G.F. CONSULTING ENGINEERS, Inc.
STRUCTURAL ENGINEERING • THRESHOLD & SPECIAL INSPECTION • SBCCI CERTIFIED PLANS EXAMINERS
LICENSES: ALABAMA, COLORADO, FLORIDA, GEORGIA, NEW MEXICO, NORTH CAROLINA, SOUTH CAROLINA.
COA 9129

Structural Calculations

New Residence for 5900 LLC

5900 SW 114 Terrace
Pinecrest, FL

2020 Florida Building Code (7TH Edition)

Fresnel E Hernandez
Digitally signed by Fresnel E Hernandez
DN: CN=Fresnel E Hernandez, dnQualifier=A01410C000001861 36F2E3A000217BB, O=GF Consulting Engineers Inc., C=US
Date: 2023.06.15 15:04:46-04'00'



Fresnel E. Hernandez P.E./S.I./P.X.
PE# 46618

NOTE:
This set of calculations was prepared. under my direct supervision, including. the results of computer software generated. results, to the extent indicated on this set of calculations.

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Design Criteria

- FBC 2020 7th Edition
- Roof Live load 30 psf
- Roof A. Dead load 20 psf
- Floor Live load 40 psf
  Balconies LL 60 psf
- Floor A. Dead load 45 psf

- Soil Bearing Capacity:
  Q_{adm} = 2000 psf
- Wind per ASCE 7-16
  - 175 mph ultimate (136 mph ASD)
  - Exposure C
  - MRH 30'
  - Parapet 1'-6"
  - Risk Cat II
  - 3.5° 12 Slope
  - K_d = 0.85
  - GCpi = +/- 0.18



Wind Load Calculations

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
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COMPONENT AND CLADDING DESIGN
WIND PRESSURES (PSF)
ROOF SURFACE PRESSURES

Area	10 sf	20 sf	100 sf	200 sf					
Negative Zone 1	-86.0	-86.0	-68.5	-68.5					
Negative Zone 2e	-115.0	-105.6	-83.8	-74.3					
Negative Zone 2r	-149.8	-135.1	-100.7	-86.0					
Negative Zone 3	-147.3	-133.2	-100.5	-86.4					
Positive All Zones	51.1	44.1	27.9	27.9					
Overhang Zone 1	-104.5	-61.3	-116.2	-116.2					
Overhang Zone 2e	-133.6	-130.9	-124.7	-122.0					
Overhang Zone 2r	-168.4	-160.4	-141.6	-133.6					
Overhang Zone 3	-200.7	-178.6	-127.2	-105.0					

PARAPET SURFACE PRESSURES

SOLID PARAPET PRESSURE	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf		
CASE A: Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 2r :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0		
CASE B: Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0		
Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0		

WALL SURFACE PRESSURES

Area	10 sf	100 sf	200 sf	500 sf
NEGATIVE ZONE 4	-74.3	-64.1	-61.0	-56.9
NEGATIVE ZONE 5	-91.8	-71.3	-65.1	-56.9
POSITIVE ZONE 4 & 5	68.5	58.3	55.2	51.1

NOTES:
1) TABLE PRESSURES ARE FOR THE SQUARE FOOT (SF) TRIBUTARY AREA SHOWN. FOR OTHER TRIBUTARY AREAS, LINEARLY INTERPOLATE BETWEEN VALUES SHOWN ABOVE.
2) POSITIVE PRESSURES ACT TOWARD THE BUILDING. NEGATIVE PRESSURES ACT AWAY FROM THE BUILDING.
3) SEE DIAGRAMS FOR LOCATION OF ZONES.
4) PRESSURES SHOWN ARE ULTIMATE PRESSURES, MULTIPLY BY 0.6 FOR NOMINAL PRESSURES

a=3.0 ft

ASCE 7-16 Wind load Program (30.3 Part 1: Low-Rise Building Wall Components and Cladding)
Wall Components and Cladding *(Mean Roof Height h <= 60 ft)*

Project: LAMAS RESIDENCE -GUEST HOUSE
Prepared by: FEH of GFCE
E-mail: GFCE@GFCE.NET

Input:
ASCE 7-16 Strength Design Load & Allowable Stress Design
Wind Speed = 175 mph
Building Exposure = C (Open Terrain with Scattered Obstructions)
Internal Pressure Coefficient GCpi = 0.18 Enclosed Building
θ > Roof angle is greater than 10°
Kzt = 1.00
Kd = 0.85
Ke = 1.00
Mean Roof Height = 17 ft

Output:
Velocity Pressure Exposure Coefficient "Kz max." = 0.87
Per Table 26.10-1, page 268
LRFD uses wind speed Vult = 175 mph
Velocity Pressure (qh LRFD) = 57.98 psf = .00256*Kz*Kzt*Kd*Ke*V^2 lb/ft^2
Per eq.30.3-1, page 335
ASD uses wind speed Vasd[1] = 136 mph
Velocity Pressure (qh ASD) = 34.79 psf = .00256*Kz*Kzt*Kd*Ke*V^2*0.6 lb/ft^2

—	LRFD (Load Resistance Factor Design)			ASD (Allowable Stress Design)			(Per Figure 30.4-1, Page 335)		
Tributary Area Sq Feet	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	External Pressure Coefficients		
							Positive GCp	Negative Center GCp	Negative Corner GCp
10 ft²	68.4	-74.2	-91.6	41.0	-44.5	-55.0	1.00	-1.10	-1.40
20 ft²	65.3	-71.1	-85.4	39.2	-42.7	-51.3	0.95	-1.05	-1.29
50 ft²	61.3	-67.1	-77.3	36.8	-40.2	-46.4	0.88	-0.98	-1.15
100 ft²	58.2	-64.0	-71.1	34.9	-38.4	-42.7	0.82	-0.92	-1.05
200 ft²	55.1	-60.9	-65.0	33.1	-36.5	-39.0	0.77	-0.87	-0.94
500 ft²	51.0	-56.8	-56.8	30.6	-34.1	-34.1	0.70	-0.80	-0.80
200 ft²	55.1	-60.9	-65.0	33.1	-36.5	-39.0	0.77	-0.87	-0.94

Least Width = 24 ft a = 2.4 ft (Zone 5)

10% of least width or 0.4h, whichever is smaller but not less than 4% of least width or 3 ft.

Presented by CADDtools.com

ASCE 7-16 Wind load Program (30.3 Part 1: Low-Rise Building Wall Components and Cladding)
Wall Components and Cladding *(Mean Roof Height h <= 60 ft)*

Project: LAMAS RESIDENCE -MAIN HOUSE
Prepared by: FEH of GFCE
E-mail: GFCE@GFCE.NET

Input:
ASCE 7-16 Strength Design Load & Allowable Stress Design
Wind Speed = 175 mph
Building Exposure = C (Open Terrain with Scattered Obstructions)
Internal Pressure Coefficient GCpi = 0.18 Enclosed Building
θ > Roof angle is greater than 10°
Kzt = 1.00
Kd = 0.85
Ke = 1.00
Mean Roof Height = 30 ft

Output:
Velocity Pressure Exposure Coefficient "Kz max." = 0.98
Per Table 26.10-1, page 268
LRFD uses wind speed Vult = 175 mph
Velocity Pressure (qh LRFD) = 65.31 psf = .00256*Kz*Kzt*Kd*Ke*V^2 lb/ft^2
Per eq.30.3-1, page 335
ASD uses wind speed Vasd[1] = 136 mph
Velocity Pressure (qh ASD) = 39.18 psf = .00256*Kz*Kzt*Kd*Ke*V^2*0.6 lb/ft^2

—	LRFD (Load Resistance Factor Design)			ASD (Allowable Stress Design)			(Per Figure 30.4-1, Page 335)		
Tributary Area Sq Feet	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	External Pressure Coefficients		
							Positive GCp	Negative Center GCp	Negative Corner GCp
10 ft²	77.1	-83.6	-103.2	46.2	-50.2	-61.9	1.00	-1.10	-1.40
20 ft²	73.6	-80.1	-96.2	44.2	-48.1	-57.7	0.95	-1.05	-1.29
50 ft²	69.0	-75.5	-87.1	41.4	-45.3	-52.2	0.88	-0.98	-1.15
100 ft²	65.5	-72.1	-80.1	39.3	-43.2	-48.1	0.82	-0.92	-1.05
200 ft²	62.1	-68.6	-73.2	37.2	-41.2	-43.9	0.77	-0.87	-0.94
500 ft²	57.5	-64.0	-64.0	34.5	-38.4	-38.4	0.70	-0.80	-0.80
200 ft²	62.1	-68.6	-73.2	37.2	-41.2	-43.9	0.77	-0.87	-0.94

Least Width = 56 ft a = 5.6 ft (Zone 5)

10% of least width or 0.4h, whichever is smaller but not less than 4% of least width or 3 ft.

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GF Consulting Engineers Inc.
13170 SW 128th Street #204
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305-971-4323

JOB TITLE LAMAS RESIDENCE GARAGE

JOB NO. 23-08 **SHEET NO.**
CALCULATED BY FH **DATE** 9/6/20
CHECKED BY FH **DATE**

www.struware.com

CODE SUMMARY

Code: ASCE 7 - 16

Live Loads:

Roof	0 to 200 sf:	30 psf
	200 to 600 sf:	30 psf
	over 600 sf:	30 psf

Typical Floor	40 psf
Partitions	15 psf
Attics without storage	10 psf
Balconies (1.5 times live load)	60 psf
0	0 psf

Dead Loads:

Floor	29.8 psf
Roof	20.0 psf

Roof Snow Loads:

Design Uniform Roof Snow load		=	0.0 psf
Flat Roof Snow Load	Pf	=	0.0 psf
Balanced Snow Load	Ps	=	0.0 psf
Ground Snow Load	Pg	=	0.0 psf
Importance Factor	I	=	1.00
Snow Exposure Factor	Ce	=	1.20
Thermal Factor	Ct	=	1.00
Sloped-roof Factor	Cs	=	1.00
Drift Surcharge load	Pd	=	
Width of Snow Drift	w	=	

Earthquake Design Data:

Risk Category		=	II
Importance Factor	I	=	1.00
Mapped spectral response acceleratio	Ss	=	0.04
	S1	=	0.02
Site Class		=	D
Spectral Response Coef.	Sds	=	0.000
	Sd1	=	0.000
Seismic Design Category		=	A
Basic Structural System		=	Dual Systems w/ intermediate Moment Frames Capable of Resisting >= 25% of S
Seismic Resisting System		=	Ordinary reinforced masonry shear walls
Seismic Response Coef.	Cs	=	0.010
Response Modification Factor	R	=	3
Analysis Procedure		=	Equivalent Lateral-Force Analysis

Rain Design Data:

Rain intensity	*i* =	4.70 in/hr
Rain Load	R =	27.5 psf

Wind Design Data:

Ultimate Design Wind Speed	175 mph
Nominal Design Wind Speed	135.55 mph
Risk Category	II
Mean Roof Ht (h)	15.0 ft
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure Coef.	+/-0.18
Directionality (Kd)	0.85

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JOB TITLE LAMAS RESIDENCE GARAGE

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Component and Cladding Ultimate Wind Pressures

Roof	Surface Pressure (psf)							
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf
Negative Zone 1	-106.4	-99.3	-90.1	-83.0	-76.0	-70.4	-66.8	-66.8
Negative Zone 1'	-61.1	-61.1	-61.1	-61.1	-52.6	-45.7	-41.3	-32.8
Negative Zone 2	-140.3	-131.3	-119.3	-110.3	-101.3	-94.0	-89.4	-89.4
Negative Zone 3	-191.2	-173.2	-149.3	-131.3	-113.2	-98.7	-89.4	-89.4
Positive All Zones	27.2	25.5	23.2	21.5	21.5	21.5	21.5	21.5
Overhang Zone 1&1'	-96.2	-94.5	-92.2	-90.5	-75.9	-64.1	-56.6	-56.6
Overhang Zone 2	-130.1	-118.1	-102.2	-90.2	-78.1	-68.4	-62.2	-62.2
Overhang Zone 3	-181.0	-160.0	-132.2	-111.1	-90.1	-73.1	-62.2	-62.2

Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 10.2 psf)

	Parapet	Solid Parapet Pressure (psf)					
	Area	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf
CASE A:	Zone 2 :	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0
CASE B:	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0

Wall	Surface Pressure (psf)			
Area	10 sf	100 sf	200 sf	500 sf
Negative Zone 4	-66.2	-57.2	-54.5	-50.9
Negative Zone 5	-81.5	-63.5	-58.1	-50.9
Positive Zone 4 & 5	61.1	52.1	49.4	45.8

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JOB TITLE LAMAS RESIDENCE GUEST HOUSE

JOB NO. 23-08 **SHEET NO.**
CALCULATED BY FH **DATE** 9/6/20
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www.struware.com

CODE SUMMARY

Code: ASCE 7 - 16

Live Loads:

Roof	0 to 200 sf:	30 psf
	200 to 600 sf:	30 psf
	over 600 sf:	30 psf

Typical Floor	40 psf
Partitions	15 psf
Attics without storage	10 psf
Balconies (1.5 times live load)	60 psf
0	0 psf

Dead Loads:

Floor	29.8 psf
Roof	20.0 psf

Roof Snow Loads:

Design Uniform Roof Snow load		=	0.0 psf
Flat Roof Snow Load	Pf	=	0.0 psf
Balanced Snow Load	Ps	=	0.0 psf
Ground Snow Load	Pg	=	0.0 psf
Importance Factor	I	=	1.00
Snow Exposure Factor	Ce	=	1.20
Thermal Factor	Ct	=	1.00
Sloped-roof Factor	Cs	=	1.00
Drift Surcharge load	Pd	=	
Width of Snow Drift	w	=	

Earthquake Design Data:

Risk Category		=	II
Importance Factor	I	=	1.00
Mapped spectral response acceleratio	Ss	=	60.00
	S1	=	20.00
Site Class		=	D
Spectral Response Coef.	Sds	=	0.528
	Sd1	=	0.293
Seismic Design Category		=	D
Basic Structural System		=	Structural steel systems not specifically detailed for seismic resistance
Seismic Resisting System		=	Structural steel systems not specifically detailed for seismic resistance
Seismic Response Coef.	Cs	=	0.176
Response Modification Factor	R	=	3
Analysis Procedure		=	Equivalent Lateral-Force Analysis

Rain Design Data:

Rain intensity	*i* =	4.70 in/hr
Rain Load	R =	27.5 psf

Wind Design Data:

Ultimate Design Wind Speed	175 mph
Nominal Design Wind Speed	135.55 mph
Risk Category	II
Mean Roof Ht (h)	17.0 ft
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure Coef.	+/-0.18
Directionality (Kd)	0.85

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JOB TITLE LAMAS RESIDENCE GUEST HOUSE

JOB NO. 23-08 **SHEET NO.**
CALCULATED BY FH **DATE** 9/6/20
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Component and Cladding Ultimate Wind Pressures

Roof	Surface Pressure (psf)			
Area	10 sf	20 sf	100 sf	200 sf
Negative Zone 1	-86.0	-86.0	-68.5	-68.5
Negative Zone 2e	-115.0	-105.6	-83.8	-74.3
Negative Zone 2r	-149.8	-135.1	-100.7	-86.0
Negative Zone 3	-147.3	-133.2	-100.5	-86.4
Positive All Zones	51.1	44.1	27.9	27.9
Overhang Zone 1	-104.5	-61.3	-116.2	-116.2
Overhang Zone 2e	-133.6	-130.9	-124.7	-122.0
Overhang Zone 2r	-168.4	-160.4	-141.6	-133.6
Overhang Zone 3	-200.7	-178.6	-127.2	-105.0

Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 10.5 psf)

	Parapet	Solid Parapet Pressure (psf)					
	Area	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf
CASE A:	Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 2r :	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0
CASE B:	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0

Wall	Surface Pressure (psf)			
Area	10 sf	100 sf	200 sf	500 sf
Negative Zone 4	-74.3	-64.1	-61.0	-56.9
Negative Zone 5	-91.8	-71.3	-65.1	-56.9
Positive Zone 4 & 5	68.5	58.3	55.2	51.1

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JOB TITLE LAMAS RESIDENCE

JOB NO. 22-02 **SHEET NO.**
CALCULATED BY FH **DATE** 9/6/20
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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 2) =	0.98	h =	30.0 ft
Base pressure (qh) =	**65.5 psf**	a =	5.5 ft
Minimum parapet ht =	0.0 ft	GCpi =	+/-0.18
Roof Angle (θ) =	16.3 deg	qi = qh =	65.5 psf
Type of roof =	Hip		

Roof

	GCp +/- Gcpi				Surface Pressure (psf)				User input	
Area	10 sf	20 sf	100 sf	200 sf	10 sf	20 sf	100 sf	200 sf	10 sf	150 sf
Negative Zone 1	-1.48	-1.48	-1.18	-1.18	-96.9	-96.9	-77.2	-77.2	-96.9	-77.2
Negative Zone 2e	-1.98	-1.82	-1.44	-1.28	-129.6	-119.0	-94.4	-83.8	-129.6	-88.2
Negative Zone 2r	-2.58	-2.33	-1.73	-1.48	-168.9	-152.2	-113.5	-96.9	-168.9	-103.8
Negative Zone 3	-2.1	-1.92	-1.5	-1.33	-137.5	-125.8	-98.5	-86.8	-137.5	-91.6
Positive All Zones	0.88	0.76	0.48	0.48	57.6	49.7	31.4	31.4	57.6	31.4
Overhang Zone 1	-1.8	-1.06	-2	-2	-117.8	-69.1	-130.9	-130.9	-117.8	-130.9
Overhang Zone 2e	-2.3	-2.25	-2.15	-2.1	-150.6	-147.5	-140.5	-137.5	-150.6	-138.7
Overhang Zone 2r	-2.9	-2.76	-2.44	-2.3	-189.8	-180.7	-159.6	-150.6	-189.8	-154.3
Overhang Zone 3	-3.02	-2.7	-1.96	-1.65	-197.8	-176.9	-128.5	-107.7	-197.8	-116.4

Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 11.8 psf)

Parapet

qp = #REF!

		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	20 sf
CASE A:	Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 2r :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CASE B:	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0

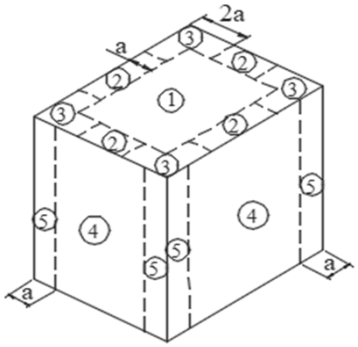
Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	20 sf	55 sf
Negative Zone 4	-1.28	-1.10	-1.05	-0.98	-83.8	-72.2	-68.7	-64.1	-80.3	-75.2
Negative Zone 5	-1.58	-1.23	-1.12	-0.98	-103.4	-80.3	-73.3	-64.1	-96.5	-86.3
Positive Zone 4 & 5	1.18	1.00	0.95	0.88	77.2	65.7	62.2	57.6	73.8	68.7

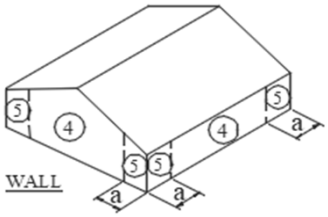
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JOB TITLE	LAMAS RESIDENCE		
JOB NO.	22-02	**SHEET NO.**	
CALCULATED BY	FH	**DATE**	9/6/20
CHECKED BY	FH	**DATE**	

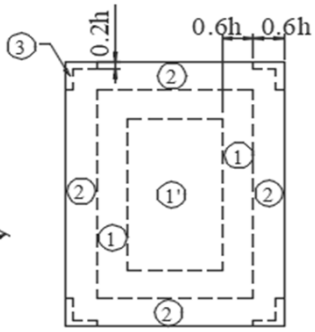
Location of C&C Wind Pressure Zones - ASCE 7-16



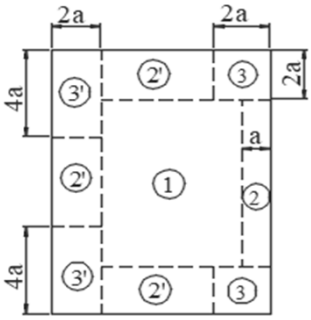
Roofs w/ θ ≤ 10°
and all walls
h > 60'



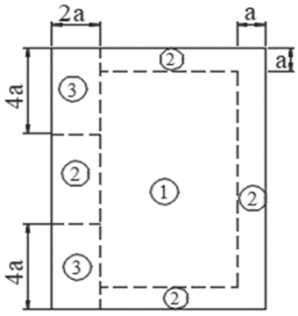
Walls h ≤ 60'
& alt design h<90'



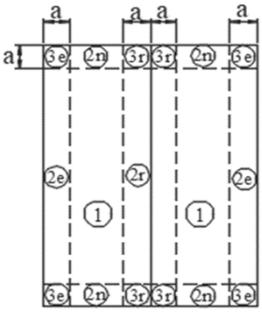
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



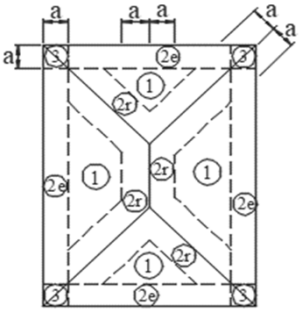
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



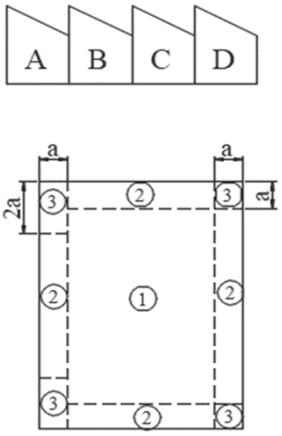
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



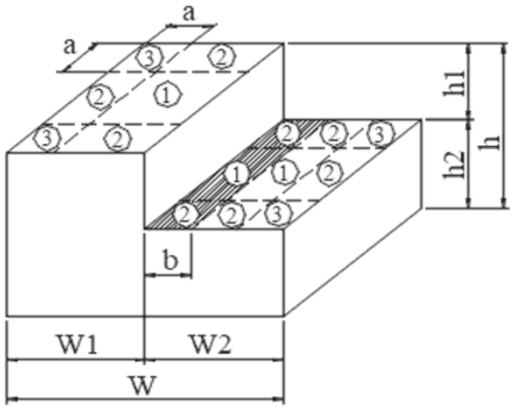
Multispan Gable &
Gable 7° < θ ≤ 45°

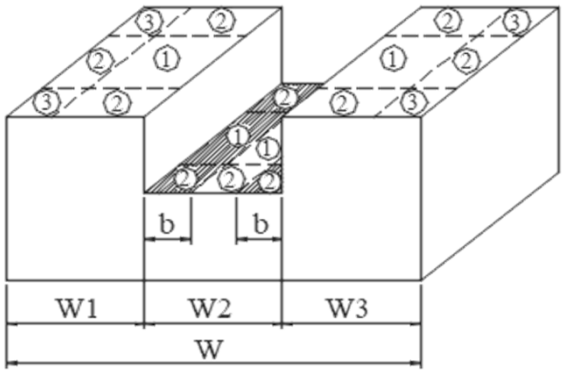


Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'





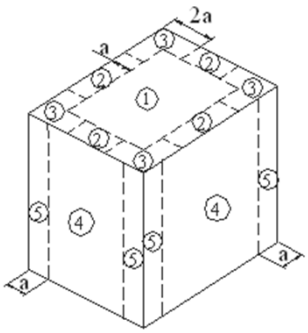
Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Note: The stepped roof zones above are as shown in ASCE 7-16 (except the upper roof zones 1 and 2 are shown at the inside edge per the notes). Prior editions didn't show zones, but the notes sent you to the low slope gable figure. The note in ASCE 7-16 still sends you to the low slope gable figure, but for some reasons the zones shown are per editions prior to ASCE 7-16. Therefore, the above zones may be a code mistake and the correct zone locations may be per the low slope gable roof shown at the top of this page.

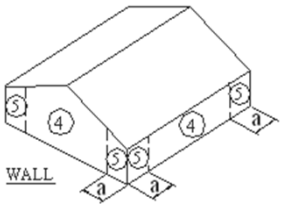
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JOB NO.	22-02	**SHEET NO.**	
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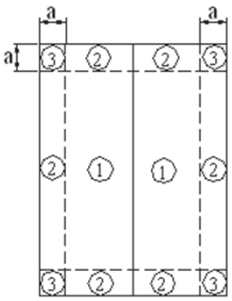
Location of C&C Wind Pressure Zones - ASCE 7-10 & earlier



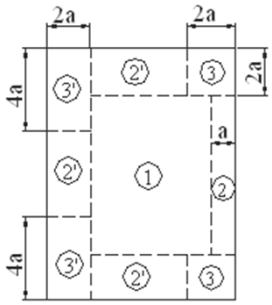
Roofs w/ θ ≤ 10°
and all walls
h > 60'



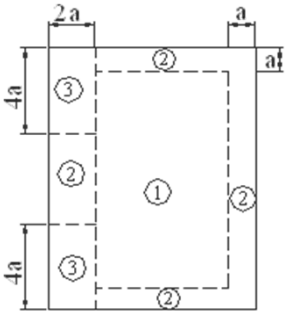
Walls h ≤ 60'
& alt design h<90'



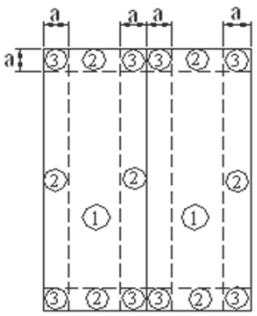
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



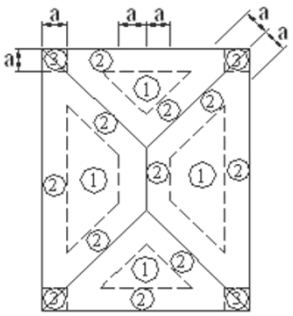
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



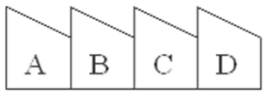
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'

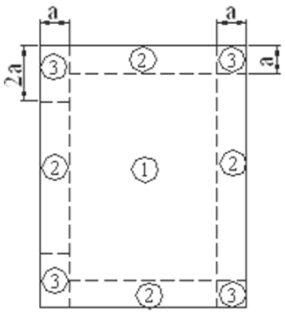


Multispan Gable &
Gable 7° < θ ≤ 45°

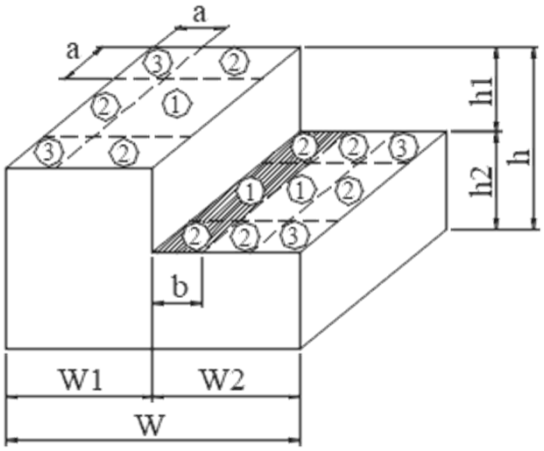


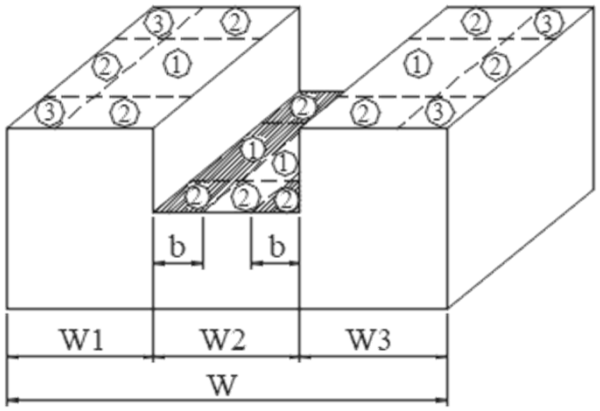
Hip 7° < θ ≤ 27°





Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'





Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

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JOB TITLE LAMAS RESIDENCE GUEST HOUSE

JOB NO. 23-08 **SHEET NO.**
CALCULATED BY FH **DATE** 9/6/20
CHECKED BY FH **DATE**

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) = 0.87
Base pressure (qh) = **58.1 psf**
GCpi = +/-0.18

Edge Strip (a) = 3.0 ft
End Zone (2a) = 6.0 ft
Zone 2 length = 12.0 ft

Wind Pressure Coefficients

	CASE A				CASE B		
		θ = 16.3 deg					
Surface	GCpf	w/-GCpi	w/+GCpi		GCpf	w/-GCpi	w/+GCpi
1	0.50	0.68	0.32		-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87		-0.69	-0.51	-0.87
3	-0.45	-0.27	-0.63		-0.37	-0.19	-0.55
4	-0.40	-0.22	-0.58		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.75	0.93	0.57		-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25		-1.07	-0.89	-1.25
3E	-0.65	-0.47	-0.83		-0.53	-0.35	-0.71
4E	-0.59	-0.41	-0.77		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1		39.4	18.4		-15.7	-36.6
2		-29.6	-50.5		-29.6	-50.5
3		-15.8	-36.7		-11.0	-31.9
4		-12.5	-33.4		-15.7	-36.6
5					33.7	12.8
6					-6.4	-27.3
1E		54.2	33.3		-17.4	-38.3
2E		-51.7	-72.6		-51.7	-72.6
3E		-27.3	-48.2		-20.3	-41.2
4E		-23.7	-44.6		-17.4	-38.3
5E					45.9	25.0
6E					-14.5	-35.4

Parapet

Windward parapet = #REF! psf (GCpn = +1.5)
Leeward parapet = #REF! psf (GCpn = -1.0)

Windward roof overhangs = 40.7 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

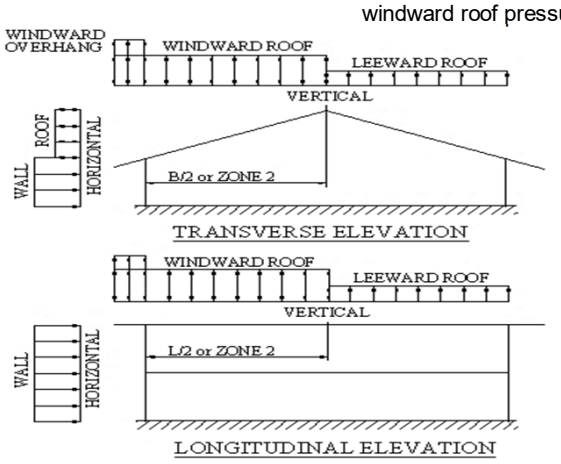
Interior Zone: Wall 51.8 psf
Roof -13.8 psf **
End Zone: Wall 77.8 psf
Roof -24.4 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 40.1 psf
End Zone: Wall 60.4 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



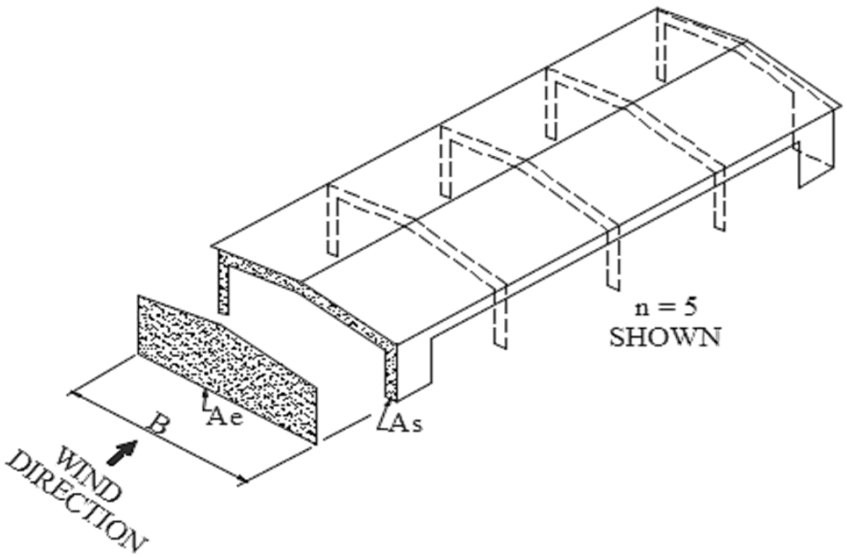
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Wind Loads - h≤60' Longitudinal Direction MWFRS On Open or Partially Enclosed Buildings with Transverse Frames and Pitched Roofs

ASCE 7-16 procedure

Base pressure (qh) = **58.1 psf**
GCpi = +/-0.18 Enclosed bldg, procdure doesn't apply
Roof Angle (θ) = 16.3 deg



B=	24.0	ft
# of frames (n) =	5	
Solid are of end wall including fascia (As) =	1,500.0	sf
Roof ridge height =	18.8	ft
Roof eave height =	15.3	ft
Total end wall area if soild (Ae) =	408.0	sf

Longidinal Directional Force (F) = pAe
p= qh [(GCpf)windward -(GCpf)leeward] K_B K_S

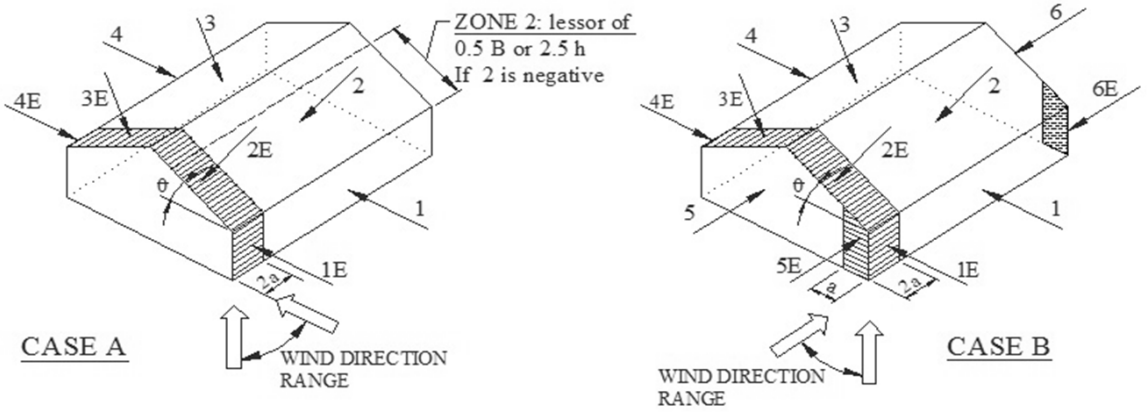
Solidarity ratio (Φ) =	3.676	
n =	5	
KB =	1.56	
KS =	13.768	
Zones 5 & 6 area =	360	sf
5E & 6E area =	48	sf
(GCpf) windward - (GCpf) leeward] =	0.731	
p =	912.5	psf

Total force to be resisted by MWFRS (F) = **372.3 kips** applied at the centroid of the end wall area Ae

Note: The longidudinal force acts in combination with roof loads calculated elsewhere for an open or partially enclosed building.

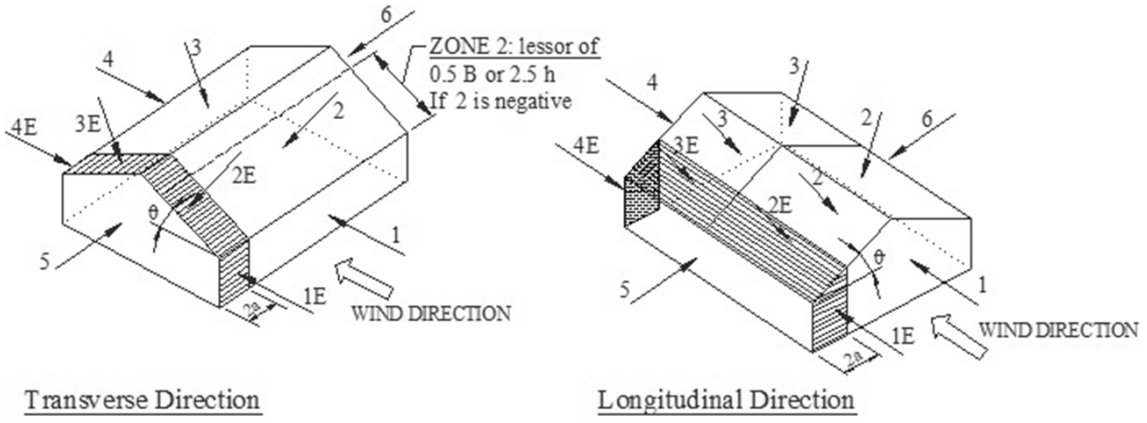
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JOB TITLE	LAMAS RESIDENCE GUEST HOUSE		
JOB NO.	23-08	**SHEET NO.**	
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NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-98 & ASCE 7-10 (& later) - MWFRS wind pressure zones



NOTE: Torsional loads are 25% of zones 1 - 4. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-02 and ASCE 7-05 - MWFRS wind pressure zones

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JOB TITLE LAMAS RESIDENCE

JOB NO. 22-02 **SHEET NO.**
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Wind Loads - MWFRS all h (Except for Open Buildings)

Kh (case 2) =	0.98			GCpi =	+/-0.18
Base pressure (q_h) =	**65.5 psf**	Bldg dim parallel to ridge =	120.0 ft	G =	0.85
Roof Angle (θ) =	16.3 deg	Bldg dim normal to ridge =	55.0 ft	qi = qh	
Roof tributary area:		h =	30.0 ft		
Wind normal to ridge =(h/2)*L:	1800 sf	ridge ht =	34.0 ft		
Wind parallel to ridge =(h/2)*L:	825 sf				

Ultimate Wind Surface Pressures (psf)

	Wind Normal to Ridge				Wind Parallel to Ridge				
	L/B = 0.46		h/L = 0.55			L/B = 2.18		h/L = 0.25	
Surface	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	44.5	see table below			0.80	44.5	see table below	
Leeward Wall (LW)	-0.50	-27.8	-39.6	-16.0		-0.29	-16.2	-28.0	-4.4
Side Wall (SW)	-0.70	-38.9	-50.7	-27.2		-0.70	-38.9	-50.7	-27.2
Leeward Roof (LR)	-0.53	-29.6	-41.4	-17.8		Included in windward roof			
Neg Windward Roof pressure	-0.65	-36.3	-48.0	-24.5	0 to h/2*	-0.90	-50.1	-61.9	-38.3
Pos/min Windward Roof press.	-0.14	-7.7	-19.5	4.1	h/2 to h*	-0.90	-50.1	-61.9	-38.3
					h to 2h*	-0.50	-27.8	-39.6	-16.0
					> 2h*	-0.30	-16.7	-28.5	-4.9
					Min press.	-0.18	-10.0	-21.8	1.8

*Horizontal distance from windward edge

Parapet

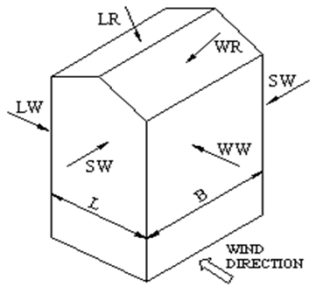
z	Kz	Kzt	qp (psf)
#REF!	#REF!	#REF!	#REF!

