SHUTLER CONSULTING ENGINEERS, Inc.

12503 Bel-Red Road, Suite 100
Bellevue, Washington 98005
(425) 450-4075 FAX (425) 450-4076

JOB Freeman Logistics Building B
SHEET NO. COVER OF
CALCULATED BY DV DATE 1-31-2024
CHECKED BY DATE
SCALE
JOB NUMBER 21-41

STRUCTURAL CALCULATIONS FOR:

FREEMAN ROAD LOGISTICS BUILDING B
PUYALLUP, WA 98371

PROPOSED BY:

VECTOR DEVELOPMENT COMPANY
KIRKLAND, WA

ARCHITECT:

SYNTHESIS PLLC
12503 BEL-RED ROAD, SUITE 100
BELLEVUE, WA 98005
(425) 646-1818

JOHN C. HEADLAND
STATE OF WASHINGTON
31757
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER
1-31-24

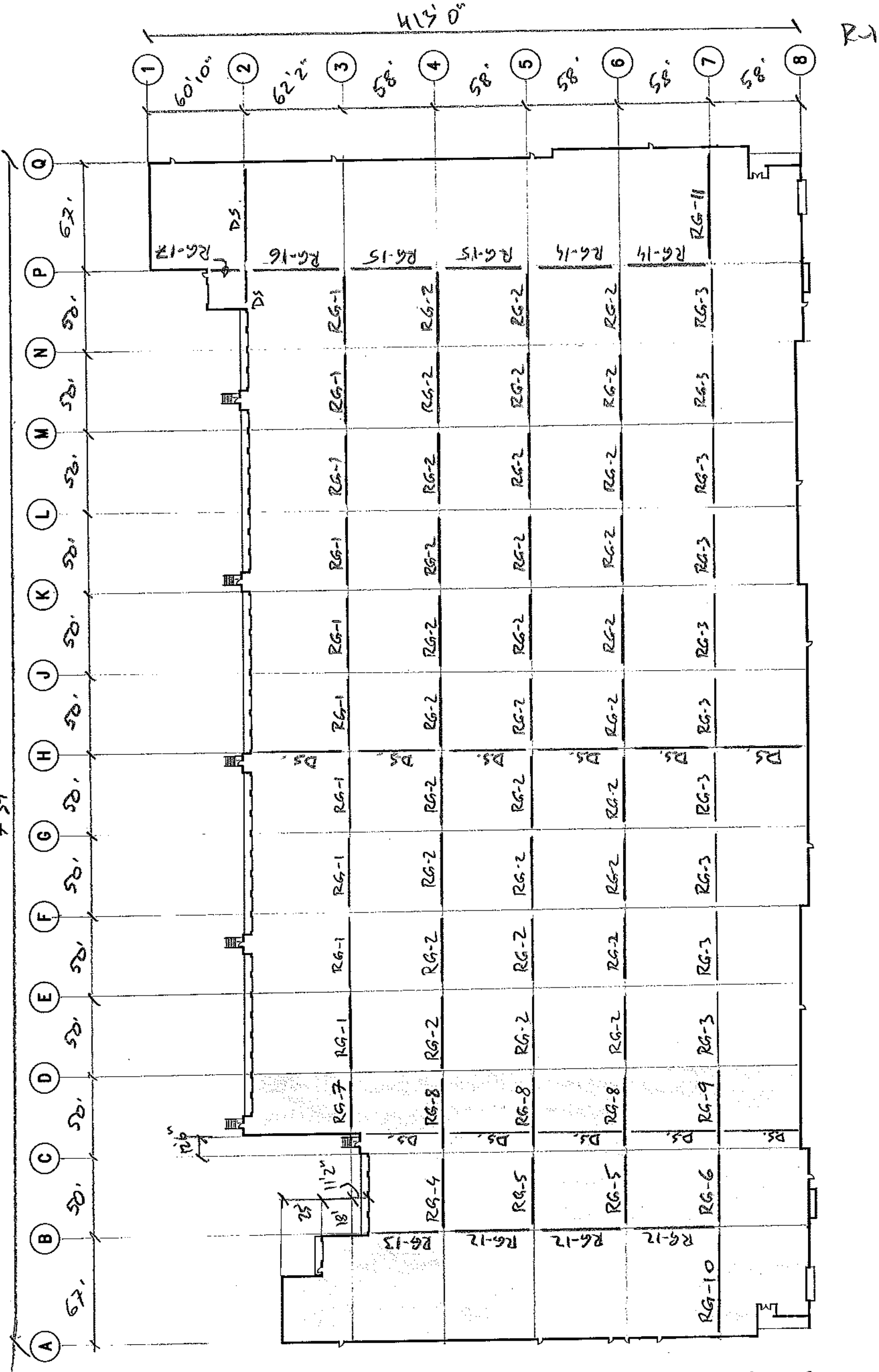
DESIGN CRITERIA:

CODE.................................. INTERNATIONAL BUILDING CODE, 2018 EDITION
ROOF LIVE LOAD.................. 25 PSF SNOW LOAD, Is = 1.0
WIND LOAD.......................... 95 MPH ZONE, EXPOSURE 'B', Iw = 1.0
SEISMIC DESIGN INFORMATION:

$S_s = 1.288$	$S_{ds} = 0.858$
$S_1 = 0.443$	$S_{d1} = 0.546$
$C_s = 0.172$	$I_e = 1.0$
$R = 5.0$	$\Omega_0 = 2.0$

SITE SOIL CLASS = F
DESIGN SITE SOIL CLASS PER GEOTECH. REPORT = D
SEISMIC DESIGN CATEGORY = "D"

FOUNDATION DESIGN PER GEOTECHNICAL REPORT #T-8565 DATED AUGUST 11, 2021, BY TERRA ASSOCIATES, INC. ALLOWABLE SOIL BEARING IS 2500 PSF. ALL FOUNDATION WORK TO BE IN ACCORDANCE WITH THIS REPORT.



K:\21\21-41 Freeman Logistics - Bldg B\0DWG\ARCH\23-8-9 pdf & cad\Vector-Freeman Rd Logistics South - Standard\xrf\B-A1.dwg, Model, 8/26/2023 12:15:33 PM, Dmitri, 1:1000

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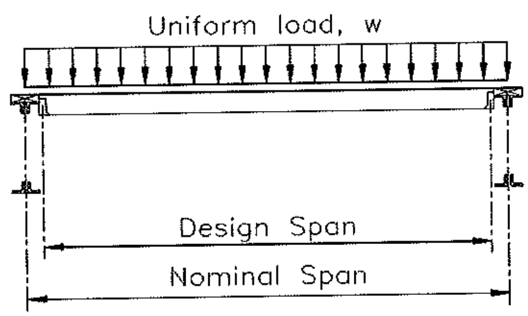
Roof Dead Load Tabulation

	Stiffener		Purlin		Girder		Dead Load to resist uplift	
Roofing (TPO)	0.30	PSF	0.30	PSF	0.30	PSF	0.30	PSF
DensDeck	2.00	PSF	2.00	PSF	2.00	PSF	0.60	PSF
Plywood	1.50	PSF	1.50	PSF	1.50	PSF	1.50	PSF
Insulation	1.75	PSF	1.75	PSF	1.75	PSF	1.00	PSF
Stiffener	1.10	PSF	1.10	PSF	1.10	PSF	1.10	PSF
Sprinkler	0.00	PSF	2.50	PSF	2.50	PSF	1.00	PSF
Purlin	0.00	PSF	2.70	PSF	2.70	PSF	1.50	PSF
Ceiling	1.55	PSF	1.35	PSF	1.35	PSF	0.00	PSF
Girder	0.00	PSF	0.00	PSF	2.00	PSF	2.00	PSF
Misc.& Mech	1.80	PSF	1.80	PSF	1.80	PSF	0.00	PSF
Solar Readiness	5.00		5.00		5.00		0.00	
Colateral*	0.00	PSF	0.00	PSF	0.00	PSF	0.00	PSF
Total	15.00	PSF	20.00	PSF	22.00	PSF	9.00	PSF

Rigid Insulation 0.24 PSF per inch, R-5.6 per inch, using 7.0 inches equals R 39.2
TPO Roofing 60 mil membrane - 0.30 PSF, 45 mil membrane - 0.21 PSF
DensDeck protection board, 1/4" thick - 1.20 PSF, 1/2" thick - 2.00 PSF, 5/8" thick - 2.50 PSF

ROOF LIVE LOAD 25 PSF **SNOW LOAD**

Stiffener design



Nominal Span	10.00	Feet
Design Span	9.63	Feet
Spacing	24	inches on center
Uniform Dead Load	30.00	PLF
Uniform Live Load	50.00	PLF
Uniform Total Load	80.00	PLF
Reaction	385.00	Lbs.
Moment	926.41	ft.-lbs.
Member size	2x6	DF #2
Width	1.50	in.
Depth	5.50	in.
A	8.25	in.^2
S	7.56	in.^3
I	20.80	in.^4

Shear stress	63	psi	<	180	psi	*	1.32	=	238	psi
Bending stress	1470	psi	<	900	psi	*	1.72	=	1548	psi
Total load deflection	0.46	in.	==>	span /	249	Based on E =	1600	ksi.		

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JOB
SHEET NO. R-3 OF
CALCULATED BY
DATE
SCALE

JOISTS

DL = 15psf
20psf, SOLAR

SL = 25psf

L	DEPTH	TL	LL	NOMENCLATURE	W_e
58'	24/56	400	250	36LH09 - 411/370	18.3 plf
58' SOLAR	24/56	450	250	36LH10 - 454/409	19.9
62'2"	40	400	250	40LH10 - 419/391	20
67'0"	24/56	400	250	40LH12 - 472/409	25
72'0"	24/56	400	250	40LH12 - 413/333	25
53'0"	24/56	400	250	40LH09 - 472/570	21
67'0" SOLAR	24/56	450	250	40LH12 - 472/409	25
42'0" SOLAR	36	450	250	36LH07 - 475/703	13.5

SOLAR

A = 2000
V = 480

$W = 2000(480)\sqrt{3} = 1662769$ WATTS

INCREASE FOR WALKWAYS & OTHER EQUIPMENT

$A = \frac{1662769(0.2)}{10} = 33{,}256$ psf $\times 1.15 = 38{,}244$ ft²

OK

$A_{TOT, 40\%} = 254{,}610(0.4) = 101{,}844$ ft²

$A_{PROVIDED} = (58' \times 3)(50' \times 5) = 43500$ ft²
$= 43{,}500$ ft² $> 38{,}244$ ft²

OK

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JOB Freeman Logistics B
SHEET No. R- 4 OF
CALCULATED BY: DV
DATE: 8-27-23
SCALE:

Girder Reactions & Sizes

Mark number	RG-1	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	60	10.00	10.00	9.81	15.02	24.83
P2	60	10.00	20.00	9.81	15.02	24.83
P3	60	10.00	30.00	9.81	15.02	24.83
P4	60	10.00	40.00	9.81	15.02	24.83
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	19.62	30.04	0.00	0.00	19.62	30.04	49.67
Right	19.62	30.04	0.00	0.00	19.62	30.04	49.67

Girder size

60 G5N 24.8K
Weight 60 PLF

Weight PLF

Weight PLF

Weight

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JOB Freeman Logistics B
SHEET No. R- 5 OF
CALCULATED BY: DV
DATE: 8-27-23
SCALE:

Girder Reactions & Sizes

Mark number	RG-~~6~~ 2	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf

Tributary width for uniform loads 0.00 ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58	10.00	10.00	12.40	14.50	26.90
P2	58	10.00	20.00	12.40	14.50	26.90
P3	58	10.00	30.00	12.40	14.50	26.90
P4	58	10.00	40.00	12.40	14.50	26.90
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	24.80	29.00	0.00	0.00	24.80	29.00	53.80
Right	24.80	29.00	0.00	0.00	24.80	29.00	53.80

Girder size

54/60 G5N 26.9K
Weight 70 PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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SHEET No. R- 6 OF
CALCULATED BY: DV
DATE: 8-27-23
SCALE:

Girder Reactions & Sizes

Mark number	RG-3	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58	10.00	10.00	9.50	14.50	24.00
P2	58	10.00	20.00	9.50	14.50	24.00
P3	58	10.00	30.00	9.50	14.50	24.00
P4	58	10.00	40.00	9.50	14.50	24.00
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	19.00	29.00	0.00	0.00	19.00	29.00	48.00
Right	19.00	29.00	0.00	0.00	19.00	29.00	48.00

Girder size

54 G5N 24.0K

Weight 63 PLF

Weight PLF

Weight PLF

Weight

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SHEET No. R- 7 OF

CALCULATED BY: DV

DATE: 8-27-23

SCALE:

Girder Reactions & Sizes

Mark number	RG-~~7~~ 4	
Span	37.50	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	60	8.75	7.50	11.21	13.14	24.36
P2	60	10.00	17.50	12.82	15.02	27.84
P3	60	10.00	27.50	12.82	15.02	27.84
P4				0.00	0.00	0.00
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	19.22	22.53	0.00	0.00	19.22	22.53	41.76
Right	17.62	20.65	0.00	0.00	17.62	20.65	38.28

Girder size

40 G4N 27.8K
Weight 55 PLF

Weight PLF

Weight PLF

Weight

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SHEET No. R- 8 OF
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SCALE:

Girder Reactions & Sizes

Mark number	RG-~~8~~ 5	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58	6.25	10.00	7.75	9.06	16.81
P2	58	5.00	12.50	6.20	7.25	13.45
P3	58	8.75	20.00	10.85	12.69	23.54
P4	58	10.00	30.00	12.40	14.50	26.90
P5	58	10.00	40.00	12.40	14.50	26.90
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	24.80	29.00	0.00	0.00	24.80	29.00	53.80
Right	24.80	29.00	0.00	0.00	24.80	29.00	53.80

Girder size

60 G6N SPECIAL
Weight 65 PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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JOB Freeman Logistics B
SHEET No. R- 9 OF
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SCALE:

Girder Reactions & Sizes

Mark number	RG-~~0~~ 6	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58	6.25	10.00	7.75	9.06	16.81
P2	58	5.00	12.50	6.20	7.25	13.45
P3	58	8.75	20.00	10.85	12.69	23.54
P4	58	10.00	30.00	12.40	14.50	26.90
P5	58	10.00	40.00	12.40	14.50	26.90
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	24.80	29.00	0.00	0.00	24.80	29.00	53.80
Right	24.80	29.00	0.00	0.00	24.80	29.00	53.80

Girder size

54 G6N SPECIAL

Weight 73 PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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SHEET No. R- 10 OF
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SCALE:

Girder Reactions & Sizes

Mark number	RG-~~10~~ 7	
Span	67.00	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	4.00	ft.

Uniform dead load	160.00	plf
Uniform live load	100.00	plf
Total uniform load	260.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	13	8.50	7.00	2.13	2.66	4.78
P2	29	10.00	17.00	5.80	7.25	13.05
P3	29	10.00	27.00	5.80	7.25	13.05
P4	29	10.00	37.00	5.80	7.25	13.05
P5	29	10.00	47.00	5.80	7.25	13.05
P6	29	10.00	57.00	5.80	7.25	13.05
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	14.89	18.61	5.36	3.35	20.25	21.96	42.21
Right	16.24	20.30	5.36	3.35	21.60	23.65	45.24

Girder size

60 G7N SPECIAL
Weight 60 PLF

Weight PLF

Weight PLF

Weight

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SHEET No. R- 11 OF
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SCALE:

Girder Reactions & Sizes

Mark number	RG-~~11~~ 8	
Span	72.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	29	6.00	2.00	2.61	4.35	6.96
P2	29	10.00	12.00	4.35	7.25	11.60
P3	29	10.00	22.00	4.35	7.25	11.60
P4	29	10.00	32.00	4.35	7.25	11.60
P5	29	10.00	42.00	4.35	7.25	11.60
P6	29	10.00	52.00	4.35	7.25	11.60
P7	13	10.00	62.00	1.88	3.13	5.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	14.88	24.80	5.58	4.50	20.46	29.30	49.76
Right	11.35	18.92	5.58	4.50	16.93	23.42	40.36

Girder size

60 G8N SPECIAL
Weight 65 PLF

Weight PLF

Weight PLF

Weight

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SHEET No. R- 12 OF
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Girder Reactions & Sizes

Mark number	RG-~~14~~ 9	
Span	58.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	36	10.00	10.00	5.40	9.00	14.40
P2	36	10.00	20.00	5.40	9.00	14.40
P3	36	10.00	30.00	5.40	9.00	14.40
P4	36	10.00	40.00	5.40	9.00	14.40
P5	36	9.00	50.00	4.86	8.10	12.96
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	12.96	21.60	4.50	3.63	17.46	25.23	42.68
Right	13.50	22.50	4.50	3.63	18.00	26.13	44.12

Girder size

54 G6N 14.4K

Weight 60 PLF

Weight PLF

Weight PLF

Weight

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JOB Freeman Logistics B
SHEET No. R- 13 OF
CALCULATED BY: DV
DATE: 8-27-23
SCALE:

Girder Reactions & Sizes

Mark number	RG-~~13~~ 10	
Span	46.83	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	180.00	plf
Uniform live load	125.00	plf
Total uniform load	305.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	34	10.00	10.00	6.70	8.38	15.08
P2	34	10.00	20.00	6.70	8.38	15.08
P3	34	10.00	30.00	6.70	8.38	15.08
P4	34	8.42	40.00	5.64	7.05	12.69
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	12.34	15.42	4.21	2.93	16.55	18.35	34.91
Right	13.40	16.75	4.21	2.93	17.62	19.68	37.29

Girder size

48 G5N SPECIAL

Weight 67 PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

SHUTLER
CONSULTING
ENGINEERS, INC.

12503 Bel-Red Road, Suite 100
Bellevue, WA 98005
(425) 450-4075 -- FAX (425) 450-4076

JOB Freeman Logistics B
SHEET No. R- 14 OF
CALCULATED BY: DV
DATE: 8-27-23
SCALE:

Girder Reactions & Sizes

Mark number	RG-~~18~~ 11	
Span	62.17	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.
Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	34	7.50	5.00	3.77	6.28	10.05
P2	34	10.00	15.00	5.03	8.38	13.40
P3	34	10.00	25.00	5.03	8.38	13.40
P4	34	10.00	35.00	5.03	8.38	13.40
P5	34	10.00	45.00	5.03	8.38	13.40
P6	34	8.43	55.00	4.24	7.06	11.30
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	14.35	23.92	4.82	3.89	19.17	27.81	46.98
Right	13.75	22.92	4.82	3.89	18.57	26.80	45.37

Girder size

60 G7N SPECIAL
Weight 70 PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

Shutler Consulting Engineers, Inc.
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Project Title:
Engineer:
Project ID:
Project Descr: R-15

Printed: 30 JAN 2024, 2:54PM

Project File: ENERCALC_20

Steel Column

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Entry Beam - AXIAL

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS6x6x1/4**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 20 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 20 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 20 ft, K = 1.0

Applied Loads

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

AXIAL LOADS . . .
Axial Load at 20.0 ft, E = 31.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.5784** : 1	
Load Combination	E Only * 2.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pu	63.20	k
0.9 * Pn	109.271	k
Mu-x	0.0	k-ft
0.9 * Mn-x :	42.0	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	42.0	k-ft
PASS Maximum Shear Stress Rati	**0.0** : 1	
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.0	k
Vn * Phi : Allowable	0.0	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
	0.000	PASS	0.00 ft	0.00	0.00	0.00	0.00	0.000	PASS	0.00 ft
E Only * 2.0	0.578	PASS	0.00 ft	1.00	1.00	102.56	102.56	0.000	PASS	0.00 ft

Steel Beam

DESCRIPTION: Entry Beam – GRAVITY

CODE REFERENCES

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

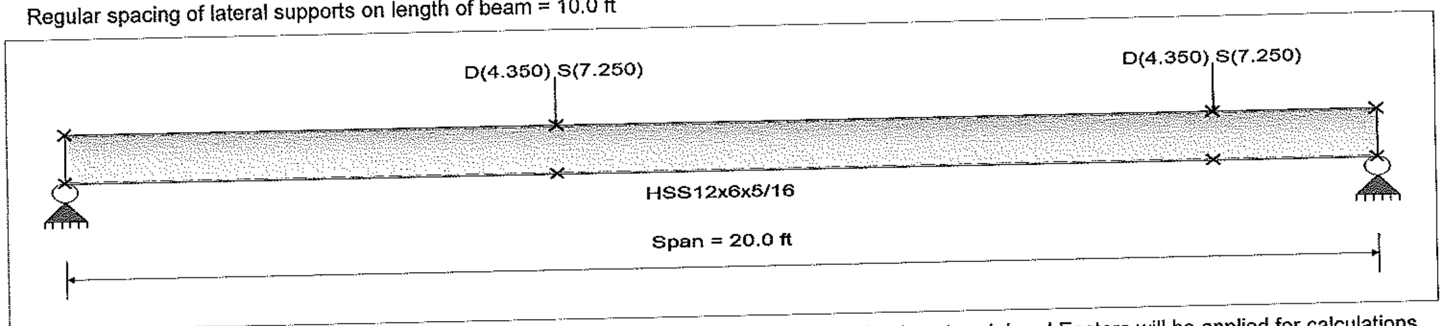
Material Properties

Analysis Method :Load Resistance Factor Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 7.50 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 10.0 ft



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 4.350, S = 7.250 k @ 7.50 ft, (Joist 1)

Point Load : D = 4.350, S = 7.250 k @ 17.50 ft, (Joist 2)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.661** : 1	Maximum Shear Stress Ratio =	**0.120** : 1
Section used for this span	**HSS12x6x5/16**	Section used for this span	**HSS12x6x5/16**
Mu : Applied	94.432 k-ft	Vu : Applied	21.025 k
Mn * Phi : Allowable	142.875 k-ft	Vn * Phi : Allowable	174.850 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	17.543 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.503 in	Ratio =	476	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.806 in	Ratio =	298	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 7.49 ft	1	0.239	0.026	34.19		34.19	158.75	142.88	1.68	1.00	4.57	194.28	174.85
Dsgn. L = 10.00 ft	1	0.239	0.026	34.19	19.05	34.19	158.75	142.88	1.21	1.00	4.57	194.28	174.85
Dsgn. L = 2.51 ft	1	0.133	0.044	19.05		19.05	158.75	142.88	1.66	1.00	7.61	194.28	174.85
+1.20D													
Dsgn. L = 7.49 ft	1	0.205	0.022	29.31		29.31	158.75	142.88	1.68	1.00	3.92	194.28	174.85
Dsgn. L = 10.00 ft	1	0.205	0.022	29.31	16.33	29.31	158.75	142.88	1.21	1.00	3.92	194.28	174.85
Dsgn. L = 2.51 ft	1	0.114	0.037	16.33		16.33	158.75	142.88	1.66	1.00	6.53	194.28	174.85
+1.20D+0.50S													
Dsgn. L = 7.49 ft	1	0.348	0.038	49.66		49.66	158.75	142.88	1.68	1.00	6.63	194.28	174.85
Dsgn. L = 10.00 ft	1	0.348	0.038	49.66	27.67	49.66	158.75	142.88	1.21	1.00	6.63	194.28	174.85
Dsgn. L = 2.51 ft	1	0.194	0.063	27.67		27.67	158.75	142.88	1.66	1.00	11.06	194.28	174.85
+1.20D+1.60S													
Dsgn. L = 7.49 ft	1	0.661	0.072	94.43		94.43	158.75	142.88	1.68	1.00	12.62	194.28	174.85
Dsgn. L = 10.00 ft	1	0.661	0.072	94.43	52.62	94.43	158.75	142.88	1.21	1.00	12.62	194.28	174.85

DESCRIPTION: Entry Beam

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 2.51 ft	1	0.368	0.120	52.62		52.62	158.75	142.88	1.66	1.00	21.03	194.28	174.85
+1.372D+0.70S													
Dsgn. L = 7.49 ft	1	0.434	0.047	61.99		61.99	158.75	142.88	1.68	1.00	8.28	194.28	174.85
Dsgn. L = 10.00 ft	1	0.434	0.047	61.99	34.54	61.99	158.75	142.88	1.21	1.00	8.28	194.28	174.85
Dsgn. L = 2.51 ft	1	0.242	0.079	34.54		34.54	158.75	142.88	1.66	1.00	13.80	194.28	174.85
+0.90D													
Dsgn. L = 7.49 ft	1	0.154	0.017	21.98		21.98	158.75	142.88	1.68	1.00	2.94	194.28	174.85
Dsgn. L = 10.00 ft	1	0.154	0.017	21.98	12.25	21.98	158.75	142.88	1.21	1.00	2.94	194.28	174.85
Dsgn. L = 2.51 ft	1	0.086	0.028	12.25		12.25	158.75	142.88	1.66	1.00	4.89	194.28	174.85
+0.7284D													
Dsgn. L = 7.49 ft	1	0.125	0.014	17.79		17.79	158.75	142.88	1.68	1.00	2.38	194.28	174.85
Dsgn. L = 10.00 ft	1	0.125	0.014	17.79	9.91	17.79	158.75	142.88	1.21	1.00	2.38	194.28	174.85
Dsgn. L = 2.51 ft	1	0.069	0.023	9.91		9.91	158.75	142.88	1.66	1.00	3.96	194.28	174.85

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Project Title:
Engineer:
Project ID:
Project Descr:

R-18

Printed: 30 JAN 2024, 4:03PM

Project File: Columns and Drag Struts.ec6

Steel Column

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Drag Strut @ Shear Wall

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **W24x76**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 51.833 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 51.833 ft, K = 1.0

Applied Loads

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

AXIAL LOADS . . .
Axial Load at 51.833 ft, E = 301.70 k
BENDING LOADS . . .
Roof: Lat. Uniform Load creating Mx-x, D = 0.0750, S = 0.1250 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9280** : 1	
Load Combination	+1.372D+0.70S+2.0E	
Location of max.above base	25.743	ft
At maximum location values are . . .		
Pu	603.40	k
0.9 * Pn	708.0	k
Mu-x	63.930	k-ft
0.9 * Mn-x :	750.0	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	107.250	k-ft
PASS Maximum Shear Stress Rati	**0.02382** : 1	
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	7.516	k
Vn * Phi : Allowable	315.480	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	5.183 k
Bottom along Y-Y	5.183 k

Maximum Load Deflections . . .

