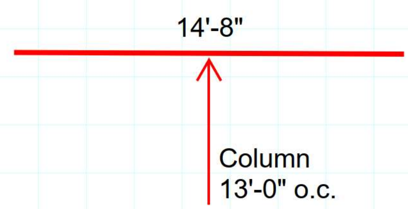
CLIENT Neri Architect PROJECT NO. SHEET NO 19
PROJECT PREP_BY AP DATE 3-8-2023
SUBJECT Car Wash Canopies CHKD BY DATE

CANOPY DESIGN:
Wind Speed = 115 mph
Exposure C
Canopies: (ASCE 7)
Kh = 0.7
qz = 0.00256*.9*0.85*115^2 = 25.9 psf
Gcp = 0.6 (upper surface & lower surface)
Gcp = -0.6 (upper surface)
Gcp = -0.6 (lower surface)
Design wind pressure, p = 1.2*26 = 31.2 psf

Dead load = 15 psf
Live load = 20 psf
Snow load = 25 psf

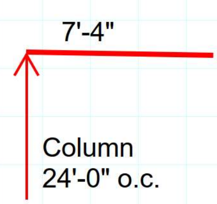
Pay station canopy:
Main beam: Span = 14'-8"
Load = 40*13' = 520 plf
M = 19.5 k-ft
HSS6x6x1/4, Mr = 22 k-ft
Def = 0.61"

Purlins: HSS5x5x1/2
Span = 13'-0", Spacing = 4'-0" max.
Load = 40*4' = 160 plf
M = 7.33 k-ft, Mr = 21.8 k-ft
Def = 0.3"



Column: HSS6x6x3/8
Ht = 10'
P = 9 k
H = 30*3'*13' = 1170 lbs
fa = 1.2 ksi
Eq H1-3 = 0.4
Def = 0.603"

Vacuum station canopy:
Main Beam: HSS6x6x1/4:
Span = 8'-0" max, Spacing = 24'-0" max
Load = 40*24' = 960 plf
M = 32.8 k-ft, Mr = 33.4 k-ft
R = 8.3 k
Def = 0.8"



Purlins: HSS5x5x1/2:
Span = 24'-0", Spacing = 4'-0" max.
M = 13.5 k-ft, Mr = 21.8 k-ft
Def = 1.9" (L/155)

Column: HSS6x6x3/8
Ht = 10'
P = 16.4 k
H = 30*3'*24' = 2160 lbs
fa = 2.2 ksi
Eq H1-3 = 0.7
Def = 1.1"

DG Structural Engineering LLC
SPREAD FOOTING DESIGN

Neri Architect
Car Wash - Canopy footing

DATE: 16-Mar-23
TIME: 02:49:04 PM
PROJ. REF:
DESIGNER: AP

LOADS :

	DEAD	LIVE	WIND	
COLUMN / PIER AXIAL LOAD	4.00	5.00	7.70	kips
COLUMN / PIER MOMENT	0.00	0.00	13.00	k.ft
COLUMN / PIER SHEAR	0.00	0.00	2.00	kips

SOIL DENSITY 120.0 pcf
CONCRETE DENSITY 150.0 pcf

GEOMETRY :

LENGTH OF FOOTING a 5.000 ft
WIDTH OF FOOTING b 5.000 ft
T/PIER EL (-ve for below floor) 0.000 ft
THICKNESS OF FOOTING 1.000 ft
DEPTH TO BTM OF FOOTING 4.000 ft
LENGTH OF COLUMN B.PL./ PIER 1.330 ft
WIDTH OF COLUMN B.PL./ PIER 1.330 ft

Dead Loads:

Footing Self Weight 3.8 kips
Pier Self Weight 0.8 kips
Soil Over Footing 8.4 kips
Column / Pier Dead Load 4.0 kips **TOTAL DL** **16.9** kips

	DEAD	LIVE	WIND	
BASE MOMENT DUE TO SHEAR	0.00	0.00	8.00	k.ft.
TOTAL MOMENT	0.00	0.00	21.00	k.ft.

BEARING PRESSURES & OVERTURNING :

	D+L	D+0.75L+0.75W	D+W	
P	21.91	26.43	24.61	
M	0.00	15.75	21.00	
DL Restoring Moment =	54.8	66.1	61.5	
Overturning FOS	N.A.	4.2	2.9	**>1.5 : OK**

BEARING PRESSURES :

ASCE Combinations	**Minimum** **psf**	**Length Acting** **ft**	**Maximum** **psf**
Dead Load + Live Load	**876.37**	**5.00**	**876**
Dead Load + 0.75Live Load + 0.75Wind Load	**301.37**	**5.00**	**1813**
Dead Load + Wind Load	**0.00**	**4.94**	**1993**

Net SBC assumed= 2000 psf
Gross allowable pressure= 2480 psf >fmax OK

DESIGN DATA :

CONCRETE STRENGTH 3.00 ksi
REINF'T YIELD STRENGTH 60.00 ksi
REINF'T EFFECTIVE DEPTH 8.25 in

ANALYSIS :

WORKING LOAD D+L 21.91
ULTIMATE LOAD. 28.29
AVG LOAD FACTOR 1.29

FLEXURE
DESIGN FOR UNIFORM PRESSURE = 1.1 fma 1712 psf
FACTORED DESIGN MOMENT 18.61 k.ft

STEEL AREA REQUIRED 0.56 in2
0.18% REINFORCEMENT 0.89 in2
STEEL AREA PROVIDED **1.53 in2** **O.K.** Bar size #: 5
No of bars: 5

BEAM SHEAR
@'d' FROM COLUMN FACE
phi.Vc= 40.67 kips
DESIGN SHEAR Vu 12.68 kips **O.K.**

PUNCHING SHEAR
phi.Vc= 131.28 kips
DESIGN SHEAR Vu 46.27 kips **O.K.**

COMMENTS