Windward parapet: #REF! (GCpn = +1.5)
Leeward parapet: #REF! (GCpn = -1.0)

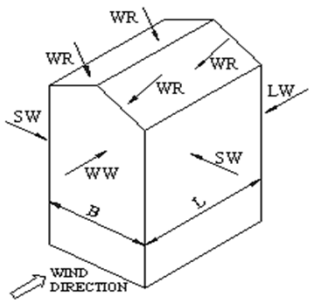
Windward roof overhangs : 44.5 psf (upward - add to windward roof pressure)

Windward Wall Pressures at "z" (psf)

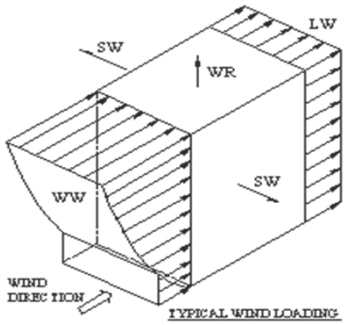
	z	Kz	Kzt	Windward Wall q_zGC_p	Windward Wall w/+q_iGC_{pi}	Windward Wall w/-q_hGC_{pi}	Combined WW + LW Wind Normal to Ridge	Combined WW + LW Wind Parallel to Ridge
	0 to 15'	0.85	1.00	38.5	26.7	50.2	66.3	54.7
	20.0 ft	0.90	1.00	40.9	29.1	52.7	68.7	57.1
	25.0 ft	0.95	1.00	42.8	31.1	54.6	70.7	59.0
h=	30.0 ft	0.98	1.00	44.5	32.7	56.3	72.3	60.7
ridge =	34.0 ft	1.01	1.00	45.7	33.9	57.5	73.5	61.9



WIND NORMAL TO RIDGE

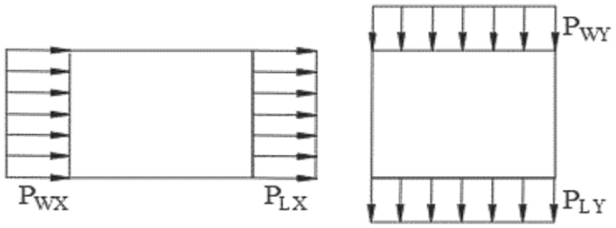


WIND PARALLEL TO RIDGE

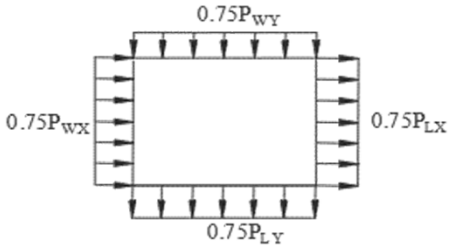


TYPICAL WIND LOADING

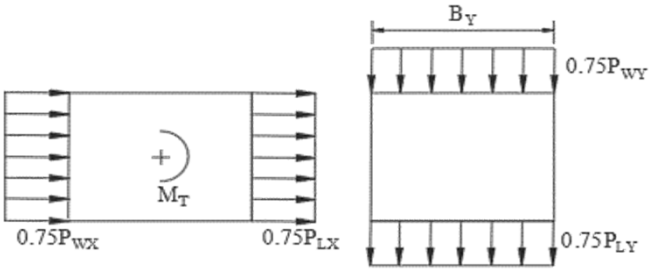
NOTE: ASCE 7 requires the application of full and partial loading of the wind pressures per the 4 cases below.



CASE 1



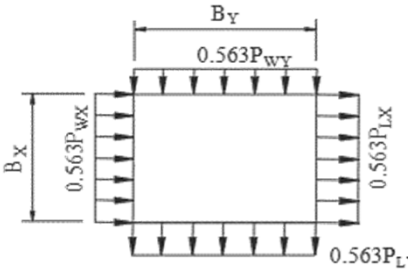
CASE 3



$M_T = 0.75(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm 0.15\ B_X$

$M_T = 0.75(P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm 0.15\ B_Y$

CASE 2



$M_T = 0.563(P_{WX}+P_{LX})B_X e_X + 0.563(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm 0.15\ B_X$ $e_Y = \pm 0.15\ B_Y$

CASE 4

Wind Forces at Floors

Building dimension (parallel with ridge) = 120.0 ft e = 18.00 ft
Total Floors = 1 Building dimension (normal to ridge) = 55.0 ft e = 8.25 ft
T/Fdn (dist below grade) = 2.0 ft L is the building dimension parallel to the wind direction

	Elevation	Height of	Wind Normal to Ridge						Wind Parallel to Ridge			
Level	Above Grade (ft)	Centroid to Fdn (ft)	L	B	Area (sf)	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	Area	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)
Equip,etc		0.00	wind on equip, screenwalls, etc =					0.0				
Parapet	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!		0.0	#REF!	#REF!		
T/Ridge		0.00			0.0	0.0		0.0	0.0	0.0		0.0
Roof	15.00	17.00	55.0	120.0	900.0	59.7	#REF!	#REF!	412.5	22.5	#REF!	#REF!
1	0.00	2.00	55.0	120.0	900.0	59.7	#REF!	#REF!	412.5	22.5	#REF!	#REF!
FDN		0.00						#REF!				#REF!

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Code Search

Code: ASCE 7 - 16

Occupancy:

Occupancy Group = R Residential

Risk Category & Importance Factors:

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

Type of Construction:

Fire Rating:

Roof =	2.0 hr
Floor =	2.0 hr

Building Geometry:

Roof angle (θ)	3.50 / 12	16.3 deg
Building length	120.0 ft	
Least width	55.0 ft	
Mean Roof Ht (h)	30.0 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0	

Live Loads:

Roof	0 to 200 sf:	20 psf	use 30.0 psf
	200 to 600 sf:	30 psf	
	over 600 sf:	30 psf	

Floor:

Typical Floor	40 psf
Partitions	15 psf
Attics without storage	10 psf
Balconies (1.5 times live load)	60 psf
0	0 psf



Roof Framing Calculations

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET

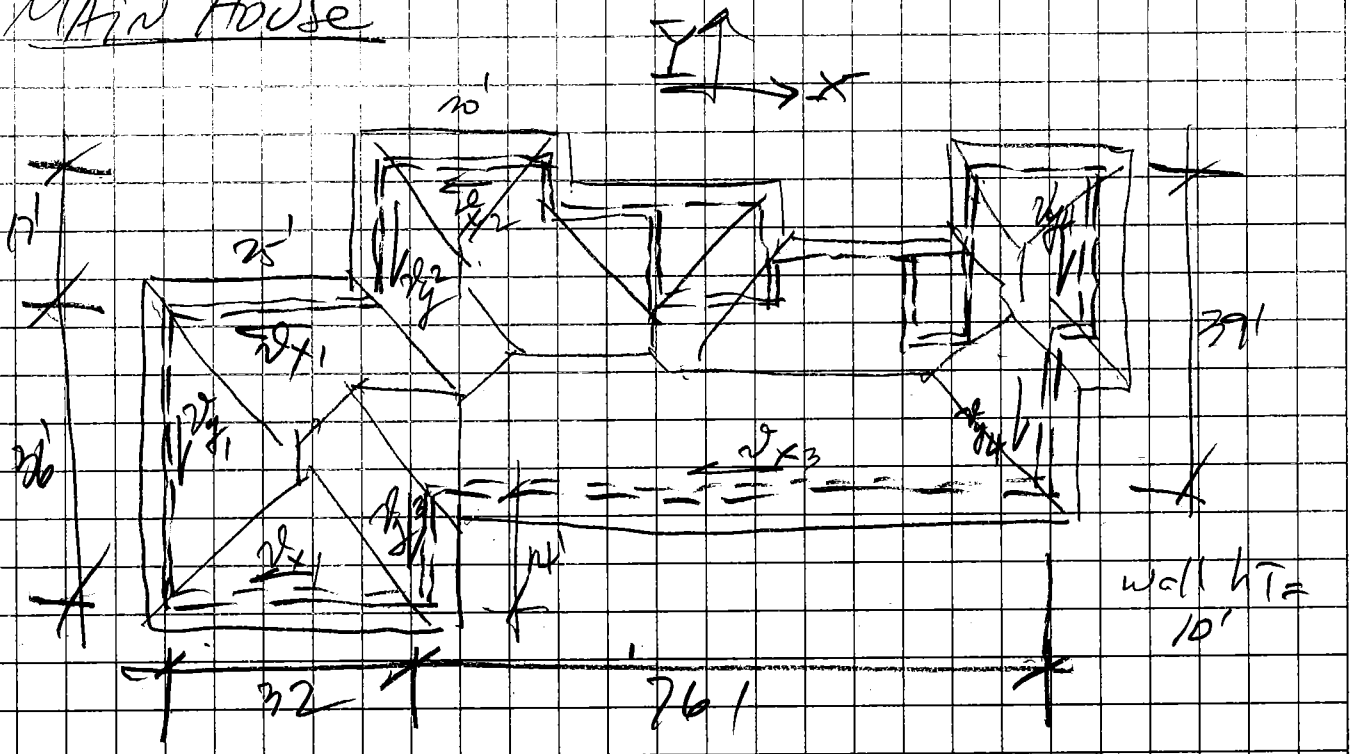
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

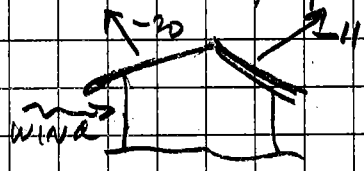
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Main House



- WW + LW = 35.3 psf
- Roof



∴ Do not add to wind direction.

Wind ⊥ X

w_{s1} = 16 × 5' × 35.3 ÷ 36 = 78.4 #/LF

$w_{y2} = w_{y3}$ = 35.3 × 5' × 56' ÷ 31 = 319 plf

w_{y4} = 35.3 × 5 × 38 ÷ 39 = 172 plf

Wind ∥ X

w_{x1} = 35.3 × 5 × 18 ÷ 25 = 127 plf

w_{x2} = 35.3 × 5 × 9 ÷ 20 = 79 plf

w_{x3} = 35.3 × 5 × 20 ÷ 76 = 46.5 plf

Side walls 16-2

41.2 × 0.6 = 24.7 psf

GF Consulting Engineers Inc.
13170 SW 128th Street #204
Miami,FL
305-971-4323

JOB TITLE LAMAS RESIDENCE

JOB NO. 22-02 SHEET NO.
CALCULATED BY FH DATE 9/6/20
CHECKED BY FH DATE

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) = 0.98
Base pressure (qh) = **65.5 psf**
GCpi = +/-0.18

Edge Strip (a) = 5.5 ft
End Zone (2a) = 11.0 ft
Zone 2 length = 27.5 ft

Wind Pressure Coefficients

Surface	CASE A θ = 16.3 deg GCpf	w/-GCpi	w/+GCpi		CASE B GCpf	w/-GCpi	w/+GCpi
1	0.50	0.68	0.32		-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87		-0.69	-0.51	-0.87
3	-0.45	-0.27	-0.63		-0.37	-0.19	-0.55
4	-0.40	-0.22	-0.58		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.75	0.93	0.57		-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25		-1.07	-0.89	-1.25
3E	-0.65	-0.47	-0.83		-0.53	-0.35	-0.71
4E	-0.59	-0.41	-0.77		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

58.9 × 0.6 = 35.3

1	44.4	20.8		-17.7	-41.2
2	-33.4	-56.9		-33.4	-56.9
3	-17.8	-41.4		-12.4	-36.0
4	-14.1	-37.6		-17.7	-41.2
5				38.0	14.4
6				-7.2	-30.8
1E	61.0	37.5		-19.6	-43.2
2E	-58.3	-81.8		-58.3	-81.8
3E	-30.8	-54.3		-22.9	-46.5
4E	-26.7	-50.2		-19.6	-43.2
5E				51.7	28.1
6E				-16.4	-39.9

Parapet

Windward parapet = #REF! psf (GCpn = +1.5)
Leeward parapet = #REF! psf (GCpn = -1.0)

Windward roof overhangs = 45.8 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

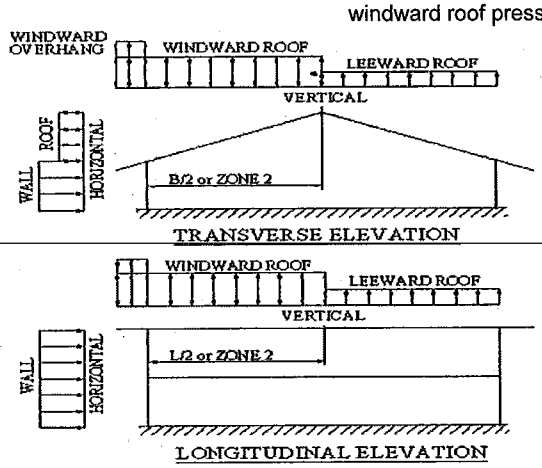
Interior Zone: Wall 58.4 psf — 35.3
Roof -15.5 psf **
End Zone: Wall 87.7 psf — 52.6
Roof -27.5 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 45.2 psf
End Zone: Wall 68.1 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



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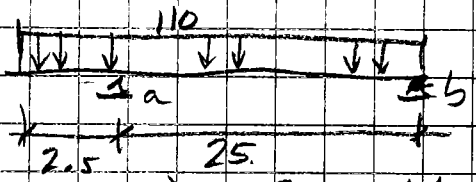
ENG: _____ SHEET: _____ OF _____

Main House Roof Trusses

T1 & T2 EA: $27.5^2 \div 3 = 250$ SF

	Net	
Z1:	40.3 psf	Avg of 47 psf
Z2e:	44.3 psf	
Z2r:	52.1 psf	
Z3:	46.1 psf	
OH2e:	64.4 psf	
OH23:	112.7 psf	

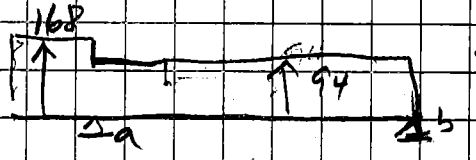
D+L



$R_a = (27.5^2 \div 2 \times 110) \div 25 = 1664\#\downarrow$

$R_b = 110 \times 27.5 - 1664 = 1361\#\downarrow$

0.6D+0.6L

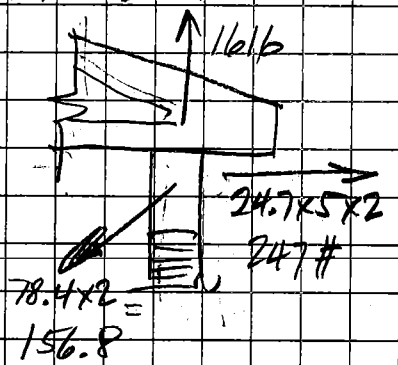


$R_a = (168 \times 2.5 \times 26.25' + 94 \times 25' \times 12.5') \div 25$

$R_a = \underline{1616\#}\uparrow$

$R_b = (168 \times 2.5' + 94 \times 25') - 1616 = 1154\#\uparrow$

Connector:



Try (D): NVHTA-16

$CSL = \frac{1616}{2645} + \frac{157}{1575} + \frac{247}{2175} = 0.82 < 1.0$

OK

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PROJECT NAME: _____

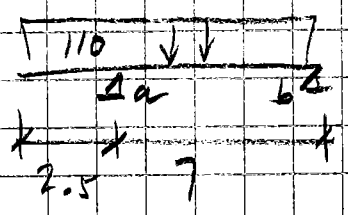
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ENG: _____ SHEET: _____ OF _____

Main House Roof (Cont.)

JACK J1

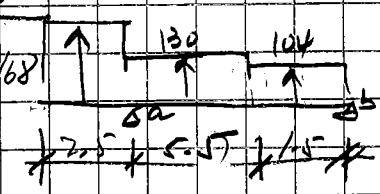
D+L



$R_a = 110 \times 9.5 \times 4.75 \div 7 = 709\ \#\downarrow$

$R_b = 110 \times 9.5 - 709 = 336\ \#\downarrow$

0.6D+0.6W



Z1: $58.1 - 6 = 52.1$

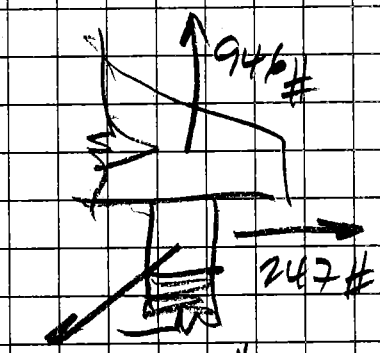
Net Z2e: $71.4 - 6 = 65.4$ psf

2eOH: $90.4 - 6 = 84.4$ psf

$R_a = (168 \times 2.5 \times 8.25 + 130 \times 5.5 \times 4.25 + 104 \times 1.5 \times 0.75) \div 7$

$R_a = 946\ \#\uparrow$

$R_b = 345\ \#\uparrow$



$127 \times 2 = 254\ \#$

Try (D)

$\frac{946}{2645} + \frac{254}{1575} + \frac{247}{2125} = 0.63$ OK

JACK J2

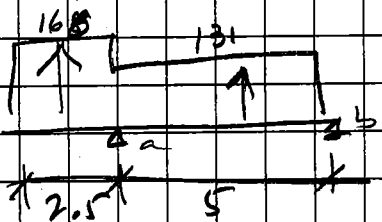
D+L (same as Guest House)

$R_a = 615\ \#\downarrow$ $R_b = 206\ \#\downarrow$

Net:

OHe: $88.5 - 6 = 82.5$ psf

Z2e: $71.4 - 6 = 65.4$ psf



$R_a = (168 \times 2.5 \times 6.25 + 131 \times 5 \times 2.5) \div 5 = 852\ \#\uparrow$

$R_b = 223\ \#\uparrow$

Use (D)

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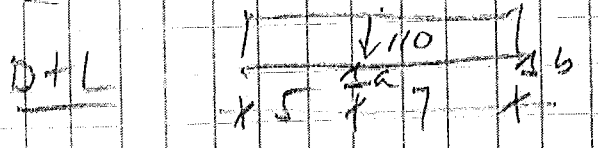
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

JACK J3

D+L



$R_a = 1131\ \#\downarrow$

$R_b = 1885\ \#\downarrow$

0.6D+W



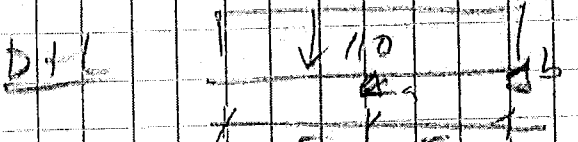
$R_a = 1522\ \#\uparrow$

$R_b = 854\ \#\uparrow$

USE (B)

JACK J4

D+L



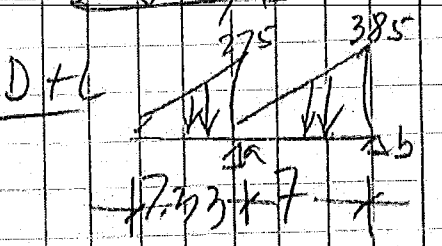
$R_a = 1100\ \#\downarrow$

$R_b = 1480\ \#\uparrow$ USE (A)

0.6D+W

HJ1 / HJ2

D+L

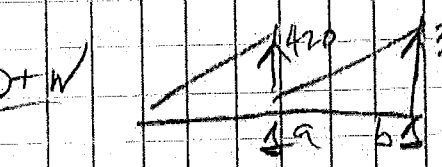


$R_a = \left(\frac{375}{2} \times 7.33 \times 9.44 + \frac{385}{2} \times 7 \times 2.33\right) \div 7$

$= 1808\ \#\downarrow$

$R_b = 542\ \#\downarrow$

0.6D+W



$R_a = \left(\frac{420}{2} \times 7.33 \times 9.44 + \frac{378}{2} \times 7 \times 2.33\right) \div 7$

$= 2516\ \#\uparrow$

$R_b = 346\ \#\uparrow$ (A)

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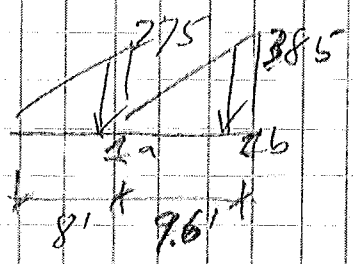
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HJ3

D+L

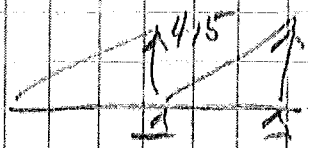


$R_2 = \left(\frac{275 \times 4}{2} \times 12.33 + \frac{385 \times 9.6}{2} \times 3.2\right) \div 9.6$

$= 1322\ \# \downarrow$

$R_6 = 1626\ \# \downarrow$

0.6D+W



$R_2 = \left(\frac{415 \times 4}{2} \times 12.33 + .588 \times \frac{9.6}{2} \times 3.2\right) \div 9.6$

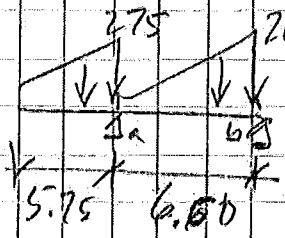
$= 2007\ \# \uparrow$

$R_6 = 2475\ \# \uparrow$

2 use A

HJ4

D+L

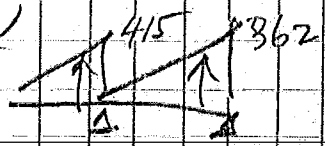


$R_2 = \left(\frac{275 \times 5.75}{2} \times 8.51 + \frac{240 \times 6.6}{2} \times 2.2\right) \div 6.6$

$= 1284\ \# \downarrow$

$R_6 = 255\ \# \downarrow$

0.6D+W



$R_2 = \left(\frac{415 \times 5.75}{2} \times 8.51 + \frac{362 \times 6.6}{2} \times 2.2\right) \div 6.6$

$= 1937\ \# \uparrow$

$R_6 = 451\ \# \uparrow$

use A

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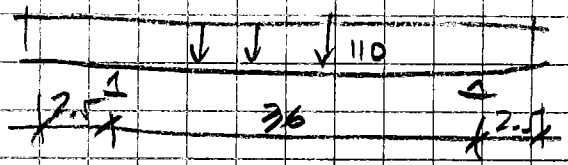
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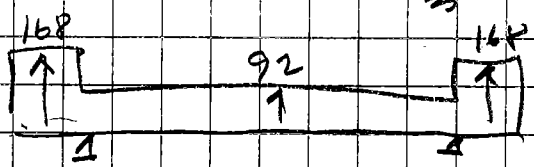
Truss T4

DL



$R_a = R_b = 2255\# \downarrow$

0.6D+0.6W E.T.A = $41 \times \frac{41}{2} = 580SF$



Net

Z1 = 40.3 psf

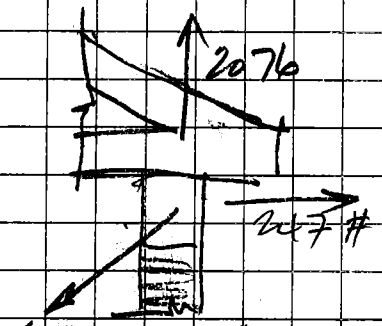
Z2e = 44.3 psf

Z2r = 52.1 psf

Z2H = 54.4

AVG. 46 psf

$R_a = R_b = 168 \times 2.5' + 18' \times 92 = 2076\# \uparrow$

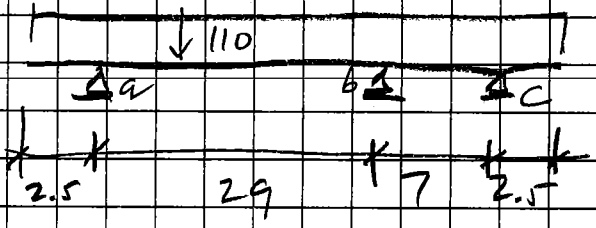


Try (B) Curve NVHTA-16H

$\frac{2076}{3117} + \frac{93}{1575} + \frac{247}{2175} = 0.84$ OK

Truss T5

DL

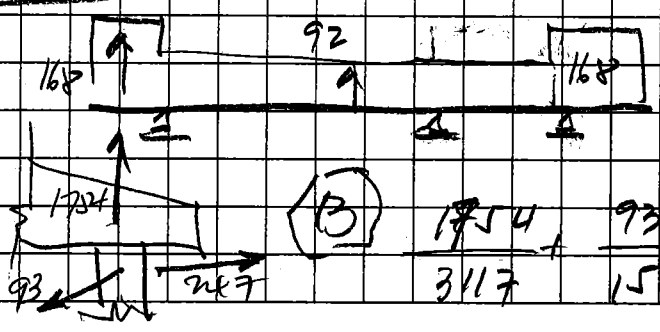


$R_a = 17' \times 110 = 1870\# \downarrow$

$R_b = 18' \times 110 = 1980\# \downarrow$

$R_c = 6' \times 110 = 660\# \downarrow$

0.6D+0.6W



$R_a = 168 \times 2.5' + 92 \times 14.5' = 1754\# \uparrow$

$R_b = 92 \times 18' = 1656\# \uparrow$

$R_c = 168 \times 2.5 + 92 \times 3.5 = 742\# \uparrow$

(B) $\frac{1754}{3117} + \frac{93}{1575} + \frac{247}{2175} = 0.74 < 1.0$ OK

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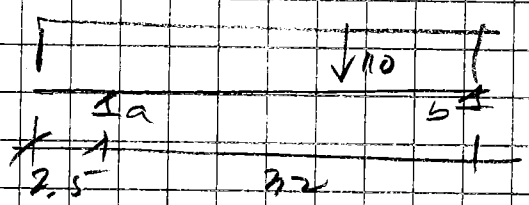
PROJECT #: _____ DATE: _____

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Main Roof

Truss T6

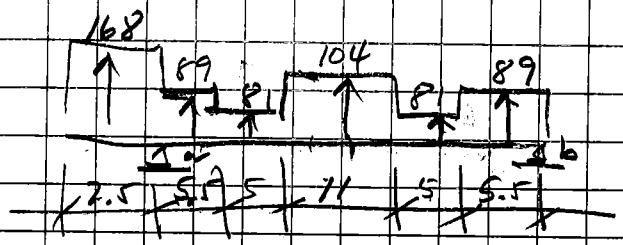
D+L



$R_a = (110 \times 34.5^2 \times 0.5) \div 32 = 2046\ \#\downarrow$

$R_b = 110 \times 34.5 - 2046 = 1749\ \#\downarrow$

0.6D+0.6W



net

21: 40.3 psf

2e: 44.3 psf

2r: 52.1 psf

0.6W+0.6W: 82.5 psf

$R_a = \dfrac{168 \times 2.5 \times 33.25 + 89 \times 5.5 \times 25.25 + 81 \times 5 \times 24 + 104 \times 11 \times 16 + 81 \times 5 \times 8 + 89 \times 5.5 \times \frac{2}{3} \times 5.5}{32}$

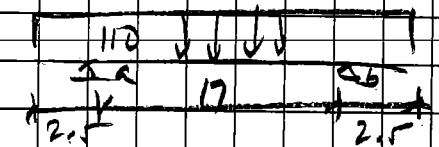
$R_a = 1903\ \#\uparrow$

$R_b = 1450\ \#\uparrow$

Truss T7

ETA: $22 \times 22 \div 3 = 161$ sf

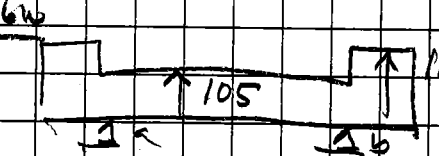
D+L



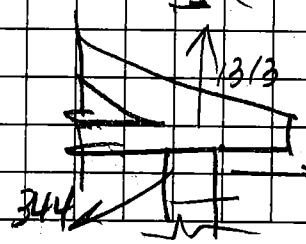
$R_a = R_b = 110 \times 11 = 1210\ \#\downarrow$

Net: 2r = 57.1 psf, 2e = 47.6 psf } ANS 2.5

0.6D+0.6W



$R_a = R_b = 168 \times 2.5 + 105 \times 8.5 = 1313\ \#\uparrow$



Try [D]

$\dfrac{1313}{2644} + \dfrac{344}{1575} + \dfrac{247}{2175} = 0.83$

OK

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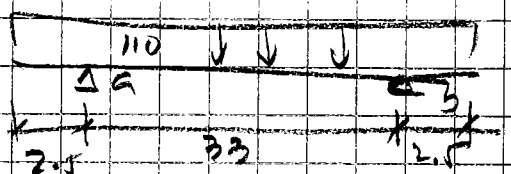
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

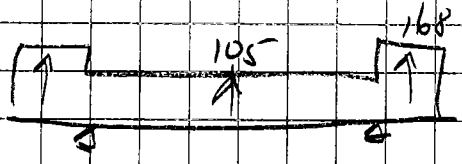
Truss T8

D+L



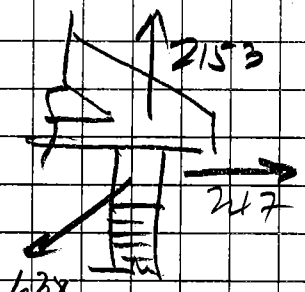
$R_a = R_b = 19 \times 110 = 2090\# \downarrow$

0.6D+0.6W



$R_a = R_b = 2153\# \uparrow$

Connection



Try (D)

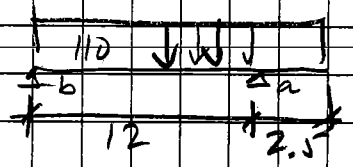
CSR $\frac{2153}{3117} + \frac{638}{1575} + \frac{247}{2175} = 1.21 > 1.0$ NG.

ADD (E)

CSR $\frac{2153}{3117+1360} + \frac{638}{1575+630} + \frac{247}{2175+125} = 0.88 < 1.0$ OK

√3

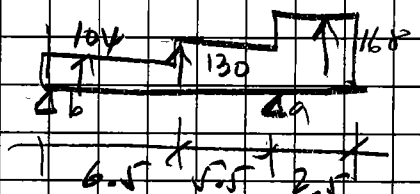
D+L



$R_a = 110 \times 14.5 \times 7.25 \div 12 = 965\# \downarrow$

$R_b = 630\# \downarrow$

0.6D+0.6W



$R_a = \frac{104 \times 6.5 \times 3.25 + 130 \times 5.5 \times 9.25 + 168 \times 2.5 \times 13.25}{12}$

$= 1198\# \uparrow$

$R_b = 613\# \uparrow$ USE (D)

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PROJECT NAME: _____

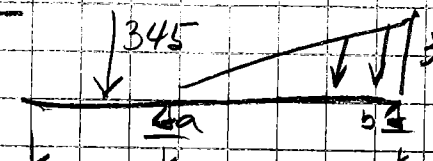
PROJECT #: _____ DATE: _____

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Main House Roof

HJ2

D+L

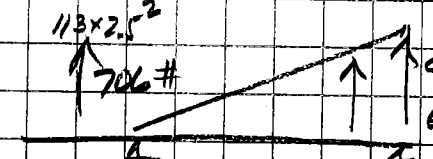


$R_a = (345 \times 11.7 + 385 \times 9.9 \times 0.5 \times 3.3) \div 9.9$

$= 1043\# \downarrow$

$R_b = 1208\# \downarrow$

0.6D+0.6W



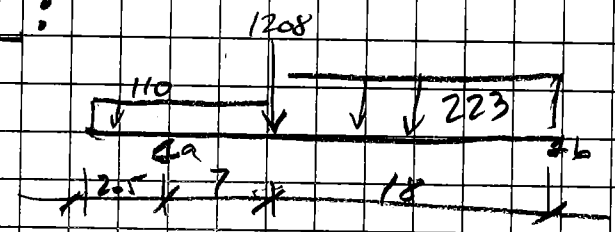
$R_a = (706 \times 11.7 + 672 \times 9.9 \times 0.5 \times 3.3) \div 9.9$

$= 1944\# \uparrow$

$R_b = 2088\# \uparrow$

I61:

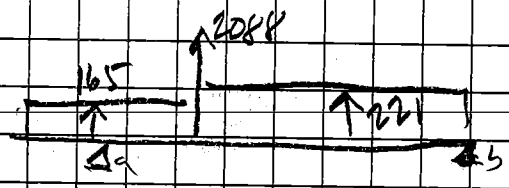
D+L



$R_a = (110 \times 9.5 \times 22.75 + 1208 \times 18 + 223 \times 18 \times 9) \div 25 = 3266\# \downarrow$

$R_b = 3001\# \downarrow$

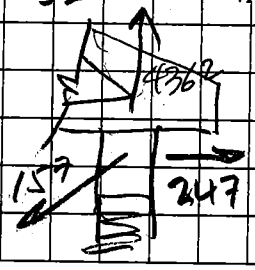
0.6D+0.6W



$R_a = (165 \times 9.5 \times 22.75 + 2088 \times 18 + 221 \times 18 \times 9) \div 25 = 4362\# \uparrow$

$R_b = 3272\# \uparrow$

Try (A) + (F)



$CSR = \frac{4362}{5397} + \frac{157}{3642} + \frac{247}{2528} = 0.94$

OK

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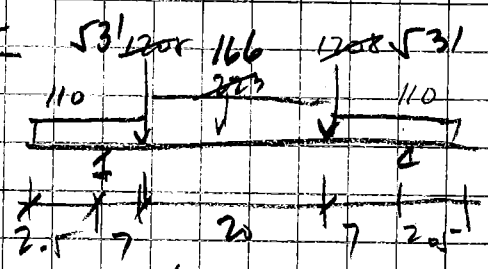
PROJECT NAME: _____

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TG2

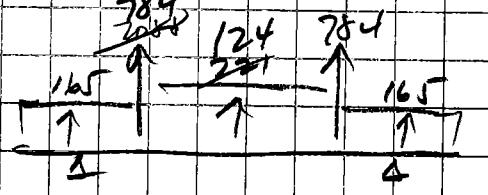
D+L



$R_a = R_b = \frac{531}{208} + 110 \times 9.5 + \frac{166}{223} \times 10$

$= 3236\# \downarrow$

0.6D+0.6L



$R_a = R_b = 784 + 165 \times 9.5 + 124 \times 10$

$= 3592\# \uparrow$

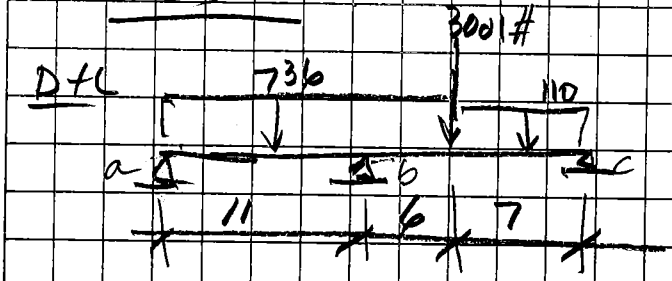
$(\frac{\#784}{2} \div 36) = 92$ psf avg.

Try (A) + (E)

$CSR \quad \frac{3592}{5397} + \frac{638}{3642} + \frac{247}{2528} = 0.92 < 1.0$

OK

TG3



D+L

$R_a = 2800\#$

$R_b = 11600\#$

$R_c = 1900\#$

Contin

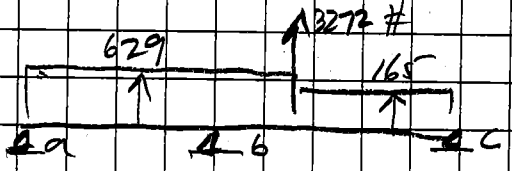
4048 #

4048 + 5200 # ↓

3000 # ↓

Two ind spans

0.6D+0.6L



$R_a = 2200\# \uparrow$

$R_b = 10800\# \uparrow$

$R_c = 2200\# \uparrow$

3460 # ↑

3460 + 5000 ↑

3300 ↑

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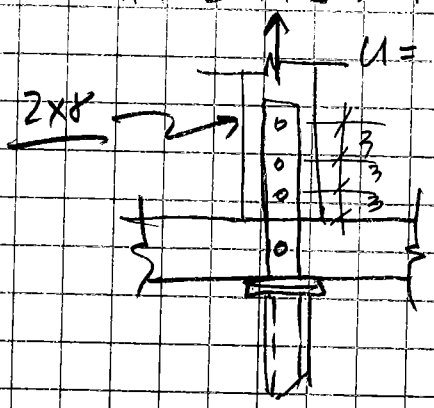
ENG: _____ SHEET: _____ OF _____

TG3 (Cont.)

Connections: R_B

$U = 10{,}800\#$

2x8



USING 2M GIRDER

$Z_{11} = 3300 \times 1.6 = 5280\#/bolt$

$\frac{3}{4}\phi$

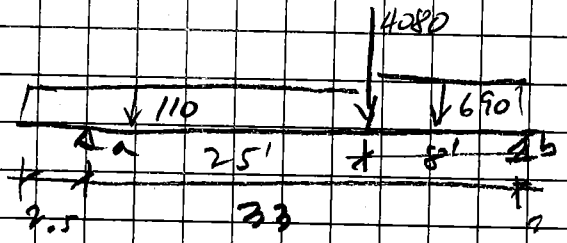
Use (3) $\frac{3}{4}$" ϕ BOLTS

$4D = 3$"

Connector R_C Use (A) Good for 3367 #

Truss TG4

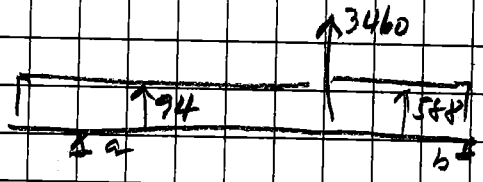
D+L



$R_a = 110 \times 27.5' \times 21.75' + 4080 \times 8' + 690 \times 8' \times 4' \div 33 = 3652\# \downarrow$

$R_b = 8973\# \downarrow$

0.6D+0.6L



$R_a = 3113\# \uparrow$ → (G)(K)

$R_b = 7636\# \uparrow$ → (L)(K)

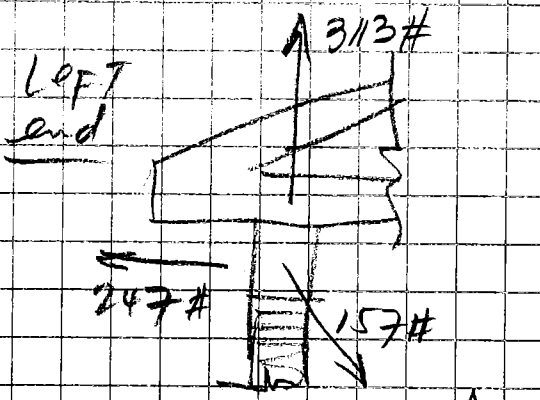
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PROJECT NAME: _____

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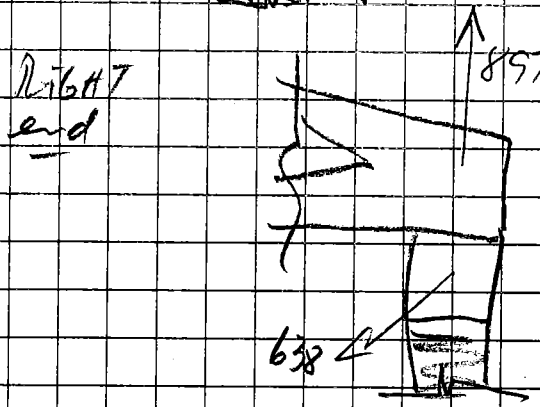
T64 Connections @ LEFT



Try (G)

$\frac{3113}{3367} + \frac{157}{475} + \frac{247}{317} = 1.04$

add Connector (K)



Try (L) + (K)

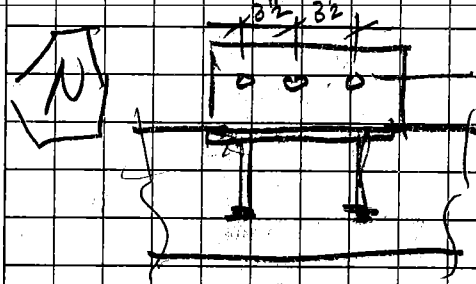
$\frac{8973}{8885+1700} + \frac{638}{2010} = 1.16$ NG.