Along Y-Y 0.5391 in at 26.090ft above base
for load combination :+D+S

Along X-X 0.0 in at 0.0ft above base
for load combination :

Kx Lx / Rx > 200

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.047	PASS	25.74 ft	1.14	1.00	64.19	62.50	0.009	PASS	0.00 ft
+1.20D	0.040	PASS	26.09 ft	1.14	1.00	64.19	62.50	0.007	PASS	0.00 ft
+1.20D+0.50S	0.068	PASS	25.74 ft	1.14	1.00	64.19	62.50	0.013	PASS	0.00 ft
+1.20D+1.60S	0.130	PASS	26.09 ft	1.14	1.00	64.19	62.50	0.024	PASS	0.00 ft
+1.372D+0.70S+2.0E	0.928	PASS	25.74 ft	1.14	1.00	64.19	62.50	0.016	PASS	0.00 ft
+0.90D	0.030	PASS	25.74 ft	1.14	1.00	64.19	62.50	0.006	PASS	0.00 ft
+0.7284D+2.0E	0.874	PASS	25.74 ft	1.14	1.00	64.19	62.50	0.004	PASS	0.00 ft

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Suite 100
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(425)-450-4075

Project Title:
Engineer:
Project ID:
Project Descr:

R-19

Steel Column

DESCRIPTION: Drag Strut @ Shear Wall

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **W24x62**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 58 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 58 ft, K = 1.0

Applied Loads

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

AXIAL LOADS . . .
Axial Load at 58.0 ft, E = 185.0 k
BENDING LOADS . . .
Roof: Lat. Uniform Load creating Mx-x, D = 0.0750, S = 0.1250 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9631** : 1	
Load Combination	+1.372D+0.70S+2.0E	
Location of max.above base	28.805	ft
At maximum location values are . . .		
Pu	370.0	k
0.9 * Pn	448.912	k
Mu-x	80.047	k-ft
0.9 * Mn-x :	512.17	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	58.80	k-ft
PASS Maximum Shear Stress Rati	**0.02751** : 1	
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	8.410	k
Vn * Phi : Allowable	305.730	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	5.80 k
Bottom along Y-Y	5.80 k

Maximum Load Deflections . . .

Along Y-Y 1.145 in at 29.195ft above base
for load combination :+D+S
Along X-X 0.0 in at 0.0ft above base
for load combination :

Kx Lx / Rx > 200

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.086	PASS	29.19 ft	1.14	1.00	75.41	86.96	0.010	PASS	0.00 ft
+1.20D	0.074	PASS	28.81 ft	1.14	1.00	75.41	86.96	0.009	PASS	0.00 ft
+1.20D+0.50S	0.125	PASS	28.81 ft	1.14	1.00	75.41	86.96	0.014	PASS	0.00 ft
+1.20D+1.60S	0.238	PASS	28.81 ft	1.14	1.00	75.41	86.96	0.028	PASS	0.00 ft
+1.372D+0.70S+2.0E	0.963	PASS	28.81 ft	1.14	1.00	75.41	86.96	0.018	PASS	0.00 ft
+0.90D	0.055	PASS	29.19 ft	1.14	1.00	75.41	86.96	0.006	PASS	0.00 ft
+0.7284D+2.0E	0.864	PASS	28.81 ft	1.14	1.00	75.41	86.96	0.005	PASS	0.00 ft

Shutler Consulting Engineers, Inc.
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Project Title:
Engineer:
Project ID:
Project Descr:

R-20

Printed: 30 JAN 2024, 3:55PM

Steel Column

Project File: Columns and Drag Struts.ec6

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Drag Strut @ Braced Frame Row

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **W24x68**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 58.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 58.0 ft, K = 1.0

Applied Loads

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

AXIAL LOADS . . .
Axial Load at 58.0 ft, E = 188.90 k
BENDING LOADS . . .
Roof: Lat. Uniform Load creating Mx-x, D = 0.150, S = 0.250 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.8678** : 1	
Load Combination	+1.372D+0.70S+2.0E	
Location of max.above base	28.805	ft
At maximum location values are . . .		
Pu	377.80	k
0.9 * Pn	578.19	k
Mu-x	160.094	k-ft
0.9 * Mn-x :	663.75	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	91.875	k-ft
PASS Maximum Shear Stress Rati	**0.0570** : 1	
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	16.820	k
Vn * Phi : Allowable	295.065	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	11.60 k
Bottom along Y-Y	11.60 k

Maximum Load Deflections . . .

Along Y-Y 1.940 in at 29.195 ft above base
for load combination : +D+S

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Kx Lx / Rx > 200

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.133	PASS	29.19 ft	1.14	1.00	72.88	64.17	0.021	PASS	0.00 ft
+1.20D	0.114	PASS	29.19 ft	1.14	1.00	72.88	64.17	0.018	PASS	0.00 ft
+1.20D+0.50S	0.193	PASS	28.81 ft	1.14	1.00	72.88	64.17	0.030	PASS	0.00 ft
+1.20D+1.60S	0.367	PASS	28.81 ft	1.14	1.00	72.88	64.17	0.057	PASS	0.00 ft
+1.372D+0.70S+2.0E	0.868	PASS	28.81 ft	1.14	1.00	72.88	64.17	0.037	PASS	0.00 ft
+0.90D	0.086	PASS	29.19 ft	1.14	1.00	72.88	64.17	0.013	PASS	0.00 ft
+0.7284D+2.0E	0.715	PASS	29.19 ft	1.14	1.00	72.88	64.17	0.011	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

R-21

Printed: 30 JAN 2024, 5:30PM

Project File: Columns and Drag Struts.ec6

Steel Beam

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 67' Drag Strut - Gravity Design

CODE REFERENCES

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

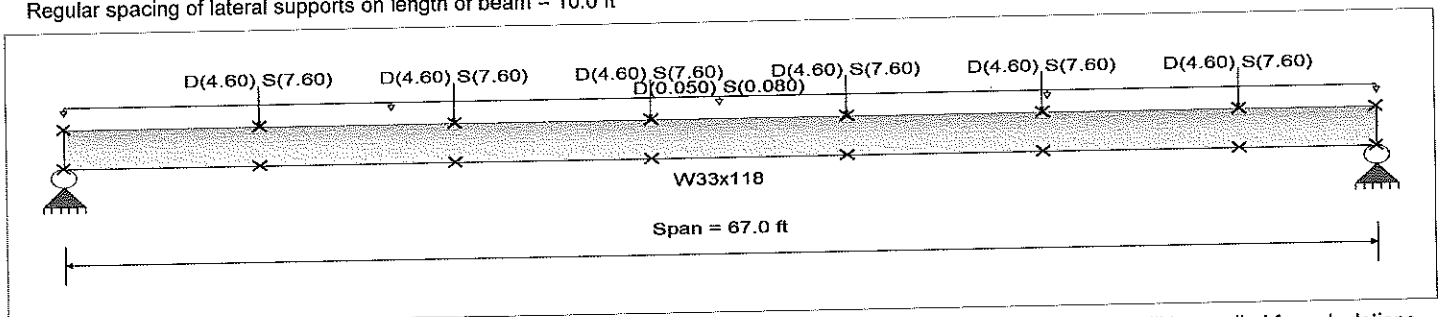
Material Properties

Analysis Method Load Resistance Factor Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 10.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 10.0 ft



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

Applied Loads

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.050, S = 0.080 k/ft, Tributary Width = 1.0 ft

Point Load : D = 4.60, S = 7.60 k @ 20.0 ft

Point Load : D = 4.60, S = 7.60 k @ 30.0 ft

Point Load : D = 4.60, S = 7.60 k @ 40.0 ft

Point Load : D = 4.60, S = 7.60 k @ 50.0 ft

Point Load : D = 4.60, S = 7.60 k @ 60.0 ft

Point Load : D = 4.60, S = 7.60 k @ 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.730** : 1	Maximum Shear Stress Ratio =	**0.126** : 1
Section used for this span	**W33x118**	Section used for this span	**W33x118**
Mu : Applied	1,093.799 k-ft	Vu : Applied	61.713 k
Mn * Phi : Allowable	1,497.433 k-ft	Vn * Phi : Allowable	488.565 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	67.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection					
Max Downward Transient Deflection	2.221 in	Ratio =	361	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	3.572 in	Ratio =	225	>=180.	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 9.95 ft	1	0.131	0.043	203.58		203.58	1,729.17	1,556.25	1.66	1.00	20.80	542.85	488.57
Dsgn. L = 9.95 ft	1	0.216	0.041	336.41	203.58	336.41	1,729.17	1,556.25	1.19	1.00	20.10	542.85	488.57

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Project Title:
Engineer:
Project ID:
Project Descr:

R-22

Steel Beam

DESCRIPTION: 67' Drag Strut - Gravity Design

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 9.95 ft	1	0.256	0.027	398.50	336.41	398.50	1,729.17	1,556.25	1.06	1.00	12.97	542.85	488.57
Dsgn. L = 9.95 ft	1	0.267	0.012	399.26	389.84	399.26	1,663.81	1,497.43	1.01	1.00	5.83	542.85	488.57
Dsgn. L = 10.15 ft	1	0.250	0.017	389.84	308.81	389.84	1,729.17	1,556.25	1.09	1.00	8.46	542.85	488.57
Dsgn. L = 9.95 ft	1	0.198	0.032	308.81	157.29	308.81	1,729.17	1,556.25	1.24	1.00	15.59	542.85	488.57
Dsgn. L = 7.08 ft	1	0.101	0.046	157.29		157.29	1,729.17	1,556.25	1.63	1.00	22.53	542.85	488.57
+1.20D													
Dsgn. L = 9.95 ft	1	0.112	0.036	174.50		174.50	1,729.17	1,556.25	1.66	1.00	17.83	542.85	488.57
Dsgn. L = 9.95 ft	1	0.185	0.035	288.35	174.50	288.35	1,729.17	1,556.25	1.19	1.00	17.23	542.85	488.57
Dsgn. L = 9.95 ft	1	0.219	0.023	341.57	288.35	341.57	1,729.17	1,556.25	1.06	1.00	11.11	542.85	488.57
Dsgn. L = 9.95 ft	1	0.229	0.010	342.23	334.15	342.23	1,663.81	1,497.43	1.01	1.00	5.00	542.85	488.57
Dsgn. L = 10.15 ft	1	0.215	0.015	334.15	264.69	334.15	1,729.17	1,556.25	1.09	1.00	7.25	542.85	488.57
Dsgn. L = 9.95 ft	1	0.170	0.027	264.69	134.82	264.69	1,729.17	1,556.25	1.24	1.00	13.37	542.85	488.57
Dsgn. L = 7.08 ft	1	0.087	0.040	134.82		134.82	1,729.17	1,556.25	1.63	1.00	19.31	542.85	488.57
+1.20D+0.50S													
Dsgn. L = 9.95 ft	1	0.189	0.062	294.25		294.25	1,729.17	1,556.25	1.66	1.00	30.06	542.85	488.57
Dsgn. L = 9.95 ft	1	0.312	0.059	486.25	294.25	486.25	1,729.17	1,556.25	1.19	1.00	29.06	542.85	488.57
Dsgn. L = 9.95 ft	1	0.370	0.038	575.99	486.25	575.99	1,729.17	1,556.25	1.06	1.00	18.75	542.85	488.57
Dsgn. L = 9.95 ft	1	0.385	0.017	577.09	563.47	577.09	1,663.81	1,497.43	1.01	1.00	8.43	542.85	488.57
Dsgn. L = 10.15 ft	1	0.362	0.025	563.47	446.36	563.47	1,729.17	1,556.25	1.09	1.00	12.22	542.85	488.57
Dsgn. L = 9.95 ft	1	0.287	0.046	446.36	227.35	446.36	1,729.17	1,556.25	1.24	1.00	22.53	542.85	488.57
Dsgn. L = 7.08 ft	1	0.146	0.067	227.35		227.35	1,729.17	1,556.25	1.63	1.00	32.56	542.85	488.57
+1.20D+1.60S													
Dsgn. L = 9.95 ft	1	0.358	0.117	557.71		557.71	1,729.17	1,556.25	1.66	1.00	56.96	542.85	488.57
Dsgn. L = 9.95 ft	1	0.592	0.113	921.61	557.71	921.61	1,729.17	1,556.25	1.19	1.00	55.09	542.85	488.57
Dsgn. L = 9.95 ft	1	0.701	0.073	1,091.70	921.61	1,091.70	1,729.17	1,556.25	1.06	1.00	35.54	542.85	488.57
Dsgn. L = 9.95 ft	1	0.730	0.033	1,093.80	1,067.98	1,093.80	1,663.81	1,497.43	1.01	1.00	15.99	542.85	488.57
Dsgn. L = 10.15 ft	1	0.686	0.047	1,067.98	846.01	1,067.98	1,729.17	1,556.25	1.09	1.00	23.15	542.85	488.57
Dsgn. L = 9.95 ft	1	0.544	0.087	846.01	430.92	846.01	1,729.17	1,556.25	1.24	1.00	42.70	542.85	488.57
Dsgn. L = 7.08 ft	1	0.277	0.126	430.92		430.92	1,729.17	1,556.25	1.63	1.00	61.71	542.85	488.57
+1.20D+0.70S													
Dsgn. L = 9.95 ft	1	0.220	0.072	342.15		342.15	1,729.17	1,556.25	1.66	1.00	34.95	542.85	488.57
Dsgn. L = 9.95 ft	1	0.363	0.069	565.40	342.15	565.40	1,729.17	1,556.25	1.19	1.00	33.80	542.85	488.57
Dsgn. L = 9.95 ft	1	0.430	0.045	669.75	565.40	669.75	1,729.17	1,556.25	1.06	1.00	21.80	542.85	488.57
Dsgn. L = 9.95 ft	1	0.448	0.020	671.04	655.20	671.04	1,663.81	1,497.43	1.01	1.00	9.81	542.85	488.57
Dsgn. L = 10.15 ft	1	0.421	0.029	655.20	519.02	655.20	1,729.17	1,556.25	1.09	1.00	14.21	542.85	488.57
Dsgn. L = 9.95 ft	1	0.334	0.054	519.02	264.36	519.02	1,729.17	1,556.25	1.24	1.00	26.20	542.85	488.57
Dsgn. L = 7.08 ft	1	0.170	0.077	264.36		264.36	1,729.17	1,556.25	1.63	1.00	37.86	542.85	488.57
+0.90D													
Dsgn. L = 9.95 ft	1	0.084	0.027	130.87		130.87	1,729.17	1,556.25	1.66	1.00	13.37	542.85	488.57
Dsgn. L = 9.95 ft	1	0.139	0.026	216.27	130.87	216.27	1,729.17	1,556.25	1.19	1.00	12.92	542.85	488.57
Dsgn. L = 9.95 ft	1	0.165	0.017	256.18	216.27	256.18	1,729.17	1,556.25	1.06	1.00	8.34	542.85	488.57
Dsgn. L = 9.95 ft	1	0.171	0.008	256.67	250.61	256.67	1,663.81	1,497.43	1.01	1.00	3.75	542.85	488.57
Dsgn. L = 10.15 ft	1	0.161	0.011	250.61	198.52	250.61	1,729.17	1,556.25	1.09	1.00	5.44	542.85	488.57
Dsgn. L = 9.95 ft	1	0.128	0.021	198.52	101.11	198.52	1,729.17	1,556.25	1.24	1.00	10.02	542.85	488.57
Dsgn. L = 7.08 ft	1	0.065	0.030	101.11		101.11	1,729.17	1,556.25	1.63	1.00	14.48	542.85	488.57

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Project Title:
Engineer:
Project ID:
Project Descr:

R-23

Printed: 30 JAN 2024, 5:36PM

Project File: Columns and Drag Struts.ec6

Steel Beam

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: 25' Drag Strut

CODE REFERENCES

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

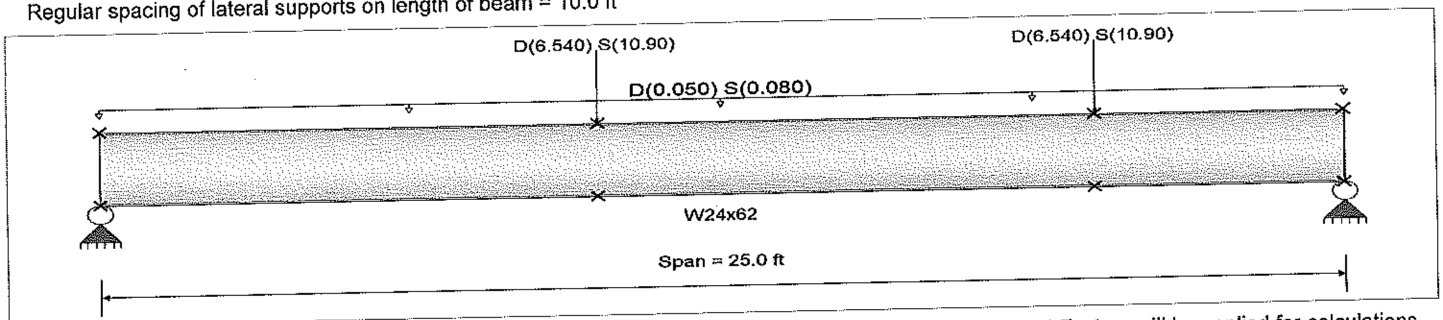
Material Properties

Analysis Method Load Resistance Factor Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 10.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 10.0 ft



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 6.540, S = 10.90 k @ 10.0 ft

Point Load : D = 6.540, S = 10.90 k @ 20.0 ft

Uniform Load : D = 0.050, S = 0.080 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.435** : 1	Maximum Shear Stress Ratio =	**0.107** : 1
Section used for this span	**W24x62**	Section used for this span	**W24x62**
Mu : Applied	216.404 k-ft	Vu : Applied	32.696 k
Mn * Phi : Allowable	497.314 k-ft	Vn * Phi : Allowable	305.730 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	25.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection					
Max Downward Transient Deflection	0.223 in	Ratio = 1,346	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio = 0	<360	n/a
Max Downward Total Deflection	0.357 in	Ratio = 840	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio = 0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 10.00 ft	1	0.137	0.027	78.50		78.50	637.50	573.75	1.65	1.00	8.20	305.73	305.73
Dsgn. L = 10.00 ft	1	0.158	0.038	78.50	58.44	78.50	552.57	497.31	1.11	1.00	11.51	305.73	305.73
Dsgn. L = 5.00 ft	1	0.102	0.039	58.44		58.44	637.50	573.75	1.65	1.00	11.86	305.73	305.73
+1.20D													
Dsgn. L = 10.00 ft	1	0.117	0.023	67.28		67.28	637.50	573.75	1.65	1.00	7.03	305.73	305.73
Dsgn. L = 10.00 ft	1	0.135	0.032	67.28	50.09	67.28	552.57	497.31	1.11	1.00	9.87	305.73	305.73
Dsgn. L = 5.00 ft	1	0.087	0.033	50.09		50.09	637.50	573.75	1.65	1.00	10.17	305.73	305.73
+1.20D+0.50S													
Dsgn. L = 10.00 ft	1	0.198	0.039	113.88		113.88	637.50	573.75	1.65	1.00	11.89	305.73	305.73
Dsgn. L = 10.00 ft	1	0.229	0.055	113.88	84.79	113.88	552.57	497.31	1.11	1.00	16.71	305.73	305.73
Dsgn. L = 5.00 ft	1	0.148	0.056	84.79		84.79	637.50	573.75	1.65	1.00	17.21	305.73	305.73
+1.20D+1.60S													

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Project Title:
Engineer:
Project ID:
Project Descr:

R-44

Printed: 30 JAN 2024, 5:36PM

Project File: Columns and Drag Struts.ec6

Steel Beam

LIC# : KW-06015511, Build:20.23.10.02

SHUTLER CONSULTING ENGINEERS

DESCRIPTION: 25' Drag Strut

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 10.00 ft	1	0.377	0.074	216.40		216.40	637.50	573.75	1.65	1.00	22.58	305.73	305.73
Dsgn. L = 10.00 ft	1	0.435	0.104	216.40	161.13	216.40	552.57	497.31	1.11	1.00	31.76	305.73	305.73
Dsgn. L = 5.00 ft	1	0.281	0.107	161.13		161.13	637.50	573.75	1.65	1.00	32.70	305.73	305.73
+1.20D+0.70S													
Dsgn. L = 10.00 ft	1	0.231	0.045	132.52		132.52	637.50	573.75	1.65	1.00	13.83	305.73	305.73
Dsgn. L = 10.00 ft	1	0.266	0.064	132.52	98.67	132.52	552.57	497.31	1.11	1.00	19.44	305.73	305.73
Dsgn. L = 5.00 ft	1	0.172	0.065	98.67		98.67	637.50	573.75	1.65	1.00	20.02	305.73	305.73
+0.90D													
Dsgn. L = 10.00 ft	1	0.088	0.017	50.46		50.46	637.50	573.75	1.65	1.00	5.27	305.73	305.73
Dsgn. L = 10.00 ft	1	0.101	0.024	50.46	37.57	50.46	552.57	497.31	1.11	1.00	7.40	305.73	305.73
Dsgn. L = 5.00 ft	1	0.065	0.025	37.57		37.57	637.50	573.75	1.65	1.00	7.63	305.73	305.73