Try (G) + (L)

$\frac{8973}{8885+3367} + \frac{638}{475} = 0.89$ OK

OR use (N)

Built up Connector



Bolts

$Z_t = 1900 \times 1.6 \times 3 = 9168\#$ ↑ capacity.

Weld 3/16" fillet good for 2.71 K/in OK

$Z_t = 1900 \times 1.6 \times 2 = 6080\#$ ↑ 2 bolts

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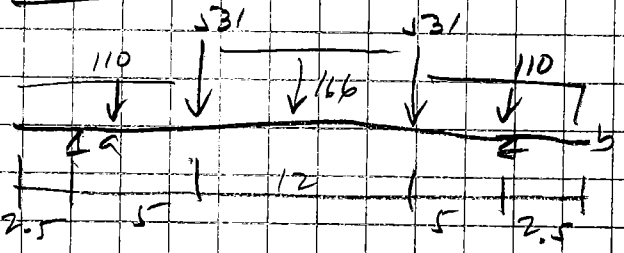
PROJECT NAME: _____

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T65

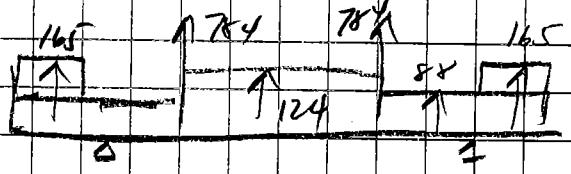
D+L



$R_A = R_B = 110 \times 7.5 + 531 + 166 \times 6$

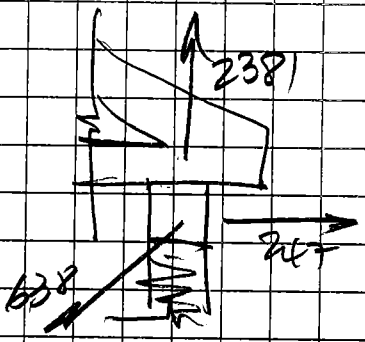
$= 2635\ \# \downarrow$

0.6D+0.6W



$R_A = R_B = 165 \times 2.5 + 88 \times 5 + 784 + 124 \times 6$

$= 2381\ \# \uparrow$

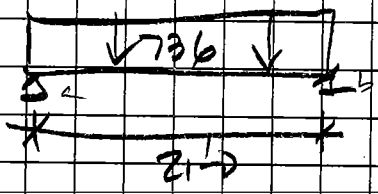


Try (A)

CSA $\frac{2381}{3367} + \frac{638}{2942} + \frac{247}{2758} = 1.01$ OK

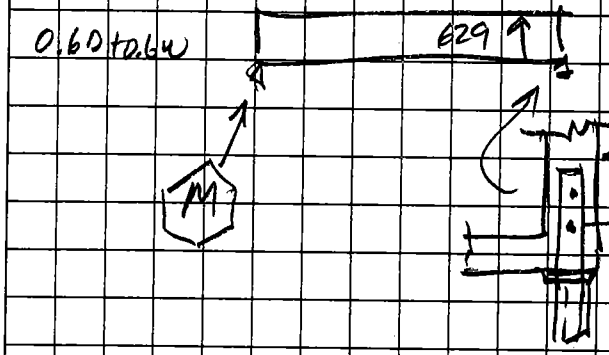
T66

D+L



$R_A = R_B = 736 \times 10.5' = 7723\ \# \downarrow$

0.6D+0.6W



$R_A = R_B = 629 \times 10.5' = 6605\ \# \uparrow$

$Z_{||} = 3300 \times 1.6 \times 2 = 10560\ \#$

> 6605

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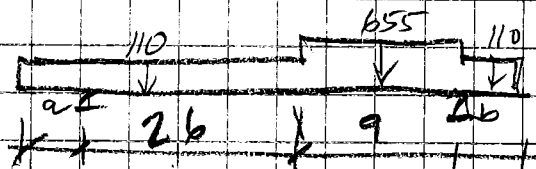
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TG7

D+L

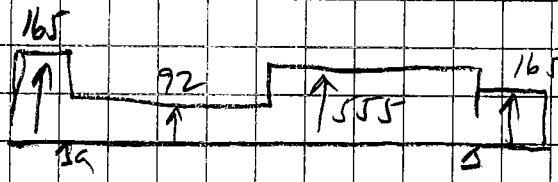


$R_a = \frac{110\times28.5\times19 + 655\times9\times4.5 - 110\times2.5\times1.25}{35}$

$R_a = 2450\#\downarrow$

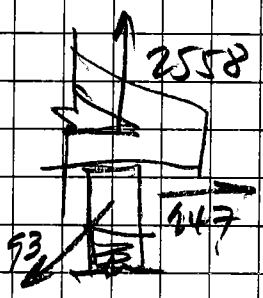
$R_b = 6855\#\downarrow$

0.6W+0.6W



$R_a = (165\times2.5\times36.25 + 92\times26\times22 + 555\times9\times4.5 - 165\times2.5\times1.25)\div35 = 2558\#\uparrow$

$R_b = 5654\#\uparrow$

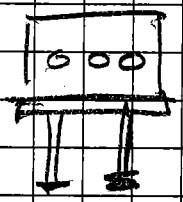
a) 

Try (A)

$CSR = \frac{2558}{3367} + \frac{93}{2542} + \frac{247}{2758}$

$CSR = 0.88$ OK

b) use: encased w/ 3 bolts

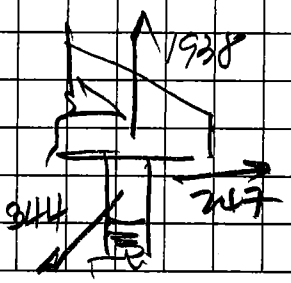


TG8 : same loadings as TG5

22/27 = 0.814

D+L $R_a = R_b = 0.814\times2635 = 2147\#\downarrow$

0.6D+0.6W $R_a = R_b = 0.814\times2381 = 1938\#\uparrow$



Try (B)

$\frac{1938}{3117} + \frac{344}{1575} + \frac{247}{2175} = 0.95$ OK

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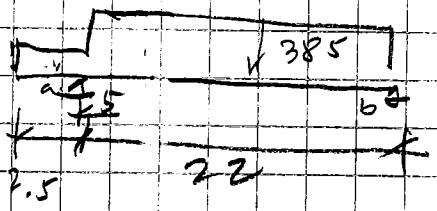
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T69

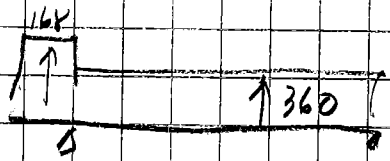
D+L



$R_a = (110 \times 2.5 \times 23.25 + 385 \times 22 \times 11) \div 22 = 4525\#$

$R_b = 4220\# \downarrow$

.6D+.6W



$R_a = (168 \times 2.5 \times 23.25 + 360 \times 22 \times 11) \div 22 = 4404\# \uparrow$

$R_b = 3536\# \uparrow$

use (N)

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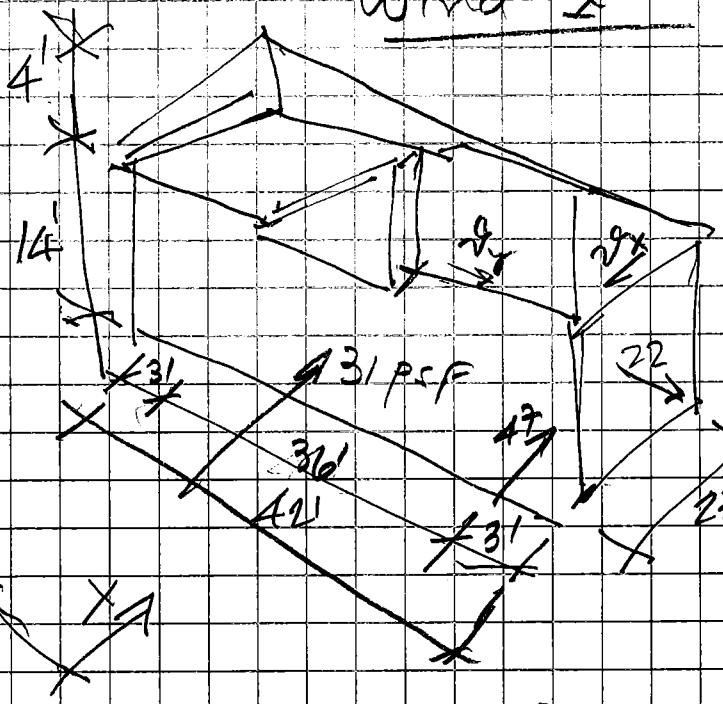
PROJECT NAME: Lamas Residence

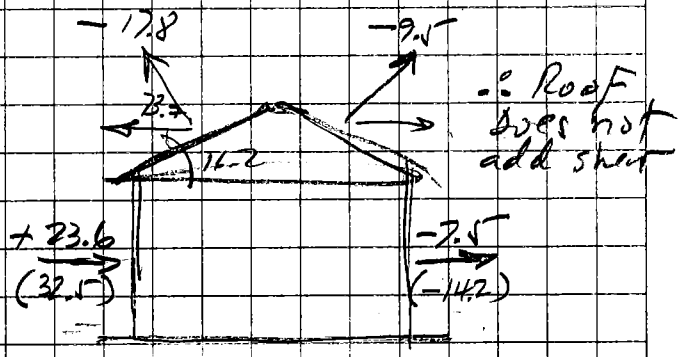
PROJECT #: _____ DATE: _____

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Roof Trusses Guest House

Wind X





∴ Roof does not add shear

$$q_x = \frac{31 \times 7' \times 18' + 47 \times 7' \times 3'}{23'}$$

$$q_x = 212.7 \text{ plf}$$

Wind Wind on Y

$$q_y = \frac{31 \times 7' \times 8.5' + 47 \times 7' \times 3'}{42'} = 67.4 \text{ plf}$$

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PROJECT NAME: _____

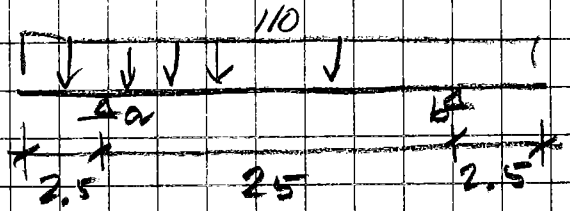
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Guest House (Cont.)

Truss T1:

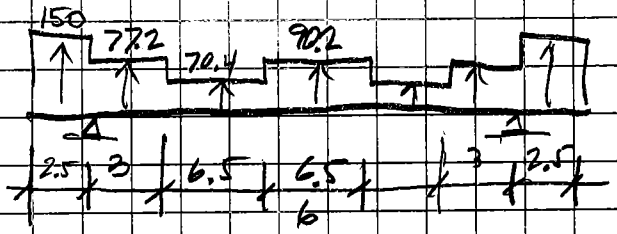
D+L:



$R_a = R_b = 110 \times 15' = 1650\#$

EWA: $25 \times \frac{25}{3} = 208$ sf

0.6D+0.6W:



net:

OH 2e: 75 psf

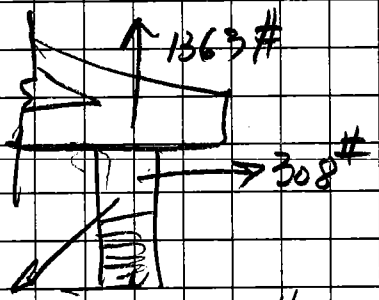
2e: 44.6 − 6 = 38.6 psf

2r: 51.6 − 6 = 45.6

1: 41.1 − 6 = 35.1

$R_a = R_b = 1363\#\uparrow$ (Avg uplift $\frac{1363}{2 \times 15} = 45$ psf)

Connector:



$67.4 \times 2 = 134.4\#$

Try MVHTA-16

$CSR = \frac{1363}{3649} + \frac{134}{1575} + \frac{308}{2175} = 0.74$ OK

(D)

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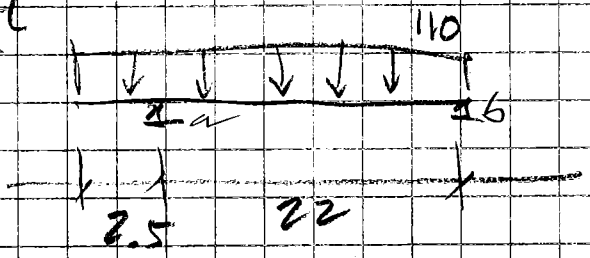
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

T3:

D+L

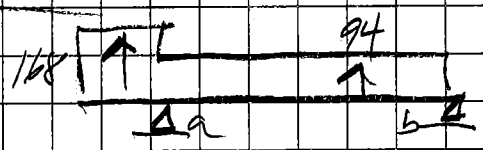


Roof Load:
Avg: 47 psf
OH2: 85 psf
D+L = 55 psf

$R_a = (110 \times 24.5 \times 12.25) \div 22 = 1500\#$ ↓

$R_b = 110 \times 24.5 - 1500 = 1195\#$ ↓

0.6D+0.6W

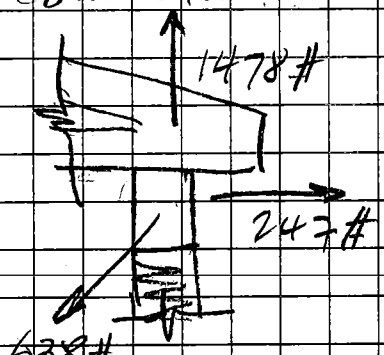


$R_a = (168 \times 2.5 \times 23.25 + 94 \times 22 \times 11) \div 22$

$R_a = 1478\#$ ↑

$R_b = (168 \times 2.5 + 94 \times 22) - 1478 = 1010\#$ ↑

Connector:



Try (D)

CSR $\frac{1478}{2649} + \frac{638}{1575} + \frac{247}{2175} = 1.08$ NG.

Try (B)

CSR $\frac{1478}{3117} + \frac{638}{1575} + \frac{247}{2175} = 0.99$ < 1.0 OK

use (B)

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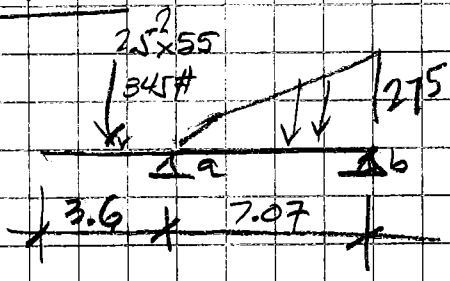
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Guest House (Cont)

HJ1

D+L



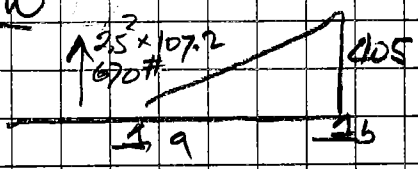
$R_a = \left(345 \times 8.87 + 275 \times \frac{7.07}{2} \times \frac{7.07}{3}\right) \div 7.07$

$= 786\#\downarrow$

$R_b = (275 \times 7.07 \times 0.5 + 345) - 786$

$= 531\#\downarrow$

0.6D+0.6W



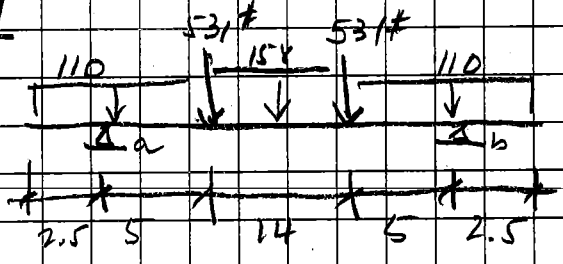
$R_a = \left(670 \times 8.87 + 405 \times \frac{7.07^2}{6}\right) \div 7.07 = 1318\#\uparrow$

$R_b = (405 \times 7.07 \times 0.5 + 670) - 1318 = 784\#\uparrow$

Use (D)

T61

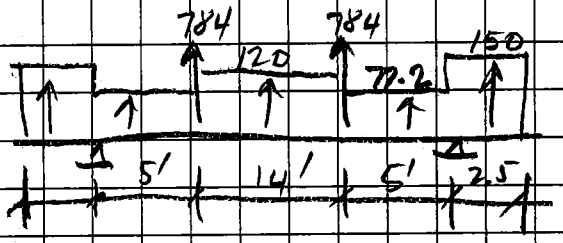
D+L



$R_a = R_b = 7.5 \times 110 + 531 + 154 \times 7$

$= 2462\#\downarrow$

0.6D+0.6W



$R_a = R_b = 150 \times 2.5 + 77.2 \times 5 + 784 + 120 \times 7' = 2385\#\uparrow$

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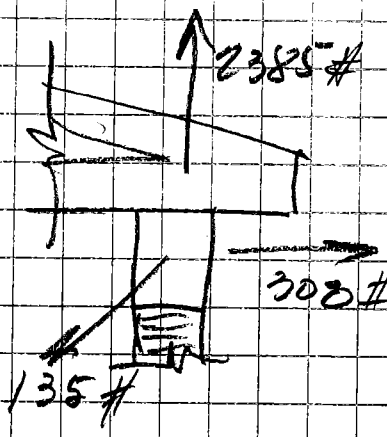
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Guest House (Cont.)

IG1 Connection



Try Ng Vue NV358-16

$$CSR = \frac{2385}{3367} + \frac{135}{4175} + \frac{308}{3117} = 0.84$$

OK

Use A

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Guest House Roof Trusses (Cont.)

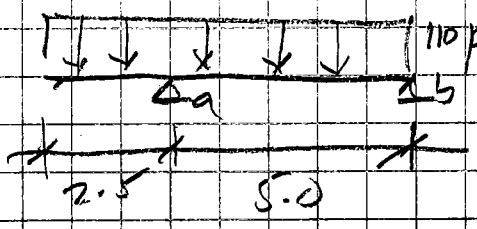
Jacks J1:

TA = 25 × 2 = 15

LL = 20 psf

DL = 35 psf

DL



$R_a = \frac{110 \times 7.5' \times 3.75'}{5} = 619\#\downarrow$

$R_b = 110 \times 7.5 - 619 = 206\#\downarrow$

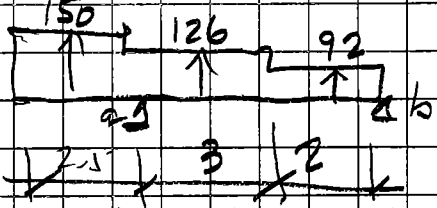
0.6D+0.6W

net

uplift: 80.7 − 6 = 75 psf

DL: 69 − 6 = 63 psf

LL: 17.6 − 6 = 46 psf

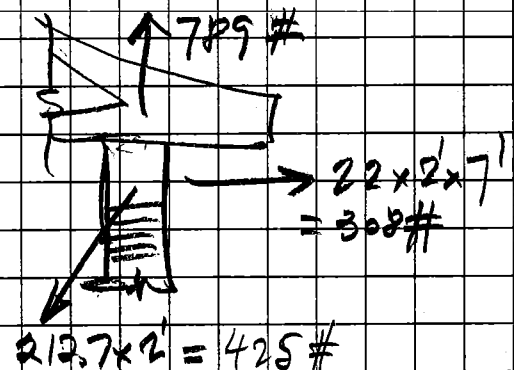


$R_a = (150 \times 2.5' \times 6.25' + 126 \times 3' \times 3.5' + 92 \times 2' \times 1') \div 5 = 789\#\uparrow$

$R_b = (150 \times 2.5 + 126 \times 3 + 92 \times 2) - 789 = 148\#\uparrow$

Connector:

Try Nu-Lue NUHTA16 w/ 18+6 Nails



$CSR = \frac{789}{2645} + \frac{425}{1575} + \frac{308}{2175} = 0.71 < 1.0$ OK

(D)

BOLTS

Table 12G BOLTS: Reference Lateral Design Values, Z, for Double Shear (three member) Connections[1,2]

for sawn lumber or SCL main member with 1/4" ASTM A 36 steel side plates



Thickness			G=0.67 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.50 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch (N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood		G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods		G=0.35 Northern Species	
Main Member t_m in	Side Member t_s in	Bolt Diameter D in	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs	$Z_{\parallel}$ lbs	$Z_{\perp}$ lbs
1-1/2	1/4	1/2	1410	730	1150	550	1050	470	1030	480	970	420	900	380	880	370	780	310	760	290	730	290
		5/8	1760	810	1440	610	1310	530	1290	520	1210	470	1130	420	1100	410	970	350	950	330	910	320
		3/4	2110	890	1730	660	1580	590	1550	560	1450	520	1350	460	1320	450	1170	370	1140	360	1100	350
		7/8	2460	960	2020	720	1840	630	1800	600	1690	550	1580	500	1540	490	1360	410	1330	390	1280	370
		1	2810	1020	2310	770	2100	680	2060	650	1930	600	1800	540	1760	530	1560	440	1520	420	1460	410
1-3/4	1/4	1/2	1640	850	1350	640	1230	550	1200	530	1130	490	1050	450	1030	430	910	360	890	340	850	330
		5/8	2050	940	1680	710	1530	610	1500	600	1410	550	1310	490	1290	480	1130	400	1110	380	1070	370
		3/4	2460	1040	2020	770	1840	680	1800	660	1690	600	1580	540	1540	530	1360	430	1330	420	1280	410
		7/8	2870	1120	2350	840	2140	740	2110	700	1970	640	1840	580	1800	570	1590	470	1550	460	1490	430
		1	3280	1190	2690	890	2450	790	2410	750	2250	700	2100	630	2060	610	1820	510	1770	490	1710	470
2-1/2	1/4	1/2	1870	1210	1720	910	1650	790	1640	760	1590	700	1500	640	1470	610	1300	510	1270	490	1220	480
		5/8	2740	1340	2400	1020	2190	880	2150	860	2010	780	1880	700	1840	690	1620	580	1580	550	1520	530
		3/4	3520	1480	2880	1110	2630	980	2580	940	2410	860	2250	770	2200	750	1950	620	1900	600	1830	580
		7/8	4100	1600	3360	1200	3060	1050	3010	1010	2820	920	2630	830	2570	810	2270	680	2210	660	2130	610
		1	4690	1700	3840	1280	3500	1130	3440	1080	3220	1000	3000	900	2940	880	2590	730	2530	700	2440	680
3-1/2	1/4	1/2	1870	1240	1720	1100	1650	1030	1640	1010	1590	970	1540	890	1530	860	1450	720	1430	680	1410	670
		5/8	2740	1720	2510	1420	2410	1230	2390	1200	2330	1090	2260	980	2230	960	2110	810	2090	770	2060	740
		3/4	3800	2070	3480	1550	3340	1370	3320	1310	3220	1210	3120	1080	3080	1050	2720	870	2660	840	2560	810
		7/8	5060	2240	4630	1680	4290	1470	4210	1410	3940	1290	3680	1160	3600	1130	3180	950	3100	920	2980	860
		1	6520	2380	5380	1790	4900	1580	4810	1510	4510	1400	4200	1280	4110	1230	3630	1020	3540	980	3410	950
5-1/4	1/4	5/8	2740	1720	2510	1610	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120
		3/4	3800	2290	3480	2000	3340	1890	3320	1850	3220	1780	3120	1610	3090	1580	2920	1300	2890	1260	2840	1220
		7/8	5060	2930	4630	2530	4440	2210	4410	2110	4280	1930	4150	1750	4110	1700	3880	1420	3840	1380	3770	1290
		1	6520	3570	5960	2680	5720	2360	5670	2260	5510	2100	5330	1880	5280	1840	4990	1520	4930	1470	4850	1420
5-1/2	1/4	5/8	2740	1720	2510	1510	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120
		3/4	3800	2290	3480	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1650	2920	1360	2890	1320	2840	1280
		7/8	5060	2930	4630	2570	4440	2310	4410	2210	4280	2020	4150	1830	4110	1780	3880	1490	3840	1440	3770	1350
		1	6520	3640	5960	2810	5720	2460	5670	2370	5510	2200	5330	1980	5280	1930	4990	1600	4930	1540	4850	1490
7-1/2	1/4	5/8	2740	1720	2510	1510	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120
		3/4	3800	2290	3480	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1670	2920	1530	2890	1500	2840	1480
		7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2100	4110	2130	3880	1960	3840	1930	3770	1840
		1	6520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2690	4990	2180	4930	2100	4850	2030
9-1/2	1/4	3/4	3800	2290	3480	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1670	2920	1530	2890	1500	2840	1480
		7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2160	4110	2130	3880	1960	3840	1930	3770	1870
		1	6520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350
11-1/2	1/4	7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2160	4110	2130	3880	1960	3840	1930	3770	1870
		1	6520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350
13-1/2	1/4	1	6520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350

1. Tabulated lateral design values, Z, for bolted connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "full-body diameter" bolts (see Appendix Table L1) with bolt bending yield strength, F_{yb}, of 45,000 psi and dowel bearing strength, F_e, of 87,000 psi for ASTM A36 steel.

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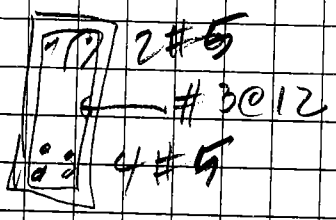
ENG: _____ SHEET: _____ OF _____

Roof Beams

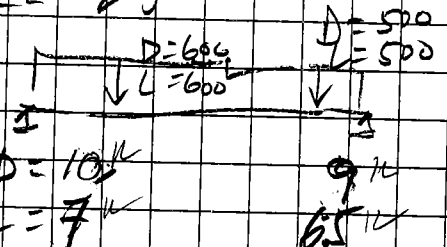
RB0: 8"x34"

L=D = 1200/2 = 600

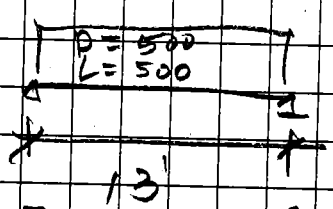
D=L = 100@5/2 = 500



L = 23'



RB1: 8x34

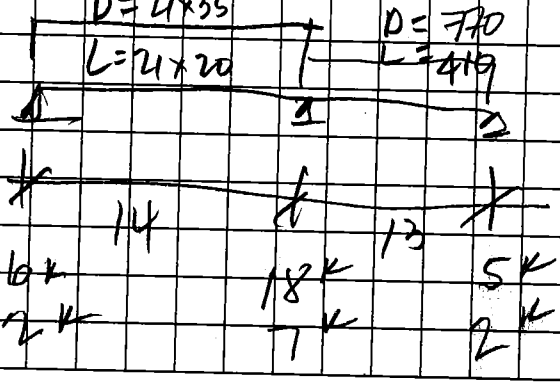


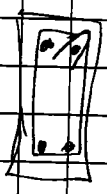


2#5 T&B
#3@12"

D 6 6
L 3 3

RB3-RB12: 8x34





2#5 T&B
#3@12"

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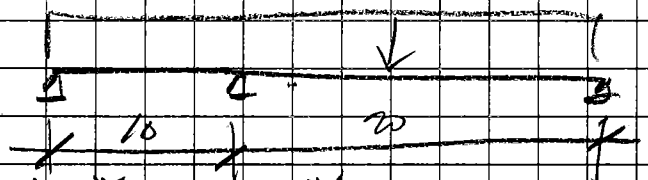
Garage Roof Beam RB9: 8x6

DL = 11.5' x 25psf

LL = 11.5 x 30psf

eyebrow: 0.67 x 1.5 x .15 = 0.145 k/f

2#6 T&B
4#4 E.F.
#3 @ 16"



	A		B
D	1.2k	19.3k	7.6k
L	0.4	7.2	3.0

GF Consulting Engineers Inc.
13170 SW 128th Street #204
Miami,FL
305-971-4323

JOB TITLE LAMAS RESIDENCE GUEST HOUSE

JOB NO. 23-08 SHEET NO.
CALCULATED BY FH DATE 9/6/20
CHECKED BY FH DATE

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) = 0.87
Base pressure (qh) = **58.1 psf**
GCpi = +/-0.18

Edge Strip (a) = 3.0 ft
End Zone (2a) = 6.0 ft
Zone 2 length = 12.0 ft

Wind Pressure Coefficients

Surface	CASE A θ = 16.3 deg GCpf	w/-GCpi	w/+GCpi	CASE B GCpf	w/-GCpi	w/+GCpi
1	0.50	0.68	0.32	-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87
3	-0.45	-0.27	-0.63	-0.37	-0.19	-0.55
4	-0.40	-0.22	-0.58	-0.45	-0.27	-0.63
5				0.40	0.58	0.22
6				-0.29	-0.11	-0.47
1E	0.75	0.93	0.57	-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25
3E	-0.65	-0.47	-0.83	-0.53	-0.35	-0.71
4E	-0.59	-0.41	-0.77	-0.48	-0.30	-0.66
5E				0.61	0.79	0.43
6E				-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1	39.4	18.4		-15.7	-36.6
2	-29.6	~~-50.5~~	× 0.6 = 17.8	-29.6	-50.5
3	-15.8	~~-36.7~~	× 0.6 = 9.5	-11.0	-31.9
4	-12.5	-33.4		-15.7	-36.6
5	51.9 × 0.6 = 31.1 psf			33.7	12.8
6				-6.4	-27.3
1E	54.2	33.3		-17.4	-38.3
2E	-51.7	-72.6		-51.7	-72.6
3E	-27.3	-48.2		-20.3	-41.2
4E	-23.7	-44.6		-17.4	-38.3
5E	77.9 × 0.6 = 46.7 psf			45.9	25.0
6E				-14.5	-35.4

Parapet

Windward parapet = #REF! psf (GCpn = +1.5)
Leeward parapet = #REF! psf (GCpn = -1.0)

Windward roof overhangs = 40.7 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

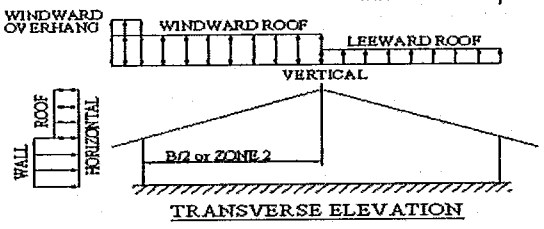
Interior Zone: Wall 51.8 psf
Roof -13.8 psf **
End Zone: Wall 77.8 psf
Roof -24.4 psf **

Longitudinal direction (parallel to L)

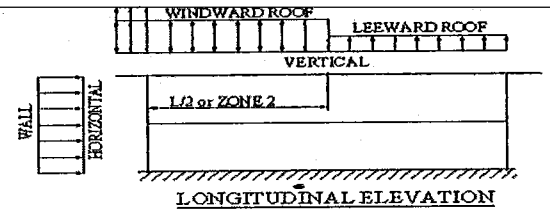
Interior Zone: Wall 40.1 psf
End Zone: Wall 60.4 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



TRANSVERSE ELEVATION



LONGITUDINAL ELEVATION

MAIN HOUSE WIND LOADING

COMPONENT AND CLADDING DESIGN
WIND PRESSURES (PSF)
ASD

ROOF SURFACE PRESSURES

Area	10 sf	20 sf	100 sf	200 sf		10	20	100	200
Negative Zone 1	-96.9	-96.9	-77.2	-77.2		58.1	58.1	46.3	46.3
Negative Zone 2e	-129.6	-119.0	-94.4	-83.8		77.8	71.4	56.6	50.3
Negative Zone 2r	-168.9	-152.2	-113.5	-96.9		101.3	91.3	68.1	58.1
Negative Zone 3	-137.5	-125.8	-98.5	-86.8		82.5	75.5	59.1	52.1
Positive All Zones	57.6	49.7	31.4	31.4		34.6	29.8	18.8	18.8
Overhang Zone 1	-117.8	-69.1	-130.9	-130.9		70.7	41.5	78.5	78.5
Overhang Zone 2e	-150.6	-147.5	-140.5	-137.5		90.4	88.5	84.3	82.5
Overhang Zone 2r	-189.8	-180.7	-159.6	-150.6		113.9	108.4	95.8	90.4
Overhang Zone 3	-197.8	-176.9	-128.5	-107.7		118.7	106.1	77.1	64.6

PARAPET SURFACE PRESSURES

SOLID PARAPET PRESSURE	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf		
CASE A: Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 2r :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0		
CASE B: Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0		
Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0		

WALL SURFACE PRESSURES

Area	10 sf	100 sf	200 sf	500 sf
NEGATIVE ZONE 4	90.3 -83.8	-72.2	-68.7	-64.1
NEGATIVE ZONE 5	62 -103.4	-80.3	-73.3	-64.1
POSITIVE ZONE 4 & 5	46.3 77.2	65.7	62.2	57.6

NOTES:
1) TABLE PRESSURES ARE FOR THE SQUARE FOOT (SF) TRIBUTARY AREA SHOWN. FOR OTHER TRIBUTARY AREAS, LINEARLY INTERPOLATE BETWEEN VALUES SHOWN ABOVE.
2) POSITIVE PRESSURES ACT TOWARD THE BUILDING. NEGATIVE PRESSURES ACT AWAY FROM THE BUILDING.
3) SEE DIAGRAMS FOR LOCATION OF ZONES.
4) PRESSURES SHOWN ARE ULTIMATE PRESSURES, MULTIPLY BY 0.6 FOR NOMINAL PRESSURES

a=5.5 ft

905F
-74.8

83.8
- 72.2
11.6

Guest House

COMPONENT AND CLADDING DESIGN
WIND PRESSURES (PSF)
ROOF SURFACE PRESSURES

ASD

Area	10 sf	20 sf	100 sf	200 sf		10	20	100	200
Negative Zone 1	-86.0	-86.0	-68.5	-68.5		51.6	51.6	41.1	41.1
Negative Zone 2e	-115.0	-105.6	-83.8	-74.3		69	63.4	50.3	44.6
Negative Zone 2r	-149.8	-135.1	-100.7	-86.0		89.9	81.1	60.4	51.6
Negative Zone 3	-147.3	-133.2	-100.5	-86.4		88.4	79.9	60.3	51.8
Positive All Zones	51.1	44.1	27.9	27.9		30.7	26.5	16.7	16.7
Overhang Zone 1	-104.5	-61.3	-116.2	-116.2		62.7	36.8	69.7	69.7
Overhang Zone 2e	-133.6	-130.9	-124.7	-122.0		80.2	78.5	74.8	73.2
Overhang Zone 2r	-168.4	-160.4	-141.6	-133.6		101.0	96.2	85	80.2
Overhang Zone 3	-200.7	-178.6	-127.2	-105.0		120.4	107.2	76.3	63

PARAPET SURFACE PRESSURES								
SOLID PARAPET PRESSURE	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf		
CASE A: Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 2r :	0.0	0.0	0.0	0.0	0.0	0.0		
Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0		
CASE B: Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0		
Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0		

WALL SURFACE PRESSURES				
Area	10 sf	100 sf	200 sf	500 sf
NEGATIVE ZONE 4	44.6 -74.3	-64.1	-61.0	-56.9
NEGATIVE ZONE 5	55.1 -91.8	-71.3	-65.1	-56.9
POSITIVE ZONE 4 & 5	41.1 68.5	58.3	55.2	51.1

NOTES:
1) TABLE PRESSURES ARE FOR THE SQUARE FOOT (SF) TRIBUTARY AREA SHOWN. FOR OTHER TRIBUTARY AREAS, LINEARLY INTERPOLATE BETWEEN VALUES SHOWN ABOVE.
2) POSITIVE PRESSURES ACT TOWARD THE BUILDING. NEGATIVE PRESSURES ACT AWAY FROM THE BUILDING.
3) SEE DIAGRAMS FOR LOCATION OF ZONES.
4) PRESSURES SHOWN ARE ULTIMATE PRESSURES, MULTIPLY BY 0.6 FOR NOMINAL PRESSURES

-	a=3.0 ft

SEE SHEET A-3 FOR THIS ENTRY ROOF

CEMENT TILE ROOF SYSTEM THROUGHOUT

EOS 4"x12"

R.D.

EOS 4"x14"

+1" ROOF HIGH POINT ABOVE TRAP DRAIN

SLOPE 1/4"/FT

+6" ROOF H ABOVE

(A) +3000 −3300

(B) +1500 −1478

(D) +709 −946

(D) 1043 −1944

(A)(E) 3266 −4262

(D) +1664 −1616

(K)(G) 3652 −3113

(B)(E) +2090 −2153

(D) +786 −1318

(A) +2635 −2381

(D) +619 −852

+6855 (N) −5654

+2255 (B) −2076

+660# (D) −742

(D) +786 −1318

+1980 (B) −1656

(B) +2147 −1938

+1749 (C) −1450

(D) +1210 −1313

(N) +4220 −3996

(D) +619 −852

(D) +965 −1198

(B) +786 −1318

+7723 (N) −6605

+8973 (N) −7636

+1664 −1616 (D)

+3236 (A)(E) −3592

+2255 (B) −2076

+2452 (A) −2558

+1870 (B) −1754

+2046 (B) −1903

+619 (D) −852

+4525 (N) −4404

(D) +786 −1318

+619 (D) −852

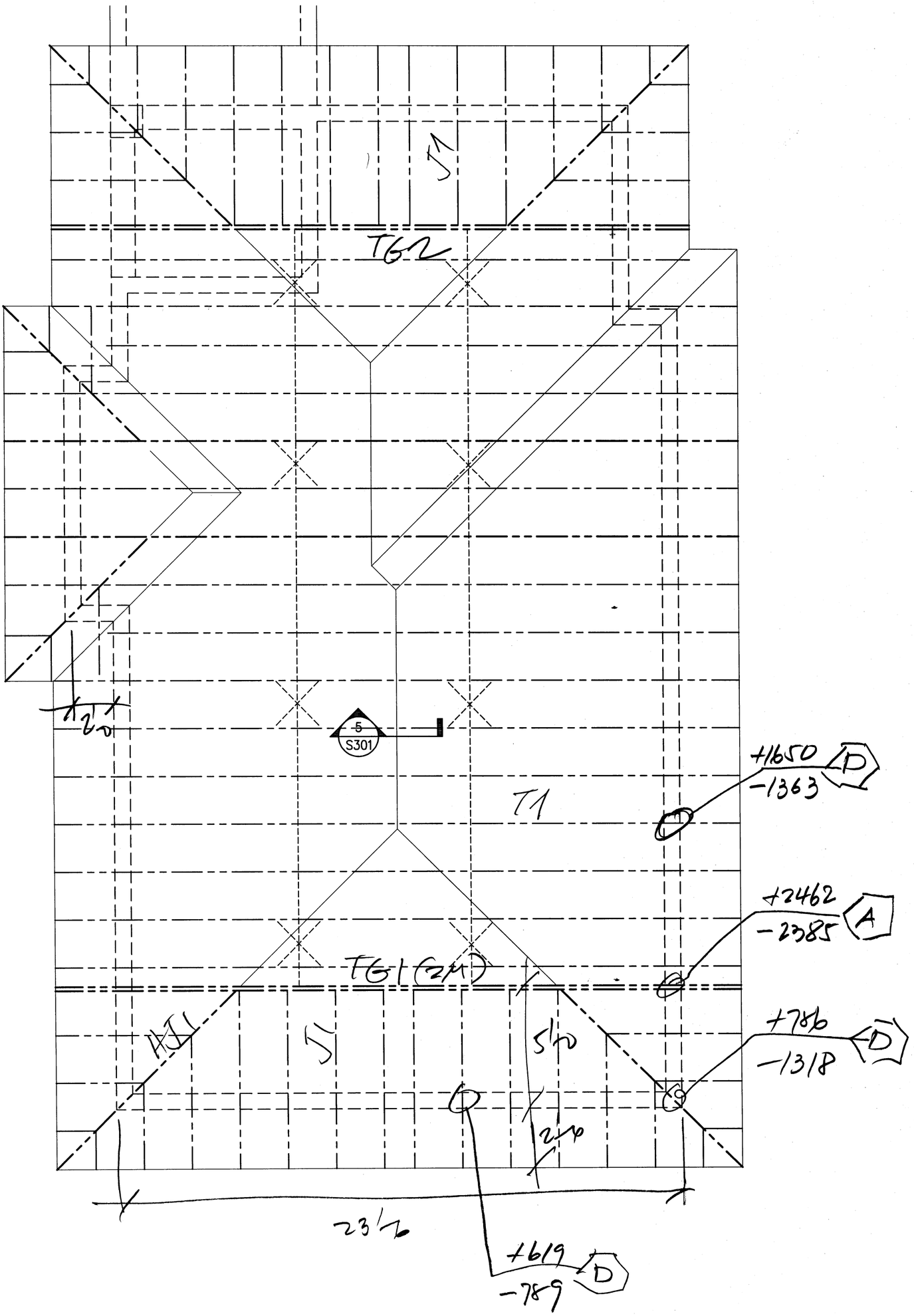
T1 T2 T3 T4 T5 T6 T7 T8 T9 T10 T11 T12 T13

TG1 TG2 TG3 TG4 TG5 TG6 TG7 TG8 TG10 TG11

HJ1 HJ2 HJ3 HJ4 HJ5

J1 J2 J3

3M 2M


T1
TG2
T1
5
S301
+1650
-1363
D
T1
+2462
-2385
A
TG1(2M)
T1
T1
5%
+786
-1318
D
2%
23%
+619
-789
D

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	34.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	13.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	5.1	7.1	5.1	7.1	
2	8.3	7.7	8.3	7.7	
3	5.1	6.1	5.1	6.1	
4	7.5	11.3	7.5	11.3	
5	5.1	6.1	5.1	6.1	
6	5.1	11.3	5.1	11.3	
7	5.1	6.1	5.1	6.1	
8	7.5	7.7	7.5	7.7	
9	5.1	6.1	5.1	6.1	
10	5.1	6.1	5.1	6.1	
11	3.1	4.6	3.1	4.6	
12	3.1	4.6	3.1	4.6	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	23	0	OK
2	0	25	0	OK
3	0	20	0	OK
4	0	37	0	OK
5	0	20	0	OK
6	0	37	0	OK
7	0	20	0	OK
8	0	25	0	OK
9	0	20	0	OK
10	0	20	0	OK
11	0	15	0	OK
12	0	15	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	3.8	-3.8	OK
2	4.2	-4.2	OK
3	3.3	-3.3	OK
4	6.1	-6.1	OK
5	3.3	-3.3	OK
6	6.1	-6.1	OK
7	3.3	-3.3	OK
8	4.2	-4.2	OK
9	3.3	-3.3	OK
10	3.3	-3.3	OK
11	2.5	-2.5	OK
12	2.5	-2.5	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		
Bot		0#5x11.00 ft		
d	2.0	3.5	2.0	in
+As	0.62	0.62	0.62	in²
c/d	0.06	0.06	0.06	
+ϕMn	88	88	88	
Top		2#5 Cont		
Top	2#5		2#5	
d'	2.0	2.0	2.0	in
-As	1.24	0.62	1.24	in²
c/d	0.09	0.06	0.09	
-ϕMn	-171	-88	-171	
As min	0.00	0.35	0.00	in²

DEFLECTIONS

Comb		
D	0.00	in
L	0.00	in
Lr, S	0.00	in
LS	0.00	in
LT	0.01	in
Ieff D	28,168	in4
Ieff L	28,168	in4
Ieff LS	28,168	in4
λ	1.78	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	24.3		-24.3	
ϕVs	26.4		-26.4	
ϕVn	50.7		-50.7	
Av/s	0.018		0.018	OK
Min Av/s	N.A.		N.A.	
s	12.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.57	OK
Max. Steel Area As max Ratio	0.24	OK
Rebars Min Spacing Smin Ratio ..	10.00	NG
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	9999	480	0.03	OK
D	Long-Term	9999	240	0.02	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

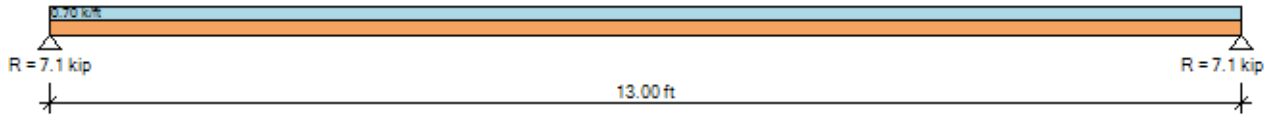
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



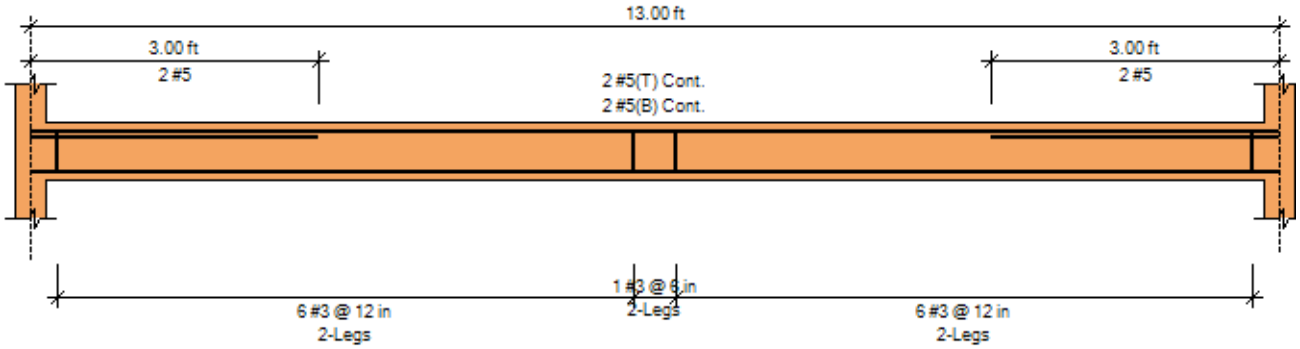
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	34.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	14.00	ft	1	Pinned
2	13.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	5.4	7.6	17.5	24.5	5.0	7.0
2	2	7.7	7.7	24.5	24.5	7.0	7.0
3	3	5.4	6.5	17.5	21.0	5.0	6.0
4	4	7.1	10.1	22.7	32.2	6.5	9.1
5	5	5.4	6.5	17.5	21.0	5.0	6.0
6	6	5.4	10.1	17.5	32.2	5.0	9.1
7	7	5.4	6.5	17.5	21.0	5.0	6.0
8	8	7.1	7.7	22.7	24.5	6.5	7.0
9	9	5.4	6.5	17.5	21.0	5.0	6.0
10	10	5.4	6.5	17.5	21.0	5.0	6.0
11	11	3.3	4.9	10.5	15.7	3.0	4.5
12	12	3.3	4.9	10.5	15.7	3.0	4.5

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	0	20	-33	17	0	OK
2	0	20	-33	17	0	OK
3	0	17	-28	14	0	OK
4	0	27	-44	22	0	OK
5	0	17	-28	14	0	OK
6	0	27	-44	22	0	OK
7	0	17	-28	14	0	OK
8	0	20	-33	17	0	OK
9	0	17	-28	14	0	OK
10	0	17	-28	14	0	OK
11	0	13	-21	11	0	OK
12	0	13	-21	11	0	OK

SHEAR FORCES Vu (kip)

Comb					
1	3.3	-8.1	7.7	-2.6	OK
2	3.4	-8.1	7.7	-2.6	OK
3	2.9	-6.9	6.6	-2.2	OK
4	4.5	-10.7	10.1	-3.4	OK
5	2.9	-6.9	6.6	-2.2	OK
6	4.5	-10.7	10.1	-3.4	OK
7	2.9	-6.9	6.6	-2.2	OK
8	3.4	-8.1	7.7	-2.6	OK
9	2.9	-6.9	6.6	-2.2	OK
10	2.9	-6.9	6.6	-2.2	OK
11	2.1	-5.2	5.0	-1.7	OK
12	2.1	-5.2	5.0	-1.7	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		2#5 Cont		
Bot		0#5x11.00 ft		0#5x11.00 ft		
d	2.0	3.5	2.0	3.5	2.0	in
+As	0.62	0.62	0.62	0.62	0.62	in²
c/d	0.06	0.06	0.06	0.06	0.06	
+ϕMn	88	88	88	88	88	
Top		2#5 Cont		2#5 Cont		
Top	0#5		0#5		0#5	
d'	2.0	2.0	2.0	2.0	2.0	in
-As	0.62	0.62	0.62	0.62	0.62	in²
c/d	0.06	0.06	0.06	0.06	0.06	
-ϕMn	-88	-88	-88	-88	-88	
As min	0.00	0.26	0.41	0.21	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	24.3		-24.3	24.3		-24.3	
ϕVs	26.4		-26.4	26.4		-26.4	
ϕVn	50.7		-50.7	50.7		-50.7	
Av/s	0.018		0.018	0.018		0.018	OK
Min Av/s	N.A.		N.A.	N.A.		N.A.	
s	12.0	12.0	12.0	12.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	

DEFLECTIONS

Comb			
D	0.00	0.00	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.01	0.00	in
Ieff D	28,168	28,168	in4
Ieff L	28,168	28,168	in4
Ieff LS	28,168	28,168	in4
λ	1.78	1.78	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.66	OK
Max. Steel Area As max Ratio	0.12	OK
Rebars Min Spacing Smin Ratio ..	0.50	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	9999	480	0.02	OK
D	Long-Term	9999	240	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

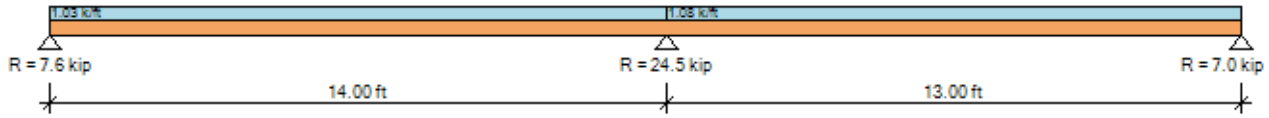


Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)



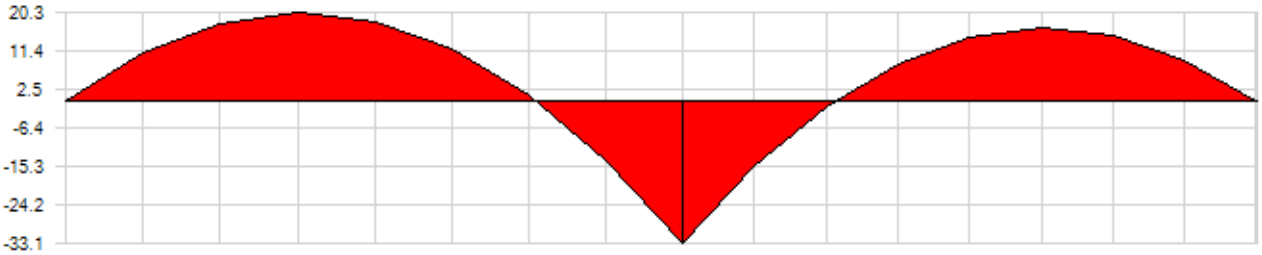
At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



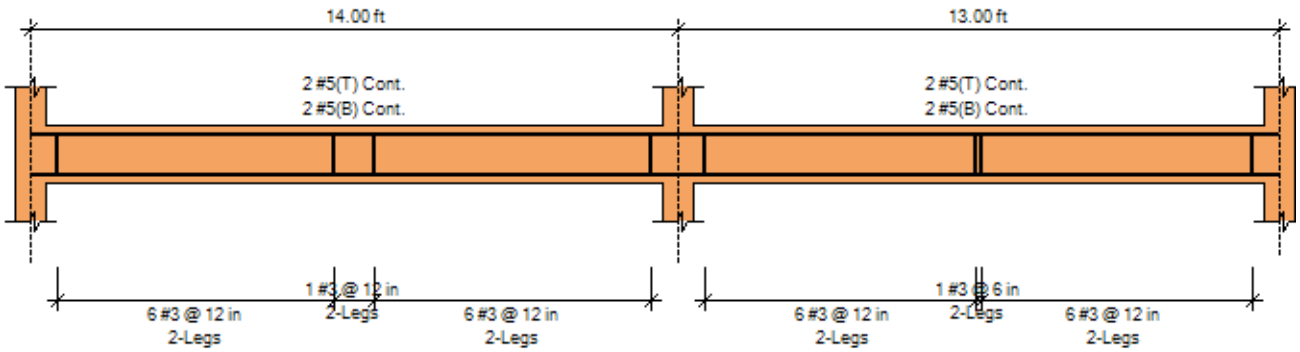
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	34.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	9.9	13.9	9.3	13.0	
2	16.5	15.2	15.4	14.2	
3	9.9	11.9	9.3	11.2	
4	14.9	22.5	13.9	20.9	
5	9.9	11.9	9.3	11.2	
6	9.9	22.5	9.3	20.9	
7	9.9	11.9	9.3	11.2	
8	14.9	15.2	13.9	14.2	
9	9.9	11.9	9.3	11.2	
10	9.9	11.9	9.3	11.2	
11	5.9	8.9	5.6	8.4	
12	5.9	8.9	5.6	8.4	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	78	0	OK
2	0	85	0	OK
3	0	66	0	OK
4	0	125	0	OK
5	0	66	0	OK
6	0	125	0	OK
7	0	66	0	OK
8	0	85	0	OK
9	0	66	0	OK
10	0	66	0	OK
11	0	50	0	OK
12	0	50	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	9.9	-9.6	OK
2	10.9	-10.4	OK
3	8.5	-8.2	OK
4	16.1	-15.4	OK
5	8.5	-8.2	OK
6	16.1	-15.4	OK
7	8.5	-8.2	OK
8	10.9	-10.4	OK
9	8.5	-8.2	OK
10	8.5	-8.2	OK
11	6.4	-6.2	OK
12	6.4	-6.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		
Bot		2#5x19.00 ft		
d	2.0	3.5	2.0	in
+As	0.62	1.24	0.62	in²
c/d	0.06	0.09	0.06	
+ϕMn	88	167	88	
Top		2#5 Cont		
Top	2#5		2#5	
d'	2.0	2.0	2.0	in
-As	1.24	0.62	1.24	in²
c/d	0.09	0.07	0.09	
-ϕMn	-171	-91	-171	
As min	0.00	0.85	0.00	in²

DEFLECTIONS

Comb		
D	0.05	in
L	0.00	in
Lr, S	0.11	in
LS	0.01	in
LT	0.11	in
Ieff D	28,964	in4
Ieff L	28,964	in4
Ieff LS	28,964	in4
λ	1.78	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	24.3		-24.3	
ϕVs	26.4		-26.4	
ϕVn	50.7		-50.7	
Av/s	0.018		0.018	OK
Min Av/s	0.007		N.A.	
s	12.0	12.0	12.0	OK
s max	16.0	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.69	OK
Max. Steel Area As max Ratio	0.24	OK
Rebars Min Spacing Smin Ratio ..	10.00	NG
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	0.75	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	2608	180	0.07	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	2558	480	0.19	OK
D	Long-Term	2558	240	0.09	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

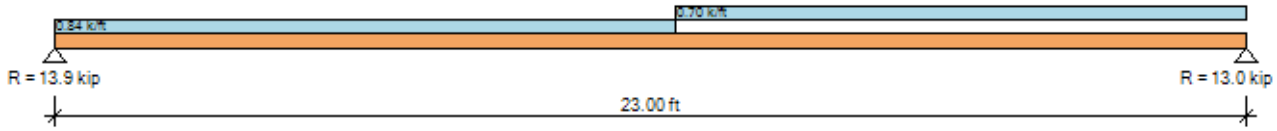
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

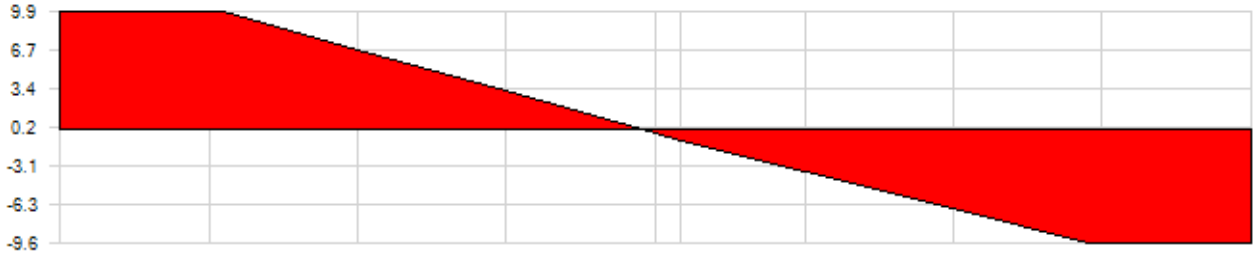
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



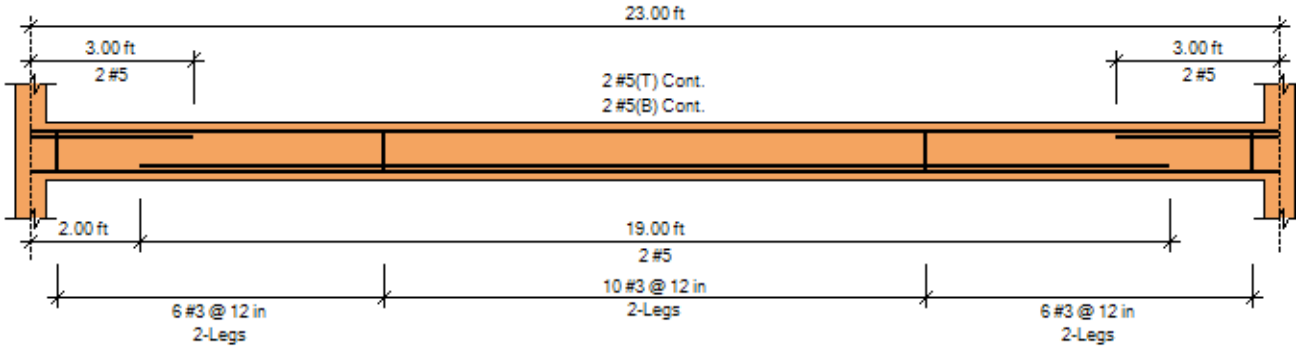
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Deep beam		NG
Total Height h	60.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	10.00	ft	1	Pinned
2	20.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	1.6	1.6	26.5	27.1	10.4	10.7
2	2	1.2	2.1	19.3	34.6	7.6	13.6
3	3	1.2	2.1	19.3	34.6	7.6	13.6
4	4	1.5	1.6	24.7	26.8	9.7	10.5
5	5	1.5	1.6	24.7	26.8	9.7	10.5
6	6	1.2	1.4	19.3	23.2	7.6	9.1
7	7	1.2	1.4	19.3	23.2	7.6	9.1
8	8	1.5	1.6	24.7	26.8	9.7	10.5
9	9	1.5	1.6	24.7	26.8	9.7	10.5
10	10	1.5	1.6	24.7	26.8	9.7	10.5
11	11	0.7	1.1	11.6	17.4	4.6	6.9
12	12	0.7	1.1	11.6	17.4	4.6	6.9

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	-3	1	-49	43	0	OK
2	-4	1	-63	55	0	OK
3	-4	1	-63	55	0	OK
4	-3	1	-49	43	0	OK
5	-3	1	-49	43	0	OK
6	-3	1	-42	37	0	OK
7	-3	1	-42	37	0	OK
8	-3	1	-49	43	0	OK
9	-3	1	-49	43	0	OK
10	-3	1	-49	43	0	OK
11	-2	1	-32	28	0	OK
12	-2	1	-32	28	0	OK

SHEAR FORCES Vu (kip)

Comb					
1	-6.0	-3.8	7.9	-3.0	OK
2	-7.7	-4.9	10.1	-3.8	OK
3	-7.7	-4.9	10.1	-3.8	OK
4	-5.9	-3.8	7.8	-3.0	OK
5	-5.9	-3.8	7.8	-3.0	OK
6	-5.2	-3.3	6.8	-2.6	OK
7	-5.2	-3.3	6.8	-2.6	OK
8	-5.9	-3.8	7.8	-3.0	OK
9	-5.9	-3.8	7.8	-3.0	OK
10	-5.9	-3.8	7.8	-3.0	OK
11	-3.9	-2.5	5.1	-1.9	OK
12	-3.9	-2.5	5.1	-1.9	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		2#6 Cont		
Bot		0#5x6.00 ft		0#5x6.00 ft		
d	2.0	2.0	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	0.88	0.88	in²
c/d	0.04	0.04	0.04	0.04	0.04	
+ϕMn	226	226	226	226	226	
Top		2#6 Cont		2#6 Cont		
Top	0#5		0#5		0#5	
d'	2.0	2.0	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	0.88	0.88	in²
c/d	0.04	0.04	0.04	0.04	0.04	
-ϕMn	-226	-226	-226	-226	-226	
As min	0.02	0.01	0.32	0.28	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	44.0		-44.0	44.0		-44.0	
ϕVs	35.9		-35.9	35.9		-35.9	
ϕVn	79.9		-79.9	79.9		-79.9	
Av/s	0.014		0.014	0.014		0.014	OK
Min Av/s	N.A.		N.A.	N.A.		N.A.	
s	16.0	6.0	16.0	16.0	5.0	16.0	OK
s max	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	

DEFLECTIONS

Comb			
D	0.00	0.00	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.00	0.01	in
Ieff D	153,720	153,720	in4
Ieff L	153,720	153,720	in4
Ieff LS	153,720	153,720	in4
λ	1.83	1.83	

LS = 0.50(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.37	OK
Max. Steel Area As max Ratio	0.10	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	9999	480	0.02	OK
D	Long-Term	9999	240	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

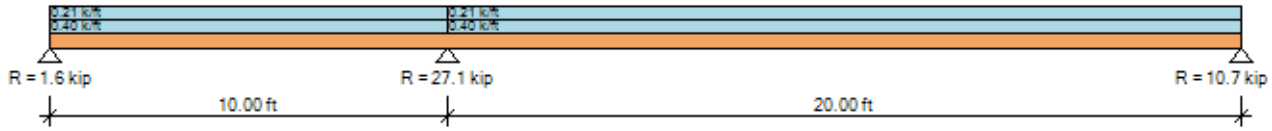


Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)



At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



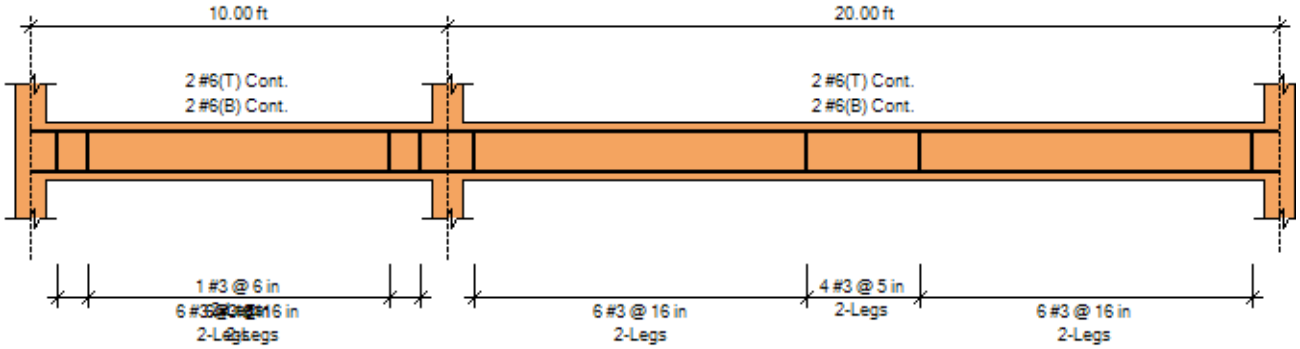
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Deep beam		NG
Total Height h	32.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	4.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	4.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	-5.0	-7.0	13.0	18.2	
2	-10.0	-8.5	23.5	20.9	
3	-5.0	-6.0	13.0	15.6	
4	-8.8	-14.0	20.9	32.4	
5	-5.0	-6.0	13.0	15.6	
6	-5.0	-14.0	13.0	32.4	
7	-5.0	-6.0	13.0	15.6	
8	-8.8	-8.5	20.9	20.9	
9	-5.0	-6.0	13.0	15.6	
10	-5.0	-6.0	13.0	15.6	
11	-3.0	-4.5	7.8	11.7	
12	-3.0	-4.5	7.8	11.7	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	-48	0	-130	OK
2	-46	0	-123	OK
3	-41	0	-111	OK
4	-56	0	-151	OK
5	-41	0	-111	OK
6	-56	0	-151	OK
7	-41	0	-111	OK
8	-46	0	-123	OK
9	-41	0	-111	OK
10	-41	0	-111	OK
11	-31	0	-83	OK
12	-31	0	-83	OK

SHEAR FORCES Vu (kip)

Comb				
1	-8.6	-7.5	7.5	OK
2	-9.9	-9.0	9.0	OK
3	-7.4	-6.5	6.5	OK
4	-15.6	-14.5	14.5	OK
5	-7.4	-6.5	6.5	OK
6	-15.6	-14.5	14.5	OK
7	-7.4	-6.5	6.5	OK
8	-9.9	-9.0	9.0	OK
9	-7.4	-6.5	6.5	OK
10	-7.4	-6.5	6.5	OK
11	-5.5	-4.8	4.8	OK
12	-5.5	-4.8	4.8	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x2.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.07	0.07	0.08	
+ϕMn	115	115	116	
Top		2#6 Cont		
Top	0#5		2#6	
d'	2.0	2.0	3.0	in
-As	0.88	0.88	1.76	in²
c/d	0.07	0.07	0.11	
-ϕMn	-115	-115	-221	
As min	0.57	0.00	0.80	in²

DEFLECTIONS

Comb			
D	0.00	0.00	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.00	0.01	in
Ieff D	24,433	25,323	in4
Ieff L	24,433	25,323	in4
Ieff LS	24,433	25,323	in4
λ	1.69	2.00	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	22.8		-22.8	22.8	
ϕVs	49.5		-49.5	24.8	
ϕVn	72.3		-72.3	47.5	
Av/s	0.037		0.037	0.018	OK
Min Av/s	N.A.		N.A.	0.007	
s	6.0	6.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	15.0	

REINFORCEMENT

Min. Steel Area As min Ratio	0.65	OK
Max. Steel Area As max Ratio	0.34	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	0.80	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	90	0.00	OK
B	Live Load	9999	180	0.00	OK
C	Long-Term	7735	240	0.03	OK
D	Long-Term	7735	120	0.02	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

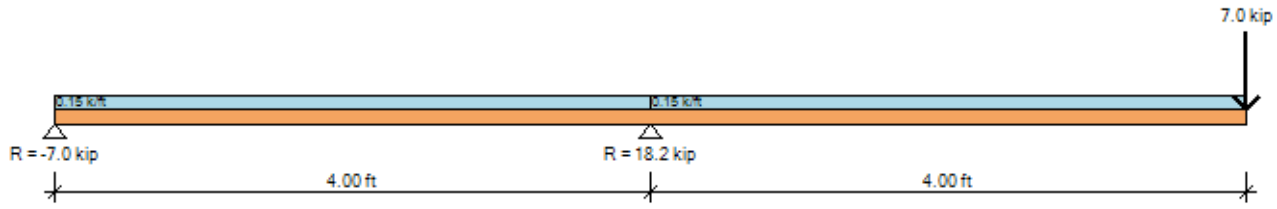


Comb				
1	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	OK

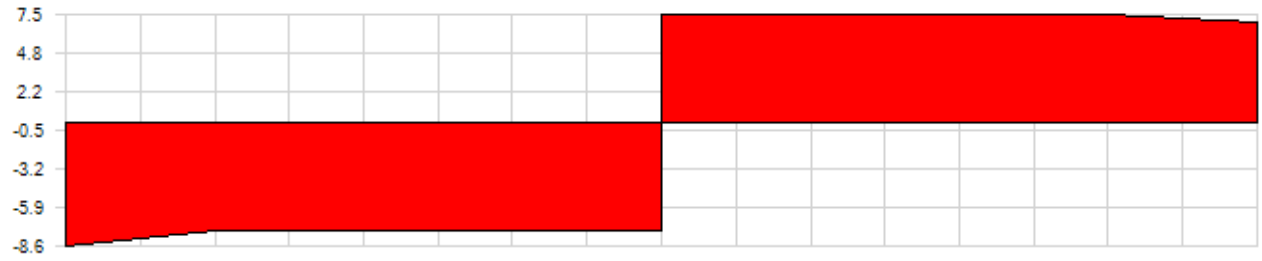
TORSION STRENGTH (k-ft)



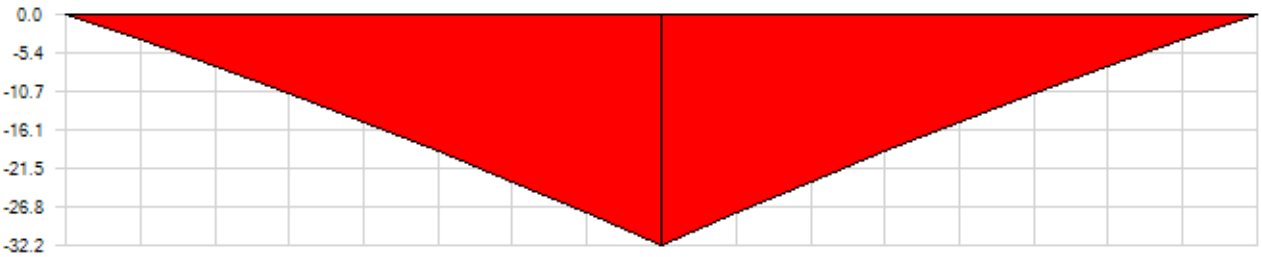
At/s (in)	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	OK



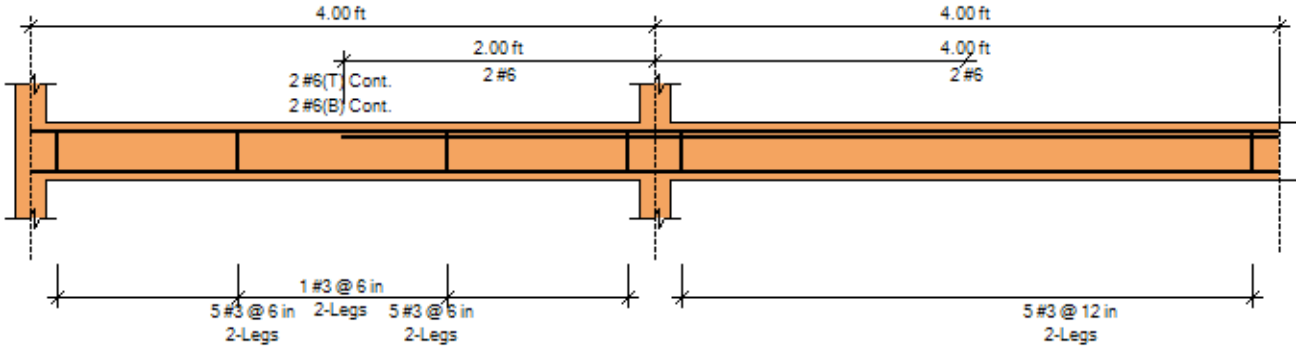
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ... Spandrel	Not deep beam		OK
Total Height h	27.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	5.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	8.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	13.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

PROPERTIES

Concrete f'c = 4.0 ksi Max. strain = 0.0030 OK Stress-strain = Hognestad

Density = 150 pcf $Ec = 57 * (f'c * 1000)^{1/2}$ $= 57 * (4000)^{1/2}$ = 3605 ksi ACI 19.2.2.1

Rupture $fr = 7.5 * (f'c * 1000)^{1/2} / 1000 = 7.5 * (4000)^{1/2} / 1000 = 0.47$ ksi ACI Eq. (19.2.3.1)

Steel fy = 60.0 ksi Stress-strain = Elasto-plastic Es = 29000 ksi ACI 19.2.2.1

Modular ratio $n = Es / Ec =$ 29000 / 3605 = 8.0

Selfweight SW = (8.0 * 27.0 + 8.0 * 5.0) / 144 * 0.15 = 0.3 k/ft

Effective width = $bfl + b + bfr =$ 0.0 + 8.0 + 8.0 = 16.0 in

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

BENDING DESIGN

Positive Bending

Max. positive moment Mu = 25.6 k-ft Comb = 1.2D+0.5L+1.6Lr Segment = Span 1

As1 = 2 #6 (Top Cont.) As6 = 0 #5 x 6.00 ft (Bot Center) As7 = 2 #6 (Bot Cont.)

As1 = 0.88 in² , d1 = 2.0 in As6 = 0.00 in² , d6 = 25.0 in As7 = 0.88 in² , d7 = 25.0 in

As1 = As1 - Al / 4 = 0.88 - 0.80 / 4 = 0.67 in² As7 = As7 - Al / 4 = 0.88 - 0.80 / 4 = 0.67 in²

Zero strain depth / d = k = 0.057 Cover d' = 2.0 in Eff. d = *h - d'* = 27.0 - 2.0 = 25.0 in

Strain1 = *(k * d - d1) / (k * d) * Max strain* = (0.057 * 25.0 - 2.0) / (0.057 * 25.0) * 0.0030 = -0.0012

Strain6 = *(k * d - d6) / (k * d) * Max strain* = (0.057 * 25.0 - 25.0) / (0.057 * 25.0) * 0.0030 = -0.0497

Strain7 = *(k * d - d7) / (k * d) * Max strain* = (0.057 * 25.0 - 25.0) / (0.057 * 25.0) * 0.0030 = -0.0497

As (in²)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (kip)	Ms = Fs(h/2-d) (k-ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.67	2.0	-0.0012	35.3	3.4	-23.8	-22.8	89.0
As6 = 0.00	25.0	-0.0497	60.0	0.0	0.0	0.0	0.0
As7 = 0.67	25.0	-0.0497	60.0	0.0	-40.4	38.7	89.0
					T = -64.1	Ms = 15.9	Is = 178.0

Conc. force fc = *C = Sum (fci) = Sum (fci * Aci)* = 64.1 kip

Conc. moment Mc = *Sum (Mci) = Sum (fci * (h / 2 - di))* = 69.0 k-ft

Nominal flexural strength Mn = *Mc + Ms* = 69.0 + 15.9 = 84.9 k-ft

Design flexural strength ϕMn = 0.9 * 84.9 = 76.4 k-ft ACI 21.2.2

Flexural ratio = *Mu / ϕMn* = 25.6 / 76.4 = 0.34 < 1.0 OK ACI 4.4.4

As min = *Max (3 * (f'c * 1000)$^{1/2}$, 200) / 1000 * b * d / fy*

As min = Max (3 * (4000)$^{1/2}$, 200) / 1000 * 8.0 * 25.0 / 60 = 0.67 in² ACI 9.6.1.2

As min = *4 / 3 * As req* = 4 / 3 * 0.23 = 0.31 in² (Controls) ACI 9.6.1.3

As min ratio = *As min / As* = 0.31 / 0.88 = 0.35 < 1.0 OK

Max. steel area for fy/Es + 0.003 tension strain = 4.41 in² > 0.88 in² OK ACI 21.2.2

Min clear spacing = *Max (diam, 1.0)* = Max (0.75, 1.0) = 1.0 in < 1.9 in OK ACI 25.2.1

Max spacing = *Smax = Min (15 * 40 / (2/3 * fy) - 2.5 * Cc, 12 * 40 / (2/3 * fy), 3 * t, 18 in)* ACI 24.3.2

= Min (15 * 40 / (2/3 * 60) - 2.5 * 1.6, 12 * 40 / (2/3 * 60), 3 * 27.0, 18) = 10.9 in > 2.8 in OK

Negative Bending

Max. negative moment Mu = 0.0 k-ft Comb = 1.2D+0.5L+1.6Lr Support = 1

As1 = 2 #6 (Bot Cont.) As6 = 0 #5 (Top Left) As7 = 2 #6 (Top Cont.)