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Project Title:
Engineer:
Project ID:
Project Descr:

R-45

Printed: 30 JAN 2024, 5:46PM

Steel Beam

Project File: Columns and Drag Struts.ec6

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: 12'-6" Drag Strut

CODE REFERENCES

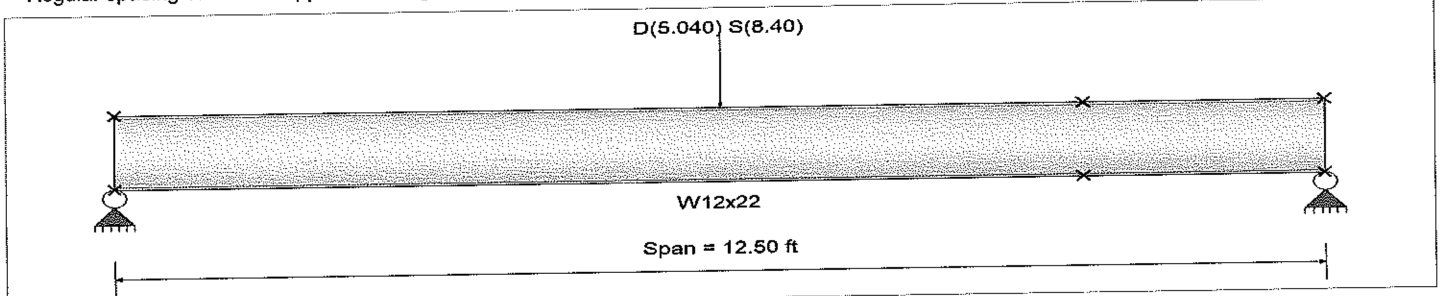
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined as a set spacing over all spans	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

First Brace starts at 10.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 10.0 ft



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 5.040, S = 8.40 k @ 6.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.777** : 1	Maximum Shear Stress Ratio =	**0.102** : 1
Section used for this span	**W12x22**	Section used for this span	**W12x22**
Mu : Applied	60.900 k-ft	Vu : Applied	9.744 k
Mn * Phi : Allowable	78.378 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	6.250 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.131 in	Ratio =	1,144	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.210 in	Ratio =	715	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 10.00 ft	1	0.281	0.037	22.05		22.05	87.09	78.38	1.34	1.00	3.53	95.94	95.94
Dsgn. L = 2.50 ft	1	0.080	0.037	8.82		8.82	122.08	109.88	1.66	1.00	3.53	95.94	95.94
+1.20D													
Dsgn. L = 10.00 ft	1	0.241	0.032	18.90		18.90	87.09	78.38	1.34	1.00	3.02	95.94	95.94
Dsgn. L = 2.50 ft	1	0.069	0.032	7.56		7.56	122.08	109.88	1.66	1.00	3.02	95.94	95.94
+1.20D+0.50S													
Dsgn. L = 10.00 ft	1	0.409	0.053	32.02		32.02	87.09	78.38	1.34	1.00	5.12	95.94	95.94
Dsgn. L = 2.50 ft	1	0.117	0.053	12.81		12.81	122.08	109.88	1.66	1.00	5.12	95.94	95.94
+1.20D+1.60S													
Dsgn. L = 10.00 ft	1	0.777	0.102	60.90		60.90	87.09	78.38	1.34	1.00	9.74	95.94	95.94
Dsgn. L = 2.50 ft	1	0.222	0.102	24.36		24.36	122.08	109.88	1.66	1.00	9.74	95.94	95.94
+1.20D+0.70S													
Dsgn. L = 10.00 ft	1	0.476	0.062	37.27		37.27	87.09	78.38	1.34	1.00	5.96	95.94	95.94
Dsgn. L = 2.50 ft	1	0.136	0.062	14.91		14.91	122.08	109.88	1.66	1.00	5.96	95.94	95.94
+0.90D													
Dsgn. L = 10.00 ft	1	0.181	0.024	14.18		14.18	87.09	78.38	1.34	1.00	2.27	95.94	95.94
Dsgn. L = 2.50 ft	1	0.052	0.024	5.67		5.67	122.08	109.88	1.66	1.00	2.27	95.94	95.94

SHUTLER CONSULTING

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JOB ______
SHEET No. R-46 OF ______
CALCULATED BY ______
DATE ______
SCALE ______

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	40.50	ft.
Height to roof	38.00	ft.
Parapet height	2.50	ft.
Wall thickness	9.25	in.
W_p	115.63	psf
Risk Category, I, II, III or IV =	II	

L_f = 250 ft. (diaphragm span)
$k_a = 1.0 + (L_f / 100)$ = 3.50
k_a = 2.0 max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S	128.7	%
Site soil class	D	
Seismic importance factor, I_e =	1.00	
F_a =	1.00	

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.287

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.858

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1713 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 998 PLF

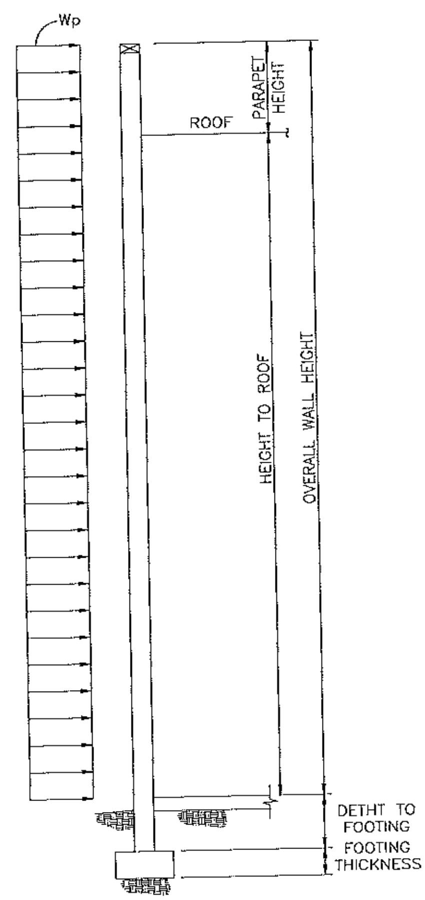
Design Seismic tie force = 1713 PLF (LRFD)
Design Seismic tie force = 1199 PLF (ASD)

Wind Criteria

Basic Wind Speed	95	MPH
Average roof height	40	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.09
K_{zt} = 1.00
p_{net30} (Pos. pressure) = 13.8 PSF
p_{net30} (Neg. pressure) = -15.2 PSF
p_{net30} (Design pressure) = -15.2 PSF
$p_{net} = \lambda * K_{zt} * p_{net30}$ = -16.6 PSF

Design Wind tie force = 358 PLF (LRFD)
Design Wind tie force = 215 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 1.00 ft.
Tie force = 1199 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

SHUTLER CONSULTING

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JOB

SHEET No. R-47 OF

CALCULATED BY

DATE

SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	40.50	ft.
Height to roof	38.00	ft.
Parapet height	2.50	ft.
Wall thickness	7.25	in.
W_p	90.63	psf
Risk Category, I, II, III or IV =	II	

L_f = 250 ft. (diaphragm span)

$k_a = 1.0 + (L_f / 100)$ = 3.50

k_a = 2.0 max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S 128.7 %

Site soil class D

Seismic importance factor, I_e = 1.00

F_a = 1.00

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.287

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.858

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.2 PSF times trib ht. = 1343 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 782 PLF

Design Seismic tie force = 1343 PLF (LRFD)

Design Seismic tie force = 940 PLF (ASD)

Wind Criteria

Basic Wind Speed	95	MPH
Average roof height	40	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.09

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 13.8 PSF

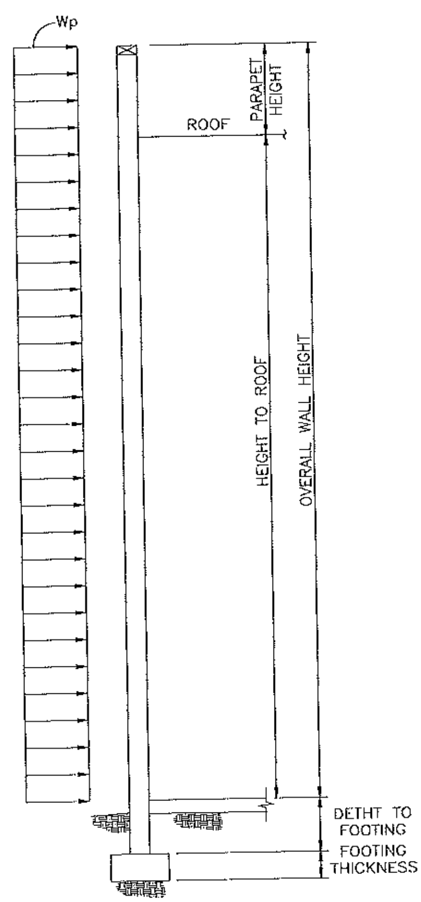
p_{net30} (Neg. pressure) = -15.2 PSF

p_{net30} (Design pressure) = -15.2 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -16.6 PSF

Design Wind tie force = 358 PLF (LRFD)

Design Wind tie force = 215 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 1.00 ft.

Tie force = 940 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

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JOB ______
SHEET No. R-48 ______ OF ______
CALCULATED BY ______
DATE ______
SCALE ______

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	42.50	ft.
Height to roof	38.00	ft.
Parapet height	4.50	ft.
Wall thickness	9.25	in.
W_p	115.63	psf
Risk Category, I, II, III or IV =	II	

L_f = 250 ft. (diaphragm span)
$k_a = 1.0 + (L_f / 100)$ = 3.50
$k_a = 2.0$ max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S	128.7	%
Site soil class	D	
Seismic importance factor, I_e =	1.00	
F_a =	1.00	

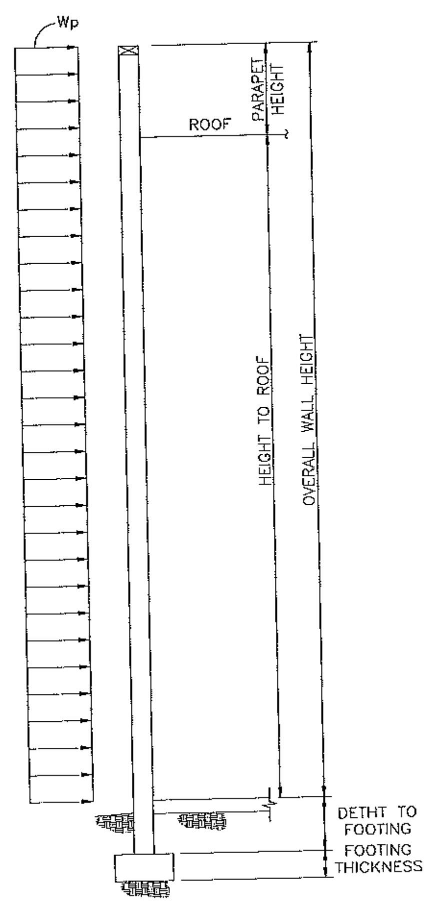
Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.287

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.858

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1886 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 1099 PLF

Design Seismic tie force = 1886 PLF (LRFD)
Design Seismic tie force = 1320 PLF (ASD)



Wind Criteria

Basic Wind Speed	95	MPH
Average roof height	40	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.09
K_{zt} = 1.00
p_{net30} (Pos. pressure) = 13.8 PSF
p_{net30} (Neg. pressure) = -15.2 PSF
p_{net30} (Design pressure) = -15.2 PSF
$p_{net} = \lambda * K_{zt} * p_{net30}$ = -16.6 PSF

Design Wind tie force = 394 PLF (LRFD)
Design Wind tie force = 236 PLF (ASD)

****** Seismic Governs ******

Tie Spacing = 1.00 ft.
Tie force = 1320 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

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JOB
SHEET No. R-49 OF
CALCULATED BY
DATE
SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	46.00	ft.
Height to roof	38.00	ft.
Parapet height	8.00	ft.
Wall thickness	7.25	in.
W_p	90.63	psf
Risk Category, I, II, III or IV =	II	

L_f = 250 ft. (diaphragm span)

$k_a = 1.0 + (L_f / 100)$ = 3.50

k_a = 2.0 max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S	128.7	%
Site soil class	D	
Seismic importance factor, I_e =	1.00	
F_a =	1.00	

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.287

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.858

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.2 PSF times trib ht. = 1732 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 1009 PLF

Design Seismic tie force = 1732 PLF (LRFD)

Design Seismic tie force = 1212 PLF (ASD)

Wind Criteria

Basic Wind Speed	95	MPH
Average roof height	40	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.09

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 13.8 PSF

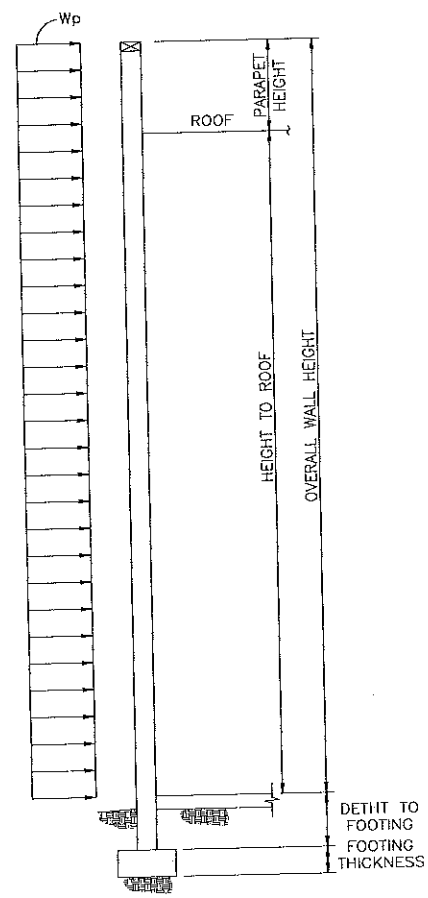
p_{net30} (Neg. pressure) = -15.2 PSF

p_{net30} (Design pressure) = -15.2 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -16.6 PSF

Design Wind tie force = 461 PLF (LRFD)

Design Wind tie force = 277 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 1.00 ft.

Tie force = 1212 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

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JOB

SHEET No. R-50 OF

CALCULATED BY

DATE

SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	36.00	ft.
Height to roof	36.00	ft.
Parapet height	0.00	ft.
Wall thickness	9.25	in.
W_p	115.63	psf
Risk Category, I, II, III or IV =	II	

L_f = 250 ft. (diaphragm span)

$k_a = 1.0 + (L_f / 100)$ = 3.50

k_a = 2.0 max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S	128.7	%
Site soil class	D	
Seismic importance factor, I_e =	1.00	
F_a =	1.00	

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.287

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.858

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1429 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 833 PLF

Design Seismic tie force = 1429 PLF (LRFD)

Design Seismic tie force = 1000 PLF (ASD)

Wind Criteria

Basic Wind Speed	95	MPH
Average roof height	40	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.09

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 13.8 PSF

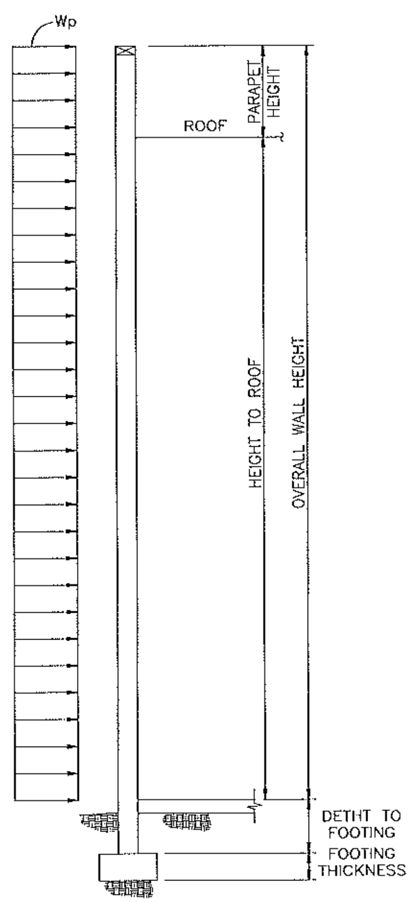
p_{net30} (Neg. pressure) = -15.2 PSF

p_{net30} (Design pressure) = -15.2 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -16.6 PSF

Design Wind tie force = 298 PLF (LRFD)

Design Wind tie force = 179 PLF (ASD)

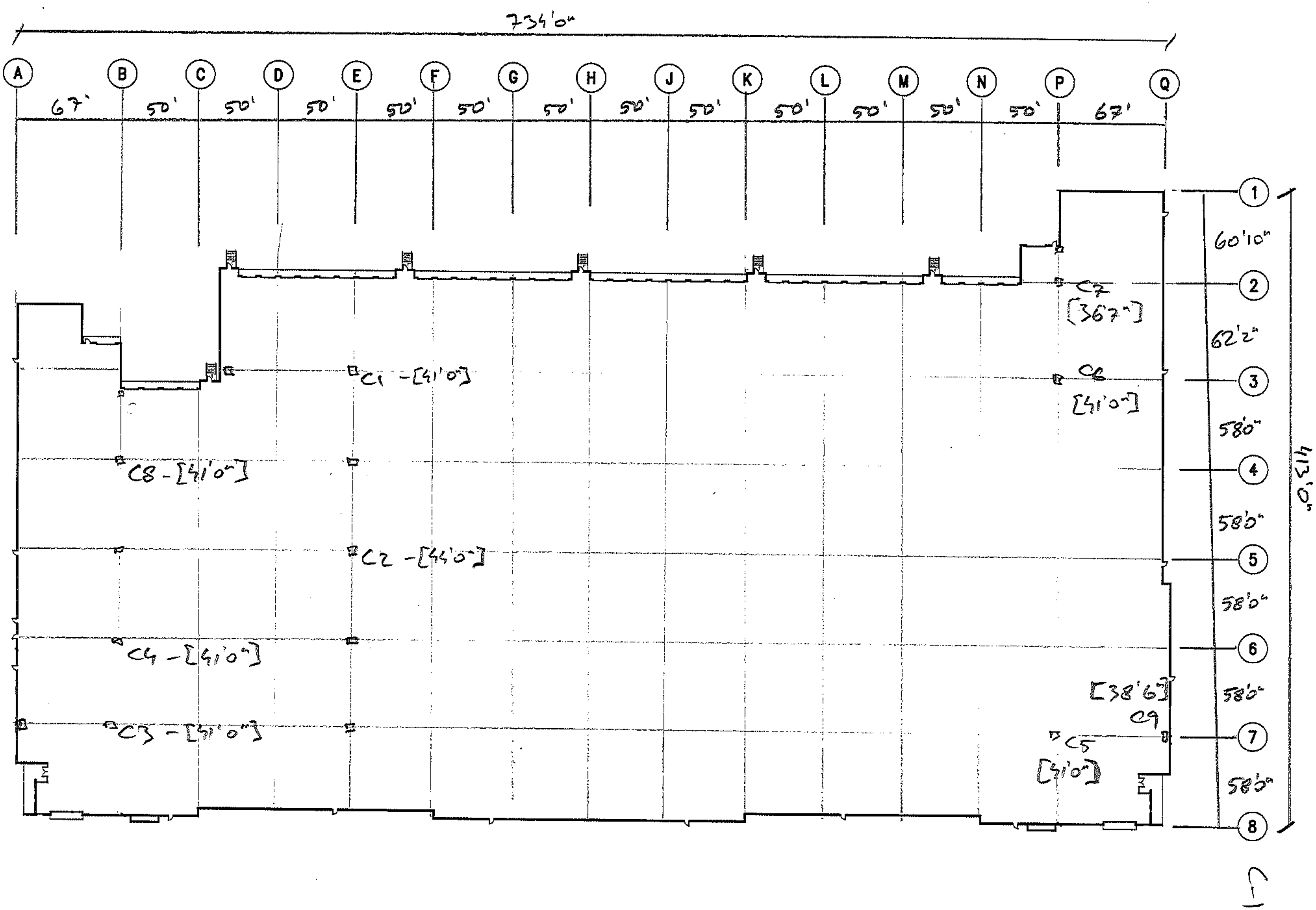


****** Seismic Governs ******

Tie Spacing = 1.00 ft.