As1 = 0.88 in² , d1 = 2.0 in As6 = 0.00 in² , d6 = 25.0 in As7 = 0.88 in² , d7 = 25.0 in

As1 = As1 - Al / 4 = 0.88 - 0.83 / 4 = 0.67 in² As7 = As7 - Al / 4 = 0.88 - 0.83 / 4 = 0.67 in²

Zero strain depth / d = k = 0.076 Cover d' = 2.0 in Eff. d = *h - d'* = 27.0 - 2.0 = 25.0 in

Strain1 = *(k * d - d1) / (k * d) * Max strain* = (0.076 * 25.0 - 25.0) / (0.076 * 25.0) * 0.0030 = -0.0001

Strain6 = *(k * d - d6) / (k * d) * Max strain* = (0.076 * 25.0 - 2.0) / (0.076 * 25.0) * 0.0030 = -0.0363

Strain7 = *(k * d - d7) / (k * d) * Max strain* = (0.076 * 25.0 - 2.0) / (0.076 * 25.0) * 0.0030 = -0.0363

As (in²)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (kip)	Ms = Fs(h/2-d) (k-ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.67	2.0	-0.0001	4.1	3.3	-2.8	-2.7	89.0
As6 = 0.00	25.0	-0.0363	60.0	2.2	0.0	0.0	0.0
As7 = 0.67	25.0	-0.0363	60.0	0.9	-40.4	38.7	89.0
					T = -43.2	Ms = 36.0	Is = 178.0

Conc. force fc = *C = Sum (fci) = Sum (fci * Aci)* = 43.2 kip

Conc. moment Mc = *Sum (Mci) = Sum (fci * (h / 2 - di))* = 45.7 k-ft

Nominal flexural strength Mn = *Mc + Ms* = 45.7 + 36.0 = 81.7 k-ft

Design flexural strength ϕMn = 0.9 * 81.7 = 73.5 k-ft ACI 21.2.2

Flexural ratio = *Mu / ϕMn* = 0.0 / 73.5 = 0.00 < 1.0 OK ACI 4.4.4

As min = *Max (3 * (f'c * 1000)$^{1/2}$, 200) / 1000 * b * d / fy*

As min = Max (3 * (4000)$^{1/2}$, 200) / 1000 * 8.0 * 25.0 / 60 = 1.33 in² ACI 9.6.1.2

As min = *4 / 3 * As req* = 4 / 3 * 0.00 = 0.00 in² (Controls) ACI 9.6.1.3

As min ratio = *As min / As* = 0.00 / 0.88 = 0.00 < 1.0 OK

Max. steel area for fy/Es + 0.003 tension strain = 4.41 in² > 0.88 in² OK ACI 21.2.2

Min clear spacing = *Max (diam, 1.0)* = Max (0.75, 1.0) = 1.0 in < 1.88 in OK ACI 25.2.1

Max spacing = *Smax = Min (15 * 40 / (2/3 * fy) - 2.5 * Cc, 12 * 40 / (2/3 * fy), 3 * t, 18 in)* ACI 24.3.2

= Min (15 * 40 / (2/3 * 60) - 2.5 * 1.6, 12 * 40 / (2/3 * 60), 3 * 27.0, 18) = 10.9 in > 2.75 in OK

SHEAR DESIGN

$\lambda s = \sqrt{2} / (1 + d / 10) = \sqrt{2} / (1 + 25.0 / 10) = 0.76$ ACI Eq. (22.5.5.1.3)

Max. $\phi Vs = \phi * 8 * (f'c * 1000)^{1/2} * b * d =$ 0.75 * 8 * (4000)½ * 8.0 * 25.0 / 1000 = 75.9 kip ACI 22.5.1.2

Min. Av/s ratio = *Max (0.75 * (f'c * 1000)½, 50) * b / (fy * 1000)* ACI 9.6.3.4

Min. Av/s ratio = Max (0.75 * (4000)½, 50) * 8.0 / 60000 = 0.007 in²/in

Positive Shear

Max. positive shear Vu = 4.9 kip Comb = 1.2D+0.5L+1.6Lr Segment = Span 1

Vc1 = *2 * (f'c * 1000)½ * b * d / 1000* = 2 * (4000)½ * 8.0 * 25.0 / 1000 = 25.3 kip ACI Table 22.5.5.1

Vc2 = *8 * λs * ρ⅓ * (f'c * 1000)½ * b * d / 1000* = 8 * 0.76 * (0.0044)⅓ * (4000)½ * 8.0 * 25.0 / 1000 = 12.8 kip

Vu = 4.9 kip < $\phi\sqrt{f'c} * b * d$ = 9.5 kip

No stirrups required ACI 9.6.3.1

Negative Shear

Max. Negative shear Vu = -7.5 kip Comb = 1.2D+0.5L+1.6Lr Segment = Span 1

Vc1 = *2 * (f'c * 1000)½ * b * d / 1000* = 2 * (4000)½ * 8.0 * 25.0 / 1000 = 25.3 kip ACI Table 22.5.5.1

Vc2 = *8 * λs * ρ⅓ * (f'c * 1000)½ * b * d / 1000* = 8 * 0.76 * (0.0044)⅓ * (4000)½ * 8.0 * 25.0 / 1000 = 12.8 kip

Vu = 7.5 kip < $\phi\sqrt{f'c} * b * d$ = 9.5 kip

No stirrups required ACI 9.6.3.1

TORSION DESIGN

*x1 = b - 2 * (cover + db / 2) =* 8.0 - 2 * (2.0 + 0.38 / 2) = 3.6 in

*y1 = h - 2 * (cover + db / 2) =* 27.0 - 2 * (2.0 + 0.38 / 2) = 22.6 in

*Aoh = x1 * y1 =* 3.6 * 22.6 = 82.0 in² *Ph = 2 * (x1 + y1) =* 2 * (3.6 + 22.6) = 52.5 in

*Acp = b * (h - tf) + beff * tf =* 8.0 * (27.0 - 5.0) + 16.0 * 5.0 = 256.0 in²

*Pcp = 2 * (beff + h) =* 2 * (16.0 + 27.0) = 86.0 in

Torsional threshold *ϕTth = ϕ√(f'c * 1000) / 1000 * Acp² / Pcp / 12* ACI 22.7.4.1

= 0.75 * √4000 / 1000 * 256.0² / 86.0 / 12 = 3.0 k-ft

Cracking torsional moment *ϕTcr = 4 * ϕTth =* 4 * 3.0 = 12.0 k-ft ACI 22.7.5.1

*Min Atransv / s = Max (0.375 * √(f'c * 1000) , 25) * b / (fy * 1000)* ACI 9.6.4.2

= Max (0.375 * √4000 , 25) * 8.0 / 60000 = 0.0033 in²/in

Positive Torsion

Max. positive torsion Tu = 5.5 k-ft Comb = 1.2D+0.5L+1.6Lr Segment = Span 1

Tu = 5.5 k-ft > ϕTth = 3.0 k-ft , Torsional effects must be considered ACI 22.7.4.1

Torsion cannot be reduced Use Tu = 5.5 k-ft ACI 22.7.3

Crushing limit = *ϕ (Vc * 1000 / (b * d) + 8 * √(f'c * 1000))* ACI 22.7.7.1

= 0.75 * (25.3 * 1000 / (8.0 * 25.0) + 8 * √4000) = 474.3 psi

Combined stress =*√((Vc * 1000 / (b * d))² + (Tu * 1000 * 12 * Ph / (1.7 * Aoh²)²)* ACI 22.7.7.1

= √((25.3 * 1000 / (8.0 * 25.0))² + (5.5 * 1000 * 12 * 52.5 / (1.7 * 82.0²)²) = 325.9 psi < 474.3 psi OK

Transverse reinf. *At/s = Tu * 12 / (2 * ϕ * Cot 45 * 0.85 * Aoh * fy)* ACI 22.7.6.1

= 5.5 * 12 / (2 * 0.75 * 1.0 * 0.85 * 82.0 * 60.0) = 0.0104 in²/in

Total transv. *Av/s / 2 + At/s =* 0.0262 / 2 + 0.0104 = 0.0236 in²/in > 0.0033 in²/in OK ACI 22.7.6.1

Stirrups max. spacing =*Min (s (shear), ph / 8, 12.0)* ACI 22.7.6.1

= Min (24.0, 52.5 / 8, 12.0) = 6.6 in >= s = 6.0 in OK

*Min Al = 5 * √(f'c * 1000) / 1000 * Acp / fy - Max (At/s, 25 * b / (fy * 1000)) * Ph* ACI 9.6.4.3

= 5 * √4000 / 1000 * 256.0 / 60.0 - Max (0.0104 , 25 * 8.0 / 60000) * 52.5 = 0.80 in²

Longitudinal reinf. *Al req = At/s * Ph * Cot² 45* = 0.0104 * 52.5 * 1.0 = 0.55 in² ACI 22.7.6.1

Since Al = 0.55 in² < Min Al = 0.80 in² , therefore use Al req = 0.80 in²

Use 3 #4 Continuous each side Al = 0.60 in² > 0.80 / 4 = 0.20 in² OK

2 #6 (Top Cont.) = 0.88 in² > 0.20 in² OK 2 #6 (Bot Cont.) = 0.88 in² > 0.20 in² OK

Negative Torsion

Max. negative torsion Tu = 8.2 k-ft Comb = 1.2D+0.5L+1.6Lr Segment = Span 1

Tu = 8.2 k-ft > ϕTth = 3.0 k-ft , Torsional effects must be considered ACI 22.7.4.1

Torsion cannot be reduced Use Tu = 8.2 k-ft ACI 22.7.3

Crushing limit = *ϕ (Vc * 1000 / (b * d) + 8 * √(f'c * 1000))* ACI 22.7.7.1

= 0.75 * (25.3 * 1000 / (8.0 * 25.0) + 8 * √4000) = 474.3 psi

Combined stress =*√((Vc * 1000 / (b * d))² + (Tu * 1000 * 12 * Ph / (1.7 * Aoh²)²)* ACI 22.7.7.1

= √((25.3 * 1000 / (8.0 * 25.0))² + (8.2 * 1000 * 12 * 52.5 / (1.7 * 82.0²)²) = 470.9 psi < 474.3 psi OK

Transverse reinf. *At/s = Tu * 12 / (2 * ϕ * Cot 45 * 0.85 * Aoh * fy)* ACI 22.7.6.1

= 8.2 * 12 / (2 * 0.75 * 1.0 * 0.85 * 82.0 * 60.0) = 0.0158 in²/in

Total transv. *Av/s / 2 + At/s* = 0.0262 / 2 + 0.0158 = 0.0262 in²/in > 0.0033 in²/in OK ACI 22.7.6.1

Stirrups max. spacing =*Min (s (shear), ph / 8, 12.0)* ACI 22.7.6.1

= Min (24.0, 52.5 / 8, 12.0) = 6.6 in >= s = 6.0 in OK

*Min Al = 5 * √(f'c * 1000) / 1000 * Acp / fy - Max (At/s, 25 * b / (fy * 1000)) * Ph* ACI 9.6.4.3

= 5 * √4000 / 1000 * 256.0 / 60.0 - Max (0.0158 , 25 * 8.0 / 60000) * 52.5 = 0.52 in²

Longitudinal reinf. *Al req = At/s * Ph * Cot² 45* = 0.0158 * 52.5 * 1.0 = 0.83 in² ACI 22.7.6.1

Since Al req = 0.8 in² > Min Al = 0.52 in² , therefore use Al req = 0.83 in²

Use 3 #4 Continuous each side Al = 0.60 in² > 0.83 / 4 = 0.21 in² OK

2 #6 (Top Cont.) = 0.88 in² > 0.21 in² OK 2 #6 (Bot Cont.) = 0.88 in² > 0.21 in² OK

DEFLECTIONS

Segment = Span 1 c = Neutral axis depth , *a1 * c² + a2 * c + a3 = 0*

-At mid-span

a1 = b =8.0

*a2 = 2 * n * (bot As cont + bot As center) + 2 * (n - 1) * top As cont + 2 * tf * (bf - b)*

= 2 * 8.0 * (0.88 + 0.00 + 2 * (8.0 - 1) * 0.88 + 2 * 5.0 * (16.0 - 8.0) = 98.7

*a3 = - 2 * n * (bot As cont * (h - d') + bot As center * (h - d') - top As cont * d') - tf² * (bf - b) + top As cont * tf*

= - 2 * 8.0 * (0.88 * (27.0 - 2.0) + 0.00 * (27.0 - 2.0) - 0.88 * 2.0) - 5.0² * (16.0 - 8.0) + 0.88 * 5.0 = -426.3

Solving the quadratic equation, c = 3.8 in

*Igross = I1 + I2 + A1 d² + A2 d² + (n-1) (top As cont * (cTop-d')² + bot As cont * (cBot-d')² + bot As center * (cBot-d')²)*

= 83 + 13122 + 40 * 9.1² + 216 * 1.9² + (8.0-1) (0.9 * (13.5-2.0)² + 0.9 * (13.5-2.0)² + 0.0 * (13.5-2.0)²) = 18984 in4

*Icr = b * c³ / 3 + (n - 1) * top As cont * (c - d')² + n * (bot As cont * (h - c - d')² + bot As center * (h - c - d')²)*

= 8.0 * 3.8³ / 3 + (8.0 - 1) * 0.88 * (3.8 - 2.0)² + 8.0 * (0.88 * (27.0 - 3.8 - 2.0)² + 0.00 * (27.0 - 3.8 - 2.0)²) = 3494 in4

*Mcr = Igross * fr / Ctop / 12* = 18984 * 0.47 / 13.5 / 12 = 48.7 k-ft ACI Eq. (24.2.3.5b)

Mcr / Ma D = 48.7 / Abs(14.0) = 3.48

Mcr / Ma D+L = 48.7 / Abs(14.0 + 0.0 + 5.5) = 3.48

Mcr / Ma LS = 48.7 / Abs(14.0 + 0.30 * (0.0 + 5.5)) = 3.11

*Ie D = Min (Igross, (Mcr / Ma D)³ * Igross + (1 - (Mcr / Ma D)³ * Icr))* ACI Eq. (24.2.3.5a)

= Min (18984, (3.48)³ * 18984 + (1 - (3.48)³ * 3494)) = 18984 in4

*Ie D+L = Min (Igross, (Mcr / Ma D+L)³ * Igross + (1 - (Mcr / Ma D+L)³ * Icr))* ACI Eq. (24.2.3.5a)

= Min (18984, (3.48)³ * 18984 + (1 - (3.48)³ * 3494)) = 18984 in4

*Ie LS = Min (Igross, (Mcr / Ma LS)³ * Igross + (1 - (Mcr / Ma LS)³ * Icr))* ACI Eq. (24.2.3.5a)

= Min (18984, (3.11)³ * 18984 + (1 - (3.11)³ * 3494)) = 18984 in4

Average Ie and deflections — ACI 24.2.3.6

Ie Avg D = Ie D center = 18984 in4

Ie Avg D+L = Ie D+L center = 18984 in4

Ie Avg LS = Ie LS center = 18984 in4

*Delta D = L / 4 * (M left * L / 2 + 2 * M center * L + M right * L / 2 + A + B) / (6 * Ec * Ie Avg D / 12)*

= 13.0 / 4 * (0 * 13.0 / 2 + 2 * 14 * 13.0 + 0 * 13.0 / 2 + 0.0 + 0.0 / (6 * 3605 * 18984 / 12) = 0.01 in

*Delta D+L = L / 4 * (M left * L / 2 + 2 * M center * L + M right * L / 2 + A + B) / (6 * Ec * Ie Avg D+L / 12)*

= 13.0 / 4 * (0 * 13.0 / 2 + 2 * 20 * 13.0 + 0 * 13.0 / 2 + 0.0 + 0.0 / (6 * 3605 * 18984 / 12) = 0.01 in

Delta L = Delta D+L - Delta D =0.01 - 0.01 = 0.00 in

*Delta D+LS = L / 4 * (M left * L / 2 + 2 * M center * L + M right * L / 2 + A + B) / (6 * Ec * Ie Avg LS / 12)*

= 13.0 / 4 * (0 * 13.0 / 2 + 2 * 16 * 13.0 + 0 * 13.0 / 2 + 0.0 + 0.0 / (6 * 3605 * 18984 / 12) = 0.01 in

Delta LS = Delta D+LS - Delta D =0.01 - 0.01 = 0.00 in

*λ = 2.0 / (1 + 50 * top As cont / (b * (h - d'))* = 2.0 / (1 + 50 * 0.88 / (8.0 * (27.0 - 2.0))) = 1.64 — ACI Eq. (24.2.4.1.1)

*Delta LT = Delta L + λ * (Delta D + Delta LS)* =0.00 + 1.64 * (0.01 + 0.00) = 0.01 in

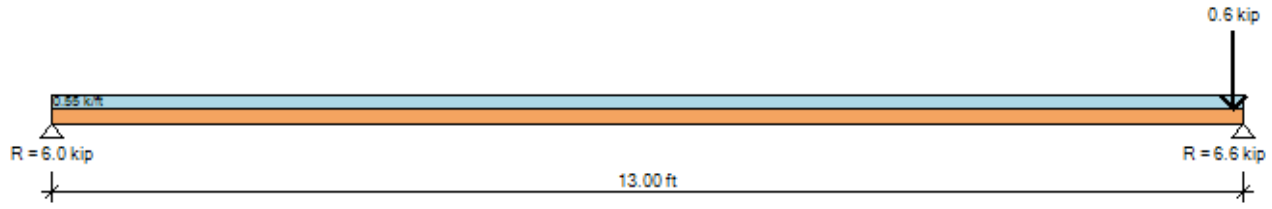
L / 180 = 13.0 / 180 * 12 = 0.87 in	> Delta L = 0.00 in	OK for Type A	ACI Table 24.2.2
L / 360 = 13.0 / 360 * 12 = 0.43 in	> Delta L = 0.00 in	OK for Type B	ACI Table 24.2.2
L / 480 = 13.0 / 480 * 12 = 0.33 in	> Delta LT = 0.01 in	OK for Type C	ACI Table 24.2.2
L / 240 = 13.0 / 240 * 12 = 0.65 in	> Delta LT = 0.01 in	OK for Type D	ACI Table 24.2.2

Type A - Flat roofs not supporting or attached to non-structural elements likely to be damaged.

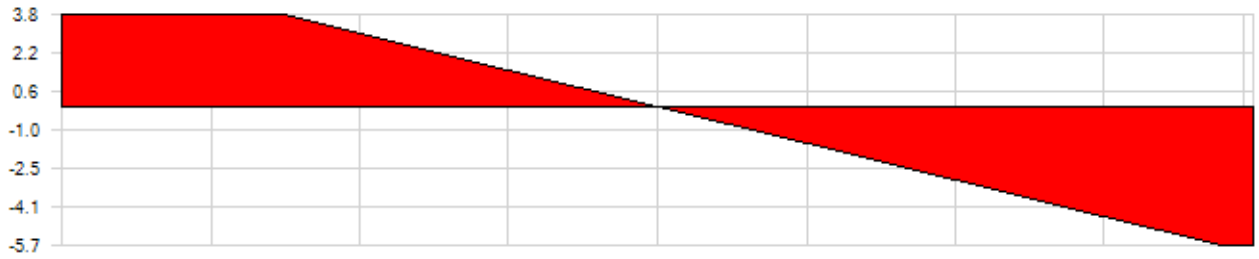
Type B - Floors not supporting or attached to non-structural elements likely to be damaged.

Type C - Roofs or floors supporting or attached to non-structural elements likely to be damaged.

Type D - Roofs or floors supporting or attached to non-structural elements not likely to be damaged.



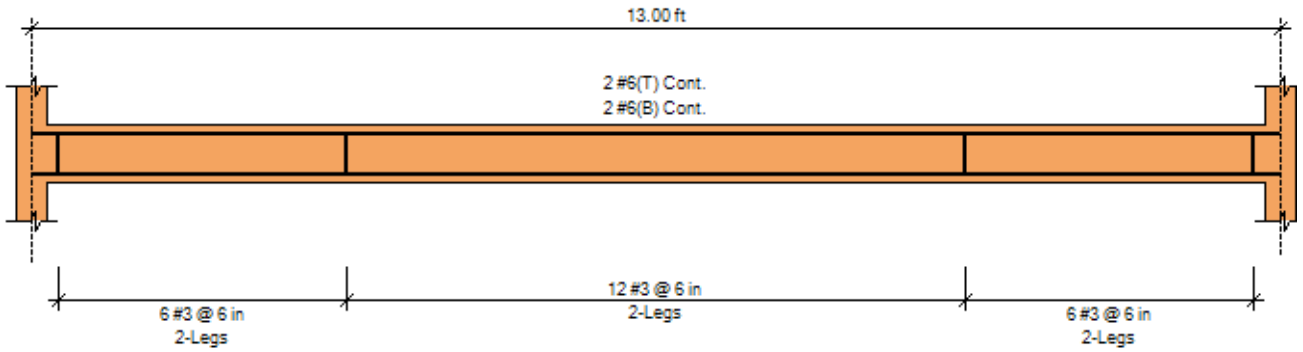
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

UNFACTORED FINAL LOADS (Selfweight calculated internally) (kip, ft, k-ft, psf)

Span 1	Start	End	Width	Dead	Live	RLive	Snow	Wind	Seismic
w1	0.00	13.00	1.00	395.00	0.00	260.00	0.00	0.00	0.00

Span 1	Dist	Dead	Live	RLive	Snow	Wind	Seismic
P1	12.90	0.4	0.0	0.4	0.0	0.0	0.0

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	32.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	5.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	8.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	13.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	4.6	6.4	5.0	7.0	
2	6.3	6.3	7.1	7.0	
3	4.6	5.5	5.0	6.0	
4	5.8	8.2	6.5	9.3	
5	4.6	5.5	5.0	6.0	
6	4.6	8.2	5.0	9.3	
7	4.6	5.5	5.0	6.0	
8	5.8	6.3	6.5	7.0	
9	4.6	5.5	5.0	6.0	
10	4.6	5.5	5.0	6.0	
11	2.7	4.1	3.0	4.5	
12	2.7	4.1	3.0	4.5	

LOAD COMBINATIONS

	SERVICE	FACTORED
1	D+L	1.4D
2	D+Lr	1.2D+1.6L+0.5Lr
3	D+S	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	1.2D+0.5L+1.6S
6	D+0.6W	1.2D+1.6Lr+0.5W
7	D+0.7E	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	0.9D+W
12	0.6D+0.7E	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	21	0	OK
2	0	21	0	OK
3	0	18	0	OK
4	0	27	0	OK
5	0	18	0	OK
6	0	27	0	OK
7	0	18	0	OK
8	0	21	0	OK
9	0	18	0	OK
10	0	18	0	OK
11	0	13	0	OK
12	0	13	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	3.6	-6.1	OK
2	3.6	-6.0	OK
3	3.1	-5.2	OK
4	4.6	-7.8	OK
5	3.1	-5.2	OK
6	4.6	-7.8	OK
7	3.1	-5.2	OK
8	3.6	-6.0	OK
9	3.1	-5.2	OK
10	3.1	-5.2	OK
11	2.3	-3.9	OK
12	2.3	-3.9	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	0.61	0.61	0.61	in²
c/d	0.05	0.05	0.05	
+ϕMn	83	83	83	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.61	0.61	0.61	in²
c/d	0.06	0.06	0.06	
-ϕMn	-81	-81	-81	
As min	0.00	0.27	0.00	in²

DEFLECTIONS

Comb		
D	0.00	in
L	0.00	in
Lr, S	0.00	in
LS	0.00	in
LT	0.01	in
Ieff D	30,721	in4
Ieff L	30,721	in4
Ieff LS	30,721	in4
λ	1.69	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	22.8		-22.8	
ϕVs	39.1		-39.1	
ϕVn	61.9		-61.9	
Av/s	0.029		0.024	OK
Min Av/s	N.A.		N.A.	
s	6.0	6.0	6.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.30	OK
Max. Steel Area As max Ratio	0.17	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	9999	480	0.02	OK
D	Long-Term	9999	240	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

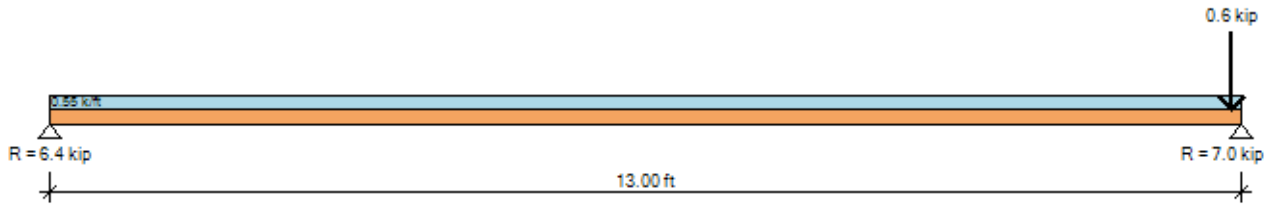
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

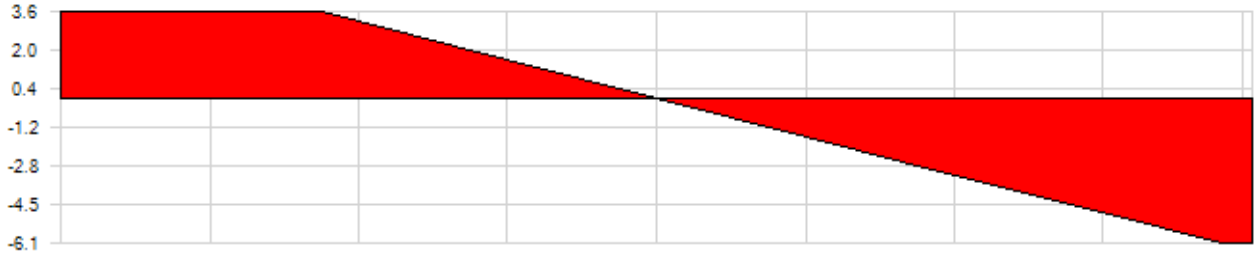
Comb			
1	0.0	-5.1	OK
2	0.0	-5.6	OK
3	0.0	-4.4	OK
4	4.9	-8.2	OK
5	0.0	-4.4	OK
6	4.9	-8.2	OK
7	0.0	-4.4	OK
8	0.0	-5.6	OK
9	0.0	-4.4	OK
10	0.0	-4.4	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0077	0.0129	
Al (in²)	1.08	0.81	
ϕTn (kip)	4.9	-8.2	
Stress (psi)	249.8	383.6	OK
Total At/s (in)	0.0222	0.0248	OK



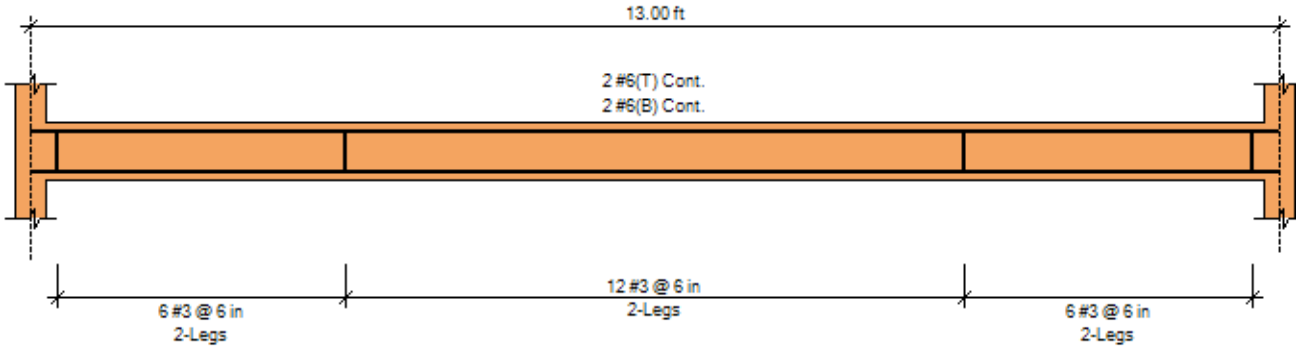
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

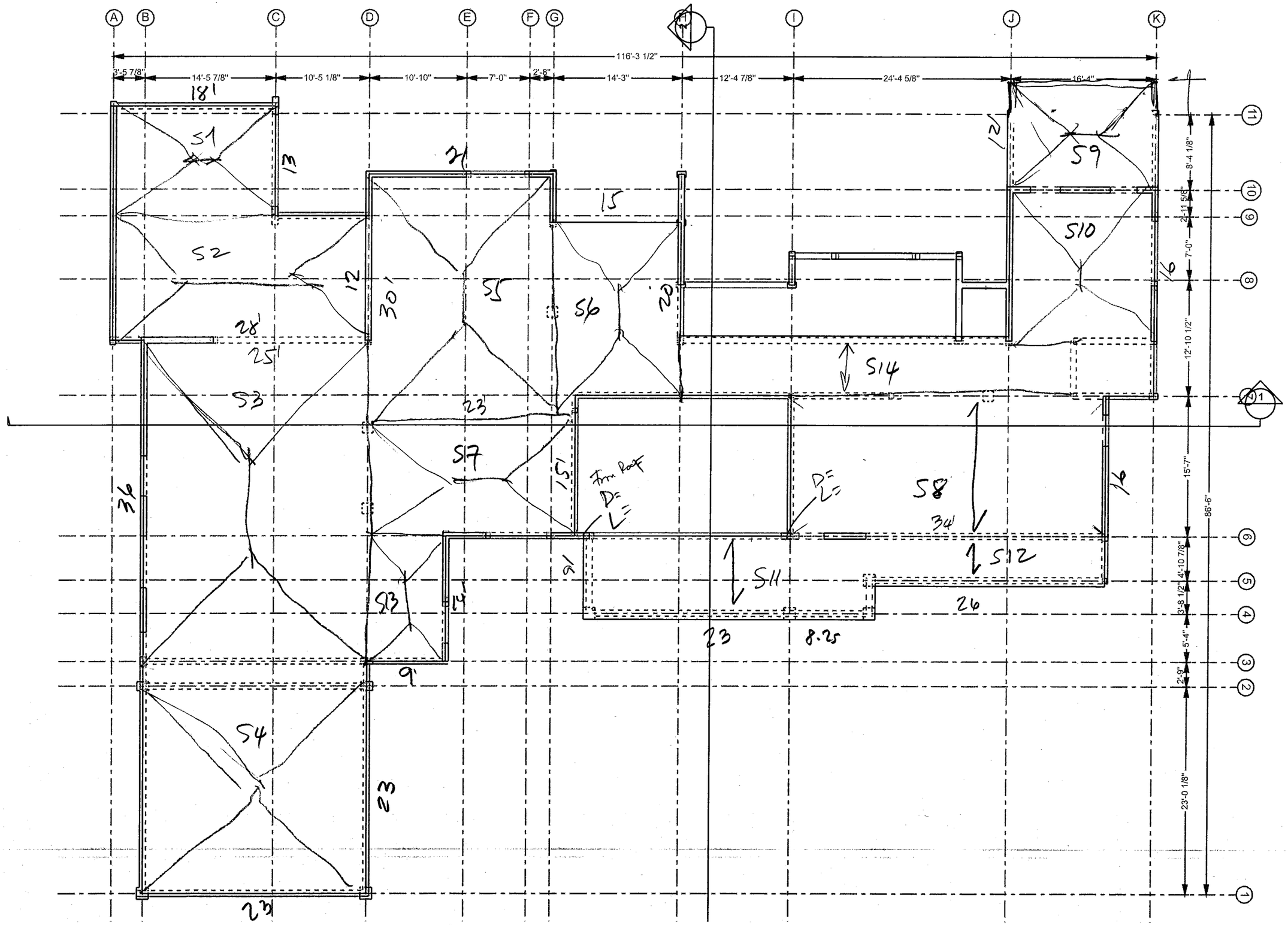


BEAM ELEVATION

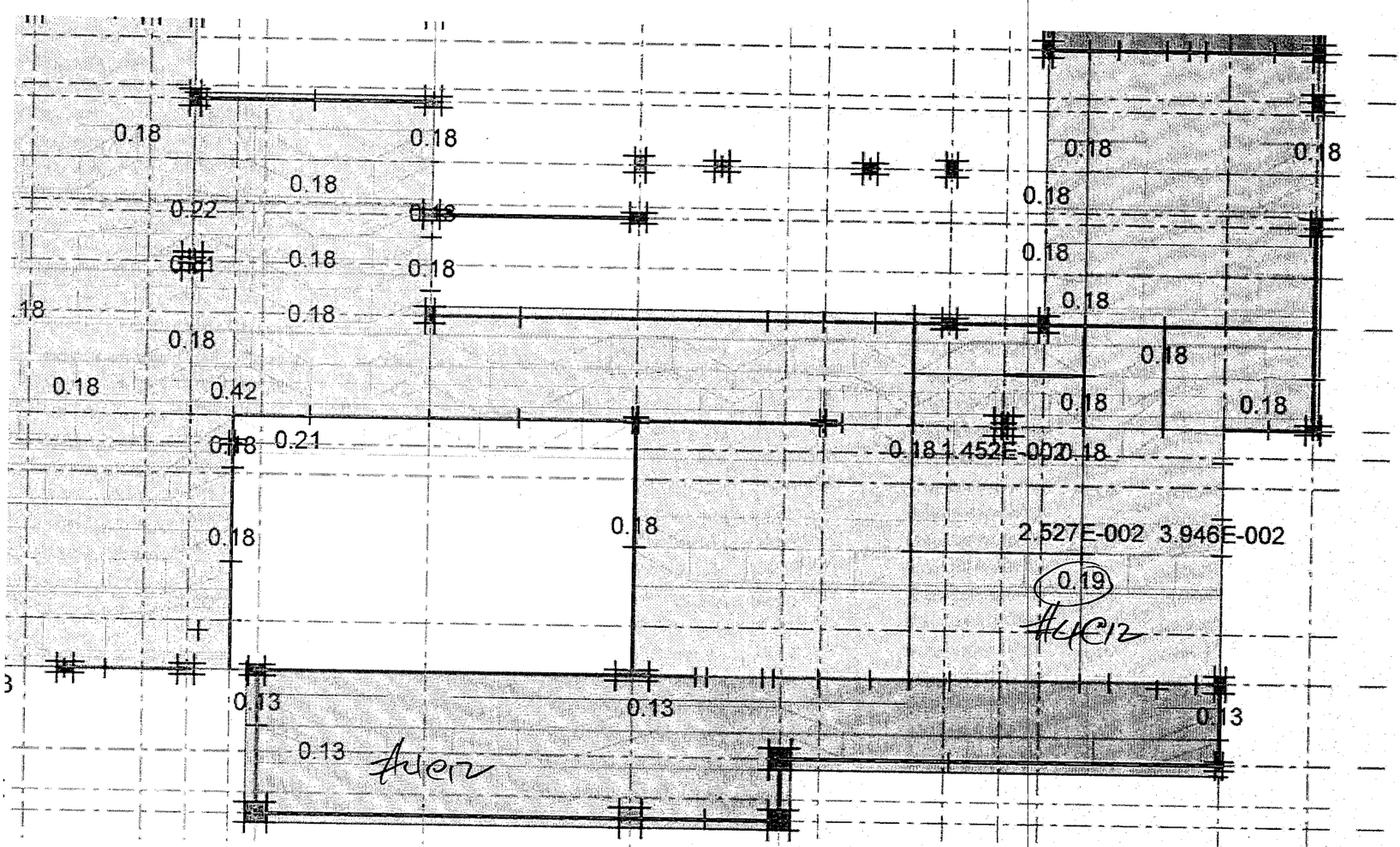


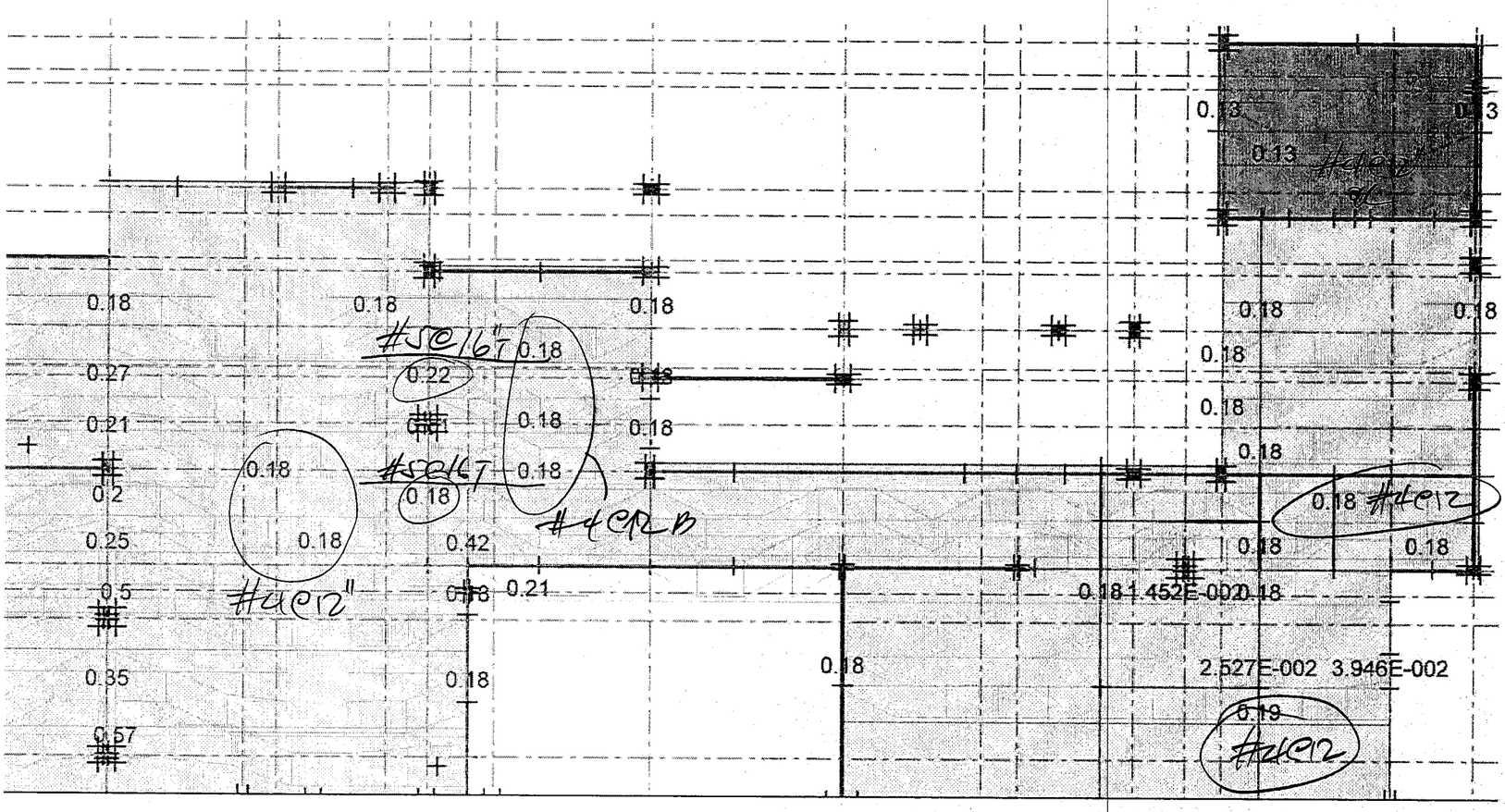
Second Floor Framing Calculations

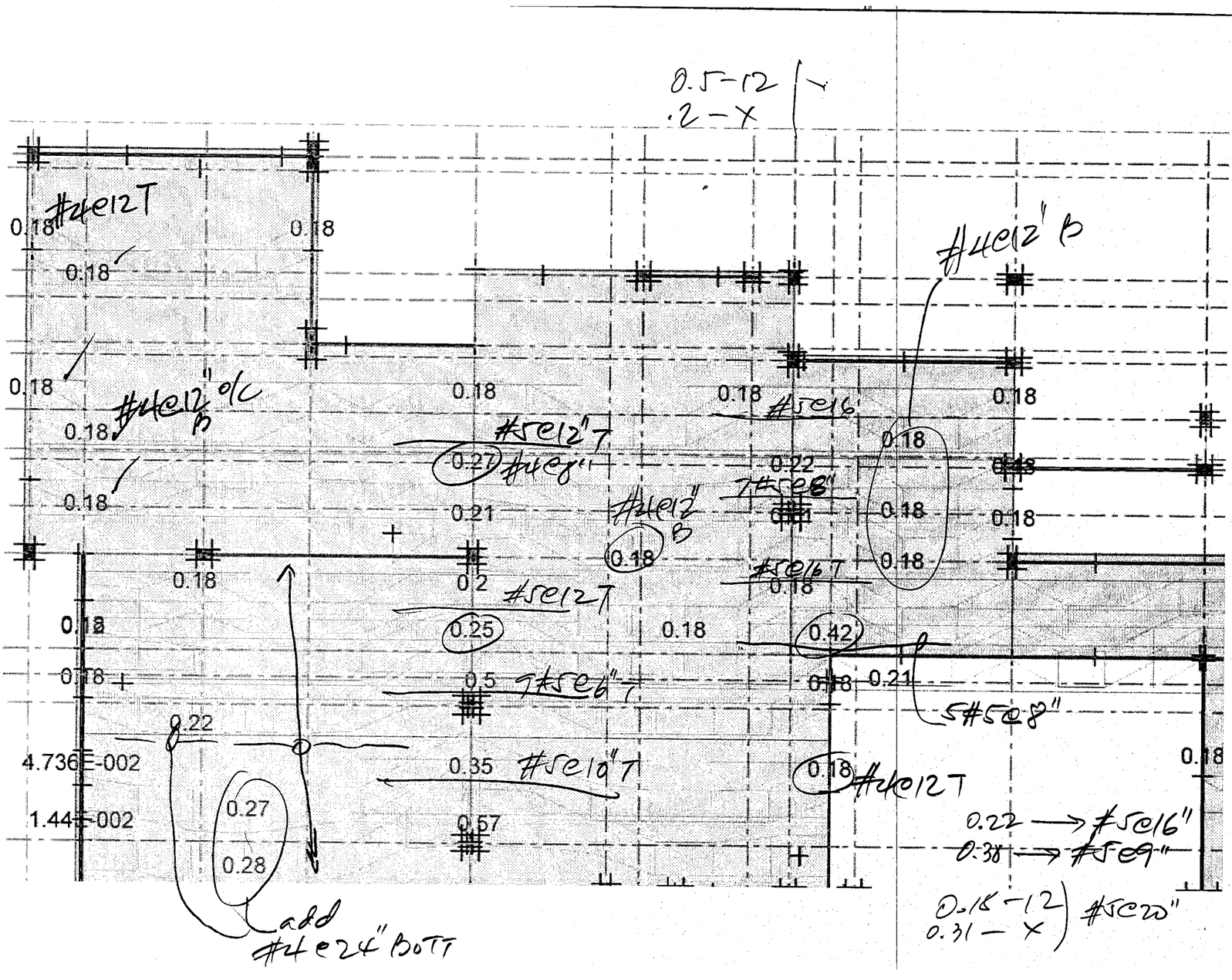
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www.gfce.net E-MAIL: GFCE@GFCE.NET

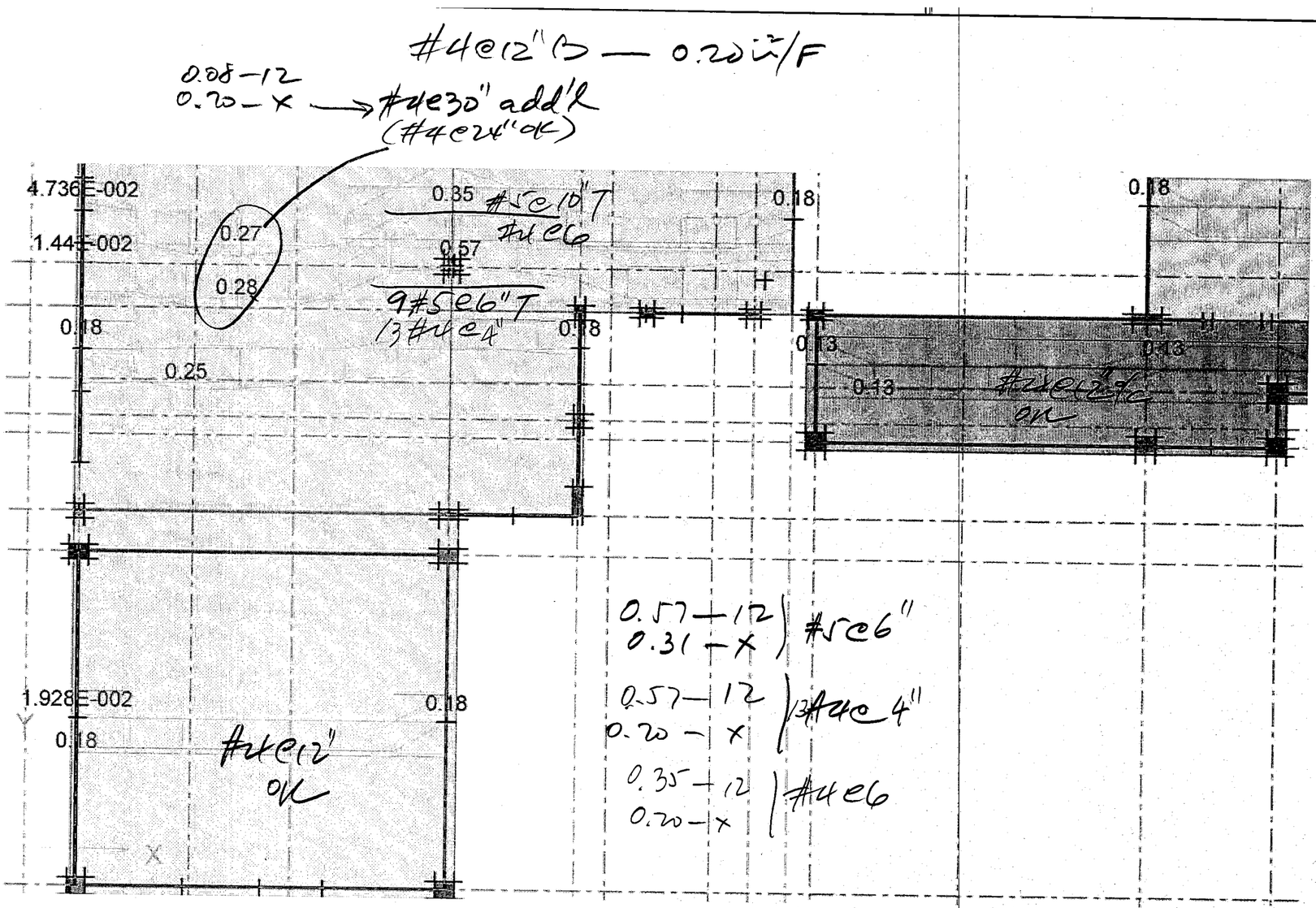

A
B
C
D
E
F
G
H
I
J
K
116'-3 1/2"
3'-5 7/8"
14'-5 7/8"
10'-5 1/8"
10'-10"
7'-0"
2'-8"
14'-3"
12'-4 7/8"
24'-4 5/8"
16'-4"
11
10
9
8
6
4
3
2
1
8'-4 1/8"
2'-11 5/8"
7'-0"
12'-10 1/2"
15'-7"
86'-6"
4'-10 7/8"
3'-8 1/2"
5'-4"
2'-9"
23'-0 1/8"
S1
S2
S3
S4
S5
S6
S7
S8
S9
S10
S11
S12
S13
S14
18'
13'
21'
15'
12'
30'
28'
25'
23'
15'
36'
14'
9'
23'
12'
20'
16'
34'
26'
23'
8.25
From Roof
D=
L=
D=
L=

0.18
0.18
0.18
#4@12 T
0.18
#4@12 B
0.18
0.18
0.23
0.18
0.25
#4@12" B
0.18
4.643E-003
0.27
#4@8"
ON #5@12 T
0.2
#4@12
5'-0
5'-0
0.25
0.3
0.18
1.762E
0.18
0.26
0.18
0.18
0.18
0.18
0.18
8.511E-002
0.18
0.13
4.87E-003
0.18
#4@12"
B
0.13
0.13
7.94E-003
#5@13" (12 x 3'-4")
#4@18" (21 x 4'-0")

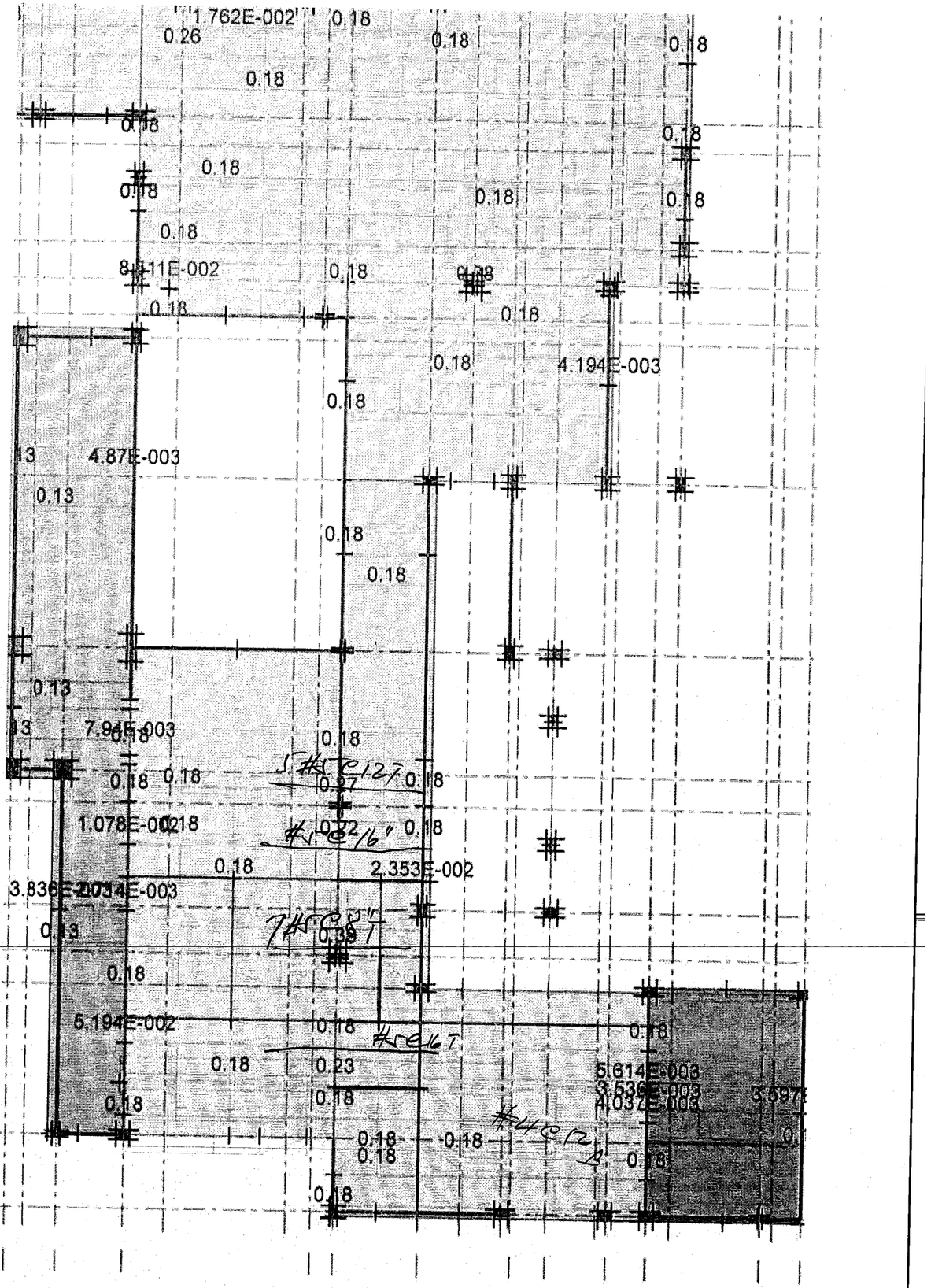

0.18
0.18
0.18
0.22
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.18
0.42
0.18
0.21
0.18
0.18
0.18
1.452E-002
0.18
2.527E-002
3.946E-002
0.19
0.13
0.13
0.13
0.13

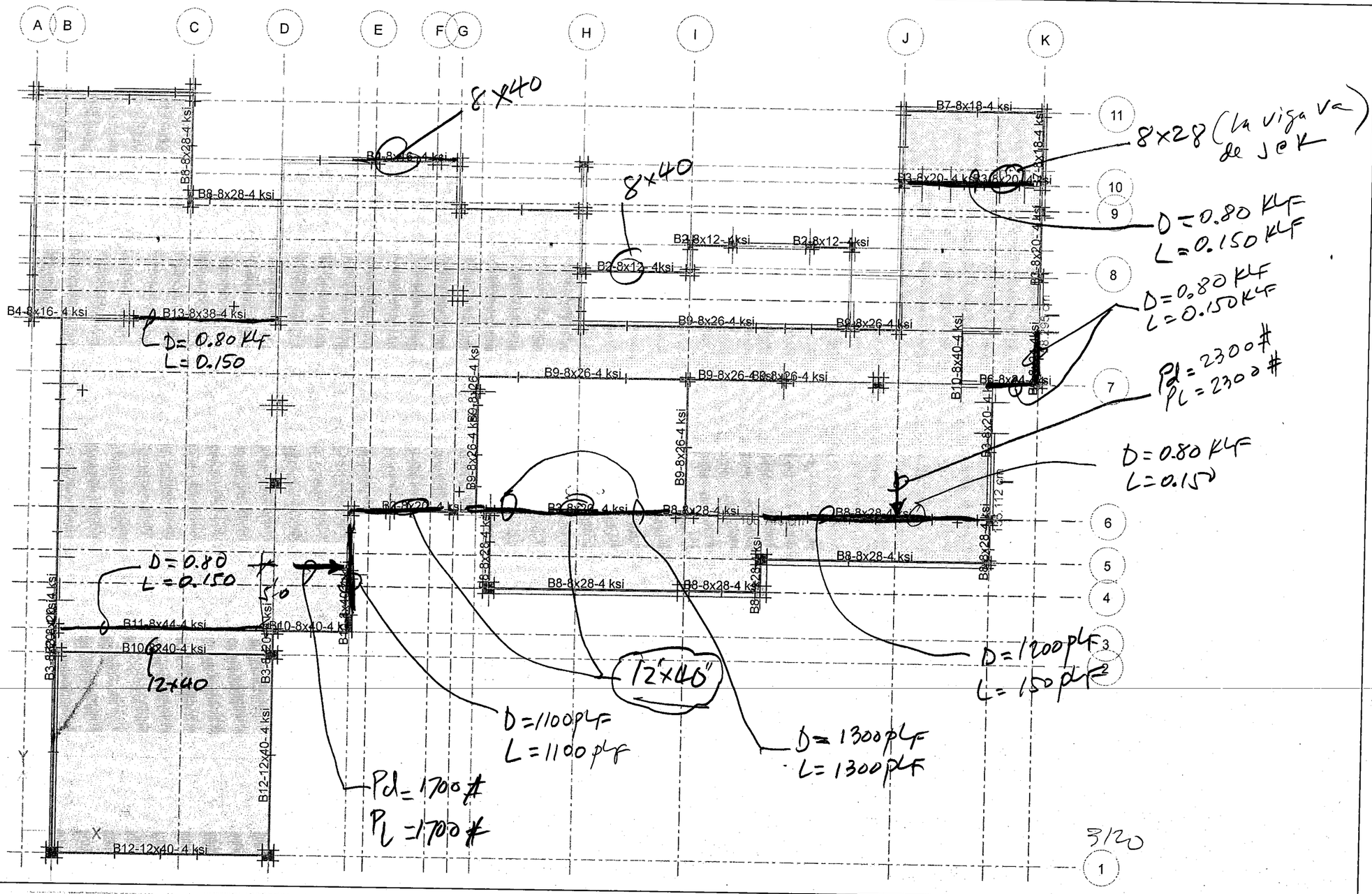

0.18
0.27
0.21
0.2
0.25
0.5
0.35
0.57
0.18
0.18
0.22
0.18
0.18
0.18
0.18
0.18
0.42
0.18
0.21
0.18
0.18
0.18
0.18
0.18

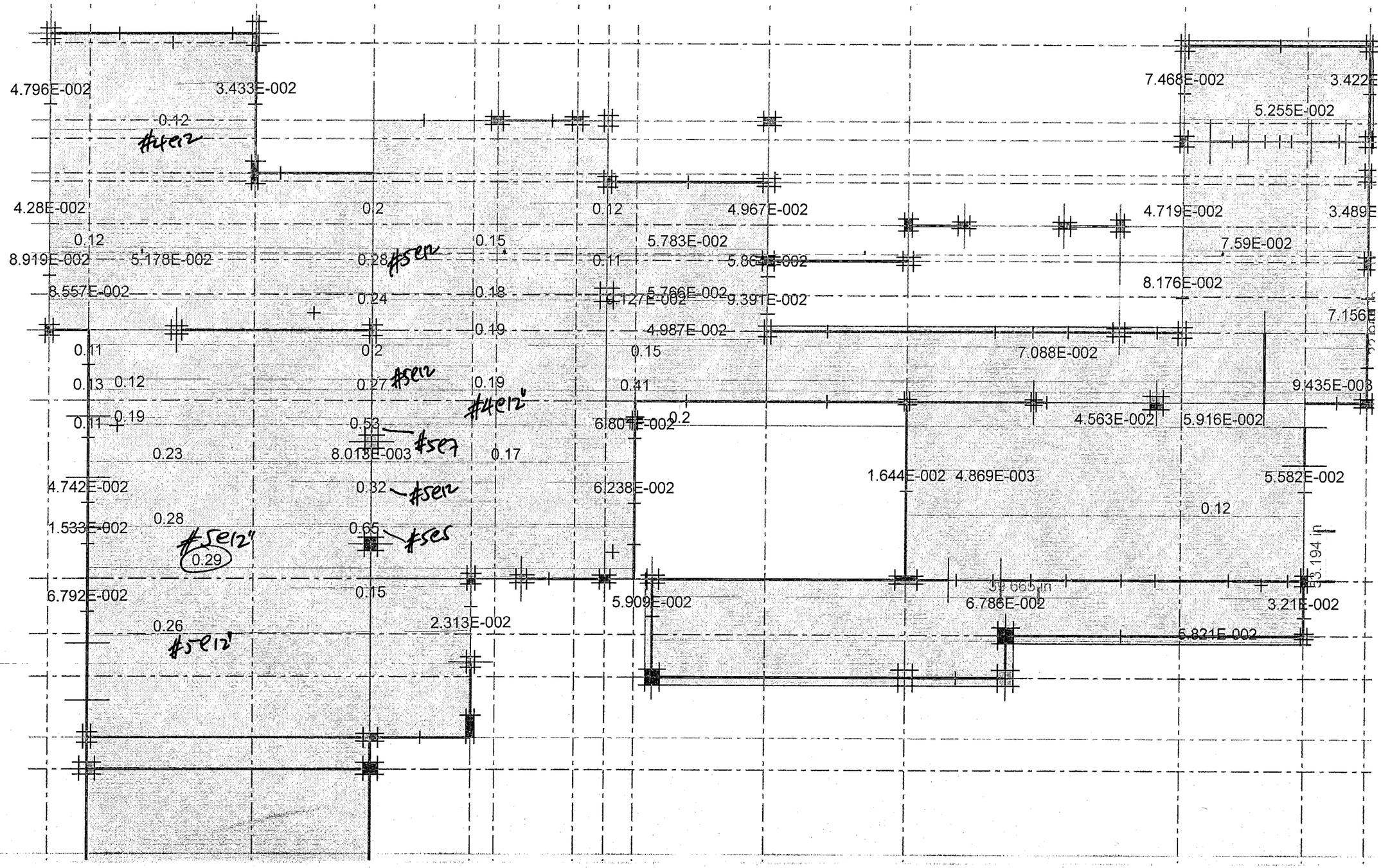

0.5-12
.2-X
#4@12T
0.18
0.18
0.18
0.18
#4@12" o/c B
0.18
0.18
0.18
#5@12"T
0.27
#4@6"
0.21
#4@12" B
0.18
0.18
0.2
#5@12T
0.25
0.18
0.18
0.5
T#5@6"T
0.22
4.736E-002
0.35
#5@10"T
1.44E-002
0.27
0.28
0.57
add #4@24" BOTT
0.18
#5@16
0.18
0.22
T#5@8"
0.18
0.18
0.18
#5@16T
0.18
#4@12" B
0.42
0.21
5#5@8"
0.18
0.18
#4@12T
0.22 → #5@16"
0.38 → #5@9"
0.18-12
0.31-X
#5@20"


#4@12" ⊃ — 0.20 k/F
0.08 — 12
0.20 — x → #4@30" add'l
(#4@24" ok)
4.736E-002
1.44E-002
0.18
0.25
0.27
0.28
0.35 #5@10" T
#4@6
0.57
9#5@6" T
13#4@4"
0.18
0.18
0.13
0.13
#4@12" T
ok
0.57 — 12
0.31 — x
#5@6"
0.57 — 12
0.20 — x
13#4@4"
0.35 — 12
0.20 — x
#4@6
1.928E-002
0.18
0.18
#4@12"
ok

0.22 #5@16"
#4@9"
0.18
0.18
0.42 7#5@8"
OR 10#4@7"
2.635E-002
0.26 #5@12"
#5@9"
#5@12"
3E-003
0.18
#5@12"
#5@12"
#5@12"
#5@12"
0.3
1.762E-002
0.18
0.18
0.18
0.26
0.18
0.18
0.18
#4@12"
0.18
0.18
0.18
6#5@9"
E-002
0.18
0.18
4#5@16"
18
0.18
0.18
4.194E-003
#5@6"
0.18
03
0.42 #5@8"
0.18 #
0.18
0.42-12 | #4@5"
2a-12 | #4@9"


1.762E-002
0.26
0.18
0.18
0.18


A
B
C
D
E
F
G
H
I
J
K
11
10
9
8
7
6
5
4
3
2
1
8x40
8x40
8x28 (la viga va de J e K
B7-8x18-4 ksi
B8-8x28-4 ksi
B4-8x16-4 ksi
B13-8x38-4 ksi
D=0.80 KLF
L=0.150
B2-8x12-4ksi
B9-8x26-4 ksi
B10-8x40-4 ksi
B6-8x24-4 ksi
B3-8x20-4 ksi
D=0.80 KLF
L=0.15
Pd=2300#
PL=2300#
D=0.80
L=0.150
B11-8x44-4 ksi
B12-12x40-4 ksi
12x40
12x40"
D=1100PLF
L=1100 plf
Pd=1700#
PL=1700#
D=1300PLF
L=1300PLF
D=1200PLF
L=150 PLF
3/20


4.796E-002
3.433E-002
0.12
#4@12
4.28E-002
0.2
0.12
4.967E-002
0.12
8.919E-002
5.178E-002
0.28
#5@12
0.15
5.783E-002
0.11
8.557E-002
0.24
0.18
5.766E-002
9.391E-002
0.19
4.987E-002
0.11
0.2
0.15
0.13
0.12
0.27
#5@12
0.19
0.41
0.11
0.19
0.53
#5@7
#4@12"
6.80[illegible]E-002
0.2
0.23
8.013E-003
0.17
4.742E-002
0.32
#5@12
6.238E-002
1.533E-002
0.28
#5@12"
0.65
#5@5
0.29
6.792E-002
0.15
5.909E-002
0.26
2.313E-002
#5@12"
7.468E-002
3.422E
5.255E-002
4.719E-002
3.489E
7.59E-002
8.176E-002
7.156E
7.088E-002
9.435E-003
4.563E-002
5.916E-002
1.644E-002
4.869E-003
5.582E-002
0.12
53.194 in
39.665 in
6.786E-002
3.21E-002
5.821E-002

PT- Slab Floor # 2

01-J

A B C D E F G H I J K

11 10 9 8 7 6 5 4 3 2

#5@2"

#4@2"

#5@2"

#4@2"

#5@2"

#4@2"

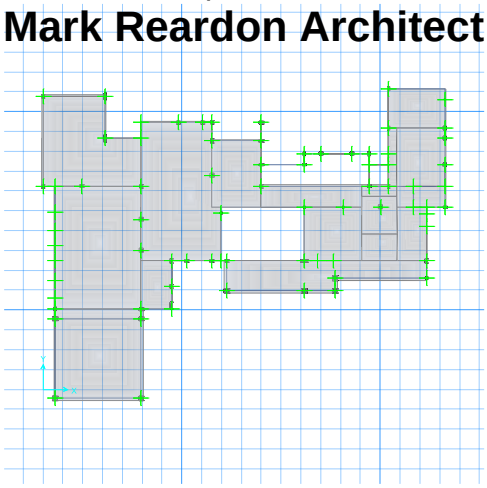
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LAMAS RESIDENCE

Project Number: May-2023

Prepared for

Mark Reardon Architect



SAFE Analysis & Design Report

Prepared by

GF CONSULTING ENGINEERS, Inc.

Model Name: Slab Floor -Level 2

Revision Number: GFC-01

23 May 2023

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Model Definition

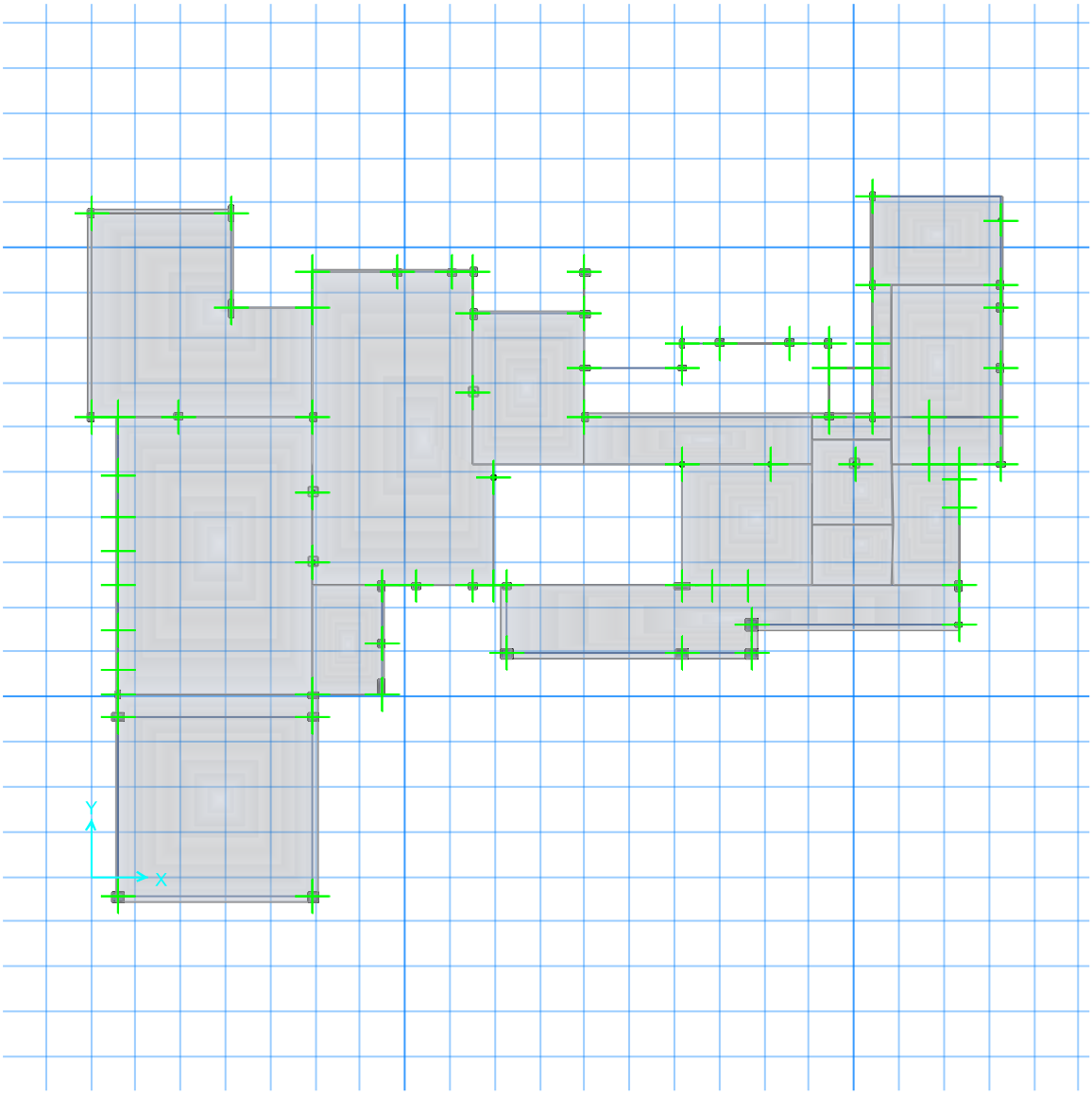


Figure 1: Finite element model

1. Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity.

1.1. Connectivity

Table 1: Concrete Slab Design Summary 02 - Span Definition Data

Table 1: Concrete Slab Design Summary 02 - Span Definition Data

Strip	SpanID	SpanLength	StartDist	GlobalX1	GlobalY1	GlobalX2	GlobalY2
		in	in	in	in	in	in
CSA1	Span 1	299.040	0.000	41.880	593.238	340.920	593.238
CSA1	Span 2	277.080	299.040	340.920	593.238	618.000	593.238
CSA2	Span 1	299.040	0.000	41.880	481.844	340.920	481.844
CSA2	Span 2	277.080	299.040	340.920	481.844	618.000	481.844
MSA1	Cantilever End	758.040	0.000	0.000	818.278	758.040	818.278
MSA2	Span 1	215.763	0.000	0.000	942.629	215.760	943.842
MSA3	Span 1	576.120	0.000	41.880	632.720	618.000	632.720
MSA3	Span 2	288.960	576.120	618.000	632.720	906.960	632.720
MSA3	Span 3	135.000	865.080	906.960	632.720	1041.960	632.720
MSA3	Span 4	13.219	1000.080	1041.960	632.720	1055.179	632.720
MSA3	Span 5	117.221	1013.299	1055.179	632.720	1172.400	632.720
MSA3	Span 6	114.840	1130.520	1172.400	632.720	1287.240	632.720
MSA3	Span 7	43.920	1245.360	1287.240	632.720	1331.160	632.720
MSA4	Cantilever Start	576.120	0.000	41.880	525.387	618.000	525.387
MSA5	Cantilever Start	404.192	0.000	41.880	411.204	446.040	416.300
MSA6	Cantilever Start	299.040	0.000	41.880	107.120	340.920	107.120
MSA7	Span 1	586.920	0.000	0.000	763.779	586.920	763.779
MSA7	Span 2	171.120	586.920	586.920	763.779	758.040	763.779
MSA8	Cantilever Start	1245.360	0.000	41.880	669.337	1287.240	669.337
MSA8	Span 1	108.360	1245.360	1287.240	669.337	1395.600	669.337
MSA9	Cantilever Start	61.920	0.000	279.000	724.000	340.920	724.000
MSA9	Span 1	246.000	61.920	340.920	724.000	586.920	724.000
MSA9	Span 2	171.120	307.920	586.920	724.000	758.040	724.000
MSB1	Span 1	276.240	0.000	128.820	-31.000	128.820	245.240
MSB1	Span 2	33.120	276.240	128.820	245.240	128.820	278.360
MSB1	Span 3	203.483	309.360	128.820	278.360	128.820	481.844
MSB1	Span 4	111.394	512.843	128.820	481.844	128.820	593.238
MSB1	Span 5	112.716	624.237	128.820	593.238	128.820	705.954
MSB1	Cantilever End	301.167	736.953	128.820	705.954	128.820	1007.120
MSB2	Span 1	276.240	0.000	191.400	-31.000	191.400	245.240
MSB2	Span 2	33.120	276.240	191.400	245.240	191.400	278.360
MSB2	Span 3	203.483	309.360	191.400	278.360	191.400	481.844
MSB2	Span 4	111.394	512.843	191.400	481.844	191.400	593.238
MSB2	Span 5	112.716	624.237	191.400	593.238	191.400	705.954
MSB2	Cantilever End	311.367	736.953	191.400	705.954	191.400	1017.320
MSB3	Span 1	276.240	0.000	242.579	-31.000	242.579	245.240
MSB3	Span 2	33.120	276.240	242.579	245.240	242.579	278.360
MSB3	Span 3	203.483	309.360	242.579	278.360	242.579	481.844
MSB3	Span 4	111.394	512.843	242.579	481.844	242.579	593.238
MSB3	Span 5	112.716	624.237	242.579	593.238	242.579	705.954
MSB3	Span 6	165.327	736.953	242.579	705.954	242.579	871.280
MSB4	Span 1	411.320	0.000	72.000	606.000	72.000	1017.320
MSB5	Span 1	203.483	0.000	393.480	278.360	393.480	481.844

Table 1: Concrete Slab Design Summary 02 - Span Definition Data

Strip	SpanID	SpanLength	StartDist	GlobalX1	GlobalY1	GlobalX2	GlobalY2
		in	in	in	in	in	in
MSB5	Span 2	111.394	203.483	393.480	481.844	393.480	593.238
MSB5	Cantilever End	333.243	314.877	393.480	593.238	393.480	926.480
MSB6	Span 1	203.483	0.000	340.920	278.360	340.920	481.844
MSB6	Span 2	0.310	203.483	340.920	481.844	340.920	482.154
MSB6	Span 3	107.520	203.793	340.920	482.154	340.920	589.674
MSB6	Span 4	3.564	311.313	340.920	589.674	340.920	593.238
MSB6	Span 5	112.716	314.877	340.920	593.238	340.920	705.954
MSB7	Cantilever Start	36.083	0.000	470.880	445.760	470.880	481.844
MSB7	Span 1	111.394	36.083	470.880	481.844	470.880	593.238
MSB7	Span 2	333.243	147.477	470.880	593.238	470.880	926.480
MSB8	Span 1	36.083	0.000	528.093	445.760	528.093	481.844
MSB8	Span 2	111.394	36.083	528.093	481.844	528.093	593.238
MSB8	Span 3	333.243	147.477	528.093	593.238	528.093	926.480
MSB9	Span 1	103.440	0.000	758.040	342.320	758.040	445.760
MSA10	Span 1	195.960	0.000	1199.640	818.278	1395.600	818.278
MSA11	Span 1	195.960	0.000	1199.640	742.856	1395.600	742.856
MSA12	Cantilever End	424.200	0.000	906.960	539.240	1331.160	539.240
MSA13	Span 1	376.080	0.000	638.022	406.294	1014.102	406.294
MSA13	Span 2	317.058	376.080	1014.102	406.294	1331.160	406.294
MSA14	Cantilever Start	199.920	0.000	1195.680	975.020	1395.600	975.020
MSA14	Cantilever End	3.960	199.920	1395.600	975.020	1399.560	975.020
MSB10	Cantilever Start	7.844	0.000	595.000	474.000	595.000	481.844
MSB10	Span 1	111.394	7.844	595.000	481.844	595.000	593.238
MSB10	Span 2	19.203	119.238	595.000	593.238	595.000	612.440
MSB10	Span 3	20.280	138.440	595.000	612.440	595.000	632.720
MSB10	Span 4	110.520	158.720	595.000	632.720	595.000	743.240
MSB10	Span 5	119.318	269.240	595.000	743.240	595.000	862.558
MSB11	Cantilever End	229.838	0.000	672.480	632.720	672.480	862.558
MSB12	Span 1	73.233	0.000	822.232	632.720	822.232	705.954
MSB13	Span 1	186.960	0.000	1000.671	445.760	1000.671	632.720
MSB13	Span 2	73.233	186.960	1000.671	632.720	1000.671	705.954
MSB14	Span 1	186.960	0.000	1040.997	445.760	1040.997	632.720
MSB14	Span 2	73.233	186.960	1040.997	632.720	1040.997	705.954
MSB15	Span 1	186.960	0.000	1078.333	445.760	1078.333	632.720
MSB15	Span 2	73.233	186.960	1078.333	632.720	1078.333	705.954
MSB16	Span 1	186.960	0.000	1172.333	445.760	1172.333	632.720
MSB16	Span 2	73.233	186.960	1172.333	632.720	1172.333	705.954
MSB17	Span 1	461.160	0.000	1250.778	445.760	1250.778	906.920
MSB18	Cantilever Start	3.937	0.000	1285.937	441.823	1285.937	445.760
MSB18	Span 1	186.960	3.937	1285.937	445.760	1285.937	632.720
MSB18	Span 2	73.234	190.897	1285.937	632.720	1285.937	705.954
MSB18	Span 3	200.966	264.131	1285.937	705.954	1285.937	906.920
MSB19	Span 1	120.645	0.000	1313.898	445.760	1313.898	566.406
MSB19	Span 2	41.461	120.645	1313.898	566.406	1313.898	607.867
MSB19	Span 3	24.854	162.106	1313.898	607.867	1313.898	632.720
MSB19	Span 4	274.200	186.960	1313.898	632.720	1313.898	906.920
MSB20	Span 1	274.200	0.000	1363.380	632.720	1363.380	906.920
MSB21	Span 1	103.440	0.000	960.531	342.320	960.531	445.760
MSB22	Span 1	58.920	0.000	1136.128	386.840	1136.128	445.760
MSB23	Span 1	136.222	0.000	1301.355	906.920	1303.811	1043.120

2. Model properties

This section provides model properties, including items such as material properties, section properties, and support properties.