Tie force = 1000 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21



K:\21\21-41 Freeman Logistics - Bldg B\ODWG\ARCH\23-8-9 pdf & cad\Vector-Freeman Rd Logistics South - Standard\xrf\B-A1.dwg, Model, 8/26/2023 12:14:08 PM, Dmitri, 1:1000

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JOB
SHEET NO. C-2 OF
CALCULATED BY
DATE
SCALE

COLUMNS

C1

	DL	LL	TL	e
RG-1	19.6k	30	49.7	7"
RG-1	19.6	30	49.7	7"
62'2" JOIST	5.1	7.8	12.9	5"
58' JOIST	4.7	7.2	11.9	5"
			124.2k	→ F7.5

C2

	DL	LL	TL	e
RG-2	24.8	29	53.8	7"
RG-2	24.8	29	53.8	7"
58' JOIST	6.2	7.3	13.5	5"
58' JOIST	6.2	7.3	13.5	5"
			161k	→ F8.5

C3

	DL	LL	TL	e
RG-3	19	29	48	10"
RG-7	18	26	44.1	10"
RG-9	21.6	23.7	45.2	7"
58' JOIST	4.4	7.3k	11.6	7"
			148.9k	→ F8.5

C9

	DL	LL	TL	e
RG-8	17	23.4	40.4	5"

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JOB

SHEET NO. C-3 OF

CALCULATED BY

DATE

SCALE

C4

	DL	LL	TL	e
RG-9	18.	26.1	44.1	10
RG-9	18	26.1	44.1	10
RG-2	24.8	29	53.8	7
67' JOIST	5	8.4	13.4	7
			155.4K → F8.5	

C5

	DL	LL	TL	e
RG-3	19	29	48	10
RG-8	20.5	29.3	49.8	10
RG-9	18	26.1	44.1	7
58' JOIST	4.4	7.3	11.6	7
			153.5K → F8.5	

C6

	DL	LL	TL	e
RG-9	18	26.1	44.1	10
RG-11	19.2	27.8	47	10
RG-1	19.6	30	49.7	7
67' JOIST	5	8.4	13.4	7
			154.2K → F8.5	

C7

	DL	LL	TL	e
RG-11	18.6	26.8	45.4	10
RG-12	7.2	10.8	18.1	10
67' D.S.	17.8	29.7	47.5K	11"
25' D.S	8.2	13.6	21.8	11"
			132.8K → F8.0	

C8

	DL	LL	TL	e
RG-9	18	26.1	44.1	10
RG-10	17.6	19.7	37.3	10
RG-1	19.6	30	49.7	7
67' JOIST	5	8.4	13.4	7
			144.5 → F8.0	

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Project Title:
Engineer:
Project ID:
Project Descr:

C-4

Printed: 30 JAN 2024, 4:12PM

Steel Column

Project File: Columns and Drag Struts.ec6

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Column 1

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 39.9167 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 39.9167 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 39.9167 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,948.19 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-1: Axial Load at 39.917 ft, Xecc = -7.0 in, D = 19.60, S = 30.0 k
RG-1: Axial Load at 39.917 ft, Xecc = 7.0 in, D = 19.60, S = 30.0 k
62'-2" Joist: Axial Load at 39.917 ft, Yecc = 5.0 in, D = 5.10, S = 7.80 k
58'-0" Joist: Axial Load at 39.917 ft, Yecc = -5.0 in, D = 4.70, S = 7.20 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6328** : 1
Load Combination +1.20D+1.60S
Location of max.above base 39.649 ft
At maximum location values are . . .
Pu 181.138 k
0.9 * Pn 287.579 k
Mu-x -0.5960 k-ft
0.9 * Mn-x : 179.551 k-ft
Mu-y 0.0 k-ft
0.9 * Mn-y : 179.551 k-ft

PASS Maximum Shear Stress Rati **0.000103** : 1
Load Combination +1.20D+1.60S
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.01503 k
Vn * Phi : Allowable 145.708 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.01044 k
Bottom along Y-Y 0.01044 k

Maximum Load Deflections . . .
Along Y-Y -0.008420 in at 23.307ft above base
for load combination :+D+S
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.249	PASS	39.65 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+1.20D	0.214	PASS	39.65 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+1.20D+0.50S	0.345	PASS	39.65 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+1.20D+1.60S	0.633	PASS	39.65 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+1.372D+0.70S	0.428	PASS	39.65 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+0.90D	0.159	PASS	0.00 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft
+0.7284D	0.129	PASS	0.00 ft	1.66	1.00	100.63	100.63	0.000	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

C-5

Steel Column

DESCRIPTION: Column 2

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name :	**HSS12x12x5/16**	Overall Column Height	43.9167 ft
Analysis Method :	Load Resistance Factor	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade		Brace condition :	
Fy : Steel Yield	50.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 43.9167 ft, K = 1.0	
E : Elastic Bending Modulus	29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 43.9167 ft, K = 1.0	

Applied Loads

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

Column self weight included : 2,143.42 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-2: Axial Load at 43.917 ft, Xecc = -7.0 in, D = 24.80, S = 29.0 k
RG-2: Axial Load at 43.917 ft, Xecc = 7.0 in, D = 24.80, S = 29.0 k
58'-0" Joist: Axial Load at 43.917 ft, Yecc = 5.0 in, D = 6.20, S = 7.30 k
58'-0" Joist: Axial Load at 43.917 ft, Yecc = -5.0 in, D = 6.20, S = 7.30 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.7848** : 1	
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pu	193.132	k
0.9 * Pn	246.083	k
Mu-x	0.0	k-ft
0.9 * Mn-x :	179.551	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	179.551	k-ft
PASS Maximum Shear Stress Rati	**0.0** : 1	
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.0	k
Vn * Phi : Allowable	0.0	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y	0.0 in	at	0.0ft	above base
for load combination :				
Along X-X	0.0 in	at	0.0ft	above base
for load combination :				

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.365	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+1.20D	0.313	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+1.20D+0.50S	0.460	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+1.20D+1.60S	0.785	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+1.372D+0.70S	0.564	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+0.90D	0.235	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft
+0.7284D	0.190	PASS	0.00 ft	1.00	1.00	110.71	110.71	0.000	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

C-6

Printed: 30 JAN 2024, 4:41PM

Project File: Columns and Drag Struts.ec6

Steel Column

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Column 3 - Unbalanced Load

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 40.917 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 40.917 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 40.917 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,997.01 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-3: Axial Load at 40.917 ft, Xecc = 10.0 in, D = 19.0, S = 29.0 k
RG-7: Axial Load at 40.917 ft, Xecc = -10.0 in, D = 18.0 k
RG-9: Axial Load at 40.917 ft, Yecc = 7.0 in, D = 21.60, S = 23.70 k
58'-0" Joist: Axial Load at 40.917 ft, Yecc = -7.0 in, D = 4.40, S = 7.30 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9577** : 1
Load Combination +1.20D+1.60S
Location of max.above base 40.642 ft
At maximum location values are . . .
Pu 173.996 k
0.9 * Pn 276.974 k
Mu-x -27.163 k-ft
0.9 * Mn-x : 179.551 k-ft
Mu-y -39.40 k-ft
0.9 * Mn-y : 179.551 k-ft

PASS Maximum Shear Stress Rati **0.006653** : 1
Load Combination +1.20D+1.60S
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.9694 k
Vn * Phi : Allowable 145.708 k

Maximum Load Reactions . .
Top along X-X 0.6110 k
Bottom along X-X 0.6110 k
Top along Y-Y 0.4790 k
Bottom along Y-Y 0.4790 k

Maximum Load Deflections . . .
Along Y-Y -0.4162 in at 23.891 ft above base
for load combination : +D+S
Along X-X -0.5309 in at 23.891 ft above base
for load combination : +D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios			Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios		
	Stress Ratio	Status	Location					Stress Ratio	Status	Location
+1.40D	0.403	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.002	PASS	0.00 ft
+1.20D	0.346	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.002	PASS	0.00 ft
+1.20D+0.50S	0.537	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.003	PASS	0.00 ft
+1.20D+1.60S	0.958	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.007	PASS	0.00 ft
+1.372D+0.70S	0.663	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.003	PASS	0.00 ft
+0.90D	0.259	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.002	PASS	0.00 ft
+0.7284D	0.171	PASS	0.00 ft	1.66	1.66	103.15	103.15	0.001	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

C-7

Printed: 30 JAN 2024, 4:42PM

Project File: Columns and Drag Struts.ec6

Steel Column

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Column 4 - Unbalanced Load

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 40.917 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 40.917 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 40.917 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,997.01 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-9: Axial Load at 40.917 ft, Xecc = 10.0 in, D = 18.0 k
RG-9: Axial Load at 40.917 ft, Xecc = -10.0 in, D = 18.0, S = 26.10 k
RG-2: Axial Load at 40.917 ft, Yecc = 7.0 in, D = 24.80, S = 29.0 k
67'-0" Joist: Axial Load at 40.917 ft, Yecc = -7.0 in, D = 5.0, S = 8.40 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9944** : 1
Load Combination +1.20D+1.60S
Location of max.above base 40.642 ft
At maximum location values are . . .
Pu 182.956 k
0.9 * Pn 276.974 k
Mu-x -32.865 k-ft
0.9 * Mn-x : 179.551 k-ft
Mu-y 34.566 k-ft
0.9 * Mn-y : 179.551 k-ft

PASS Maximum Shear Stress Rati **0.005837** : 1
Load Combination +1.20D+1.60S
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.8505 k
Vn * Phi : Allowable 145.708 k

Maximum Load Reactions . .
Top along X-X 0.5316 k
Bottom along X-X 0.5316 k
Top along Y-Y 0.5760 k
Bottom along Y-Y 0.5760 k

Maximum Load Deflections . . .
Along Y-Y -0.5004 in at 23.891 ft above base
for load combination : +D+S
Along X-X 0.4618 in at 23.891 ft above base
for load combination : S Only

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios Stress Ratio	Status	Location
+1.40D	0.422	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.003	PASS	0.00 ft
+1.20D	0.362	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.002	PASS	0.00 ft
+1.20D+0.50S	0.560	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.003	PASS	0.00 ft
+1.20D+1.60S	0.994	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.006	PASS	0.00 ft
+1.372D+0.70S	0.690	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.004	PASS	0.00 ft
+0.90D	0.271	PASS	40.64 ft	1.66	1.66	103.15	103.15	0.002	PASS	0.00 ft
+0.7284D	0.178	PASS	0.00 ft	1.66	1.66	103.15	103.15	0.001	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

C-8

Printed: 30 JAN 2024, 4:43PM

Project File: Columns and Drag Struts.ec6

Steel Column

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Column 5 - Unbalanced Load

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 39.917 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 39.917 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 39.917 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,948.20 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-3: Axial Load at 39.917 ft, Xecc = 10.0 in, D = 19.0 k
RG-8: Axial Load at 39.917 ft, Xecc = -10.0 in, D = 20.50, S = 29.30 k
RG-9: Axial Load at 39.917 ft, Yecc = 7.0 in, D = 18.0, S = 26.10 k
58'-0" Joist: Axial Load at 39.917 ft, Yecc = -7.0 in, D = 4.40, S = 7.30 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9501** : 1
Load Combination +1.20D+1.60S
Location of max.above base 39.917 ft
At maximum location values are . . .
Pu 176.938 k
0.9 * Pn 287.576 k
Mu-x -27.067 k-ft
0.9 * Mn-x : 179.551 k-ft
Mu-y 40.567 k-ft
0.9 * Mn-y : 179.551 k-ft

PASS Maximum Shear Stress Rati **0.006975** : 1
Load Combination +1.20D+1.60S
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 1.016 k
Vn * Phi : Allowable 145.708 k

Maximum Load Reactions . .
Top along X-X 0.6430 k
Bottom along X-X 0.6430 k
Top along Y-Y 0.4735 k
Bottom along Y-Y 0.4735 k

Maximum Load Deflections . . .
Along Y-Y -0.3819 in at 23.307 ft above base
for load combination :+D+S
Along X-X 0.5187 in at 23.307 ft above base
for load combination :+D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.374	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D	0.321	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D+0.50S	0.518	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.003	PASS	0.00 ft
+1.20D+1.60S	0.950	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.007	PASS	0.00 ft
+1.372D+0.70S	0.642	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.003	PASS	0.00 ft
+0.90D	0.200	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft
+0.7284D	0.162	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft

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Project Descr:

C-9

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Steel Column

Project File: Columns and Drag Struts.ec6

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Column 6 - Unbalanced Load

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 39.917 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 39.917 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 39.917 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,948.20 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-9: Axial Load at 39.917 ft, Xecc = 10.0 in, D = 18.0 k
RG-11: Axial Load at 39.917 ft, Xecc = -10.0 in, D = 19.20, S = 27.80 k
RG-1: Axial Load at 39.917 ft, Yecc = 7.0 in, D = 19.60, S = 30.0 k
67'-0" Joist: Axial Load at 39.917 ft, Yecc = -7.0 in, D = 5.0, S = 8.40 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9742**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	39.917	ft
At maximum location values are . . .		
Pu	182.418	k
0.9 * Pn	287.576	k
Mu-x	-30.380	k-ft
0.9 * Mn-x :	179.551	k-ft
Mu-y	38.267	k-ft
0.9 * Mn-y :	179.551	k-ft
PASS Maximum Shear Stress Rati	**0.006579**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.9587	k
Vn * Phi : Allowable	145.708	k

Maximum Load Reactions . .

Top along X-X	0.6054 k
Bottom along X-X	0.6054 k
Top along Y-Y	0.5290 k
Bottom along Y-Y	0.5290 k

Maximum Load Deflections . . .

Along Y-Y -0.4267 in at 23.307 ft above base
for load combination : +D+S

Along X-X 0.4884 in at 23.307 ft above base
for load combination : +D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.376	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D	0.323	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D+0.50S	0.526	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.003	PASS	0.00 ft
+1.20D+1.60S	0.974	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.007	PASS	0.00 ft
+1.372D+0.70S	0.654	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.004	PASS	0.00 ft
+0.90D	0.200	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft
+0.7284D	0.161	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft

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Project Descr:

C-10

Printed: 30 JAN 2024, 4:36PM

Steel Column

Project File: Columns and Drag Struts.ec6

LIC# : KW-06015511, Build:20.23.10.02 SHUTLER CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Column 7

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 35.5 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 35.5 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 35.5 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,732.63 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-11: Axial Load at 35.50 ft, Xecc = 10.0 in, D = 18.60, S = 26.80 k
RG-12: Axial Load at 35.50 ft, Xecc = -10.0 in, D = 7.20, S = 10.80 k
67' DS: Axial Load at 35.50 ft, Yecc = 11.0 in, D = 17.80, S = 29.70 k
25' DS: Axial Load at 35.50 ft, Yecc = -11.0 in, D = 8.20, S = 13.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9059**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	35.262	ft
At maximum location values are . . .		
Pu	193.679	k
0.9 * Pn	335.722	k
Mu-x	-33.944	k-ft
0.9 * Mn-x :	179.551	k-ft
Mu-y	-32.514	k-ft
0.9 * Mn-y :	179.551	k-ft
PASS Maximum Shear Stress Rati	**0.006607**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.9626	k
Vn * Phi : Allowable	145.708	k

Maximum Load Reactions . .

Top along X-X	0.6432 k
Bottom along X-X	0.6432 k
Top along Y-Y	0.6636 k
Bottom along Y-Y	0.6636 k

Maximum Load Deflections . . .

Along Y-Y -0.3766 in at 20.728ft above base
for load combination :+D+S

Along X-X -0.3650 in at 20.728ft above base
for load combination :+D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.349	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.003	PASS	0.00 ft
+1.20D	0.217	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.002	PASS	0.00 ft
+1.20D+0.50S	0.489	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.003	PASS	0.00 ft
+1.20D+1.60S	0.906	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.007	PASS	0.00 ft
+1.372D+0.70S	0.608	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.004	PASS	0.00 ft
+0.90D	0.163	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.002	PASS	0.00 ft
+0.7284D	0.132	PASS	35.26 ft	1.66	1.66	89.50	89.50	0.001	PASS	0.00 ft

DESCRIPTION: Column 8 - Unbalanced Load

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 39.917 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 39.917 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 39.917 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,948.20 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-9: Axial Load at 39.917 ft, Xecc = 10.0 in, D = 18.0, S = 26.10 k
RG-10: Axial Load at 39.917 ft, Xecc = -10.0 in, D = 17.60 k
RG-1: Axial Load at 39.917 ft, Yecc = 7.0 in, D = 19.60, S = 30.0 k
67' Joist: Axial Load at 39.917 ft, Yecc = -7.0 in, D = 5.0, S = 8.40 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9429**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	39.917	ft
At maximum location values are . . .		
Pu	177.778	k
0.9 * Pn	287.576	k
Mu-x	-30.380	k-ft
0.9 * Mn-x :	179.551	k-ft
Mu-y	-35.20	k-ft
0.9 * Mn-y :	179.551	k-ft
PASS Maximum Shear Stress Rati	**0.006052**	: 1
Load Combination	+1.20D+1.60S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.8818	k
Vn * Phi : Allowable	145.708	k

Maximum Load Reactions . .

Top along X-X	0.5532 k
Bottom along X-X	0.5532 k
Top along Y-Y	0.5290 k
Bottom along Y-Y	0.5290 k

Maximum Load Deflections . . .

Along Y-Y -0.4267 in at 23.307 ft above base
for load combination :+D+S

Along X-X -0.4463 in at 23.307 ft above base
for load combination :+D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.364	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D	0.312	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.002	PASS	0.00 ft
+1.20D+0.50S	0.509	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.003	PASS	0.00 ft
+1.20D+1.60S	0.943	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.006	PASS	0.00 ft
+1.372D+0.70S	0.633	PASS	39.92 ft	1.66	1.66	100.63	100.63	0.004	PASS	0.00 ft
+0.90D	0.194	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft
+0.7284D	0.157	PASS	0.00 ft	1.66	1.66	100.63	100.63	0.001	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr:

C-12

DESCRIPTION: Column 9

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS8x8x5/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 37.4167 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 37.4167 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 37.4167 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,189.57 lbs * Dead Load Factor
AXIAL LOADS . . .
RG-8: Axial Load at 37.417 ft, Xecc = 5.0 in, D = 17.0, S = 23.40 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.8439** : 1
Load Combination	+1.20D+1.60S
Location of max.above base	37.417 ft
At maximum location values are . . .	
Pu	59.267 k
0.9 * Pn	96.170 k
Mu-x	0.0 k-ft
0.9 * Mn-x :	94.125 k-ft
Mu-y	-24.10 k-ft
0.9 * Mn-y :	94.125 k-ft
PASS Maximum Shear Stress Rati	**0.006901** : 1
Load Combination	+1.20D+1.60S
Location of max.above base	0.0 ft
At maximum location values are . . .	
Vu : Applied	0.6441 k
Vn * Phi : Allowable	93.328 k

Maximum Load Reactions . .

Top along X-X	0.4499 k
Bottom along X-X	0.4499 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X -1.062 in at 21.847 ft above base
for load combination : +D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.358	PASS	37.42 ft	1.00	1.66	143.45	143.45	0.003	PASS	0.00 ft
+1.20D	0.307	PASS	37.42 ft	1.00	1.66	143.45	143.45	0.002	PASS	0.00 ft
+1.20D+0.50S	0.475	PASS	37.42 ft	1.00	1.66	143.45	143.45	0.004	PASS	0.00 ft
+1.20D+1.60S	0.844	PASS	37.42 ft	1.00	1.66	143.45	143.45	0.007	PASS	0.00 ft
+1.372D+0.70S	0.586	PASS	37.42 ft	1.00	1.66	143.45	143.45	0.005	PASS	0.00 ft
+0.90D	0.170	PASS	0.00 ft	1.00	1.66	143.45	143.45	0.002	PASS	0.00 ft
+0.7284D	0.138	PASS	0.00 ft	1.00	1.66	143.45	143.45	0.001	PASS	0.00 ft

SHUTLER CONSULTING ENGINEERS, INC
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JOB

SHEET NO. C-13 OF

CALCULATED BY

DATE

SCALE

BASE PL

HSS 12x12

$P = 161^k$

$w = \frac{161^k}{18^2} = 0.497\ ksi$

$M_1 = \frac{0.497(3)^2}{2} = 2.24\ ^{k''}/_{''}$

$M_2 = \frac{0.497(12)^2}{8} - 2.24 = 6.71\ ^{k''}/_{''}$

$t = \sqrt{\frac{6M}{27}} = 1.22'' \Rightarrow$ use $1\frac{1}{4}''$ OK

HSS 8x8

$P = 40.5^k$

$w = \frac{40.5^k}{11.5 \times 14} = 0.251\ ^{k}/_{''}$

$M_1 = \frac{0.251(8)^2}{8} = 2.01\ ^{k''}/_{''}$

$t = \sqrt{\frac{6M}{27}} = 0.67'' \Rightarrow \frac{3}{4}''$ OK

SHUTLER CONSULTING ENGINEERS, INC.
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Job Name : Freeman Logistics Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-26-2024
Sheet No.: F.1

Spread footing design program, Beta Version 1.0, latest revision 4-11-2018

****
* SPREAD FOOTING DESIGN *
****

Description:

DESIGN CRITERIA

Allowable soil bearing capacity	= 2,500.00 psf
Fy of reinforcing steel	= 60,000.00 psi.
f'c of concrete	= 3,500.00 psi.
Column size used in design	= 12.00 inches
Base plate size used in design	= 18.00 inches
Dead load	= 20.00 psf.
Live load	= 0.00 psf.
Snow load	= 25.00 psf.
Cover to bottom of footing	= 3.00 inches

Critical design load case is case 3
Load case 3, 1.2D + 1.6(Lr or S or R) + (0.5L or 0.5W)
Combined ultimate load factor = 1.42

NOTES

Load combinations per ASCE 7-16, Section 2.3, combinations 1 through 3. Design is for vertical loads due to dead load, live load and snow loads only, seismic and/or wind loads are not included in the assumed load combinations. Exception number one in ASCE 7-16, Section 2.3 states "The load factor on L in combinations 3 and 4 is permitted to equal 0.5 for all occupancies in which Lo in Chapter 4, Table 4.3-1, is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly"

FOOTING SCHEDULE
*** Allowable load is allowable applied load (Kips). ***
(Allow. Load = Ftg area * allow. soil bearing - ftg weight)

Footing Size	Thick-ness	Required Area of steel	Bar size options No.	Size	No.	Size	Allow. Load
F - 6.00	12.00	1.85 in.^2	5	# 6	4	# 7	84.60
F - 6.50	13.00	2.15 in.^2	5	# 6	4	# 7	98.76
F - 7.00	14.00	2.48 in.^2	6	# 6	5	# 7	113.93
F - 7.50	15.00	2.84 in.^2	7	# 6	5	# 7	130.08
F - 8.00	16.00	3.23 in.^2	8	# 6	6	# 7	147.20
F - 8.50	17.00	3.63 in.^2	9	# 6	7	# 7	165.27
F - 9.00	18.00	4.07 in.^2	10	# 6	7	# 7	184.28
F - 9.50	19.00	4.53 in.^2	8	# 7	6	# 8	204.19
F -10.00	20.00	5.01 in.^2	9	# 7	7	# 8	225.00
F -10.50	21.00	5.53 in.^2	10	# 7	8	# 8	246.68
F -11.00	23.00	6.07 in.^2	11	# 7	8	# 8	267.71
F -11.50	24.00	6.62 in.^2	12	# 7	9	# 8	290.95
F -12.00	25.00	7.20 in.^2	12	# 7	10	# 8	315.00

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Bellevue, Washington 98005
(425) 450-4075

Job Name : Freeman Logistics Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-26-2024
Sheet No.: F-2

Description:

FOOTING SHEAR

Footing Size	Thick-ness	d (in.)	Beam Shear (Kips) Vu	Beam Shear (Kips) Phi Vc	Punching Shear (Kips) Vu	Punching Shear (Kips) Phi Vc	Bo (in.)
F - 6.00	12.00	7.50	37.33	47.92	115.50	119.80	90.00
F - 6.50	13.00	8.50	44.30	58.84	136.59	141.81	94.00
F - 7.00	14.00	9.50	51.85	70.82	159.40	165.24	98.00
F - 7.50	15.00	10.50	60.00	83.86	183.94	190.08	102.00
F - 8.00	16.00	11.50	68.74	97.97	210.22	216.35	106.00
F - 8.50	17.00	12.50	78.07	113.15	238.22	244.04	110.00
F - 9.00	18.00	13.50	88.00	129.38	267.94	273.15	114.00
F - 9.50	19.00	14.50	98.52	146.69	299.40	303.67	118.00
F -10.00	20.00	15.50	109.63	165.06	332.59	335.62	122.00
F -10.50	21.00	16.50	121.33	184.49	367.50	368.99	126.00
F -11.00	23.00	18.50	130.37	216.71	402.51	439.98	134.00
F -11.50	24.00	19.50	143.11	238.80	440.83	477.61	138.00
F -12.00	25.00	20.50	156.44	261.96	480.88	516.65	142.00

FOOTING STEEL

Footing Size	Thick-ness	d (in.)	Design Steel Mu	Design Steel As	Minimum Steel (Temp steel)	As Provided
F - 6.00	12.00	7.50	10.03	1.85	1.73 in.^2	2.21
F - 6.50	13.00	8.50	12.25	2.15	2.03 in.^2	2.21
F - 7.00	14.00	9.50	14.69	2.48	2.35 in.^2	2.65
F - 7.50	15.00	10.50	17.36	2.84	2.70 in.^2	3.01
F - 8.00	16.00	11.50	20.25	3.23	3.07 in.^2	3.53
F - 8.50	17.00	12.50	23.36	3.63	3.47 in.^2	3.98
F - 9.00	18.00	13.50	26.69	4.07	3.89 in.^2	4.21
F - 9.50	19.00	14.50	30.25	4.53	4.33 in.^2	4.71
F -10.00	20.00	15.50	34.03	5.01	4.80 in.^2	5.41
F -10.50	21.00	16.50	38.03	5.53	5.29 in.^2	6.01
F -11.00	23.00	18.50	42.25	5.72	6.07 in.^2	6.28
F -11.50	24.00	19.50	46.69	6.27	6.62 in.^2	7.07
F -12.00	25.00	20.50	51.36	6.84	7.20 in.^2	7.22

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JOB ____________

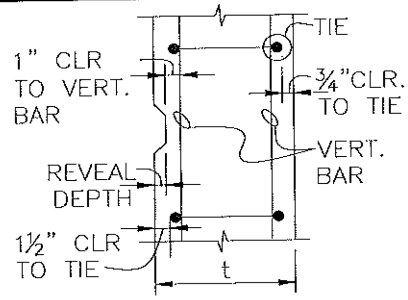
SHEET No. W-1 OF ____

CALCULATED BY ____________

DATE ____________

SCALE ____________

PRECAST TILT-UP WALL PANEL DESIGN



Concrete Strength (f'_c) = 5,000 psi.
Reinforcing Steel (f_y) = 60,000 psi.
Max. depth of reveal 0.75 in.