2.1. Material properties

Table 2: Material Properties 03 - Concrete

Table 2: Material Properties 03 - Concrete

Material	E	U	A	UnitWt	Fc	LtWtConc	UserModRup
	lb/in2		1/F	lb/in3	lb/in2		
CMU=2.5 ksi	2489084.91	0.200000	5.5000E-06	6.5029E-02	2489.08	No	No
CSAC30	3867766.32	0.200000	5.5000E-06	8.6806E-02	4351.13	No	No
MAT1-fc=5 ksi	4030508.35	0.200000	5.5000E-06	8.6806E-02	5000.00	No	No
MAT3-fc=4ksi	4030508.35	0.200000	5.5000E-06	8.6806E-02	4000.00	No	No
MAT4	4030508.35	0.200000	5.5000E-06	0.0000E+00	0.00	No	No

Table 3: Material Properties 04 - Rebar

Table 3: Material Properties 04 - Rebar

Material	E	UnitWt	Fy	Fu
	lb/in2	lb/in3	lb/in2	lb/in2
CSA-G30.18Gr400-fy=60ksi	29007545.04	2.8356E-01	58015.09	72518.86
MAT2-fy=60 ksi	29007545.04	2.8356E-01	60000.00	75000.00

Table 4: Material Properties 05 - Tendon

Table 4: Material Properties 05 - Tendon

Material	E	UnitWt	Fy	Fu
	lb/in2	lb/in3	lb/in2	lb/in2
A416MGr186	28500000.00	2.8356E-01	245113.76	269770.17

2.2. Section properties

Table 5: Slab Properties 02 - Solid Slabs

Table 5: Slab Properties 02 - Solid Slabs

Slab	Type	MatProp	Thickness	Ortho
			in	
CAP-14x14-20in	Drop	CSA-G30.18Gr400-fy=60ksi	20.0000	No
Cap-14x14-4A50	Stiff	CSA-G30.18Gr400-fy=60ksi	1.0000	No
Col_Stiff	Stiff	MAT3-fc=4ksi	13.7795	No
Slab-H=6in-4 ksi	Slab	MAT3-fc=4ksi	6.0000	No
Slab-H=8 in-4 ksi	Slab	MAT3-fc=4ksi	8.0000	No

Table 5: Slab Properties 02 - Solid Slabs

Slab	Type	MatProp	Thickness	Ortho
			in	
SLAB1	Slab	CSAC30	7.8740	No
Stiff1	Stiff	MAT1-fc=5 ksi	13.7795	No

Table 6: Beam Properties 02 - Rectangular Beam

Table 6: Beam Properties 02 - Rectangular Beam

Beam	MatProp	Depth	WidthTop	WidthBot
		in	in	in
B1-12x22-4 ksi	MAT3-fc=4ksi	22.0000	12.0000	12.0000
B10-8x40-4 ksi	MAT3-fc=4ksi	40.0000	7.8740	7.8740
B11-8x44-4 ksi	MAT3-fc=4ksi	44.0000	7.8740	7.8740
B12-12x40- 4 ksi	MAT3-fc=4ksi	40.0000	7.8740	7.8740
B13-8x38-4 ksi	MAT3-fc=4ksi	38.0000	7.8740	7.8740
B14-8x48-fc 4ksi	MAT3-fc=4ksi	48.0000	8.0040	8.0040
B2-8x12- 4ksi	MAT3-fc=4ksi	11.8110	7.8740	7.8740
B3-8x20- 4 ksi	MAT3-fc=4ksi	19.6850	7.8740	7.8740
B4-8x16- 4 ksi	MAT3-fc=4ksi	15.7480	7.8740	7.8740
B5-10x20-4 ksi	MAT1-fc=5 ksi	19.6850	9.8425	9.8425
B6-8x24-4ksi	MAT3-fc=4ksi	24.0000	8.0040	8.0040
B7-8x18-4 ksi	MAT3-fc=4ksi	18.0000	7.8740	7.8740
B8-8x28-4 ksi	MAT3-fc=4ksi	27.9600	8.0040	8.0040
B9-8x26-4 ksi	MAT3-fc=4ksi	26.0000	8.0040	8.0040
BEAM1	CSAC30	23.6220	11.8110	11.8110
bF-8x8-4 ksi	MAT3-fc=4ksi	8.0000	8.0000	8.0000

Table 7: Beam Properties 06 - Design Data

Table 7: Beam Properties 06 - Design Data

Beam	MatRebarL	MatRebarS	FlngWOpt	CoverTop	CoverBot	NoDesign
				in	in	
B1-12x22-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	2.2000	2.2000	No
B10-8x40-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B11-8x44-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B12-12x40- 4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B13-8x38-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B14-8x48-fc 4ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B2-8x12- 4ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B3-8x20- 4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B4-8x16- 4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B5-10x20-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B6-8x24-4ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No

Table 7: Beam Properties 06 - Design Data

Beam	MatRebarL	MatRebarS	FlngWOpt	CoverTop in	CoverBot in	NoDesign
B7-8x18-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B8-8x28-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
B9-8x26-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No
BEAM1	CSA-G30.18Gr400-fy=60ksi	CSA-G30.18Gr400-fy=60ksi	Analysis Property	2.9528	2.9528	No
bF-8x8-4 ksi	MAT2-fy=60 ksi	MAT2-fy=60 ksi	Analysis Property	1.5748	1.5748	No

Table 8: Tendon Properties

Table 8: Tendon Properties

TendonProp	MatProp	StrandArea in2
TENDON1	A416MGr186	0.15

Table 9: Column Properties 02 - Rectangular

Table 9: Column Properties 02 - Rectangular

Column	MatProp	SecDim2 in	SecDim3 in	AutoRigid	AutoDrop	IncludeCap
C1-8x12-4 ksi	MAT3-fc=4ksi	11.8110	7.8740	Yes	No	No
C2-16x16-4 ksi	MAT3-fc=4ksi	15.7480	15.7480	Yes	No	No
C3-8x16- 4 ksi	MAT3-fc=4ksi	15.9960	8.0040	Yes	No	No
C4-8x24-4ksi	MAT3-fc=4ksi	8.0040	24.0000	Yes	No	No
C5-12x16-4 ksi	MAT3-fc=4ksi	15.9960	12.0000	Yes	No	No
COL1	CSAC30	23.6220	11.8110	No	No	No
HS-EQ--14x14in	MAT3-fc=4ksi	14.0000	14.0000	Yes	No	No
HS-Eq--18x18 in	MAT3-fc=4ksi	18.0000	18.0000	Yes	No	No
HS-Eq--20x20 in	MAT3-fc=4ksi	20.0000	20.0000	Yes	No	No
HSS-6x6x1/4	MAT2-fy=60 ksi	6.0000	6.0000	Yes	No	No
HSS4x4-1/4	MAT2-fy=60 ksi	4.0000	4.0000	Yes	No	No

Table 10: Wall Properties

Table 10: Wall Properties

Wall	MatProp	Thickness in	AutoRigid	OutOfPlane
cmu=8in-	CMU=2.5 ksi	8.0000	No	No
WALL1	CSAC30	11.8110	No	Yes
WW2-8in	MAT1-fc=5 ksi	8.0000	Yes	Yes

2.3. Support properties

Table 11: Soil Properties

Table 11: Soil Properties

Soil	Subgrade	NonlinOpt
	lb/in3	
SOIL1	7.3679E+01	Compression Only

Table 12: Spring Properties - Point

Table 12: Spring Properties - Point

Spring	Ux	Uy	Uz	Rx	Ry	Rz	NonlinOpt
	lb/in	lb/in	lb/in	lb-in/rad	lb-in/rad	lb-in/rad	
PSPR1	0.00	0.00	5.600E-002	0.00	0.00	0.00	None (Linear)

Table 13: Spring Properties - Line

Table 13: Spring Properties - Line

Spring	VertStiff	RotStiff	NonlinOpt
	lb/in/in	lb/rad	
LSPR1	1.422E-003	2.20	None (Linear)

3. Model assignments

This section provides model assignments, including assignments to slabs, beams, and joints.

3.1. Slab assignments

Table 14: Slab Property Assignments

Table 14: Slab Property Assignments

Area	SlabProp
2	Slab-H=6in-4 ksi
143	Cap-14x14-4 A50
213	Cap-14x14-4 A50
214	CAP-14x14-20in
215	CAP-14x14-20in
262	Slab-H=8 in-4 ksi
266	Slab-H=8 in-4 ksi
267	Slab-H=8 in-4 ksi
270	Slab-H=8 in-4 ksi
273	Slab-H=8 in-4 ksi
276	Slab-H=8 in-4 ksi
440	Slab-H=6in-4 ksi
549	Slab-H=8 in-4 ksi
550	Slab-H=8 in-4 ksi

Table 14: Slab Property Assignments

Area	SlabProp
1341	Slab-H=8 in-4 ksi
1344	Slab-H=8 in-4 ksi
1346	Slab-H=8 in-4 ksi
1347	Slab-H=8 in-4 ksi
1348	Slab-H=8 in-4 ksi
1350	Slab-H=8 in-4 ksi

Table 15: Slab Local Axes

Table 15: Slab Local Axes

Area	Angle Degrees
599	90.000
602	90.000
603	90.000
606	90.000
607	90.000
610	90.000
616	90.000
617	90.000
618	90.000
619	90.000
620	90.000
621	90.000
622	90.000
623	90.000

Table 15: Slab Local Axes

Area	Angle Degrees
624	90.000
625	90.000
626	90.000
635	90.000
648	90.000
649	90.000
658	90.000

Table 16: Slab Edge Releases

Table 16: Slab Edge Releases

Area	Edge	Shear	Moment
2	1	No	Yes
2	2	No	Yes
2	3	No	Yes
2	4	No	Yes

Table 17: Slab Line Releases

Table 17: Slab Line Releases

Line	ShearL	MomentL	ShearR	MomentR
5	No	Yes	No	Yes
51	No	Yes	No	No
57	No	Yes	No	No
65	No	Yes	No	Yes
67	No	Yes	No	No
68	No	Yes	No	No
70	No	Yes	No	No
89	No	Yes	No	No
90	No	Yes	No	No
96	No	Yes	No	No
97	No	Yes	No	No
99	No	Yes	No	Yes
100	No	Yes	No	No

3.2. Beam assignments

Table 18: Beam Property Assignments

Table 18: Beam Property Assignments

Line	BeamProp
1	B7-8x18-4 ksi
3	B8-8x28-4 ksi

Table 18: Beam Property Assignments

Line	BeamProp
4	B8-8x28-4 ksi
23	B4-8x16- 4 ksi
25	B8-8x28-4 ksi
26	B8-8x28-4 ksi
27	B10-8x40-4 ksi
42	B9-8x26-4 ksi
43	B9-8x26-4 ksi
45	B10-8x40-4 ksi
46	B2-8x12- 4ksi
47	B2-8x12- 4ksi
58	B8-8x28-4 ksi
61	B9-8x26-4 ksi
62	B9-8x26-4 ksi
63	B9-8x26-4 ksi
64	B12-12x40- 4 ksi
65	B8-8x28-4 ksi
69	B3-8x20- 4 ksi
71	B3-8x20- 4 ksi
72	B8-8x28-4 ksi
74	B9-8x26-4 ksi
75	B9-8x26-4 ksi
76	B9-8x26-4 ksi
84	B3-8x20- 4 ksi
85	B12-12x40- 4 ksi
86	B12-12x40- 4 ksi
87	B3-8x20- 4 ksi
88	B10-8x40-4 ksi
91	B11-8x44-4 ksi
92	B10-8x40-4 ksi
93	B10-8x40-4 ksi
94	B12-12x40- 4 ksi
95	B3-8x20- 4 ksi
98	B13-8x38-4 ksi
99	B14-8x48-fc 4ksi
100	B8-8x28-4 ksi
101	B8-8x28-4 ksi
102	B8-8x28-4 ksi
105	B10-8x40-4 ksi
111	B6-8x24-4ksi
113	B6-8x24-4ksi
117	B10-8x40-4 ksi
119	B7-8x18-4 ksi
120	B7-8x18-4 ksi

Table 19: Beam Property Modifiers

Table 19: Beam Property Modifiers

Line	Area	As2	As3	J	I22	I33	Weight
95	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
96	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
97	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000

Table 19: Beam Property Modifiers

Line	Area	As2	As3	J	I22	I33	Weight
5	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
51	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
57	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
67	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
68	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
70	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
89	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000
90	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000

Table 20: Beam End Releases

Table 20: Beam End Releases

Line	TI	M2I	M3I	TJ	M2J	M3J
1	No	Yes	Yes	Yes	Yes	Yes
3	Yes	Yes	Yes	No	Yes	Yes
4	Yes	Yes	Yes	No	Yes	Yes
5	Yes	No	No	No	No	No
25	No	Yes	Yes	Yes	Yes	Yes
26	No	Yes	Yes	Yes	Yes	Yes
42	Yes	Yes	Yes	No	No	No
43	No	No	No	Yes	Yes	Yes
51	Yes	No	No	No	No	No
57	Yes	No	No	No	No	No
58	Yes	Yes	Yes	No	No	Yes
61	Yes	Yes	Yes	No	No	No
63	Yes	Yes	Yes	No	No	No
67	Yes	No	No	No	No	No
68	Yes	No	No	No	No	No
70	Yes	No	No	No	No	No
72	Yes	Yes	Yes	No	Yes	Yes
74	Yes	Yes	Yes	No	Yes	Yes
85	No	Yes	Yes	Yes	Yes	Yes
86	Yes	Yes	Yes	No	No	No
87	No	No	No	Yes	Yes	Yes
88	Yes	Yes	Yes	No	Yes	Yes
89	Yes	No	No	No	No	No
90	Yes	No	No	No	No	No
91	Yes	Yes	Yes	No	No	No
92	No	No	No	Yes	Yes	Yes
93	Yes	Yes	Yes	No	No	No
96	Yes	Yes	Yes	No	Yes	Yes
97	Yes	Yes	Yes	No	Yes	Yes
98	No	No	No	Yes	Yes	Yes
99	No	No	No	No	Yes	Yes
101	Yes	Yes	Yes	No	No	No
102	No	No	No	Yes	Yes	Yes
105	No	Yes	Yes	Yes	Yes	Yes
117	No	No	No	Yes	Yes	Yes

3.3. Column assignments

Table 21: Column Property Assignments

Table 21: Column Property Assignments

Line	ColProp
6	C1-8x12-4 ksi
7	C1-8x12-4 ksi
8	C2-16x16-4 ksi
9	C4-8x24- 4ksi
10	C4-8x24- 4ksi
11	C1-8x12-4 ksi
12	C1-8x12-4 ksi
13	C1-8x12-4 ksi
14	C1-8x12-4 ksi
15	C2-16x16-4 ksi
16	C1-8x12-4 ksi
17	HSS-6x6x1/4
18	C2-16x16-4 ksi
19	C2-16x16-4 ksi
20	C1-8x12-4 ksi
21	C1-8x12-4 ksi
22	C1-8x12-4 ksi
29	C1-8x12-4 ksi
30	C1-8x12-4 ksi
31	C1-8x12-4 ksi
32	C1-8x12-4 ksi
33	C1-8x12-4 ksi
34	C1-8x12-4 ksi
35	C1-8x12-4 ksi
36	C1-8x12-4 ksi
37	C1-8x12-4 ksi
38	C1-8x12-4 ksi
39	C1-8x12-4 ksi
40	C1-8x12-4 ksi
41	C1-8x12-4 ksi
48	C1-8x12-4 ksi
49	HSS4x4-1/4
52	C4-8x24- 4ksi
53	C2-16x16-4 ksi
54	C5-12x16-4 ksi
55	C5-12x16-4 ksi
56	C2-16x16-4 ksi
59	C1-8x12-4 ksi
60	C3-8x16- 4 ksi
77	HSS4x4-1/4
78	HSS4x4-1/4
79	HSS4x4-1/4
80	HSS4x4-1/4
81	HSS4x4-1/4
82	HSS4x4-1/4
83	HSS4x4-1/4
103	C4-8x24- 4ksi
104	C1-8x12-4 ksi

Table 21: Column Property Assignments

Line	ColProp
106	C1-8x12-4 ksi
107	C1-8x12-4 ksi
108	C1-8x12-4 ksi
109	C1-8x12-4 ksi
110	C1-8x12-4 ksi
115	C1-8x12-4 ksi

3.4. Wall assignments

Table 22: Wall Property Assignments

Table 22: Wall Property Assignments

Area	WallProp
1	cmu=8in-
3	cmu=8in-
4	cmu=8in-
16	cmu=8in-
18	cmu=8in-
48	cmu=8in-
50	cmu=8in-
51	cmu=8in-
53	cmu=8in-
54	cmu=8in-
56	cmu=8in-
57	cmu=8in-
59	cmu=8in-
74	cmu=8in-
76	cmu=8in-
105	cmu=8in-
107	cmu=8in-
108	cmu=8in-
110	cmu=8in-
114	cmu=8in-
116	cmu=8in-
117	cmu=8in-
119	cmu=8in-
120	cmu=8in-
122	cmu=8in-
123	cmu=8in-
125	cmu=8in-
126	cmu=8in-
128	cmu=8in-
129	cmu=8in-
131	cmu=8in-
132	cmu=8in-
134	cmu=8in-
162	cmu=8in-
164	cmu=8in-

Table 22: Wall Property Assignments

Area	WallProp
168	cmu=8in-
170	cmu=8in-
174	cmu=8in-
176	cmu=8in-
177	cmu=8in-
179	cmu=8in-
189	cmu=8in-
191	cmu=8in-
628	cmu=8in-
630	cmu=8in-
631	cmu=8in-
633	cmu=8in-
634	cmu=8in-
636	cmu=8in-
637	cmu=8in-
639	cmu=8in-
953	cmu=8in-
955	cmu=8in-
956	cmu=8in-
958	cmu=8in-
959	cmu=8in-
961	cmu=8in-
965	cmu=8in-
967	cmu=8in-
1053	cmu=8in-
1055	cmu=8in-
1057	cmu=8in-
2667	cmu=8in-
2669	cmu=8in-
2671	cmu=8in-
2675	cmu=8in-

Table 23: Wall Property Modifiers

Table 23: Wall Property Modifiers

Area	f11	f22	f12	m11	m22	m12	v13	v23	Weight
953	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
955	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
956	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
958	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
959	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
961	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
965	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
967	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
1055	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
2667	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000
2669	1.000000	1.000000	1.000000	1.000000	0.000000	1.000000	1.000000	1.000000	1.000000

3.5. Support assignments

Table 24: Point Restraint Assignments

Table 24: Point Restraint Assignments

Point	Ux	Uy	Uz	Rx	Ry	Rz
21	Yes	Yes	Yes	Yes	Yes	Yes
22	Yes	Yes	Yes	Yes	Yes	Yes
65	Yes	Yes	Yes	Yes	Yes	Yes
76	Yes	Yes	Yes	Yes	Yes	Yes
77	Yes	Yes	Yes	Yes	Yes	Yes
127	Yes	Yes	Yes	Yes	Yes	Yes
193	Yes	Yes	Yes	Yes	Yes	Yes
207	Yes	Yes	Yes	Yes	Yes	Yes
214	Yes	Yes	Yes	Yes	Yes	Yes
219	Yes	Yes	Yes	Yes	Yes	Yes
226	Yes	Yes	Yes	Yes	Yes	Yes
227	Yes	Yes	Yes	Yes	Yes	Yes
296	Yes	Yes	Yes	Yes	Yes	Yes
406	Yes	Yes	Yes	Yes	Yes	Yes
412	Yes	Yes	Yes	Yes	Yes	Yes
426	Yes	Yes	Yes	Yes	Yes	Yes
430	Yes	Yes	Yes	Yes	Yes	Yes
440	Yes	Yes	Yes	Yes	Yes	Yes
450	Yes	Yes	Yes	Yes	Yes	Yes
457	Yes	Yes	Yes	Yes	Yes	Yes
458	Yes	Yes	Yes	Yes	Yes	Yes
466	Yes	Yes	Yes	Yes	Yes	Yes
467	Yes	Yes	Yes	Yes	Yes	Yes
476	Yes	Yes	Yes	Yes	Yes	Yes
481	Yes	Yes	Yes	Yes	Yes	Yes
486	Yes	Yes	Yes	Yes	Yes	Yes
487	Yes	Yes	Yes	Yes	Yes	Yes
495	Yes	Yes	Yes	Yes	Yes	Yes
528	Yes	Yes	Yes	Yes	Yes	Yes
534	Yes	Yes	Yes	Yes	Yes	Yes
540	Yes	Yes	Yes	Yes	Yes	Yes
579	Yes	Yes	Yes	Yes	Yes	Yes
585	Yes	Yes	Yes	Yes	Yes	Yes
615	Yes	Yes	Yes	Yes	Yes	Yes
621	Yes	Yes	Yes	Yes	Yes	Yes
628	Yes	Yes	Yes	Yes	Yes	Yes
629	Yes	Yes	Yes	Yes	Yes	Yes
666	Yes	Yes	Yes	Yes	Yes	Yes
672	Yes	Yes	Yes	Yes	Yes	Yes
673	Yes	Yes	Yes	Yes	Yes	Yes
690	Yes	Yes	Yes	Yes	Yes	Yes
696	Yes	Yes	Yes	Yes	Yes	Yes
702	Yes	Yes	Yes	Yes	Yes	Yes
708	Yes	Yes	Yes	Yes	Yes	Yes
726	Yes	Yes	Yes	Yes	Yes	Yes
1286	Yes	Yes	Yes	Yes	Yes	Yes
1591	Yes	Yes	Yes	Yes	Yes	Yes
1596	Yes	Yes	Yes	Yes	Yes	Yes
1602	Yes	Yes	Yes	Yes	Yes	Yes

Table 24: Point Restraint Assignments

Point	Ux	Uy	Uz	Rx	Ry	Rz
1607	Yes	Yes	Yes	Yes	Yes	Yes
1612	Yes	Yes	Yes	Yes	Yes	Yes
2376	Yes	Yes	Yes	Yes	Yes	Yes
2377	Yes	Yes	Yes	Yes	Yes	Yes
2386	Yes	Yes	Yes	Yes	Yes	Yes
2395	Yes	Yes	Yes	Yes	Yes	Yes
2396	Yes	Yes	Yes	Yes	Yes	Yes
2404	Yes	Yes	Yes	Yes	Yes	Yes
3282	Yes	Yes	Yes	Yes	Yes	Yes
3287	Yes	Yes	Yes	Yes	Yes	Yes
3292	Yes	Yes	Yes	Yes	Yes	Yes
3297	Yes	Yes	Yes	Yes	Yes	Yes
3304	Yes	Yes	Yes	Yes	Yes	Yes
3309	Yes	Yes	Yes	Yes	Yes	Yes
3628	Yes	Yes	Yes	Yes	Yes	Yes
3633	Yes	Yes	Yes	Yes	Yes	Yes
3640	Yes	Yes	Yes	Yes	Yes	Yes
3645	Yes	Yes	Yes	Yes	Yes	Yes
3646	Yes	Yes	Yes	Yes	Yes	Yes
3665	Yes	Yes	Yes	Yes	Yes	Yes
3666	Yes	Yes	Yes	Yes	Yes	Yes
4020	Yes	Yes	Yes	Yes	Yes	Yes
9884	Yes	Yes	Yes	Yes	Yes	Yes
9887	Yes	Yes	Yes	Yes	Yes	Yes
9893	Yes	Yes	Yes	Yes	Yes	Yes
12023	Yes	Yes	Yes	Yes	Yes	Yes

4. Model loading

This section provides model loading information, including load patterns, load cases, and load combinations.

4.1. Load patterns

Table 25: Load Patterns

Table 25: Load Patterns

LoadPat	Type	SelfWtMult
Live	LIVE	0.000000
Lr	LIVE	0.000000
SDL	SUPER DEAD	0.000000
Dead	DEAD	1.000000
PT-FINAL	PRESTRESS-FINAL	0.000000
PT-TRANSFER	PRESTRESS-TRANSFER	0.000000
PT-FINAL1	PRESTRESS-FINAL	0.000000
PT-TRANSFER1	PRESTRESS-TRANSFER	0.000000

Table 26: Load Assignments - Surface Loads

Table 26: Load Assignments - Surface Loads

Area	LoadPat	Dir	UnifLoad	A	B	C
			lb/in2	lb/in3	lb/in3	lb/in2
1341	Live	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
1341	SDL	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
1344	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1344	Live	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
1346	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1346	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1347	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1347	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1348	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1350	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
1350	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
2	Live	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
2	SDL	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
440	SDL	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
440	Live	Gravity	0.208	0.0000E+00	0.0000E+00	0.000
262	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
262	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
266	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
266	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
267	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
267	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
270	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
270	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
273	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
273	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
276	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
276	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
549	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000

Table 26: Load Assignments - Surface Loads

Area	LoadPat	Dir	UnifLoad	A	B	C
			lb/in2	lb/in3	lb/in3	lb/in2
549	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
550	Live	Gravity	0.278	0.0000E+00	0.0000E+00	0.000
550	SDL	Gravity	0.278	0.0000E+00	0.0000E+00	0.000

Table 27: Load Assignments - Line Objects - Distributed Loads, Part 1 of 2

Table 27: Load Assignments - Line Objects - Distributed Loads, Part 1 of 2

Line	LoadPat	Type	Dir	RelDistA	RelDistB	AbsDistA	AbsDistB
						in	in
64	SDL	Force	Gravity	0.0000	1.0000	0.000	268.938
64	Live	Force	Gravity	0.0000	1.0000	0.000	268.938
72	Live	Force	Gravity	0.0000	1.0000	0.000	195.960
72	SDL	Force	Gravity	0.0000	1.0000	0.000	195.960
91	SDL	Force	Gravity	0.0000	1.0000	0.000	299.040
91	Live	Force	Gravity	0.0000	1.0000	0.000	299.040
93	SDL	Force	Gravity	0.0000	1.0000	0.000	79.261
93	Live	Force	Gravity	0.0000	1.0000	0.000	79.261
98	SDL	Force	Gravity	0.0000	1.0000	0.000	207.000
98	Live	Force	Gravity	0.0000	1.0000	0.000	207.000
44	Live	Force	Gravity	0.0000	1.0000	0.000	108.360
44	SDL	Force	Gravity	0.0000	1.0000	0.000	108.360
50	Live	Force	Gravity	0.0000	1.0000	0.000	73.804
50	SDL	Force	Gravity	0.0000	1.0000	0.000	73.804
99	SDL	Force	Gravity	0.5917	1.0000	190.610	322.130
99	Live	Force	Gravity	0.5917	1.0000	190.610	322.130
99	Live	Force	Gravity	0.0000	0.5917	0.000	190.610
99	SDL	Force	Gravity	0.0000	0.5917	0.000	190.610

Table 27: Load Assignments - Line Objects - Distributed Loads, Part 2 of 2

Table 27: Load Assignments - Line Objects - Distributed Loads, Part 2 of 2

Line	LoadPat	FOverLA	FOverLB
		lb/in	lb/in
64	SDL	108.33	108.33
64	Live	108.33	108.33
72	Live	12.50	12.50
72	SDL	66.67	66.67
91	SDL	66.67	66.67
91	Live	12.50	12.50
93	SDL	91.67	91.67
93	Live	91.67	91.67
98	SDL	66.67	66.67
98	Live	12.50	12.50
44	Live	12.50	12.50
44	SDL	66.67	66.67
50	Live	12.50	12.50
50	SDL	66.67	66.67
99	SDL	66.67	66.67
99	Live	12.50	12.50
99	Live	12.50	12.50

Table 27: Load Assignments - Line Objects - Distributed Loads, Part 2 of 2

Line	LoadPat	FOverLA	FOverLB
		lb/in	lb/in
99	SDL	100.00	100.00

Table 28: Load Assignments - Point Loads, Part 1 of 2

Table 28: Load Assignments - Point Loads, Part 1 of 2

Point	LoadPat	Fx	Fy	Fgrav
		lb	lb	lb
766	Live	0.00	0.00	-2300.00
766	SDL	0.00	0.00	-2300.00
767	SDL	0.00	0.00	-1700.00
767	Live	0.00	0.00	-1700.00

Table 28: Load Assignments - Point Loads, Part 2 of 2

Table 28: Load Assignments - Point Loads, Part 2 of 2

Point	Mx	My	Mz	XDim	YDim
	lb-in	lb-in	lb-in	in	in
766	0.00	0.00	0.00	8.0400	8.0400
766	0.00	0.00	0.00	8.0400	8.0400
767	0.00	0.00	0.00	8.0400	8.0400
767	0.00	0.00	0.00	8.0400	8.0400

4.2. Load cases

Table 29: Load Cases 02 - Static

Table 29: Load Cases 02 - Static

LoadCase	InitialCond	AType
D	Zero	Linear
L	Zero	Linear
Lr	Zero	Linear
PT-FINAL	Zero	Linear
PT-TRANSFER	Zero	Linear
PT-FINAL1	Zero	Linear
PT-TRANSFER1	Zero	Linear

Table 30: Load Cases 05 - Hyperstatic

Table 30: Load Cases 05 - Hyperstatic

LoadCase	LinCase	SuppType	SpringSF
PT-FINAL-HP	PT-FINAL	Springs	1.000000
PT-FINAL1-HP	PT-FINAL1	Springs	1.000000

Table 31: Load Cases 06 - Loads Applied

Table 31: Load Cases 06 - Loads Applied

LoadCase	LoadPat	SF
D	Dead	1.000000
D	SDL	1.000000
L	Live	1.000000
Lr	Lr	1.000000
PT-FINAL	PT-FINAL	1.000000
PT-TRANSFER	PT-TRANSFER	1.000000
PT-FINAL1	PT-FINAL1	1.000000
PT-TRANSFER1	PT-TRANSFER1	1.000000

4.3. Load combinations

Table 32: Load Combinations

Table 32: Load Combinations

Combo	Load	SF	Type	DSStrength	DSServInit	DSServNorm	DSServLong
Comb=1.0 D + 1.0 L+1.0 Lr	D	1.000000	Linear Add	No	No	Yes	No
Comb=1.0 D + 1.0 L+1.0 Lr	L	1.000000					
Comb=1.0 D + 1.0 L+1.0 Lr	Lr	1.000000					
Comb=1.2 D + 1.6 L +1.6 Lr	D	1.200000	Linear Add	Yes	No	No	No
Comb=1.2 D + 1.6 L +1.6 Lr	L	1.600000					
Comb=1.2 D + 1.6 L +1.6 Lr	Lr	1.600000					
DCONI1	D	1.000000	Linear Add	No	Yes	No	No
DCONI1	PT-TRANSFER	1.000000					
DCONI1	PT-TRANSFER1	1.000000					
DCONL1	D	1.000000	Linear Add	No	No	No	Yes
DCONL1	PT-FINAL	1.000000					
DCONL1	PT-FINAL1	1.000000					
DCONL2	D	1.000000	Linear Add	No	No	No	Yes
DCONL2	L	0.500000					
DCONL2	Lr	0.500000					
DCONL2	PT-FINAL	1.000000					
DCONL2	PT-FINAL1	1.000000					

Table 32: Load Combinations

Combo	Load	SF	Type	DSStrength	DSServInit	DSServNorm	DSServLong
DCONN1	D	1.000000	Linear Add	No	No	Yes	No
DCONN1	PT-FINAL	1.000000					
DCONN1	PT-FINAL1	1.000000					
DCONN2	D	1.000000	Linear Add	No	No	Yes	No
DCONN2	L	1.000000					
DCONN2	Lr	1.000000					
DCONN2	PT-FINAL	1.000000					
DCONN2	PT-FINAL1	1.000000					
DCONU1	D	1.400000	Linear Add	Yes	No	No	No
DCONU1	PT-FINAL-HP	1.000000					
DCONU1	PT-FINAL1-HP	1.000000					
DCONU2	D	1.200000	Linear Add	Yes	No	No	No
DCONU2	L	1.600000					
DCONU2	Lr	1.600000					
DCONU2	PT-FINAL-HP	1.000000					
DCONU2	PT-FINAL1-HP	1.000000					

Analysis Results

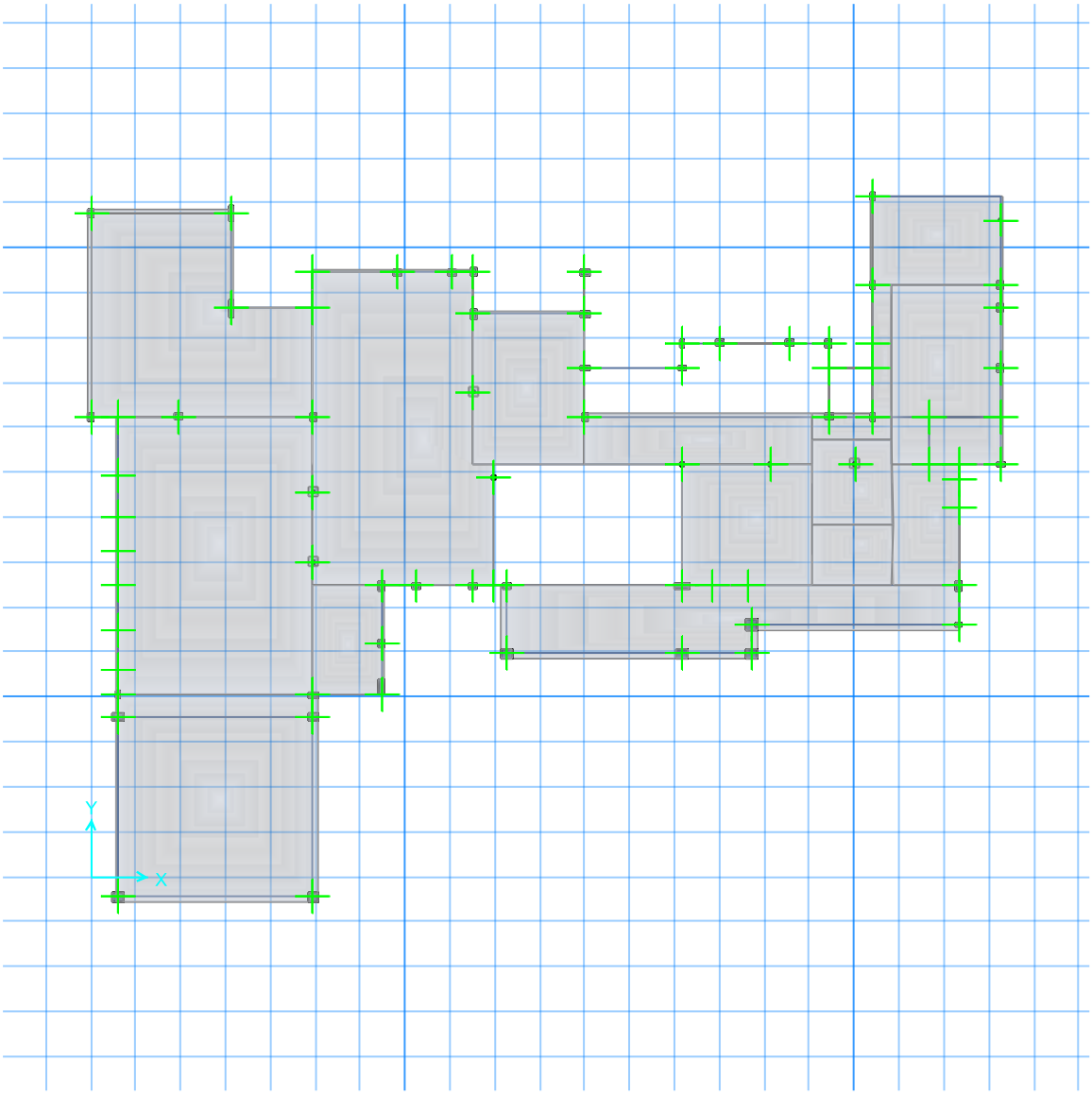


Figure 2: Deformed shape

5. Analysis results

5.1. Support results

This section provides support results, including items such as column, support, and spring reactions, .