Minimum steel and maximum spacing...

Panel Thickness	Maximum Spacing (3*t or 18" max)	Minimum Horiz. Reinf. In^2 / foot	Minimum Horiz. Reinf. Size & Spacing	Minimum Vertical Reinf. In^2 / foot	Minimum Vertical Reinf. Size & Spacing
6.25 in.	18.00	0.150 in^2/ft.	4 @ 15.71 in. o.c.	0.090 in^2/ft.	4 @ 18.00 in. o.c.
7.25 in.	18.00	0.174 in^2/ft.	4 @ 13.54 in. o.c.	0.104 in^2/ft.	4 @ 18.00 in. o.c.
9.25 in.	18.00	0.222 in^2/ft.	4 @ 10.61 in. o.c.	0.133 in^2/ft.	4 @ 17.69 in. o.c.
11.25 in.	18.00	0.270 in^2/ft.	5 @ 13.64 in. o.c.	0.162 in^2/ft.	5 @ 18.00 in. o.c.

Depth to centroid of steel

Panels without ties...

Cover, outside face 1.00 in. (from reveal)
Cover, Inside face 0.75 in.

Depth to centroid of reinforcing steel (No reveal)			
Panel Thickness	Reveal Depth	Max. Bar Size	d (in.)
6.25 in.	0.00	5	4.94 in.
7.25 in.	0.00	6	5.88 in.
9.25 in.	0.00	7	7.81 in.
11.25 in.	0.00	7	9.81 in.

Depth to centroid of reinforcing steel (Maximum reveal depth = 0.75 inches)			
Panel Thickness	Reveal Depth	Max. Bar Size	d (in.)
6.25 in.	0.75	5	4.19 in.
7.25 in.	0.75	6	5.13 in.
9.25 in.	0.75	7	7.06 in.
11.25 in.	0.75	7	9.06 in.

Outside Face (See Diagram Above)
1-1/2" clear to tie **$d = t - (1.5 + \text{tie } d_B + 0.5 \text{ vert } d_B)$**
1" clear from reveal to vert bar **$d = t - (3/4 + 1 + 0.5 \text{ vert } d_B)$**

Inside Face (See Diagram Above)
3/4" clear to tie **$d = t - (3/4 + 3/4 + \text{tie } d_B + 0.5 \text{ vert } d_B)$**
(Use lesser of 3 conditions)

Panels with ties...

Cover, Outside face 1.00 in. (from face of panel to tie)
Cover, inside face 0.75 in. (from face of panel to tie)

Depth to centroid of steel (No. 3 ties with no reveal)			
Panel Thickness	Max. Bar Size	Reveal Depth	d (in.)
6.25 in.	5	0.00	4.56 in.
7.25 in.	6	0.00	5.50 in.
9.25 in.	7	0.00	7.44 in.
11.25 in.	7	0.00	9.44 in.

Cover, Outside face 1.500 in. (from face of panel to tie)
Cover, Outside face 1.00 in. (from reveal to vert bar)
Cover, inside face 0.75 in. (from face of panel to tie)

Depth to centroid of steel (No. 3 ties with 0.75 inch reveal)			
Panel Thickness	Max. Bar Size	Reveal Depth	d (in.)
6.25 in.	5	0.75	4.06 in.
7.25 in.	6	0.75	5.00 in.
9.25 in.	7	0.75	6.94 in.
11.25 in.	7	0.75	8.94 in.

File: PC panel cover, 4-3-13

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-2

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 1 - 46" Leg w/ 8'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 7.83 feet
Uniform dead load on design section	= 413.00 # / ft.
Uniform live load on design section	= 688.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 13,029.92 # / ft.
Total design load on design section	= 21,650.75 # / ft.
Steel ratio (As / (b * d))	= 0.0191
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 171.78 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.00583
Maximum axial stress (94.10 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-2

Description: Panel 1 - 46" Leg w/ 8'-0" Opening

10 Number 6 Bars
Results shown are for 46.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 32.82 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,037.22 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.05 in.	Mu	= 22.60 in-k
Ultimate Defl.=	0.27 in.	Phi * Mn	= 1,075.68 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 90.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,054.11 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.13 in.	Mu	= 68.28 in-k
Ultimate Defl.=	0.82 in.	Phi * Mn	= 1,086.88 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.60 in.	Mu	= 225.28 in-k
Ultimate Defl.=	2.70 in.	Phi * Mn	= 1,086.88 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.61 in.	Mu	= 316.93 in-k
Ultimate Defl.=	3.82 in.	Phi * Mn	= 1,074.51 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.98 in.	Mu	= 571.86 in-k
Ultimate Defl.=	6.88 in.	Phi * Mn	= 1,076.97 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.77 in.	Mu	= 291.06 in-k
Ultimate Defl.=	3.52 in.	Phi * Mn	= 1,068.87 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.53 in.	Mu	= 556.71 in-k
Ultimate Defl.=	6.71 in.	Phi * Mn	= 1,074.72 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.67 in.	Mu	= 266.26 in-k
Ultimate Defl.=	3.24 in.	Phi * Mn	= 1,058.62 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.20 in.	Mu	= 467.19 in-k
Ultimate Defl.=	5.69 in.	Phi * Mn	= 1,052.73 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: U-4

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 1 - 102" Leg w/ 8'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 413.00 # / ft.
Uniform live load on design section	= 688.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 20,801.27 # / ft.
Total design load on design section	= 34,563.77 # / ft.
Steel ratio (As / (b * d))	= 0.0086
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 380.91 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01585
Maximum axial stress (67.75 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: U-5

Description: Panel 1 - 102" Leg w/ 8'-0" Opening

10 Number 6 Bars
Results shown are for 102.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 52.40 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,134.19 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu	= 37.75 in-k
Ultimate Defl.=	0.32 in.	Phi * Mn	= 1,208.44 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 144.67 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,166.74 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.09 in.	Mu	= 115.39 in-k
Ultimate Defl.=	0.96 in.	Phi * Mn	= 1,230.15 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.35 in.	Mu	= 380.74 in-k
Ultimate Defl.=	3.17 in.	Phi * Mn	= 1,230.15 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.35 in.	Mu	= 528.76 in-k
Ultimate Defl.=	4.46 in.	Phi * Mn	= 1,206.18 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.75 in.	Mu	= 957.02 in-k
Ultimate Defl.=	8.06 in.	Phi * Mn	= 1,210.94 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.39 in.	Mu	= 482.93 in-k
Ultimate Defl.=	4.11 in.	Phi * Mn	= 1,195.26 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.48 in.	Mu	= 929.00 in-k
Ultimate Defl.=	7.84 in.	Phi * Mn	= 1,206.57 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.37 in.	Mu	= 437.39 in-k
Ultimate Defl.=	3.77 in.	Phi * Mn	= 1,175.44 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.11 in.	Mu	= 763.26 in-k
Ultimate Defl.=	6.62 in.	Phi * Mn	= 1,164.07 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-6

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 3 - Typical Panel

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 1.00 feet
Uniform dead load on design section	= 75.00 # / ft.
Uniform live load on design section	= 125.00 # / ft.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 1,664.10 # / ft.
Total design load on design section	= 1,864.10 # / ft.
Steel ratio (As / (b * d))	= 0.0042
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 44.81 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.03622
Maximum axial stress (31.21 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-7

Description: Panel 3 - Typical Panel

5 Bar @ 14.74 inches o.c.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	0.76 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	65.79 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.00 in.	Mu =	0.55 in-k
Ultimate Defl.=	0.07 in.	Phi * Mn =	72.18 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	2.10 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	66.30 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.01 in.	Mu =	1.47 in-k
Ultimate Defl.=	0.18 in.	Phi * Mn =	71.77 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.19 in.	Mu =	20.12 in-k
Ultimate Defl.=	2.52 in.	Phi * Mn =	71.77 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.19 in.	Mu =	37.23 in-k
Ultimate Defl.=	4.69 in.	Phi * Mn =	71.40 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	0.40 in.	Mu =	72.07 in-k
Ultimate Defl.=	8.99 in.	Phi * Mn =	72.11 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.24 in.	Mu =	36.56 in-k
Ultimate Defl.=	4.61 in.	Phi * Mn =	71.23 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.65 in.	Mu =	71.65 in-k
Ultimate Defl.=	8.95 in.	Phi * Mn =	72.04 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.24 in.	Mu =	33.61 in-k
Ultimate Defl.=	4.32 in.	Phi * Mn =	69.80 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.39 in.	Mu =	60.02 in-k
Ultimate Defl.=	7.79 in.	Phi * Mn =	68.98 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-8

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 4 - 27" Leg w/ 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 5.58 feet
Uniform dead load on design section	= 188.00 # / ft.
Uniform live load on design section	= 313.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 11,119.33 # / ft.
Total design load on design section	= 13,916.41 # / ft.
Steel ratio (As / (b * d))	= 0.0065
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 172.42 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.02131
Maximum axial stress (75.81 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Panel 4 - 27" Leg w/ 3'-4" Opening

4 Number 5 Bars
Results shown are for 27.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	12.12 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	448.00 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	7.74 in-k
Ultimate Defl.=	0.10 in.	Phi * Mn =	501.66 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	33.46 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	456.96 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	21.47 in-k
Ultimate Defl.=	0.27 in.	Phi * Mn =	502.90 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.20 in.	Mu =	109.52 in-k
Ultimate Defl.=	1.39 in.	Phi * Mn =	502.90 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.20 in.	Mu =	182.21 in-k
Ultimate Defl.=	2.34 in.	Phi * Mn =	496.32 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.29 in.	Mu =	429.01 in-k
Ultimate Defl.=	5.46 in.	Phi * Mn =	501.68 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.24 in.	Mu =	175.47 in-k
Ultimate Defl.=	2.26 in.	Phi * Mn =	493.32 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.98 in.	Mu =	424.99 in-k
Ultimate Defl.=	5.42 in.	Phi * Mn =	500.48 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.23 in.	Mu =	164.92 in-k
Ultimate Defl.=	2.17 in.	Phi * Mn =	480.77 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.85 in.	Mu =	376.27 in-k
Ultimate Defl.=	5.02 in.	Phi * Mn =	473.58 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-10

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 4 - 48" Leg w/ 3'-4" & 12'-0" Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 11.67 feet
Uniform dead load on design section	= 188.00 # / ft.
Uniform live load on design section	= 313.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 23,242.45 # / ft.
Total design load on design section	= 29,089.12 # / ft.
Steel ratio (As / (b * d))	= 0.0079
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 306.53 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01718
Maximum axial stress (89.14 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.:

Description: Panel 4 - 48" Leg w/ 3'-4" & 12'-0" Openings

6 Number 6 Bars
Results shown are for 48.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	25.34 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	953.01 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	16.29 in-k
Ultimate Defl.=	0.10 in.	Phi * Mn =	1,062.64 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	69.94 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	971.34 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.04 in.	Mu =	45.23 in-k
Ultimate Defl.=	0.28 in.	Phi * Mn =	1,065.17 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.23 in.	Mu =	230.73 in-k
Ultimate Defl.=	1.44 in.	Phi * Mn =	1,065.17 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.23 in.	Mu =	383.25 in-k
Ultimate Defl.=	2.41 in.	Phi * Mn =	1,051.74 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.50 in.	Mu =	903.20 in-k
Ultimate Defl.=	5.65 in.	Phi * Mn =	1,062.67 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.28 in.	Mu =	368.89 in-k
Ultimate Defl.=	2.33 in.	Phi * Mn =	1,045.62 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.18 in.	Mu =	894.56 in-k
Ultimate Defl.=	5.60 in.	Phi * Mn =	1,060.23 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.27 in.	Mu =	346.08 in-k
Ultimate Defl.=	2.23 in.	Phi * Mn =	1,020.00 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.03 in.	Mu =	788.82 in-k
Ultimate Defl.=	5.14 in.	Phi * Mn =	1,005.30 in-k

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Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-12

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 4 - 48" Leg w/ 12'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 10.00 feet
Uniform dead load on design section	= 188.00 # / ft.
Uniform live load on design section	= 313.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 19,916.41 # / ft.
Total design load on design section	= 24,926.41 # / ft.
Steel ratio (As / (b * d))	= 0.0064
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 306.53 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.02163
Maximum axial stress (76.38 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
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Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-13

Description: Panel 4 - 48" Leg w/ 12'-0" Opening

7 Number 5 Bars
Results shown are for 48.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	21.71 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	784.80 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	13.92 in-k
Ultimate Defl.=	0.10 in.	Phi * Mn =	881.07 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	59.93 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	800.88 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	38.64 in-k
Ultimate Defl.=	0.28 in.	Phi * Mn =	883.30 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.20 in.	Mu =	197.12 in-k
Ultimate Defl.=	1.42 in.	Phi * Mn =	883.30 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.20 in.	Mu =	327.78 in-k
Ultimate Defl.=	2.39 in.	Phi * Mn =	871.48 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.32 in.	Mu =	772.09 in-k
Ultimate Defl.=	5.58 in.	Phi * Mn =	881.10 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.24 in.	Mu =	315.58 in-k
Ultimate Defl.=	2.31 in.	Phi * Mn =	866.10 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.03 in.	Mu =	764.79 in-k
Ultimate Defl.=	5.54 in.	Phi * Mn =	878.95 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.23 in.	Mu =	296.31 in-k
Ultimate Defl.=	2.22 in.	Phi * Mn =	843.59 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.90 in.	Mu =	675.63 in-k
Ultimate Defl.=	5.12 in.	Phi * Mn =	830.69 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-14

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 13 - 21" Leg w/ 9'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 36.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 6.25 feet
Uniform dead load on design section	= 466.00 # / ft.
Uniform live load on design section	= 777.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 11,056.64 # / ft.
Total design load on design section	= 18,825.39 # / ft.
Steel ratio (As / (b * d))	= 0.0084
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 134.11 in.-k
Delta Cracked	= 0.43 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.45 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01499
Maximum axial stress (137.44 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-15

Description: Panel 13 - 21" Leg w/ 9'-0" Opening

4 Number 5 Bars
Results shown are for 21.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	9.17 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	450.54 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	6.11 in-k
Ultimate Defl.=	0.08 in.	Phi * Mn =	502.08 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	25.35 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	474.58 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	18.25 in-k
Ultimate Defl.=	0.23 in.	Phi * Mn =	518.47 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu =	122.95 in-k
Ultimate Defl.=	1.53 in.	Phi * Mn =	518.47 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu =	201.56 in-k
Ultimate Defl.=	2.58 in.	Phi * Mn =	500.88 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.73 in.	Mu =	475.83 in-k
Ultimate Defl.=	6.06 in.	Phi * Mn =	503.98 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.36 in.	Mu =	190.69 in-k
Ultimate Defl.=	2.47 in.	Phi * Mn =	492.85 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.41 in.	Mu =	467.34 in-k
Ultimate Defl.=	5.98 in.	Phi * Mn =	500.77 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.32 in.	Mu =	177.96 in-k
Ultimate Defl.=	2.35 in.	Phi * Mn =	478.94 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.25 in.	Mu =	406.06 in-k
Ultimate Defl.=	5.44 in.	Phi * Mn =	470.95 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: N-16

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 13 - 42" Leg w/ 2x 9'-0" Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
esign height, distance between floor and roof.	= 36.00 feet
ixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 466.00 # / ft.
Uniform live load on design section	= 777.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 22,113.28 # / ft.
Total design load on design section	= 37,650.78 # / ft.
Steel ratio (As / (b * d))	= 0.0084
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 268.21 in.-k
Delta Cracked	= 0.43 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.45 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01499
Maximum axial stress (137.44 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-22-2024
Sheet No.: W-17

Description: Panel 13 - 42" Leg w/ 2x 9'-0" Openings

8 Number 5 Bars
Results shown are for 42.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	18.35 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	901.08 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	12.23 in-k
Ultimate Defl.=	0.08 in.	Phi * Mn =	1,004.17 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	50.69 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	949.17 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	36.49 in-k
Ultimate Defl.=	0.23 in.	Phi * Mn =	1,036.93 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu =	245.90 in-k
Ultimate Defl.=	1.53 in.	Phi * Mn =	1,036.93 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu =	403.12 in-k
Ultimate Defl.=	2.58 in.	Phi * Mn =	1,001.75 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.73 in.	Mu =	951.66 in-k
Ultimate Defl.=	6.06 in.	Phi * Mn =	1,007.96 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.36 in.	Mu =	381.37 in-k
Ultimate Defl.=	2.47 in.	Phi * Mn =	985.69 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.41 in.	Mu =	934.68 in-k
Ultimate Defl.=	5.98 in.	Phi * Mn =	1,001.55 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.32 in.	Mu =	355.92 in-k
Ultimate Defl.=	2.35 in.	Phi * Mn =	957.87 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.25 in.	Mu =	812.12 in-k
Ultimate Defl.=	5.44 in.	Phi * Mn =	941.90 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-18

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 12 - 27" Leg w/ 8'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 36.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 6.25 feet
Uniform dead load on design section	= 38.00 # / ft.
Uniform live load on design section	= 63.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 12,501.95 # / ft.
Total design load on design section	= 13,133.20 # / ft.
Steel ratio (As / (b * d))	= 0.0065
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 172.42 in.-k
Delta Cracked	= 0.43 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.45 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.02124
Maximum axial stress (77.71 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-19

Description: Panel 12 - 27" Leg w/ 8'-0" Opening

4 Number 5 Bars
Results shown are for 27.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	2.74 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	444.05 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.00 in.	Mu =	1.74 in-k
Ultimate Defl.=	0.02 in.	Phi * Mn =	504.40 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	7.55 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	446.07 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.01 in.	Mu =	4.68 in-k
Ultimate Defl.=	0.06 in.	Phi * Mn =	497.83 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.18 in.	Mu =	94.77 in-k
Ultimate Defl.=	1.15 in.	Phi * Mn =	497.83 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.18 in.	Mu =	181.58 in-k
Ultimate Defl.=	2.21 in.	Phi * Mn =	496.34 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.23 in.	Mu =	443.43 in-k
Ultimate Defl.=	5.33 in.	Phi * Mn =	503.43 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.23 in.	Mu =	180.09 in-k
Ultimate Defl.=	2.19 in.	Phi * Mn =	495.67 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.98 in.	Mu =	442.54 in-k
Ultimate Defl.=	5.32 in.	Phi * Mn =	503.16 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.23 in.	Mu =	170.65 in-k
Ultimate Defl.=	2.12 in.	Phi * Mn =	482.54 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.89 in.	Mu =	395.18 in-k
Ultimate Defl.=	4.98 in.	Phi * Mn =	475.01 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-20

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 40 - 70" Leg w/ 8'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 9.83 feet
Uniform dead load on design section	= 456.00 # / ft.
Uniform live load on design section	= 760.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 16,358.12 # / ft.
Total design load on design section	= 28,311.40 # / ft.
Steel ratio (As / (b * d))	= 0.0113
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 261.41 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01150
Maximum axial stress (81.24 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-21

Description: Panel 40 - 70" Leg w/ 8'-0" Opening

9 Number 6 Bars
Results shown are for 70.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	45.50 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	999.21 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu =	32.58 in-k
Ultimate Defl.=	0.34 in.	Phi * Mn =	1,054.97 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	125.66 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,026.21 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.12 in.	Mu =	100.95 in-k
Ultimate Defl.=	1.03 in.	Phi * Mn =	1,073.78 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.41 in.	Mu =	311.14 in-k
Ultimate Defl.=	3.18 in.	Phi * Mn =	1,073.78 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.41 in.	Mu =	419.41 in-k
Ultimate Defl.=	4.35 in.	Phi * Mn =	1,053.93 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.97 in.	Mu =	752.95 in-k
Ultimate Defl.=	7.79 in.	Phi * Mn =	1,057.16 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.52 in.	Mu =	380.14 in-k
Ultimate Defl.=	3.96 in.	Phi * Mn =	1,044.87 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.59 in.	Mu =	729.16 in-k
Ultimate Defl.=	7.56 in.	Phi * Mn =	1,053.54 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.43 in.	Mu =	344.13 in-k
Ultimate Defl.=	3.62 in.	Phi * Mn =	1,029.68 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.22 in.	Mu =	599.26 in-k
Ultimate Defl.=	6.35 in.	Phi * Mn =	1,020.97 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-22

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 40 - 102" Leg w/ 8'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 456.00 # / ft.
Uniform live load on design section	= 760.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 20,801.27 # / ft.
Total design load on design section	= 36,001.27 # / ft.
Steel ratio (As / (b * d))	= 0.0086
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 380.91 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01576
Maximum axial stress (70.89 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-23

Description: Panel 40 - 102" Leg w/ 8'-0" Opening

10 Number 6 Bars
Results shown are for 102.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 57.86 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,136.12 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu	= 42.06 in-k
Ultimate Defl.=	0.35 in.	Phi * Mn	= 1,210.35 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 159.79 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,172.05 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.10 in.	Mu	= 130.88 in-k
Ultimate Defl.=	1.09 in.	Phi * Mn	= 1,235.42 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.36 in.	Mu	= 403.37 in-k
Ultimate Defl.=	3.35 in.	Phi * Mn	= 1,235.42 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.36 in.	Mu	= 541.08 in-k
Ultimate Defl.=	4.56 in.	Phi * Mn	= 1,208.96 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.81 in.	Mu	= 972.12 in-k
Ultimate Defl.=	8.17 in.	Phi * Mn	= 1,213.26 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.39 in.	Mu	= 489.55 in-k
Ultimate Defl.=	4.16 in.	Phi * Mn	= 1,196.90 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.51 in.	Mu	= 940.62 in-k
Ultimate Defl.=	7.93 in.	Phi * Mn	= 1,208.44 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.37 in.	Mu	= 441.78 in-k
Ultimate Defl.=	3.80 in.	Phi * Mn	= 1,176.67 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.12 in.	Mu	= 767.96 in-k
Ultimate Defl.=	6.65 in.	Phi * Mn	= 1,165.07 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-24

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 42 - 36" Leg w/ 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.25 feet
Design height, distance between floor and roof.	= 38.25 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 4.67 feet
Uniform dead load on design section	= 503.00 # / ft.
Uniform live load on design section	= 838.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 7,832.19 # / ft.
Total design load on design section	= 14,094.66 # / ft.
Steel ratio (As / (b * d))	= 0.0098
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 134.44 in.-k
Delta Cracked	= 0.64 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.60 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01359
Maximum axial stress (78.97 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-25

Description: Panel 42 - 36" Leg w/ 3'-4" Opening

4 Number 6 Bars
Results shown are for 36.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	23.84 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	450.68 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu =	17.24 in-k
Ultimate Defl.=	0.37 in.	Phi * Mn =	478.09 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	65.83 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	465.19 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.12 in.	Mu =	54.36 in-k
Ultimate Defl.=	1.16 in.	Phi * Mn =	488.57 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	155.68 in-k
Ultimate Defl.=	3.33 in.	Phi * Mn =	488.57 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	202.04 in-k
Ultimate Defl.=	4.39 in.	Phi * Mn =	477.89 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.86 in.	Mu =	359.61 in-k
Ultimate Defl.=	7.79 in.	Phi * Mn =	479.31 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.41 in.	Mu =	181.28 in-k
Ultimate Defl.=	3.96 in.	Phi * Mn =	473.02 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.46 in.	Mu =	347.09 in-k
Ultimate Defl.=	7.54 in.	Phi * Mn =	477.37 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.39 in.	Mu =	163.43 in-k
Ultimate Defl.=	3.62 in.	Phi * Mn =	465.40 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.08 in.	Mu =	283.27 in-k
Ultimate Defl.=	6.31 in.	Phi * Mn =	461.03 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-26

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 55 - 60" Leg w/ 15'-0" Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 4.00 feet
esign height, distance between floor and roof.	= 38.50 feet
ixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 503.00 # / ft.
Uniform live load on design section	= 838.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 29,430.18 # / ft.
Total design load on design section	= 46,192.68 # / ft.
Steel ratio (As / (b * d))	= 0.0073
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 383.16 in.-k
Delta Cracked	= 0.50 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01760
Maximum axial stress (116.90 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-27

Description: Panel 55 - 60" Leg w/ 15'-0" Openings

7 Number 6 Bars
Results shown are for 60.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 72.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,134.11 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu	= 50.37 in-k
Ultimate Defl.=	0.28 in.	Phi * Mn	= 1,273.81 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 200.52 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,186.93 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.10 in.	Mu	= 148.90 in-k
Ultimate Defl.=	0.82 in.	Phi * Mn	= 1,306.10 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.29 in.	Mu	= 395.95 in-k
Ultimate Defl.=	2.17 in.	Phi * Mn	= 1,306.10 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.29 in.	Mu	= 527.68 in-k
Ultimate Defl.=	2.96 in.	Phi * Mn	= 1,267.40 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.81 in.	Mu	= 1,175.52 in-k
Ultimate Defl.=	6.56 in.	Phi * Mn	= 1,277.45 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.31 in.	Mu	= 480.57 in-k
Ultimate Defl.=	2.73 in.	Phi * Mn	= 1,249.75 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.40 in.	Mu	= 1,146.70 in-k
Ultimate Defl.=	6.42 in.	Phi * Mn	= 1,270.40 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.29 in.	Mu	= 437.57 in-k
Ultimate Defl.=	2.54 in.	Phi * Mn	= 1,213.53 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.14 in.	Mu	= 963.01 in-k
Ultimate Defl.=	5.66 in.	Phi * Mn	= 1,192.74 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-28

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 58 - 73" Leg w/ 7'-6" & 11'-7.5" Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 6,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
esign height, distance between floor and roof.	= 37.00 feet
ixity coefficient	= 0.85
Tributary width for design loads	= 15.69 feet
Uniform dead load on design section	= 60.00 # / ft.
Uniform live load on design section	= 100.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 22,359.48 # / ft.
Total design load on design section	= 24,869.88 # / ft.
Steel ratio (As / (b * d))	= 0.0169
Modulus of rupture (7.5 * (f'c^.5))	= 580.95 psi.
Modulus of elasticity (Concrete)	= 4,415.20 ksi.
n = Es / Ec	= 6.57
Cracking Moment	= 298.63 in.-k
Delta Cracked	= 0.60 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.00915
Maximum axial stress (68.75 psi.) is less than .06 * f'c = 360.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-29

Description: Panel 58 - 73" Leg w/ 7'-6" & 11'-7.5" Openings

14 Number 6 Bars
Results shown are for 73.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	9.56 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,501.17 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	6.15 in-k
Ultimate Defl.=	0.04 in.	Phi * Mn =	1,574.01 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	26.39 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,506.61 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.02 in.	Mu =	16.69 in-k
Ultimate Defl.=	0.11 in.	Phi * Mn =	1,569.04 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	260.60 in-k
Ultimate Defl.=	1.79 in.	Phi * Mn =	1,569.04 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	489.48 in-k
Ultimate Defl.=	3.37 in.	Phi * Mn =	1,565.05 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.70 in.	Mu =	937.29 in-k
Ultimate Defl.=	6.44 in.	Phi * Mn =	1,573.21 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.69 in.	Mu =	482.78 in-k
Ultimate Defl.=	3.33 in.	Phi * Mn =	1,563.23 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.49 in.	Mu =	933.34 in-k
Ultimate Defl.=	6.41 in.	Phi * Mn =	1,572.49 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.64 in.	Mu =	454.95 in-k
Ultimate Defl.=	3.16 in.	Phi * Mn =	1,547.02 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.29 in.	Mu =	823.76 in-k
Ultimate Defl.=	5.74 in.	Phi * Mn =	1,537.73 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-30

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 58 - 85" Leg w/ 7'-6" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 6,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 10.83 feet
Uniform dead load on design section	= 60.00 # / ft.
Uniform live load on design section	= 100.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 15,433.60 # / ft.
Total design load on design section	= 17,166.40 # / ft.
Steel ratio (As / (b * d))	= 0.0062
Modulus of rupture (7.5 * (f'c^.5))	= 580.95 psi.
Modulus of elasticity (Concrete)	= 4,415.20 ksi.
n = Es / Ec	= 6.57
Cracking Moment	= 347.72 in.-k
Delta Cracked	= 0.60 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.02894
Maximum axial stress (40.75 psi.) is less than .06 * f'c = 360.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-31

Description: Panel 58 - 85" Leg w/ 7'-6" Opening

6 Number 6 Bars
Results shown are for 85.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	6.60 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	689.22 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.00 in.	Mu =	4.44 in-k
Ultimate Defl.=	0.05 in.	Phi * Mn =	747.58 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	18.22 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	693.56 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.01 in.	Mu =	12.01 in-k
Ultimate Defl.=	0.14 in.	Phi * Mn =	743.59 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu =	187.50 in-k
Ultimate Defl.=	2.15 in.	Phi * Mn =	743.59 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu =	351.36 in-k
Ultimate Defl.=	4.05 in.	Phi * Mn =	740.38 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.01 in.	Mu =	676.00 in-k
Ultimate Defl.=	7.74 in.	Phi * Mn =	746.93 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.29 in.	Mu =	346.04 in-k
Ultimate Defl.=	4.00 in.	Phi * Mn =	738.92 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.20 in.	Mu =	672.87 in-k
Ultimate Defl.=	7.71 in.	Phi * Mn =	746.35 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.28 in.	Mu =	323.15 in-k
Ultimate Defl.=	3.79 in.	Phi * Mn =	725.91 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.99 in.	Mu =	582.12 in-k
Ultimate Defl.=	6.89 in.	Phi * Mn =	718.46 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-32

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 59 - 30" Leg w/ 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.10 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 3.50 feet
Design height, distance between floor and roof.	= 36.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 4.67 feet
Uniform dead load on design section	= 435.00 # / ft.
Uniform live load on design section	= 725.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.25 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 8,046.45 # / ft.
Total design load on design section	= 13,463.65 # / ft.
Steel ratio (As / (b * d))	= 0.0117
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 112.03 in.-k
Delta Cracked	= 0.58 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.48 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01090
Maximum axial stress (89.80 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-23-2024
Sheet No.: W-33

Description: Panel 59 - 30" Leg w/ 3'-4" Opening

4 Number 6 Bars
Results shown are for 30.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	20.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	442.77 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu =	14.69 in-k
Ultimate Defl.=	0.31 in.	Phi * Mn =	469.98 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	56.95 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	454.92 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.11 in.	Mu =	44.91 in-k
Ultimate Defl.=	0.94 in.	Phi * Mn =	478.12 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.37 in.	Mu =	133.00 in-k
Ultimate Defl.=	2.78 in.	Phi * Mn =	478.12 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.37 in.	Mu =	178.86 in-k
Ultimate Defl.=	3.79 in.	Phi * Mn =	469.20 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.73 in.	Mu =	320.25 in-k
Ultimate Defl.=	6.77 in.	Phi * Mn =	470.92 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.47 in.	Mu =	162.37 in-k
Ultimate Defl.=	3.46 in.	Phi * Mn =	465.14 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.26 in.	Mu =	310.67 in-k
Ultimate Defl.=	6.58 in.	Phi * Mn =	469.29 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.38 in.	Mu =	147.03 in-k
Ultimate Defl.=	3.17 in.	Phi * Mn =	457.86 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.93 in.	Mu =	255.74 in-k
Ultimate Defl.=	5.54 in.	Phi * Mn =	453.68 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-34

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 61 - 30" Leg w/ 20'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 6,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30) (p = 1.09 * 1.00 * 15.20)	= 16.57 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 3.50 feet
Design height, distance between floor and roof.	= 36.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 435.00 # / ft.
Uniform live load on design section	= 725.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 27,479.00 # / ft.
Total design load on design section	= 41,979.00 # / ft.
Steel ratio (As / (b * d))	= 0.0171
Modulus of rupture (7.5 * (f'c^.5))	= 580.95 psi.
Modulus of elasticity (Concrete)	= 4,415.20 ksi.
n = Es / Ec	= 6.57
Cracking Moment	= 209.87 in.-k
Delta Cracked	= 0.45 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.48 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.00798
Maximum axial stress (211.76 psi.) is less than .06 * f'c = 360.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-35

Description: Panel 61 - 30" Leg w/ 20'-0" Opening

6 Number 7 Bars
Results shown are for 30.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 62.80 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,246.76 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.05 in.	Mu	= 42.53 in-k
Ultimate Defl.=	0.24 in.	Phi * Mn	= 1,361.99 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 173.46 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,287.30 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.14 in.	Mu	= 124.49 in-k
Ultimate Defl.=	0.70 in.	Phi * Mn	= 1,385.31 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.60 in.	Mu	= 339.10 in-k
Ultimate Defl.=	1.91 in.	Phi * Mn	= 1,385.31 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.61 in.	Mu	= 459.91 in-k
Ultimate Defl.=	2.62 in.	Phi * Mn	= 1,355.93 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.96 in.	Mu	= 1,028.26 in-k
Ultimate Defl.=	5.84 in.	Phi * Mn	= 1,364.58 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.68 in.	Mu	= 421.24 in-k
Ultimate Defl.=	2.41 in.	Phi * Mn	= 1,342.50 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.43 in.	Mu	= 1,004.85 in-k
Ultimate Defl.=	5.72 in.	Phi * Mn	= 1,359.23 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.59 in.	Mu	= 385.44 in-k
Ultimate Defl.=	2.24 in.	Phi * Mn	= 1,313.09 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.18 in.	Mu	= 851.98 in-k
Ultimate Defl.=	4.98 in.	Phi * Mn	= 1,296.17 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-30-2024
Sheet No.: W-36

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 61 - 60" Leg w/ 20'-0" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 6,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 3.50 feet
esign height, distance between floor and roof.	= 36.50 feet
ixity coefficient	= 0.85
Tributary width for design loads	= 15.00 feet
Uniform dead load on design section	= 435.00 # / ft.
Uniform live load on design section	= 725.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 32,974.80 # / ft.
Total design load on design section	= 50,374.80 # / ft.
Steel ratio (As / (b * d))	= 0.0073
Modulus of rupture (7.5 * (f'c^.5))	= 580.95 psi.
Modulus of elasticity (Concrete)	= 4,415.20 ksi.
n = Es / Ec	= 6.57
Cracking Moment	= 419.73 in.-k
Delta Cracked	= 0.45 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.48 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.02137
Maximum axial stress (127.06 psi.) is less than .06 * f'c = 360.00 psi.
Phi Mn is greater than M cracked for all cases.

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Description: Panel 61 - 60" Leg w/ 20'-0" Opening

7 Number 6 Bars
Results shown are for 60.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 75.36 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,145.90 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu	= 51.07 in-k
Ultimate Defl.=	0.24 in.	Phi * Mn	= 1,305.63 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 208.15 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,201.85 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.08 in.	Mu	= 148.87 in-k
Ultimate Defl.=	0.69 in.	Phi * Mn	= 1,338.22 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu	= 405.50 in-k
Ultimate Defl.=	1.89 in.	Phi * Mn	= 1,338.22 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.25 in.	Mu	= 552.71 in-k
Ultimate Defl.=	2.64 in.	Phi * Mn	= 1,297.17 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.61 in.	Mu	= 1,234.21 in-k
Ultimate Defl.=	5.84 in.	Phi * Mn	= 1,309.25 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.27 in.	Mu	= 507.01 in-k
Ultimate Defl.=	2.45 in.	Phi * Mn	= 1,278.46 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.16 in.	Mu	= 1,207.05 in-k
Ultimate Defl.=	5.74 in.	Phi * Mn	= 1,301.78 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.25 in.	Mu	= 464.83 in-k
Ultimate Defl.=	2.30 in.	Phi * Mn	= 1,237.58 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.96 in.	Mu	= 1,027.72 in-k
Ultimate Defl.=	5.17 in.	Phi * Mn	= 1,214.13 in-k

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-25-2024
Sheet No.: W-28

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Panel 94 - 62" Leg w/ 14'-8" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure catagory (B, C or D)	---> B
Lamda based on a mean roof height	= 38.50 feet
Net design wind pressure based on inputed trib area	= 100.00 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lamda * Iw * P net 30)	= 16.57 psf.
(p = 1.09 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.70 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.68 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 2.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 503.00 # / ft.
Uniform live load on design section	= 838.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 8.25 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 26,539.55 # / ft.
Total design load on design section	= 43,302.05 # / ft.
Steel ratio (As / (b * d))	= 0.0071
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 395.94 in.-k
Delta Cracked	= 0.50 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controled for all load cases, minimum steel strain is 0.01849
Maximum axial stress (106.55 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Bldg B
Job No. : 21-41
Engr: : DAV
Date: : 1-25-2024
Sheet No.: W-39

Description: Panel 94 - 62" Leg w/ 14'-8" Opening

7 Number 6 Bars
Results shown are for 62.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 72.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,136.16 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu	= 48.84 in-k
Ultimate Defl.=	0.27 in.	Phi * Mn	= 1,262.76 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 200.52 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,189.20 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.09 in.	Mu	= 144.59 in-k
Ultimate Defl.=	0.79 in.	Phi * Mn	= 1,297.20 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.28 in.	Mu	= 384.48 in-k
Ultimate Defl.=	2.10 in.	Phi * Mn	= 1,297.20 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.28 in.	Mu	= 513.73 in-k
Ultimate Defl.=	2.88 in.	Phi * Mn	= 1,258.27 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.74 in.	Mu	= 1,140.09 in-k
Ultimate Defl.=	6.35 in.	Phi * Mn	= 1,266.70 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.30 in.	Mu	= 468.01 in-k
Ultimate Defl.=	2.65 in.	Phi * Mn	= 1,240.52 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

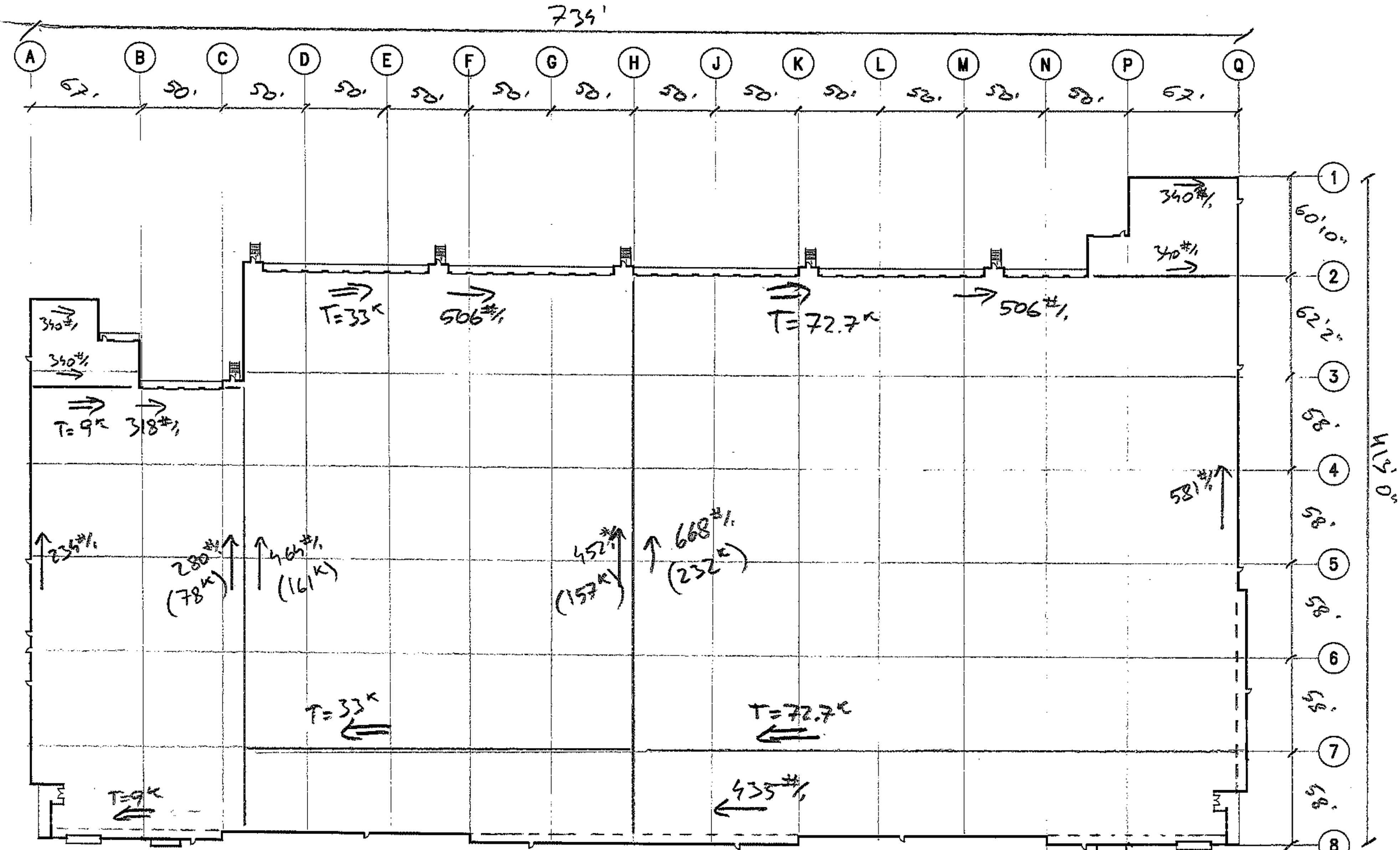
Service Defl. =	2.32 in.	Mu	= 1,112.50 in-k
Ultimate Defl.=	6.23 in.	Phi * Mn	= 1,259.61 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.28 in.	Mu	= 429.37 in-k
Ultimate Defl.=	2.48 in.	Phi * Mn	= 1,207.05 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.09 in.	Mu	= 948.57 in-k
Ultimate Defl.=	5.56 in.	Phi * Mn	= 1,187.85 in-k



L-1

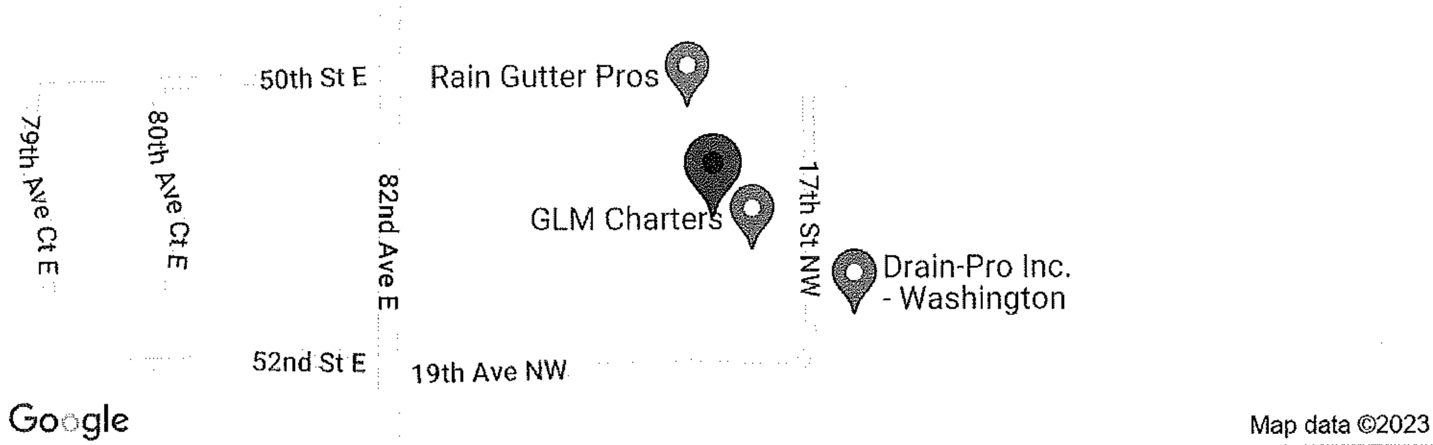
K:\21\21-41 Freeman Logistics - Bldg B\0DWG\ARCH\23-8-9 pdf & cad\Vector-Freeman Rd Logistics South - Standard\xrf\B-A1.dwg, Model, 8/26/2023 12:15:33 PM, Dmitri, 1:1000

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

L-2

OSHPD

Latitude, Longitude: 47.21082380, -122.31747047



Date	8/29/2023, 9:38:59 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.287	MCE_R ground motion. (for 0.2 second period)
S_1	0.443	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.287	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.858	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.287	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.408	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.443	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.493	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.545	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.899	Mapped value of the risk coefficient at a period of 1 s
C_V	1.357	Vertical coefficient

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JOB
SHEET No. L-3 OF
CALCULATED BY
DATE
SCALE

CALCULATION OF SEISMIC DESIGN PARAMETERS BASED ON IBC 2018 & ASCE 7-16

(For Site Class A, B, C, D, D-Default, E with period less than 0.50 seconds)

Seismic Criteria:

Site Specific Report		Yes	
Risk Category, I, II, III or IV (Table 1.5-1 of ASCE 7-16)	=	II	
Seismic importance factor, I_e (Table 1.5-2 of ASCE 7-16)	=	1.00	
Mapped spectral response for short periods, S_S (Site specific)	=	128.70	%
Mapped spectral response acceleration at a period of 1 second, S_1 (Site specific)	=	44.30	%
Site soil class (From soils report)	=	D	
Response modification factor, R (Table 12.2-1 of ASCE 7-16)	=	5.00	
Ω_o (Table 12.2-1 of ASCE 7-16, footnote b for flexible diaphragms)	=	2.00	
C_t (Table 12.8-2 of ASCE 7-16)	=	0.02	
x (Table 12.8-2 of ASCE 7-16)	=	0.75	
Average roof height	=	38.00	ft.
Long period transition period, T_L (Figure 22-14 of ASCE 7-16)	=	6.00	
$T_a = C_t * (h_n)^x$	=	0.306	Sec.
F_a (Table 1613.2.3(1) of IBC 2018)	=	1.000	
F_v (Table 1613.2.3(2) of IBC 2018)	=	1.857	
Maximum spectral response acceleration at short periods, $S_{MS} = F_a * S_S$	=	1.287	
Maximum spectral response acceleration at 1-second, $S_{M1} = F_v * S_1$	=	0.823	
Design spectral response acceleration at short periods, $S_{DS} = 2/3 * S_{MS}$	=	0.858	
Design spectral response acceleration at 1-second, $S_{D1} = 2/3 * S_{M1}$	=	0.548	
Seismic Design Category based on short period response acceleration	=	D	
Seismic Design Category based on 1-second period response acceleration	=	D	
Seismic Design Category, choose most severe of above two.	=	D	
$C_S = S_{DS} / (R/ I_e)$	=	0.172	Controls
If $T < T_L$ then $C_{s(max)} = S_{D1} / (T * (R/Ie))$	=	0.358	
If $T > T_L$ then $C_{S(max)} = S_{D1} * T_L / (T^2 * (R / Ie))$	=	N.A.	
$C_{S(min)} = 0.044 * S_{DS} * I_e$ => 0.01	=	0.038	
If S_1 is equal to or greater than 0.6g, then $C_{S(min)} = 0.50 * S_1 / (R/Ie)$	=	N.A.	

Seismic Design Force

$V = C_S * W$ = 0.172 W

Diaphragm Shear force ASCE 7-16 section 12.10.1.1

$F_{px\ (min)} = 0.2 * S_{DS} * I_e * W_{PX}$ = 0.172 W_{PX}

$F_{px\ (max)} = 0.4 * S_{DS} * I_e * W_{PX}$ = 0.343 W_{PX}

File:Seismic cover ASCE 7-16.xls, modified 04-05-21

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JOB
SHEET NO. L-4 OF
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SCALE

BASE SHEAR

ROOF A = 254610 ft²
SOLAR A = 43500 ft²

PERIMETER = 2402'

ROOF wt = 254610(17) + 43500(5) = 4546^k
WALL wt = 2402(150)(9.25/12 + 9.25/12)/2(40.5') = 10032
14578^k

14578(0.172) = 2507^k

L-5

ROOF DIAPHRAGM NAILING DIAGRAM

MARK	SHEATHING	STIFFENER AT PLYWOOD JOINT	CONTINUOUS EDGE	'OTHER' EDGE	ALLOWABLE SHEAR
I	$^{15}/_{32}$"	2x6	6"o.c.	6"o.c.	320#/FT
II	$^{15}/_{32}$"	2x6	4"o.c.	4"o.c.	425#/FT
III	$^{15}/_{32}$"	2x6	3"o.c.	3"o.c.	568#/FT
IV	$^{15}/_{32}$"	3x6	3"o.c.	3"o.c.	640#/FT
V	$^{15}/_{32}$"	3x6	2½"o.c. STAGGERED	2½"o.c. STAGGERED	720#/FT

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LATERAL - TRANSVERSE DIRECTION

$F_p = 0.172(0.7)W_c = 0.12W_c$

ROOF = $(15+2)(333 \times 67 - 25^2)$ = 369
PANEL = $150(67')(38.5/2 + 2')[7.25 + 9.25]/12$ = 294
662 × 0.12 = 79.5^k

$W = \frac{79.5^k}{67'} = 1.19^{k/'}$

ROOF = $(15+2)(62.5' \times 279')$ = 296
SOLAR = $5\ (62.5' \times 174)$ = 54
PANEL = $150(9.25/12)(62.5)(36/2 + 38.5/2 + 2)$ = 284
634 × 0.12 = 76.1^k

$W = \frac{76.1}{62.5} = 1.22^{k/'}$

ROOF = $(15+2)(137.5' \times 347.17')$ = 812 | 812
SOLAR = $5\ (174' \times 137.5)$ = | 120^k
PANEL = $150(9.25/12)(137.5')(36/2 + 38.5/2 + 2)$ = 624 | 624
1436 | 1556^k

$W_1 = \frac{1436(0.12)}{137.5} = 1.25^{k/'}$ $W_2 = \frac{1556(0.12)}{137.5} = 1.36^{k/'}$

ROOF = $(15+2)(50 \times 352.17')$ = 299 | 299
SOLAR = $5\ (50 \times 174)$ = | 43.5
PANEL = $150(9.25/12)(50')(36/2 + 38.5/2 + 2)$ = 227 | 227
526^k | 570^k

$W_1 = \frac{526(0.12)}{50'} = 1.26^{k/'}$ $W_2 = \frac{570(0.12)}{50} = 1.37^{k/'}$

ROOF = $(15+2)(92 \times 413 - 25' \times 35.83')$ = 631
PANEL = $150(9.25/12)(92')(38.5/2 + 2) \times 2$ = 452
1083 × 0.12 = 130^k

$W = \frac{130^k}{92'} = 1.41^{k/'}$

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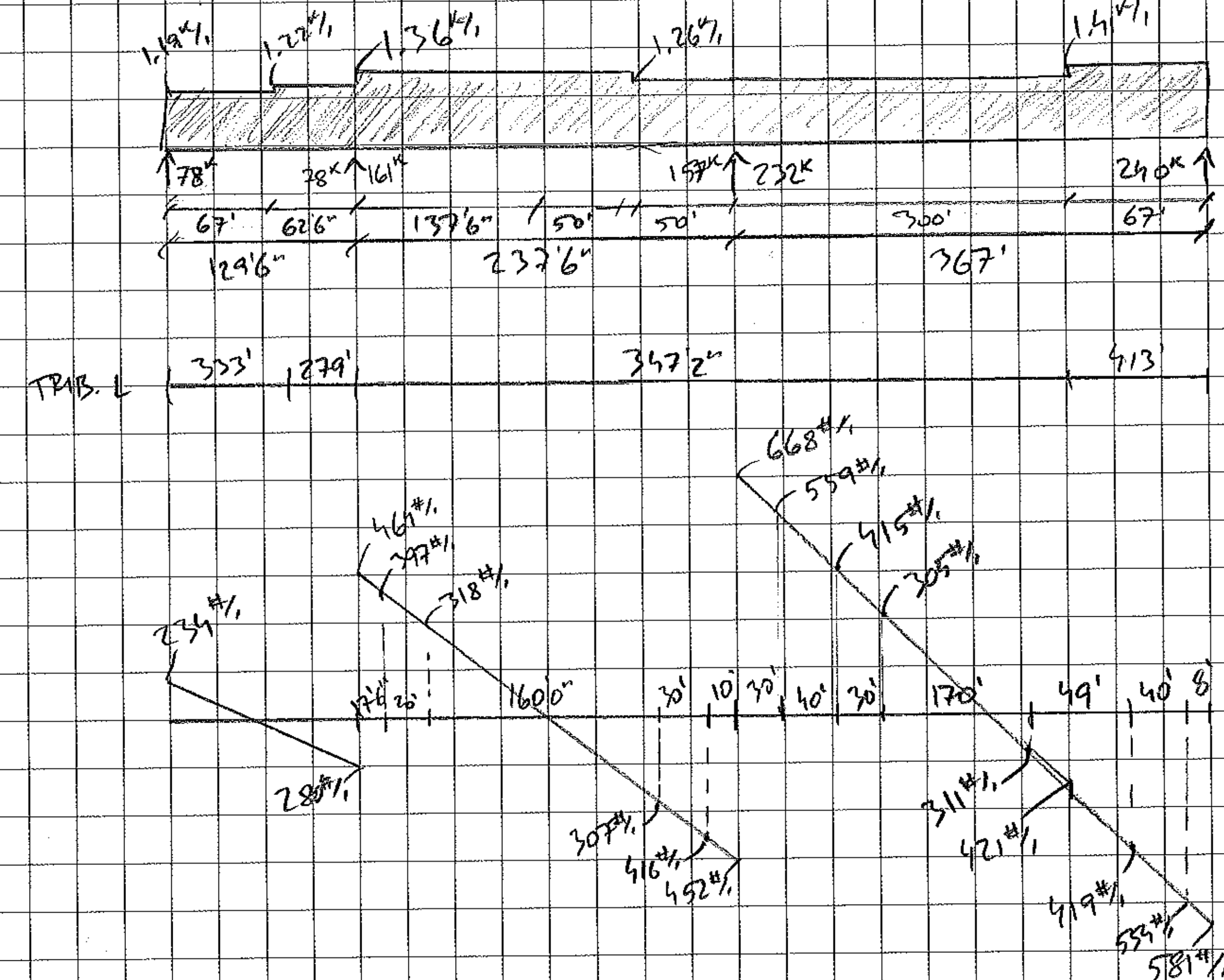
LATERAL LOADS - TRANSVERSE DIRECTION



M_{max}:	2524 k'	$\frac{9527 \text{ k'}}{294.167'}$	$\frac{21382 \text{ k'}}{294.167'}$
F	9.05 k	= 32.4 k	= 72.7 k

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JOB
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SCALE

LATERAL LOADS — LONGITUDINAL DIRECTION

ROOF = (15+2)(62.17' × 604.5') = 645^K
PANEL = 150 (9.25"/12)(62.17')(38.5'/2 + 2')×2 = 305
957 × 0.12 = 114^K

$w = \frac{114^K}{62.17'} = 1.83^K/'$

ROOF = (17)(58' × 734') = 724
PANEL = 150 (9.25/12)(58')(38.5/2 + 2)×2 = 285
1009 × 0.12 = 121

$w = \frac{121^K}{58'} = 2.09^K/'$

ROOF = 17 (174 × 734') = 2171
SOLAR = 5 (174 × 250') = 218
PANEL = 150 (9.25/12)(174')(38.5/2 + 2)×2 = 855
3244 × 0.12 = 389^K

$w = \frac{389^K}{174} = 2.24^K/'$

ROOF = 17 (60.83' × 92' − 25 × 35.83') = 79.9^K
PANEL = 150 (9.25/12)(60.83')(38.5/2 + 2)×2 = 299
379^K × 0.12 = 45.5^K

$w = \frac{45.5^K}{60.83'} = 0.75^K/'$

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JOB

SHEET NO. L-9 OF

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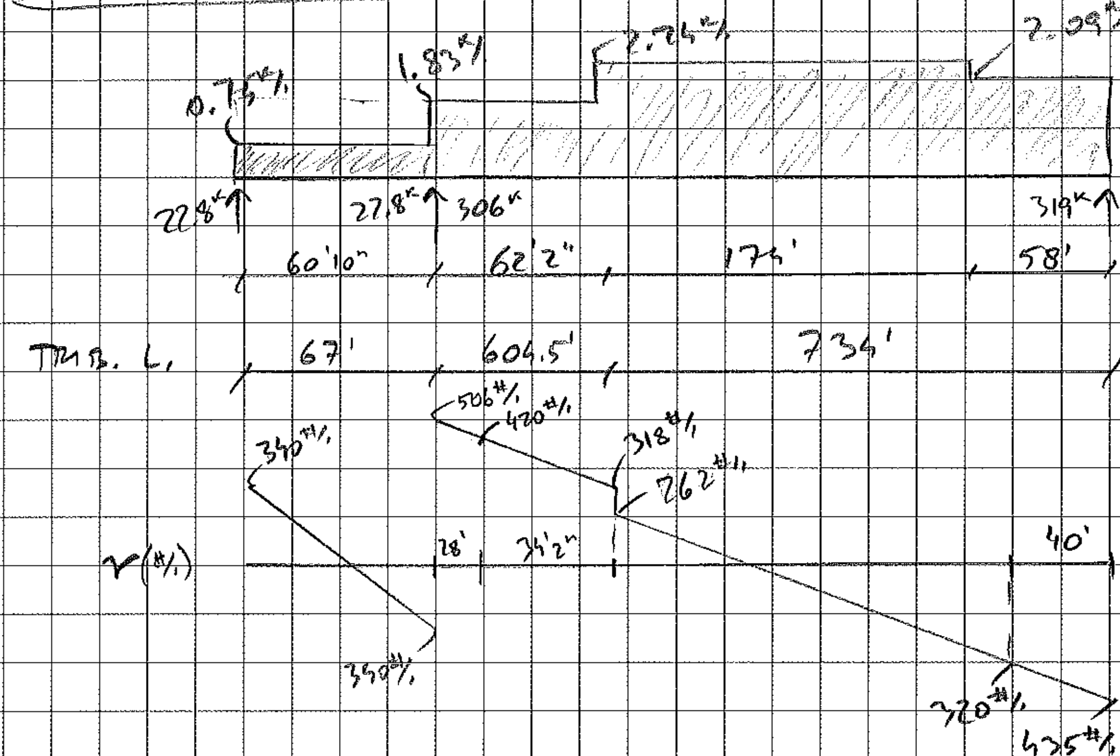
DATE

SCALE

LATERAL LOADS - LONGITUDINAL DIR.



$M_{max} = 347^k$

$T = \frac{347^k}{67'} = 5^k$

$M_{max} = 23708^{\#/'}$

$T = \frac{23708}{734'} = 32.2^k$

Panel Forces - Grid 2

V_{in} 506 lb/ft V_{DS} 846 lb/ft

Panel	Rigidity	L (ft)	Drag (k)	V_{out} (k)	$V_{out,tot}$ (k)	Drag (k)	Em (k)	As (in^2)	
DS	0	12.5	6	0.00	0.0	6	18.1	0.33	
13	10.4	25	19	-16.99	-17.0	2	5.7	0.10	
14	10.4	25	32	-16.99	-34.0	-2	-6.7	0.12	
15	10.4	25	44	-16.99	-51.0	-7	-19.2	0.35	
16	10.4	25	57	-16.99	-68.0	-11	-31.6	0.58	
DS	0	12.5	63	0.00	-68.0	-5	-13.5	0.25	
18	10.4	25	76	-16.99	-85.0	-9	-25.9	0.48	
19	10.4	25	89	-16.99	-102.0	-13	-38.3	0.71	
20	10.4	25	101	-16.99	-119.0	-18	-50.7	0.94	CHORD CONTROLLED As = 5-#6
21	10.4	25	114	-16.99	-135.9	-22	-63.1	1.17	
DS	0	12.5	120	0.00	-135.9	-16	-45.1	0.83	
23	10.4	25	133	-16.99	-152.9	-20	-57.5	1.06	
24	10.4	25	145	-16.99	-169.9	-24	-69.9	1.29	
25	10.4	25	158	-16.99	-186.9	-29	-82.3	1.52	
26	10.4	25	171	-16.99	-203.9	-33	-94.7	1.75	
DS	0	12.5	177	0.00	-203.9	-27	-76.6	1.42	
28	10.4	25	190	-16.99	-220.9	-31	-89.0	1.65	
29	10.4	25	202	-16.99	-237.9	-36	-101.4	1.88	
30	10.4	25	215	-16.99	-254.9	-40	-113.9	2.11	6-#6
31	10.4	25	228	-16.99	-271.9	-44	-126.3	2.34	6-#6
DS	0	12.5	234	0.00	-271.9	-38	-108.2	2.00	6-#6
33	10.4	25	247	-16.99	-288.9	-42	-120.6	2.23	6-#6
34	10.4	25	259	-16.99	-305.9	-47	-133.0	2.46	6-#6
DS	0	92	306	0.00	-305.9	0	0.0	0.00	
	187.2								

Panel Forces - Grid 3

V_{in} 318 lb/ft V_{DS} 658 lb/ft

Panel	Rigidity	L (ft)	Drag (k)	V_{out} (k)	$V_{out,tot}$ (k)	Drag (k)	Em (k)	As (in^2)	
DS	0	67	44	0.00	0.0	44	126.0	2.33	6-#6
6	10.4	25	52	-31.98	-32.0	20	57.3	1.06	6-#6
7	10.4	25	60	-31.98	-64.0	-4	-11.4	0.21	
DS	0	12.5	64	0.00	-64.0	0	0.0	0.00	
	20.8								

GRID 2

$T = 72.7^k$ $T_u = 104^k$

$A_{s,req} = 1.92 in^2$ — 5-#6

Panel Forces - Grid A

V_{in} 234 lb/ft

Panel	Rigidity	L (ft)	Drag (k)	V_{out} (k)	$V_{out,tot}$ (k)	Drag (k)	Em (k)	As (in^2)
DS	0	33	8	0.00	0.0	8	22.1	0.41
89	9.1	25	14	-4.38	-4.4	9	26.3	0.49
90	9.1	25	19	-4.38	-8.8	11	30.5	0.56
91	14.2	25	25	-6.83	-15.6	10	27.7	0.51
92	14	25	31	-6.73	-22.3	9	25.2	0.47
93	14.6	25	37	-7.02	-29.3	8	21.8	0.40
94	14.2	25	43	-6.83	-36.2	7	19.0	0.35
95	14.4	25	49	-6.93	-43.1	6	15.9	0.29
96	14.6	25	55	-7.02	-50.1	4	12.6	0.23
97	14.6	25	60	-7.02	-57.1	3	9.2	0.17
98	14.4	25	66	-6.93	-64.1	2	6.2	0.11
99	14.2	25	72	-6.83	-70.9	1	3.4	0.06
100	14.6	25	78	-7.02	-77.9	0	0.0	0.00
	162							

CHORD CONTROLLED USE 4-#5

Panel Forces - Grid Q

V_{in} 581 lb/ft

Panel	Rigidity	L (ft)	Drag (k)	V_{out} (k)	$V_{out,tot}$ (k)	Drag (k)	Em (k)	As (in^2)
42	22.8	30	17	-25.52	-25.5	-8	-23.1	0.43
43	14.6	25	32	-16.34	-41.9	-10	-28.3	0.52
44	14.6	25	46	-16.34	-58.2	-12	-33.5	0.62
45	14.6	25	61	-16.34	-74.5	-14	-38.7	0.72
46	14.2	25	76	-15.89	-90.4	-15	-42.6	0.79
47	14.4	25	90	-16.12	-106.5	-16	-47.1	0.87
48	14.6	25	105	-16.34	-122.9	-18	-52.3	0.97
49	14.6	25	119	-16.34	-139.2	-20	-57.5	1.06
50	14.4	25	134	-16.12	-155.3	-22	-62.0	1.15
51	14.2	25	148	-15.89	-171.2	-23	-65.9	1.22
52	14.6	25	163	-16.34	-187.6	-25	-71.1	1.32
53	14.4	25	177	-16.12	-203.7	-26	-75.7	1.40
54	14.2	25	192	-15.89	-219.6	-28	-79.6	1.47
55	9.1	25	206	-10.18	-229.8	-24	-67.2	1.24
56	9.1	25	221	-10.18	-240.0	-19	-54.8	1.01
DS	0	33	240	0.00	-240.0	0	0.0	0.00
	214.4							

CHORD CONTROLLED USE 4-#6

4-#6

CHORD: 32.2^k $T_n \approx 46^k$

$A_s = 0.85\ in^2 \rightarrow 3\text{-}\#5$

L-12

Panel Forces - Grid 8

V_{in} 435 lb/ft

Panel	Rigidity	L (ft)	Drag (k)	V_{out} (k)	$V_{out,tot}$ (k)	Drag (k)	Em (k)	As (in^2)	
DS	0	37.5	16	0.00	0.0	16	46.6	0.86	
60	8.2	27.5	28	-8.32	-8.3	20	57.0	1.06	4-#5
61	14.4	25	39	-14.61	-22.9	16	46.3	0.86	
62	18	27	51	-18.27	-41.2	10	27.7	0.51	
63	10.5	25	62	-10.66	-51.9	10	28.3	0.52	
64	10.5	25	73	-10.66	-62.5	10	28.9	0.54	
65	10.5	25	84	-10.66	-73.2	10	29.6	0.55	
66	10.3	25	94	-10.45	-83.6	11	30.8	0.57	
67	10.5	25	105	-10.66	-94.3	11	31.4	0.58	
68	10.5	25	116	-10.66	-104.9	11	32.0	0.59	
69	14.4	25	127	-14.61	-119.6	7	21.3	0.39	
70	14.2	25	138	-14.41	-134.0	4	11.2	0.21	
71	14.6	25	149	-14.82	-148.8	0	0.0	0.00	
72	14.4	25	160	-14.61	-163.4	-4	-10.7	0.20	
73	14.4	25	171	-14.61	-178.0	-7	-21.4	0.40	
74	14.6	25	181	-14.82	-192.8	-11	-32.7	0.61	
75	14.2	25	192	-14.41	-207.2	-15	-42.8	0.79	
76	14.4	25	203	-14.61	-221.9	-19	-53.5	0.99	
77	10.5	25	214	-10.66	-232.5	-18	-52.8	0.98	
78	10.5	25	225	-10.66	-243.2	-18	-52.2	0.97	
79	10.3	25	236	-10.45	-253.6	-18	-51.0	0.94	
80	10.5	25	247	-10.66	-264.3	-18	-50.4	0.93	
81	10.5	25	258	-10.66	-274.9	-17	-49.8	0.92	
82	10.5	25	268	-10.66	-285.6	-17	-49.1	0.91	
83	10.6	22	278	-10.76	-296.4	-18	-52.5	0.97	
84	14.4	25	289	-14.61	-311.0	-22	-63.2	1.17	4-#5
85	8.2	27.5	301	-8.32	-319.3	-18	-52.8	0.98	
DS	0	42.5	319	0.00	-319.3	0	0.0	0.00	
	314.6								

Chord = N/A

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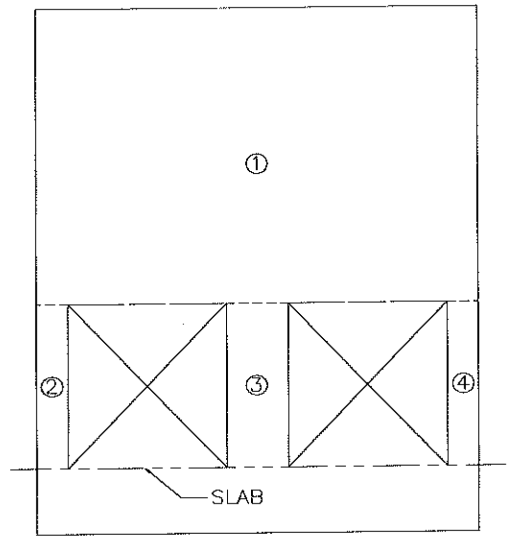
JOB
SHEET No. L-13 OF
CALCULATED BY
DATE
SCALE JOB No.

Description: Panel 6

Overall Wall Height	36.00	Feet
Overall Wall Length	25.00	Feet
Opening One Height	10.00	Feet
Opening One Length	9.00	Feet
Distance From left edge to opening one	1.75	Feet
Opening Two Height	10.00	Feet
Opening Two Length	9.00	Feet
Distance between openings	3.50	Feet
Wall thickness	9.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	36.00	Ft.	25.00	Ft.	1.440	C	0.040	25.111
Deduct Bottom Strip	10.00	Ft.	25.00	Ft.	0.400	F	0.003	323.106
							0.037	

Add pier 2	10.00	Ft.	1.75	Ft.	5.714	F	0.499	2.005
Add pier 3	10.00	Ft.	3.50	Ft.	2.857	F	0.078	12.805
Add pier 4	10.00	Ft.	1.75	Ft.	5.714	F	0.499	2.005
Rigidity Piers 2, 3 & 4								16.814
Deflection of Piers 2, 3 & 4 = 1 / Rigidity $_{2, 3 \& 4}$							0.059	

Wall Deflection = Solid wall - bottom strip + piers 2, 3, & 4	0.096	
Wall Rigidity = 1 / wall deflection		10.395

Panel Weight 83.250 kip

File: Rigidity2.xls, modified 4-2-08

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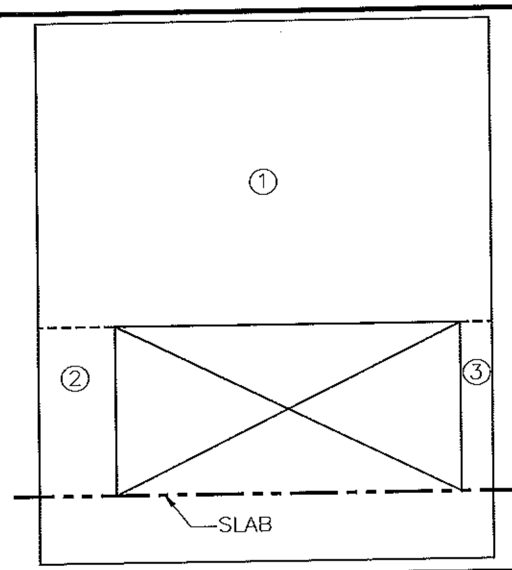
JOB
SHEET No. 1-14 OF
CALCULATED BY
DATE
SCALE JOB No.

Description: Panel 42

Overall Wall Height	40.50	Feet
Overall Wall Length	30.00	Feet
Opening Height	7.12	Feet
Opening Length	3.33	Feet
Distance From left edge to opening	13.33	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.50	Ft.	30.00	Ft.	1.350	C	0.043	23.043
Deduct Bottom Strip	7.12	Ft.	30.00	Ft.	0.237	F	0.002	441.296
							0.041	

Add pier 2	7.12	Ft.	13.33	Ft.	0.534	F	0.005	182.465
Add pier 3	7.12	Ft.	13.34	Ft.	0.534	F	0.005	182.529
Sum of Rigidities for piers 2, & 3								364.993
Deflection of Piers 2, & 3 = 1 / $R_{2, \& 3}$							0.003	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.044	
Wall Rigidity = 1 / wall deflection		22.794

File: Rigidity2.xls, modified 4-2-08

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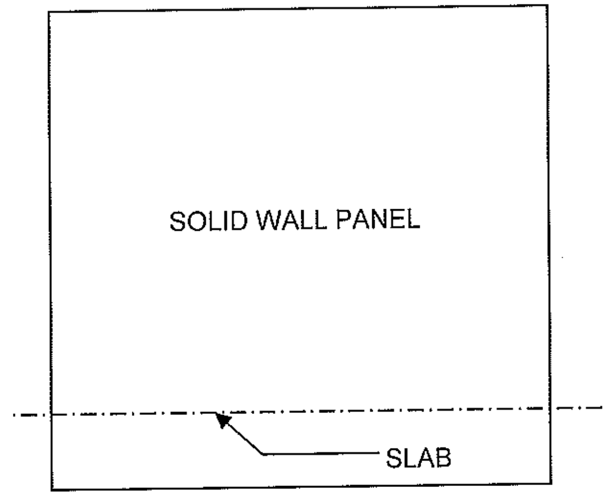
JOB ______
SHEET No. L-15 OF ______
CALCULATED BY ______
DATE ______
SCALE ______ JOB No. ______

Description: Panel 43

Overall Wall Height	40.50	Feet
Overall Wall Length	25.00	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.50	Ft.	25.00	Ft.	1.620	C	0.068	14.639

Panel Weight 91.758 kip

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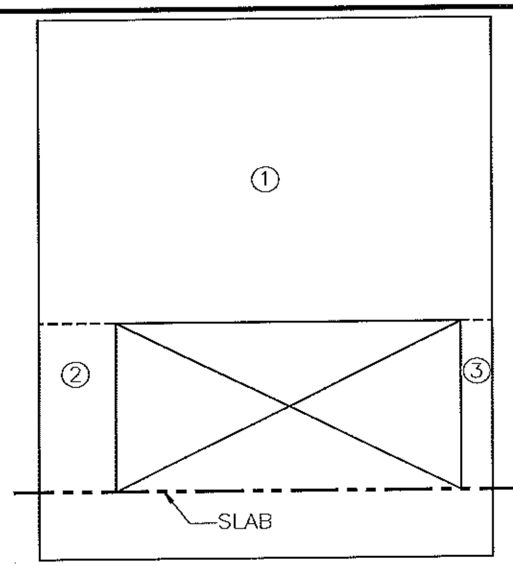
JOB ______
SHEET No. L-16 OF ______
CALCULATED BY ______
DATE ______
SCALE ______ JOB No. ______

Description: Panel 46

Overall Wall Height	40.50	Feet
Overall Wall Length	25.00	Feet
Opening Height	7.12	Feet
Opening Length	3.33	Feet
Distance From left edge to opening	10.83	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.50	Ft.	25.00	Ft.	1.620	C	0.068	14.639
Deduct Bottom Strip	7.12	Ft.	25.00	Ft.	0.285	F	0.003	364.789
							0.066	

Add pier 2	7.12	Ft.	10.83	Ft.	0.657	C	0.010	103.012
Add pier 3	7.12	Ft.	10.84	Ft.	0.657	C	0.010	103.078
Sum of Rigidities for piers 2, & 3								206.090
Deflection of Piers 2, & 3 = 1 / $R_{2, \& 3}$							0.005	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.070	
Wall Rigidity = 1 / wall deflection		14.200

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JOB ______

SHEET No. L-17 OF ______

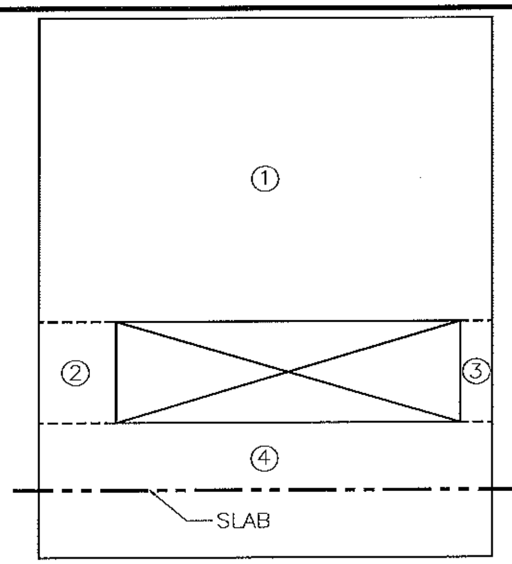
CALCULATED BY ______

DATE ______

SCALE ______ JOB No. ______

Description: Panel 47

Overall Wall Height	40.50	Feet
Overall Wall Length	25.00	Feet
Height to bottom of opening	24.00	Feet
Opening Height	5.00	Feet
Opening Length	8.00	Feet
Distance From left edge to opening	8.50	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.50	Ft.	25.00	Ft.	1.620	C	0.068	14.639
Deduct Middle Strip	5.00	Ft.	25.00	Ft.	0.200	F	0.002	526.484
							0.066	

Add pier 2	5.00	Ft.	8.50	Ft.	0.588	F	0.006	162.633
Add pier 3	5.00	Ft.	8.50	Ft.	0.588	F	0.006	162.633
Sum of Rigidities for piers 2, & 3								325.266
Deflection of Piers 2, & 3 = 1 / $R_{2,\&3}$							0.003	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.069	
Wall Rigidity = 1 / wall deflection		14.392

Panel Weight 88.133 kip

File: Rigidity2.xls, modified 4-2-08

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JOB ______________________

SHEET No. L-18 OF ______

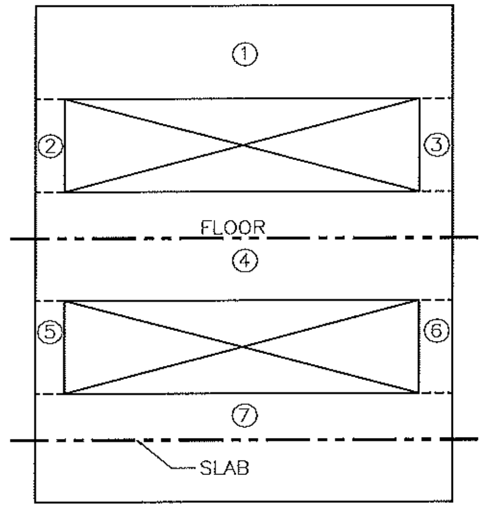
CALCULATED BY ______________________

DATE ______________________

SCALE ______________ JOB No. ______

Description: Panel 60

Overall Wall Height	40.00 Feet
Overall Wall Length	27.50 Feet
Height to bottom of lower opening	0.00 Feet
Height of lower opening	10.00 Feet
Length of lower opening	20.00 Feet
Distance from left edge to lower opening	2.50 Feet
Height to bottom of upper opening from slab	14.00 Feet
Height of upper opening	10.00 Feet
Length of upper opening	11.00 Feet
Distance from left edge to upper opening	2.50 Feet
Wall thickness	9.25 inches
Modulus of Elasticity (E)	4415.20 Ksi
F'c of concrete	6000.00 Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.00	Ft.	27.50	Ft.	1.455	C	0.041	24.495
Deduct upper window$	10.00	Ft.	27.50	Ft.	0.364	F	0.003	358.568
Deduct lower window $	10.00	Ft.	27.50	Ft.	0.364	F	0.003	358.568
							0.035	

Add pier 2	10.00	Ft.	2.50	Ft.	4.000	F	0.186	5.374
Add pier 3	10.00	Ft.	14.00	Ft.	0.714	F	0.006	162.888
Sum of Rigidities for piers 2 & 3								168.261
Deflection of Piers 2 & 3 = 1 / $R_{2\&3}$							0.006	

Add pier 5	10.00	Ft.	2.50	Ft.	4.000	C	0.656	1.524
Add pier 6	10.00	Ft.	5.00	Ft.	2.000	C	0.093	10.748
Sum of Rigidities for piers 5 & 6								12.271
Deflection of Piers 5 & 6 = 1 / $R_{5\&6}$							0.081	

Wall Deflection = Solid wall - upper & lower window strip + piers 2 & 3 + piers 5 & 6	0.123	
Wall Rigidity = 1 / wall deflection		8.151

Panel Weight 91.344 kip

File: Rigidity2.xls, modified 5-31-2012

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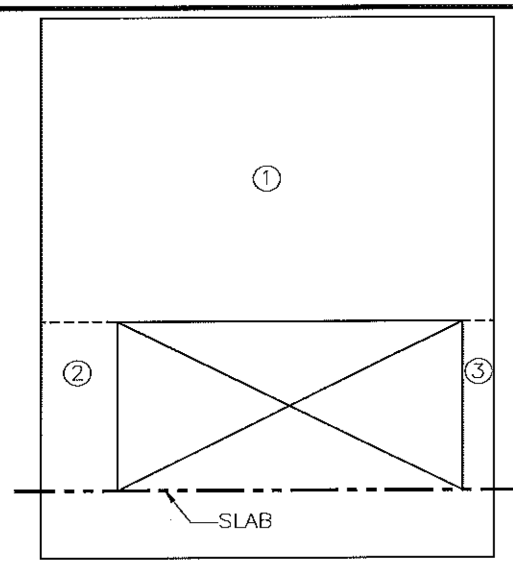
JOB ____
SHEET No. L-19 OF ____
CALCULATED BY ____
DATE ____
SCALE ____ JOB No. ____

Description: Panel 62

Overall Wall Height	40.00	Feet
Overall Wall Length	27.00	Feet
Opening Height	7.12	Feet
Opening Length	3.33	Feet
Distance From left edge to opening	2.50	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.00	Ft.	27.00	Ft.	1.481	C	0.055	18.343
Deduct Bottom Strip	7.12	Ft.	27.00	Ft.	0.264	F	0.003	395.457
							0.052	

Add pier 2	7.12	Ft.	2.50	Ft.	2.848	C	0.315	3.171
Add pier 3	7.12	Ft.	21.17	Ft.	0.336	C	0.004	275.677
Sum of Rigidities for piers 2, & 3								278.848
Deflection of Piers 2, & 3 = 1 / $R_{2, \& 3}$							0.004	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.056	
Wall Rigidity = 1 / wall deflection		17.994

File: Rigidity2.xls, modified 4-2-08

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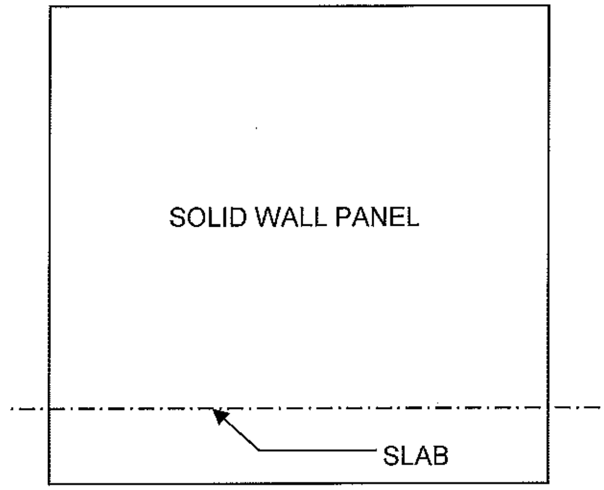
JOB ____
SHEET No. L-20 OF ____
CALCULATED BY ____
DATE ____
SCALE ____ JOB No. ____

Description: Panel 63

Overall Wall Height	46.00	Feet
Overall Wall Length	25.00	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	46.00	Ft.	25.00	Ft.	1.840	C	0.095	10.517

Panel Weight 104.219 kip

File: Rigidity2.xls, modified 4-2-08

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JOB ______

SHEET No. L-21 OF ______

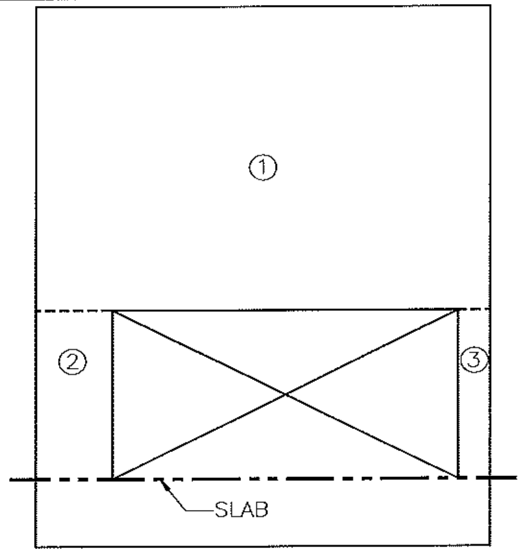
CALCULATED BY ______

DATE ______

SCALE ______ JOB No. ______

Description: Panel 66

Overall Wall Height	46.00	Feet
Overall Wall Length	25.00	Feet
Opening Height	7.12	Feet
Opening Length	3.33	Feet
Distance From left edge to opening	10.83	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	46.00	Ft.	25.00	Ft.	1.840	C	0.095	10.517
Deduct Bottom Strip	7.12	Ft.	25.00	Ft.	0.285	F	0.003	364.789
							0.092	

Add pier 2	7.12	Ft.	10.83	Ft.	0.657	C	0.010	103.012
Add pier 3	7.12	Ft.	10.84	Ft.	0.657	C	0.010	103.078
Sum of Rigidities for piers 2, & 3								206.090
Deflection of Piers 2, & 3 = 1 / $R_{2,\&3}$							0.005	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.097	
Wall Rigidity = 1 / wall deflection		10.288

File: Rigidity2.xls, modified 4-2-08

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JOB ____________________

SHEET No. L-22 ______ OF ______

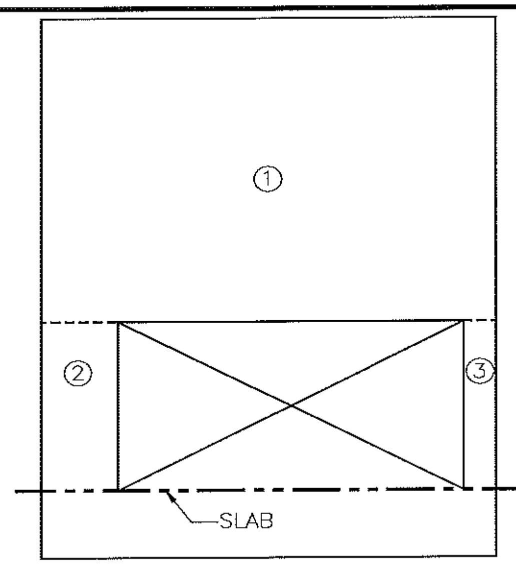
CALCULATED BY ____________________

DATE ____________________

SCALE ____________ JOB No. ____________

Description: Panel 83

Overall Wall Height	40.00	Feet
Overall Wall Length	22.00	Feet
Opening Height	7.12	Feet
Opening Length	3.33	Feet
Distance From left edge to opening	2.50	Feet
Wall thickness	7.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.00	Ft.	22.00	Ft.	1.818	C	0.092	10.852
Deduct Bottom Strip	7.12	Ft.	22.00	Ft.	0.324	F	0.003	318.571
							0.089	

Add pier 2	7.12	Ft.	2.50	Ft.	2.848	C	0.315	3.171
Add pier 3	7.12	Ft.	16.17	Ft.	0.440	C	0.005	192.549
Sum of Rigidities for piers 2, & 3								195.720
Deflection of Piers 2, & 3 = 1 / $R_{2, \& 3}$							0.005	

Wall Deflection = Solid wall - middle strip + piers 2 & 3	0.094	
Wall Rigidity = 1 / wall deflection		10.625

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JOB

SHEET No. L-23 OF

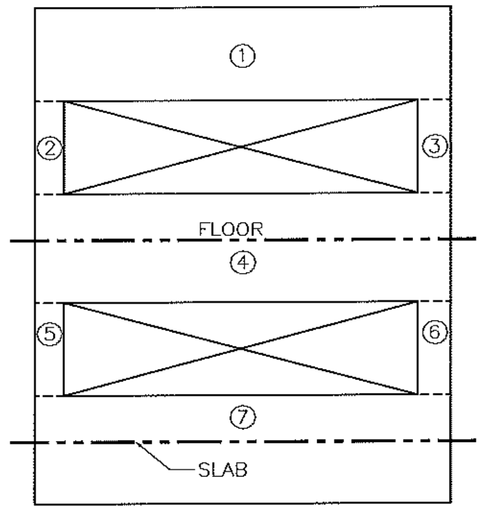
CALCULATED BY

DATE

SCALE JOB No.

Description: Panel 89

Overall Wall Height	42.50	Feet
Overall Wall Length	25.00	Feet
Height to bottom of lower opening	0.00	Feet
Height of lower opening	10.00	Feet
Length of lower opening	15.00	Feet
Distance from left edge to lower opening	5.00	Feet
Height to bottom of upper opening from slab	16.50	Feet
Height of upper opening	7.00	Feet
Length of upper opening	15.00	Feet
Distance from left edge to upper opening	5.00	Feet
Wall thickness	9.25	inches
Modulus of Elasticity (E)	4415.20	Ksi
F'c of concrete	6000.00	Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	42.50	Ft.	25.00	Ft.	1.700	C	0.061	16.500
Deduct upper windowS	7.00	Ft.	25.00	Ft.	0.280	F	0.002	473.815
Deduct lower window S	10.00	Ft.	25.00	Ft.	0.400	F	0.003	323.106
							0.055	

Add pier 2	7.00	Ft.	5.00	Ft.	1.400	F	0.017	58.814
Add pier 3	7.00	Ft.	5.00	Ft.	1.400	F	0.017	58.814
Sum of Rigidities for piers 2 & 3								117.628
Deflection of Piers 2 & 3 = 1 / $R_{2\,\&\,3}$							0.009	

Add pier 5	10.00	Ft.	5.00	Ft.	2.000	C	0.093	10.748
Add pier 6	10.00	Ft.	5.00	Ft.	2.000	C	0.093	10.748
Sum of Rigidities for piers 5 & 6								21.495
Deflection of Piers 5 & 6 = 1 / $R_{5\,\&\,6}$							0.047	

Wall Deflection = Solid wall - upper & lower window strip + piers 2 & 3 + piers 5 & 6	0.110	
Wall Rigidity = 1 / wall deflection		9.056

Panel Weight 93.367 kip

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JOB

SHEET No. L-24 OF

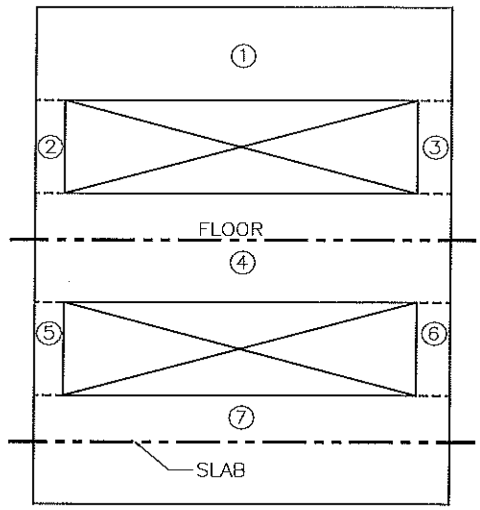
CALCULATED BY

DATE

SCALE JOB No.

Description: Panel 92

Overall Wall Height	40.50 Feet
Overall Wall Length	25.00 Feet
Height to bottom of lower opening	0.00 Feet
Height of lower opening	7.33 Feet
Length of lower opening	14.67 Feet
Distance from left edge to lower opening	5.17 Feet
Height to bottom of upper opening from slab	24.00 Feet
Height of upper opening	5.00 Feet
Length of upper opening	8.00 Feet
Distance from left edge to upper opening	8.50 Feet
Wall thickness	9.25 inches
Modulus of Elasticity (E)	4415.20 Ksi
F'c of concrete	6000.00 Psi



Pier	Height		Length		Ht/Length	Fixed (F) Cant (C)	Delta	Rigidity
Solid Wall	40.50	Ft.	25.00	Ft.	1.620	C	0.054	18.678
Deduct upper window$	5.00	Ft.	25.00	Ft.	0.200	F	0.001	671.721
Deduct lower window $	7.33	Ft.	25.00	Ft.	0.293	C	0.002	416.356
							0.050	

Add pier 2	5.00	Ft.	8.50	Ft.	0.588	F	0.005	207.497
Add pier 3	5.00	Ft.	8.50	Ft.	0.588	F	0.005	207.497
Sum of Rigidities for piers 2 & 3								414.995
Deflection of Piers 2 & 3 = 1 / $R_{2\,\&\,3}$							0.002	

Add pier 5	7.33	Ft.	5.17	Ft.	1.419	C	0.038	26.027
Add pier 6	7.33	Ft.	5.16	Ft.	1.420	C	0.038	25.978
Sum of Rigidities for piers 5 & 6								52.005
Deflection of Piers 5 & 6 = 1 / $R_{5\,\&\,6}$							0.019	

Wall Deflection = Solid wall - upper & lower window strip + piers 2 & 3 + piers 5 & 6	0.071	
Wall Rigidity = 1 / wall deflection		14.028

Panel Weight 100.007 kip

File: Rigidity2.xls, modified 5-31-2012