Table 33: Nodal Reactions

Table 33: Nodal Reactions

Point	OutputCase	Fx	Fy	Fz	Mx	My	Mz
		lb	lb	lb	lb-in	lb-in	lb-in
21	D	-121.69	-382.85	2971.21	2148.79	-1236.74	-3.66
22	D	-1.09	1297.33	6313.84	-3929.48	-464.14	-69.33
65	D	-147.60	1657.39	26524.34	225.43	-6236.42	-9.71
76	D	3804.93	-296.81	11274.61	-2594.80	13783.11	-6.068E-002
77	D	-868.48	-254.37	20202.07	10034.59	15745.88	-4.07
127	D	-1313.29	-1039.05	8892.55	38623.92	-4135.47	-87.07
193	D	-353.68	-129.16	27626.64	3909.78	-14827.60	-76.47
207	D	6.437E-002	-1079.31	3443.63	3327.68	3.45	2.930E-002
214	D	-147.89	-775.54	2200.27	2242.19	-1203.47	-0.14
219	D	-1333.38	-460.59	11019.84	18115.65	-3452.96	12.66
226	D	-569.47	-78.17	3173.78	2902.23	-7427.52	14.39
227	D	-901.82	-1382.29	7676.12	3553.29	-5059.78	20.14
296	D	-153.68	-408.54	13644.99	5044.31	-6456.15	0.59
406	D	-298.02	212.96	6650.61	-1588.48	-12463.69	-121.41
412	D	-2.71	-220.86	2728.52	-152.29	4288.34	-56.71
426	D	-60.52	-198.04	11054.47	273.99	-3241.71	-38.44
430	D	102.92	843.32	15221.05	992.01	3698.28	64.95
440	D	-52.68	261.44	4787.86	-416.87	-2924.59	-59.80
450	D	-0.35	-265.72	3794.65	-166.75	1.87	-358.35
457	D	370.40	-3.32	4266.30	337.37	605.06	-226.13
458	D	-382.08	1.05	3715.82	-128.87	-701.73	27.20
466	D	0.35	677.94	8063.27	-2282.39	21.89	-0.24
467	D	571.16	876.65	9705.34	-2063.30	14927.43	-4.054E-002
476	D	30.21	949.91	6279.52	-1994.14	961.90	-162.15
481	D	593.88	-222.87	11505.51	677.96	23633.22	-61.12
486	D	46.12	-415.64	3209.04	96.58	1834.77	21.04
487	D	1360.97	-286.84	6685.72	269.92	15174.12	0.24
495	D	-178.67	608.20	8276.31	-786.24	-7642.10	-80.80
528	D	-549.77	-740.14	15177.45	-563.21	-21811.85	-70.40
534	D	-542.71	1079.81	12821.67	-830.86	-21565.86	-52.52
540	D	-302.61	-1141.50	11824.38	2401.88	-12115.87	-67.08
579	D	599.91	-833.06	6017.40	1400.54	24000.97	4.73
585	D	-220.50	19.85	9819.29	-730.25	-8763.56	-15.79
615	D	-2331.45	-436.41	12362.93	-6.52	1787.81	-0.40
621	D	1.312E-002	1124.93	3768.97	-1231.49	-1.87	0.27
628	D	-0.50	-1702.69	5482.16	2548.95	-20.94	-3.44
629	D	-592.92	124.31	15510.96	3154.74	-24257.37	-72.34
666	D	-3832.02	248.91	33773.13	-10745.31	-158937.82	71.15
672	D	7249.06	0.23	21309.41	-11.69	17058.28	1.81
673	D	3807.51	0.31	5474.79	-12.57	18598.19	-1.94
690	D	3989.16	1070.52	11751.69	-14419.55	152637.55	-337.64
696	D	4168.02	-187.54	15622.03	-11043.85	159791.39	-10.60
702	D	-3835.98	-411.28	48694.10	15700.88	-151514.11	9.31
708	D	-4148.44	2538.76	28118.61	-99751.51	-163559.10	140.44

Table 33: Nodal Reactions

Point	OutputCase	Fx	Fy	Fz	Mx	My	Mz
		lb	lb	lb	lb-in	lb-in	lb-in
726	D	568.20	52.39	13043.80	-2118.61	22281.61	-6.66
1286	D	-1841.41	312.80	49159.45	-12591.29	-73242.09	64.48
1591	D	2009.00	645.49	11298.30	-28666.91	68485.36	-247.40
1596	D	-1578.54	604.14	16465.13	-26956.28	-68981.88	-17.93
1602	D	-865.04	51.18	10341.98	-1871.28	-35220.62	-86.44
1607	D	2762.27	524.00	16252.10	-23470.99	96895.71	-197.64
1612	D	6.52	389.06	2398.03	-18317.31	-8506.89	81.94
2376	D	-220.62	32.04	3990.51	-1662.93	-9075.48	36.73
2377	D	-246.81	90.18	7499.23	-3967.96	-4050.53	67.86
2386	D	-1544.91	329.04	10267.67	-13258.35	-5136.32	-10.05
2395	D	1317.10	194.25	10475.74	-7882.16	326.02	44.85
2396	D	2005.83	0.24	9502.41	-7.31	5374.17	-0.10
2404	D	943.72	61.22	22324.76	-2796.32	17086.49	-153.76
3282	D	-238.74	26.88	24746.09	-1123.21	-9654.94	-9.59
3287	D	-2.92	-60.92	27187.04	2349.62	-247.18	-23.70
3292	D	-6.36	-6.412E-011	22874.05	-2679.53	-371.47	-30.16
3297	D	-5.48	-3.24	23857.14	-6.57	-341.05	-4.58
3304	D	-213.06	1.569E-009	38645.55	-1318.84	-8627.20	0.26
3309	D	-73.91	-2.734E-012	30115.44	-7.76	-3073.30	-0.74
3628	D	-120.13	2.497E-010	27984.97	-840.70	-4916.45	-2.91
3633	D	-2.996E-003	-141.72	3656.75	814.38	-0.25	-6.331E-003
3640	D	-7.524E-004	-60.24	6567.24	-55.88	-0.28	-1.673E-003
3645	D	2.597E-004	7.11	7836.08	245.75	-0.19	-1.530E-003
3646	D	-4.377E-004	77.06	4286.24	-338.22	-0.11	-6.159E-003
3665	D	2.947E-004	735.15	4436.31	1099.66	0.67	4.006E-003
3666	D	5.386E-004	-227.37	8031.36	2455.91	-0.11	7.650E-003
4020	D	-52.34	951.17	-5667.90	-107112.24	-2617.49	475.51
9884	D	-2830.85	-9.574E-002	1994.24	2.70	-20700.21	-0.31
9887	D	-414.05	0.22	4580.26	-5.85	-4159.05	0.74
9893	D	-1924.93	338.01	9132.31	1021.54	-3509.69	0.22
12023	D	-32.86	-4.70	2992.29	390.60	-1911.91	-74.73
21	L	-90.97	2.11	-391.97	269.05	-346.70	-3.03
22	L	-3.24	181.91	454.81	-698.92	-232.75	-9.07
65	L	-21.47	278.47	5467.45	295.74	-945.95	-6.50
76	L	746.85	15.64	1333.92	-518.71	2766.75	-5.234E-003
77	L	-160.79	-93.23	4065.17	3683.55	3138.79	-2.01
127	L	-256.84	-185.05	1126.62	6848.17	-946.24	-32.15
193	L	-83.64	-4.89	4363.31	-70.81	-3514.22	-17.76
207	L	2.177E-002	-234.63	246.30	783.69	1.20	1.141E-002
214	L	-116.88	-88.95	-386.73	480.33	-342.17	-2.878E-002
219	L	-219.27	-134.97	1790.73	5321.71	-694.55	1.31
226	L	-174.06	-22.84	-54.21	862.92	-1800.27	5.05
227	L	-154.92	-343.49	804.31	1133.10	-1201.34	10.30
296	L	-43.00	-257.32	2663.08	1784.44	-1790.30	-1.06
406	L	-89.04	31.12	346.75	-301.07	-3684.30	-19.47
412	L	-0.82	24.64	-166.62	-120.63	1321.32	-17.10
426	L	-27.16	-112.97	1374.45	263.02	-1270.47	-8.53
430	L	10.20	99.60	2603.91	343.62	319.36	19.44
440	L	-4.18	-7.40	-219.82	46.04	-362.79	-35.02
450	L	1.87	7.94	-22.50	74.51	77.55	-73.97
457	L	-4.29	-1.12	47.78	124.68	0.97	-43.05
458	L	2.34	0.17	-140.74	-19.07	-23.97	5.16
466	L	8.770E-002	105.04	861.77	-383.73	5.34	-5.428E-002
467	L	290.15	95.10	831.50	-114.46	4247.58	-2.887E-003

Table 33: Nodal Reactions

Point	OutputCase	Fx	Fy	Fz	Mx	My	Mz
		lb	lb	lb	lb-in	lb-in	lb-in
476	L	8.85	83.55	601.44	-90.35	309.64	-34.61
481	L	135.92	-22.22	1497.22	65.55	5416.51	-13.79
486	L	13.39	1.25	-152.22	-90.40	535.78	3.83
487	L	312.48	-3.16	184.87	-81.25	4111.53	4.705E-002
495	L	-28.43	30.54	498.72	-29.82	-1231.83	-15.86
528	L	-125.84	-134.34	2623.44	-1.38	-4985.11	-14.76
534	L	-126.86	177.84	2132.83	-29.83	-5033.07	-10.95
540	L	-80.74	-174.25	1824.15	359.71	-3218.22	-13.00
579	L	132.95	-138.49	832.50	306.56	5327.29	-1.40
585	L	-52.04	3.13	1837.13	-110.18	-2066.58	-3.47
615	L	-543.18	-46.91	2290.76	-32.16	621.27	-9.835E-002
621	L	5.108E-003	231.16	628.55	-314.87	-0.43	6.214E-002
628	L	-0.14	-253.87	799.14	410.69	-4.89	-0.81
629	L	-92.06	-47.21	1619.41	667.98	-3791.78	-16.39
666	L	-2215.21	42.68	22446.54	-1844.04	-86272.58	7.85
672	L	926.05	5.672E-002	2234.19	-2.90	1952.55	0.45
673	L	850.39	7.832E-002	-310.93	-3.11	2627.89	-0.48
690	L	705.94	210.19	1411.85	-3724.44	26779.91	-41.72
696	L	810.23	4.63	2434.66	-2351.83	30843.61	1.47
702	L	-760.19	-22.60	7829.24	758.23	-30238.45	6.73
708	L	-758.70	415.14	4020.43	-16432.03	-30136.64	36.71
726	L	123.43	15.11	2321.05	-611.34	4821.00	-0.50
1286	L	-400.19	98.38	11574.07	-3950.48	-16034.68	13.39
1591	L	478.00	261.61	1667.59	-9897.07	16704.03	91.04
1596	L	-350.85	284.11	3009.81	-11539.57	-15096.53	2.54
1602	L	-141.10	4.04	1268.12	-111.69	-5784.49	-20.31
1607	L	450.81	76.12	1891.42	-3494.79	15560.19	-42.58
1612	L	15.01	63.91	-204.29	-3030.45	-1123.37	5.27
2376	L	-74.34	-5.08	1796.15	103.75	-3042.95	9.22
2377	L	-244.88	22.31	807.45	-979.07	-1489.24	17.84
2386	L	-400.48	93.38	1574.28	-3750.63	-1856.38	-7.29
2395	L	1061.81	58.54	3974.92	-2347.34	3045.10	-25.37
2396	L	1716.74	6.339E-002	3337.03	-2.35	5937.95	5.820E-002
2404	L	1338.10	26.26	10746.10	-1032.68	14271.07	29.71
3282	L	-69.36	7.74	5958.22	-316.23	-2800.21	-5.15
3287	L	0.92	-20.74	6836.04	814.52	11.37	-4.51
3292	L	-1.75	-1.599E-011	6068.08	-699.24	-91.37	-5.92
3297	L	0.60	-5.24	4659.39	185.56	-1.42	-1.07
3304	L	-60.20	4.846E-010	10826.80	-407.64	-2437.09	2.672E-002
3309	L	-19.55	1.011E-012	8118.56	-6.52	-812.93	-0.81
3628	L	-34.30	7.273E-011	7969.02	-243.85	-1398.14	-0.73
3633	L	-1.326E-003	-58.76	369.41	219.01	-7.234E-002	-1.511E-003
3640	L	-3.671E-004	-42.25	904.13	12.85	-8.334E-002	-5.070E-004
3645	L	1.759E-005	42.02	1246.06	-89.92	-5.838E-002	-8.160E-004
3646	L	-1.950E-004	17.83	664.94	-93.93	-3.268E-002	-2.286E-003
3665	L	1.181E-004	131.69	730.40	195.32	0.21	5.730E-004
3666	L	8.756E-005	4.96	1272.75	333.93	-3.535E-002	9.003E-004
4020	L	-12.06	244.21	1377.06	6537.97	-639.75	108.69
9884	L	-783.21	-2.543E-002	70.18	0.69	-5365.33	-8.058E-002
9887	L	-154.21	6.083E-002	857.58	-1.60	-1111.94	0.20
9893	L	-393.48	19.24	1359.21	243.38	-762.80	4.705E-002
12023	L	-4.68	-2.02	238.37	127.57	-285.00	-14.32

Table 34: Integrated Wall Forces

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
1	D	LinStatic	5.69	6256.48	50790.44	-690947.09	228.12	10.96
3	D	LinStatic	8.631E-014	-4.863E-012	8.014E-012	-5.126E-010	2.258E-013	-1.681E-011
4	D	LinStatic	5.15	-2269.21	23422.64	-41977.90	206.45	-3.65
16	D	LinStatic	7600.40	-0.45	41785.32	19.88	-192518.71	-19.42
18	D	LinStatic	-1.889E-010	6.690E-002	136.50	1.74	6281.96	-0.53
48	D	LinStatic	0.92	-4302.07	67326.24	-947710.86	34.04	-30.23
50	D	LinStatic	-0.17	328.95	-134.14	-11285.39	4.376E-003	13.06
51	D	LinStatic	-8.649E-003	-2228.26	6087.65	8902.78	2.57	7.31
53	D	LinStatic	0.12	-329.16	70.89	-5251.35	0.91	-5.16
54	D	LinStatic	-3488.10	-1.83	24273.53	76.01	-526713.70	-66.20
56	D	LinStatic	4.807E-002	0.22	63.24	0.63	4108.70	-1.64
57	D	LinStatic	-1344.82	-6.957E-002	2806.75	1.86	-56956.80	-1.24
59	D	LinStatic	7.298E-003	-1.796E-002	-11.55	-0.13	185.00	-0.65
74	D	LinStatic	-0.27	-3244.91	16776.03	54832.64	-12.45	-4.60
76	D	LinStatic	-7.298E-003	1.796E-002	11.55	369.21	-6.056E-003	0.60
105	D	LinStatic	-0.24	184.18	6384.15	-22772.10	-7.46	-10.36
107	D	LinStatic	1.167E-002	77.17	-7.97	199.97	-1.453E-002	-0.72
108	D	LinStatic	-0.46	600.48	12869.11	-53743.17	-19.27	4.24
110	D	LinStatic	-1.167E-002	-77.17	7.97	384.50	1.453E-002	1.58
114	D	LinStatic	-2.863E-002	-3.80	4452.69	-4808.16	-1.53	-2.63
116	D	LinStatic	-4.815E-017	-6.977E-013	-6.936E-013	-3.984E-011	-1.453E-015	-3.689E-015
117	D	LinStatic	-85.04	-2.221E-002	11743.46	1.42	11825.46	2.20
119	D	LinStatic	-5.577E-014	-4.886E-017	-1.489E-012	6.914E-017	1.544E-011	9.823E-016
120	D	LinStatic	0.41	727.12	7387.21	-13285.74	12.56	-10.32
122	D	LinStatic	8.790E-002	24.99	10.52	338.07	-10.56	-0.90
123	D	LinStatic	7.348E-002	1922.67	13170.75	-41734.17	0.89	-4.82
125	D	LinStatic	0.16	-124.17	-57.29	-1893.73	-10.37	-7.95
126	D	LinStatic	2.21	454.42	18143.30	-5773.76	94.71	-41.81
128	D	LinStatic	-0.35	98.89	-23.54	1187.35	1.51	-9.80
129	D	LinStatic	0.24	-257.09	3707.95	-900.78	3.86	2.60
131	D	LinStatic	2.126E-002	-1.79	-11.20	65.59	-10.03	0.57
132	D	LinStatic	0.15	247.90	12339.47	-49655.51	-2.57	-14.54
134	D	LinStatic	-3.801E-002	2.14	-23.31	-709.51	-10.29	6.18
162	D	LinStatic	-1.11	303.17	13764.31	-3854.75	-44.59	-0.11
164	D	LinStatic	-8.655E-016	8.261E-013	-2.022E-012	2.200E-013	-1.417E-013	-1.325E-013
168	D	LinStatic	-0.48	-1384.74	19703.71	60061.57	-22.27	15.61
170	D	LinStatic	4.129E-015	-3.225E-012	5.855E-012	1.629E-011	5.828E-015	5.900E-014
174	D	LinStatic	-4.571E-002	688.48	7557.46	-997.19	-3.74	0.33
176	D	LinStatic	5.001E-016	3.744E-013	3.893E-012	-5.774E-011	-1.472E-016	-1.793E-015
177	D	LinStatic	-4.49	-5912.81	38863.21	105593.76	-184.12	0.30
179	D	LinStatic	-3.144E-014	-3.416E-012	6.891E-012	-2.418E-010	-4.568E-013	2.104E-012
189	D	LinStatic	20098.53	1.18	51752.85	-48.53	512347.76	2.38
191	D	LinStatic	1.089E-010	-1.718E-014	-3.664E-011	-3.323E-014	2.290E-010	-1.085E-013
628	D	LinStatic	-2447.70	0.66	14663.61	-27.21	-135240.73	6.44
630	D	LinStatic	4.445E-012	-1.080E-014	-5.671E-012	-1.311E-014	-1.176E-011	-1.728E-013
631	D	LinStatic	2294.95	0.22	11441.37	-9.31	49938.35	-0.76
633	D	LinStatic	308.05	-1.422E-003	-216.83	-5.041E-003	5186.72	-2.325E-002
634	D	LinStatic	1648.33	0.13	7930.13	-5.38	29664.17	-0.48
636	D	LinStatic	-308.05	1.422E-003	216.83	5.041E-003	353.54	-1.309E-002
637	D	LinStatic	276.86	9.271E-002	10182.80	-2.63	-11896.73	1.55
639	D	LinStatic	0.12	-6.772E-002	104.83	0.75	-1169.72	1.89
953	D	LinStatic	-2.868E-017	-590.20	14581.32	11780.10	-7.528E-016	-2.03

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
955	D	LinStatic	-7.343E-016	85.83	-13.08	-10905.73	7.424E-016	-1.96
956	D	LinStatic	-2.080E-016	-314.71	12048.90	-3852.63	-5.270E-015	3.653E-002
958	D	LinStatic	4.895E-016	-85.90	-123.43	-4743.80	-1.284E-015	-0.59
959	D	LinStatic	1.252E-016	289.85	11133.42	-6134.26	8.469E-016	9.128E-002
961	D	LinStatic	-2.683E-017	2.661E-013	-5.888E-012	2.022E-012	9.248E-017	4.901E-016
965	D	LinStatic	5.737E-017	69.58	16444.11	-14107.48	-9.787E-016	3.801E-002
967	D	LinStatic	0.00	3.678E-012	-8.159E-012	1.055E-010	-5.251E-016	-5.693E-016
1053	D	LinStatic	-2.105E-002	-12.26	19651.43	-60952.66	-1.99	7.82
1055	D	LinStatic	-1.147E-017	3981.39	51817.38	23669.42	-1.176E-016	0.19
1057	D	LinStatic	-7401.90	-2.95	30271.30	122.39	-589637.02	-60.54
2667	D	LinStatic	1.680E-016	-343.54	15691.40	-10710.16	3.200E-015	8.852E-002
2669	D	LinStatic	-1.205E-016	145.01	11074.96	-4959.60	-2.033E-015	9.077E-002
2671	D	LinStatic	-5162.24	-6.453E-002	10568.68	4.44	-163411.90	2.92
2675	D	LinStatic	-6800.10	0.26	27123.47	-13.10	-149932.54	-17.24
1	L	LinStatic	1.15	1780.50	3082.64	-145555.65	45.87	-0.34
3	L	LinStatic	1.106E-014	-5.502E-013	1.337E-012	-1.021E-011	1.041E-013	-1.827E-012
4	L	LinStatic	1.20	-402.59	3175.07	-1033.47	47.99	-0.16
16	L	LinStatic	1704.28	-0.18	7068.79	7.77	-50997.01	-7.44
18	L	LinStatic	-4.826E-011	2.637E-002	24.89	0.36	1145.65	-0.25
48	L	LinStatic	0.40	-1260.76	11762.70	-232134.68	15.50	-5.68
50	L	LinStatic	-6.774E-002	79.18	-29.96	-2542.04	2.220E-003	5.19
51	L	LinStatic	-3.703E-004	-442.96	-140.91	16.24	0.97	2.47
53	L	LinStatic	3.730E-002	-79.25	14.44	-1190.22	0.30	-0.82
54	L	LinStatic	-701.92	-0.53	2282.55	22.11	-128539.31	-20.06
56	L	LinStatic	3.045E-002	6.134E-002	15.53	0.22	1008.54	-4.598E-002
57	L	LinStatic	-297.52	-2.008E-002	-62.49	0.56	-13709.15	-0.37
59	L	LinStatic	2.075E-003	-5.081E-003	-3.53	-3.872E-002	56.48	-0.19
74	L	LinStatic	-6.837E-002	-1084.12	2692.53	20555.48	-3.23	-1.41
76	L	LinStatic	-2.075E-003	5.081E-003	3.53	112.70	-9.355E-003	0.17
105	L	LinStatic	-7.334E-002	76.03	-9.66	-6292.98	-2.21	-3.13
107	L	LinStatic	2.906E-003	12.75	-1.24	21.99	-4.010E-003	-0.25
108	L	LinStatic	-0.15	27.57	1173.18	-11400.30	-6.18	0.86
110	L	LinStatic	-2.906E-003	-12.75	1.24	68.65	4.010E-003	0.46
114	L	LinStatic	-3.109E-004	0.51	-123.13	966.32	-0.10	-0.59
116	L	LinStatic	-8.312E-017	-1.053E-013	-5.572E-014	2.499E-012	1.383E-015	6.538E-016
117	L	LinStatic	-24.05	-5.815E-003	-89.89	0.47	4237.93	0.66
119	L	LinStatic	-1.793E-014	-1.327E-016	1.078E-013	3.196E-016	2.993E-012	2.745E-015
120	L	LinStatic	0.11	95.43	735.23	-1467.98	3.29	-2.32
122	L	LinStatic	2.191E-002	4.10	-4.31	33.62	-2.93	-0.22
123	L	LinStatic	3.384E-002	181.32	1214.09	-3805.03	0.68	-1.41
125	L	LinStatic	3.900E-002	-4.22	-4.21	-111.22	-2.78	-1.98
126	L	LinStatic	0.52	85.43	2152.25	1690.58	22.49	-9.82
128	L	LinStatic	-8.161E-002	6.436E-002	-1.04	90.96	0.42	-2.32
129	L	LinStatic	6.232E-002	2.75	-110.02	-1199.74	0.99	0.76
131	L	LinStatic	6.007E-003	-3.01	-0.53	-19.38	-2.70	7.640E-002
132	L	LinStatic	4.933E-002	46.50	486.51	-5934.77	-5.199E-002	-3.22
134	L	LinStatic	-6.799E-003	3.08	-4.89	-150.65	-2.74	1.29
162	L	LinStatic	-0.26	38.84	2321.72	331.37	-10.30	-5.197E-002
164	L	LinStatic	-8.879E-016	6.574E-013	-1.084E-012	8.264E-013	-3.386E-014	-1.351E-014
168	L	LinStatic	-0.12	-274.17	2733.14	12154.26	-5.71	4.40
170	L	LinStatic	1.817E-015	-8.406E-013	-1.938E-013	2.727E-011	-1.410E-014	4.598E-014
174	L	LinStatic	-9.520E-003	184.24	1247.46	-467.12	-0.87	9.799E-002
176	L	LinStatic	-8.636E-016	-4.258E-013	4.495E-013	-1.052E-012	-2.480E-016	-1.116E-014
177	L	LinStatic	-0.92	-971.43	4683.83	39070.61	-37.52	5.38

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
179	L	LinStatic	-7.971E-015	1.232E-013	4.492E-013	3.611E-011	-7.382E-014	4.243E-013
189	L	LinStatic	3388.45	0.29	3713.03	-12.02	82801.49	0.58
191	L	LinStatic	1.390E-011	-2.629E-015	7.508E-012	-1.254E-014	-7.577E-011	3.076E-014
628	L	LinStatic	-910.85	0.19	1962.41	-7.61	-45949.77	1.84
630	L	LinStatic	-1.953E-014	-3.036E-015	1.900E-013	-6.463E-015	-3.881E-012	-1.806E-014
631	L	LinStatic	2201.09	7.751E-002	5218.99	-3.09	55794.74	-0.45
633	L	LinStatic	174.29	1.797E-004	-125.10	2.899E-003	3077.43	2.566E-003
634	L	LinStatic	1639.82	3.981E-002	1843.87	-1.66	33155.68	-0.26
636	L	LinStatic	-174.29	-1.797E-004	125.10	-2.899E-003	119.12	2.026E-003
637	L	LinStatic	237.54	2.114E-002	608.93	-0.58	821.97	0.17
639	L	LinStatic	2.150E-002	-1.622E-002	14.98	0.17	-129.00	0.28
953	L	LinStatic	-1.434E-017	-117.59	896.96	-3031.05	-1.506E-016	-0.97
955	L	LinStatic	-3.060E-016	24.58	0.66	-2131.60	5.946E-016	-0.73
956	L	LinStatic	-7.245E-017	-128.45	1485.09	-1529.01	-1.506E-015	-2.303E-002
958	L	LinStatic	3.060E-017	-24.61	-25.55	-956.31	-5.168E-016	-0.23
959	L	LinStatic	1.147E-017	97.63	1775.16	-2106.38	7.528E-017	1.596E-002
961	L	LinStatic	-5.686E-018	-2.940E-013	-1.581E-012	-2.513E-011	6.164E-018	1.727E-016
965	L	LinStatic	1.530E-017	79.56	2620.23	-3936.16	-2.259E-016	9.989E-003
967	L	LinStatic	0.00	-1.065E-012	9.051E-014	-2.159E-011	-3.512E-016	-1.897E-016
1053	L	LinStatic	-4.911E-002	-138.92	2974.49	-10362.00	-2.23	1.60
1055	L	LinStatic	-2.868E-018	768.41	6912.12	-9346.67	-6.470E-017	8.456E-002
1057	L	LinStatic	-1743.69	-0.61	3155.59	25.37	-164484.91	-13.21
2667	L	LinStatic	5.563E-017	34.75	2493.54	-3404.36	9.222E-016	1.821E-002
2669	L	LinStatic	-3.899E-017	21.16	1677.64	-1465.26	-3.952E-016	1.176E-002
2671	L	LinStatic	-1326.37	-1.488E-002	1742.03	1.11	-39916.23	0.82
2675	L	LinStatic	-1655.38	6.985E-002	4654.37	-3.56	-18293.52	-4.76
1	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	8.67	10356.57	65880.75	-1062025.56	347.13	12.61
3	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.213E-013	-6.715E-012	1.176E-011	-6.315E-010	4.376E-013	-2.310E-011
4	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	8.10	-3367.19	33187.28	-52027.03	324.52	-4.64
16	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	11847.33	-0.83	61452.44	36.29	-312617.68	-35.22
18	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-3.039E-010	0.12	203.64	2.67	9371.39	-1.04
48	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.75	-7179.70	99611.80	-1508668.52	65.66	-45.37
50	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.31	521.43	-208.90	-17609.73	8.803E-003	23.97
51	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.097E-002	-3382.66	7079.73	10709.31	4.64	12.73
53	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.21	-521.79	108.17	-8205.98	1.57	-7.49
54	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-5308.80	-3.04	32780.33	126.59	-837719.33	-111.53

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
56	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.11	0.36	100.73	1.10	6544.11	-2.04
57	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-2089.81	-0.12	3268.12	3.13	-90282.80	-2.08
59	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.208E-002	-2.968E-002	-19.50	-0.22	312.37	-1.08
74	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.43	-5628.48	24439.28	98687.93	-20.11	-7.77
76	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.208E-002	2.968E-002	19.50	623.38	-2.224E-002	0.99
105	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.41	342.66	7645.53	-37395.29	-12.49	-17.44
107	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.865E-002	113.00	-11.54	275.15	-2.385E-002	-1.27
108	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.80	764.68	17320.03	-82732.29	-33.02	6.47
110	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.865E-002	-113.00	11.54	571.25	2.385E-002	2.64
114	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-3.486E-002	-3.74	5146.22	-4223.68	-2.00	-4.09
116	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.908E-016	-1.006E-012	-9.215E-013	-4.381E-011	4.687E-016	-3.381E-015
117	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-140.53	-3.596E-002	13948.32	2.46	20971.25	3.68
119	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-9.562E-014	-2.709E-016	-1.614E-012	5.944E-016	2.332E-011	5.571E-015
120	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.67	1025.23	10041.03	-18291.66	20.34	-16.09
122	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.14	36.55	5.71	459.48	-17.35	-1.43
123	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.14	2597.32	17747.44	-56169.05	2.15	-8.03
125	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.25	-155.76	-75.48	-2450.43	-16.88	-12.70
126	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	3.48	682.00	25215.56	-4223.58	149.63	-65.89
128	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.55	118.77	-29.90	1570.35	2.49	-15.48
129	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.39	-304.11	4273.51	-3000.52	6.21	4.33

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
131	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	3.512E-002	-6.97	-14.29	47.71	-16.35	0.81
132	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.25	371.89	15585.78	-69082.24	-3.17	-22.61
134	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-5.649E-002	7.50	-35.80	-1092.45	-16.73	9.48
162	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.74	425.95	20231.91	-4095.50	-69.98	-0.21
164	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-2.459E-015	2.043E-012	-4.161E-012	1.586E-012	-2.242E-013	-1.806E-013
168	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.78	-2100.36	28017.47	91520.70	-35.86	25.78
170	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	7.862E-015	-5.215E-012	6.716E-012	6.319E-011	-1.557E-014	1.444E-013
174	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-7.009E-002	1120.97	11064.89	-1944.02	-5.89	0.56
176	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-7.817E-016	-2.320E-013	5.391E-012	-7.097E-011	-5.734E-016	-2.001E-014
177	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-6.86	-8649.65	54129.98	189225.48	-280.97	8.96
179	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-5.048E-014	-3.902E-012	8.988E-012	-2.323E-010	-6.662E-013	3.204E-012
189	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	29539.76	1.88	68044.27	-77.46	747299.69	3.79
191	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.529E-010	-2.483E-014	-3.196E-011	-5.995E-014	1.536E-010	-8.105E-014
628	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-4394.61	1.09	20736.18	-44.83	-235808.51	10.67
630	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	5.303E-012	-1.782E-014	-6.501E-012	-2.608E-014	-2.032E-011	-2.362E-013
631	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	6275.67	0.39	22080.04	-16.12	149197.61	-1.63
633	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	648.53	-1.419E-003	-460.36	-1.412E-003	11147.95	-2.379E-002
634	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	4601.71	0.22	12466.34	-9.11	88646.09	-0.99
636	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-648.53	1.419E-003	460.36	1.412E-003	614.85	-1.247E-002
637	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	712.30	0.15	13193.65	-4.08	-12960.92	2.13

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
639	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.18	-0.11	149.76	1.18	-1610.06	2.72
953	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-5.737E-017	-896.39	18932.72	9286.45	-1.144E-015	-3.99
955	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.371E-015	142.33	-14.64	-16497.44	1.842E-015	-3.52
956	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-3.655E-016	-583.16	16834.83	-7069.57	-8.733E-015	6.982E-003
958	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	6.364E-016	-142.45	-188.99	-7222.65	-2.368E-015	-1.08
959	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	1.687E-016	504.04	16200.37	-10731.32	1.137E-015	0.14
961	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-4.129E-017	-1.511E-013	-9.595E-012	-3.778E-011	1.208E-016	8.644E-016
965	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	9.332E-017	210.80	23925.30	-23226.83	-1.536E-015	6.160E-002
967	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	0.00	2.710E-012	-9.646E-012	9.207E-011	-1.192E-015	-9.866E-016
1053	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-0.10	-236.98	28340.90	-89722.39	-5.95	11.96
1055	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-1.836E-017	6007.13	73240.24	13448.63	-2.447E-016	0.36
1057	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-11672.19	-4.52	41374.51	187.45	-970740.28	-93.77
2667	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	2.907E-016	-356.65	22819.35	-18299.17	5.315E-015	0.14
2669	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-2.069E-016	207.87	15974.18	-8295.93	-3.072E-015	0.13
2671	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-8316.89	-0.10	15469.66	7.10	-259960.25	4.81
2675	Comb=1.2 D + 1.6 L +1.6 Lr	Combination	-10808.73	0.43	39995.15	-21.41	-209188.69	-28.31
1	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	6.84	8036.98	53873.08	-836502.75	273.99	10.62
3	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	9.736E-014	-5.413E-012	9.351E-012	-5.228E-010	3.299E-013	-1.864E-011
4	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	6.35	-2671.80	26597.71	-43011.37	254.44	-3.81
16	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	9304.68	-0.63	48854.10	27.65	-243515.73	-26.87

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
18	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-2.372E-010	9.327E-002	161.40	2.10	7427.61	-0.78
48	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	1.32	-5562.83	79088.94	-1179845.54	49.55	-35.91
50	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.24	408.13	-164.10	-13827.43	6.596E-003	18.25
51	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-9.020E-003	-2671.23	5946.75	8919.01	3.55	9.78
53	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.16	-408.41	85.33	-6441.58	1.21	-5.97
54	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-4190.03	-2.36	26556.09	98.12	-655253.00	-86.26
56	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	7.851E-002	0.28	78.77	0.85	5117.25	-1.68
57	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1642.34	-8.965E-002	2744.27	2.42	-70665.95	-1.61
59	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	9.373E-003	-2.304E-002	-15.07	-0.17	241.48	-0.84
74	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.34	-4329.03	19468.55	75388.12	-15.68	-6.01
76	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-9.373E-003	2.304E-002	15.07	481.91	-1.541E-002	0.77
105	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.32	260.21	6374.49	-29065.08	-9.67	-13.49
107	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	1.458E-002	89.92	-9.20	221.96	-1.854E-002	-0.98
108	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.61	628.04	14042.30	-65143.47	-25.45	5.11
110	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.458E-002	-89.92	9.20	453.16	1.854E-002	2.04
114	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-2.894E-002	-3.29	4329.56	-3841.84	-1.63	-3.21
116	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.313E-016	-8.030E-013	-7.494E-013	-3.734E-011	-7.028E-017	-3.036E-015
117	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-109.09	-2.803E-002	11653.57	1.89	16063.39	2.85
119	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-7.370E-014	-1.816E-016	-1.381E-012	3.888E-016	1.844E-011	3.727E-015
120	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.52	822.55	8122.45	-14753.72	15.85	-12.63

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
122	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.11	29.09	6.20	371.69	-13.48	-1.12
123	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.11	2103.99	14384.84	-45539.20	1.57	-6.23
125	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.19	-128.39	-61.50	-2004.95	-13.14	-9.92
126	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	2.73	539.85	20295.55	-4083.18	117.20	-51.63
128	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.43	98.96	-24.57	1278.31	1.94	-12.13
129	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.30	-254.34	3597.93	-2100.52	4.85	3.36
131	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	2.727E-002	-4.80	-11.73	46.22	-12.73	0.65
132	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.20	294.41	12825.98	-55590.28	-2.62	-17.77
134	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-4.480E-002	5.23	-28.20	-860.16	-13.03	7.47
162	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.36	342.01	16086.02	-3523.38	-54.89	-0.16
164	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.753E-015	1.484E-012	-3.106E-012	1.046E-012	-1.756E-013	-1.460E-013
168	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-0.61	-1658.91	22436.85	72215.83	-27.98	20.01
170	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	5.946E-015	-4.065E-012	5.661E-012	4.357E-011	-8.277E-015	1.050E-013
174	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-5.523E-002	872.72	8804.92	-1464.31	-4.62	0.43
176	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-3.635E-016	-5.138E-014	4.343E-012	-5.879E-011	-3.952E-016	-1.296E-014
177	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-5.41	-6884.24	43547.04	144664.36	-221.64	5.68
179	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-3.941E-014	-3.293E-012	7.341E-012	-2.057E-010	-5.306E-013	2.529E-012
189	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	23486.98	1.47	55465.88	-60.55	595149.25	2.96
191	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	1.228E-010	-1.981E-014	-2.913E-011	-4.578E-014	1.533E-010	-7.779E-014
628	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-3358.55	0.85	16626.01	-34.82	-181190.50	8.28

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
630	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	4.426E-012	-1.384E-014	-5.481E-012	-1.958E-014	-1.564E-011	-1.908E-013
631	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	4496.03	0.30	16660.36	-12.40	105733.09	-1.21
633	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	482.34	-1.242E-003	-341.93	-2.143E-003	8264.15	-2.068E-002
634	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	3288.15	0.17	9774.00	-7.04	62819.85	-0.74
636	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-482.34	1.242E-003	341.93	2.143E-003	472.67	-1.106E-002
637	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	514.40	0.11	10791.73	-3.21	-11074.76	1.72
639	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.15	-8.394E-002	119.81	0.93	-1298.72	2.17
953	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-4.302E-017	-707.80	15478.28	8749.06	-9.034E-016	-3.00
955	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.040E-015	110.42	-12.42	-13037.34	1.337E-015	-2.69
956	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-2.804E-016	-443.15	13533.99	-5381.64	-6.776E-015	1.350E-002
958	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	5.201E-016	-110.51	-148.98	-5700.10	-1.801E-015	-0.82
959	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	1.367E-016	387.49	12908.59	-8240.64	9.222E-016	0.11
961	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-3.251E-017	-2.792E-014	-7.469E-012	-2.310E-011	9.865E-017	6.628E-016
965	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	7.266E-017	149.14	19064.34	-18043.64	-1.205E-015	4.800E-002
967	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	0.00	2.613E-012	-8.069E-012	8.392E-011	-8.763E-016	-7.590E-016
1053	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-7.016E-002	-151.18	22625.92	-71314.66	-4.22	9.43
1055	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.434E-017	4749.81	58729.50	14322.75	-1.823E-016	0.27
1057	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-9145.59	-3.56	33426.89	147.76	-754121.93	-73.74
2667	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	2.237E-016	-308.79	18184.94	-14114.52	4.122E-015	0.11
2669	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-1.595E-016	166.17	12752.61	-6424.85	-2.428E-015	0.10

Table 34: Integrated Wall Forces

Area	OutputCase	CaseType	Fx	Fy	Fz	Mx	My	Mz
			lb	lb	lb	lb-in	lb-in	lb-in
2671	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-6488.62	-7.941E-002	12310.71	5.55	-203328.13	3.74
2675	Comb=1.0 D + 1.0 L+1.0 Lr	Combination	-8455.48	0.33	31777.84	-16.66	-168226.06	-22.00

5.2. Structure results

Table 35: Sum Of Reactions, Part 1 of 2

Table 35: Sum Of Reactions, Part 1 of 2

OutputCase	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
	lb	lb	lb	lb-in	lb-in	lb-in
D	-5.714E-003	-0.20	1370382.70	816711536	-837616478	-29.73
L	-1.411E-003	-4.732E-002	240068.05	133334470	-147995430	-9.10
Comb=1.2 D + 1.6 L +1.6 Lr	-9.114E-003	-0.32	2028568.12	1193388994	-1241932461	-50.24
Comb=1.0 D + 1.0 L+1.0 Lr	-7.125E-003	-0.25	1610450.75	950046005	-985611908	-38.83

Table 35: Sum Of Reactions, Part 2 of 2

Table 35: Sum Of Reactions, Part 2 of 2

OutputCase	GlobalX	GlobalY	GlobalZ
	in	in	in
D	0.000	0.000	0.000
L	0.000	0.000	0.000
Comb=1.2 D + 1.6 L +1.6 Lr	0.000	0.000	0.000
Comb=1.0 D + 1.0 L+1.0 Lr	0.000	0.000	0.000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
1	2447	D	0.000000	0.000000	0.003414	0.000000	0.000000	0.000000
2	727	D	0.000000	0.000000	-0.004021	0.000000	0.000000	0.000000
3	~629	D	0.000000	0.000000	-0.004183	0.000000	0.000000	0.000000
4	~492	D	0.000000	0.000000	0.003064	0.000000	0.000000	0.000000
5	~500	D	0.000000	0.000000	0.003396	0.000000	0.000000	0.000000
6	~516	D	0.000000	0.000000	0.003660	0.000000	0.000000	0.000000
7	~601	D	0.000000	0.000000	0.003801	0.000000	0.000000	0.000000
8	~547	D	0.000000	0.000000	0.004254	0.000000	0.000000	0.000000
9	~599	D	0.000000	0.000000	0.004601	0.000000	0.000000	0.000000
10	~563	D	0.000000	0.000000	0.003634	0.000000	0.000000	0.000000
11	~571	D	0.000000	0.000000	0.002838	0.000000	0.000000	0.000000
12	~1213	D	0.000000	0.000000	-0.001190	0.000000	0.000000	0.000000
13	~1098	D	0.000000	0.000000	-0.006836	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
14	~1108	D	0.000000	0.000000	-0.017250	0.000000	0.000000	0.000000
16	~1128	D	0.000000	0.000000	-0.031603	0.000000	0.000000	0.000000
17	~1137	D	0.000000	0.000000	-0.032198	0.000000	0.000000	0.000000
18	~1145	D	0.000000	0.000000	-0.032283	0.000000	0.000000	0.000000
19	~1154	D	0.000000	0.000000	-0.028644	0.000000	0.000000	0.000000
20	~1164	D	0.000000	0.000000	-0.018765	0.000000	0.000000	0.000000
21	~1217	D	0.000000	0.000000	-0.002404	0.000000	0.000000	0.000000
22	~1182	D	0.000000	0.000000	0.001064	0.000000	0.000000	0.000000
23	~1000	D	0.000000	0.000000	-0.084473	0.000000	0.000000	0.000000
24	~479	D	0.000000	0.000000	-0.059231	0.000000	0.000000	0.000000
25	~486	D	0.000000	0.000000	-0.084472	0.000000	0.000000	0.000000
26	~494	D	0.000000	0.000000	-0.109813	0.000000	0.000000	0.000000
27	~502	D	0.000000	0.000000	-0.133350	0.000000	0.000000	0.000000
28	~518	D	0.000000	0.000000	-0.166027	0.000000	0.000000	0.000000
29	~518	D	0.000000	0.000000	-0.166027	0.000000	0.000000	0.000000
30	~525	D	0.000000	0.000000	-0.167795	0.000000	0.000000	0.000000
31	~557	D	0.000000	0.000000	-0.111509	0.000000	0.000000	0.000000
32	~651	D	0.000000	0.000000	-0.091696	0.000000	0.000000	0.000000
33	~648	D	0.000000	0.000000	-0.073135	0.000000	0.000000	0.000000
34	60	D	0.000000	0.000000	-0.006396	0.000000	0.000000	0.000000
35	~1195	D	0.000000	0.000000	-0.009011	0.000000	0.000000	0.000000
36	~1109	D	0.000000	0.000000	-0.026023	0.000000	0.000000	0.000000
38	~1129	D	0.000000	0.000000	-0.049039	0.000000	0.000000	0.000000
39	~1138	D	0.000000	0.000000	-0.050007	0.000000	0.000000	0.000000
40	~1146	D	0.000000	0.000000	-0.050189	0.000000	0.000000	0.000000
41	~1155	D	0.000000	0.000000	-0.044795	0.000000	0.000000	0.000000
42	~1165	D	0.000000	0.000000	-0.029928	0.000000	0.000000	0.000000
43	~1217	D	0.000000	0.000000	-0.002404	0.000000	0.000000	0.000000
44	~1183	D	0.000000	0.000000	0.002774	0.000000	0.000000	0.000000
45	~1002	D	0.000000	0.000000	-0.152429	0.000000	0.000000	0.000000
46	~1040	D	0.000000	0.000000	-0.072065	0.000000	0.000000	0.000000
47	~487	D	0.000000	0.000000	-0.101313	0.000000	0.000000	0.000000
48	~495	D	0.000000	0.000000	-0.131976	0.000000	0.000000	0.000000
49	~503	D	0.000000	0.000000	-0.160260	0.000000	0.000000	0.000000
50	~519	D	0.000000	0.000000	-0.197784	0.000000	0.000000	0.000000
51	~519	D	0.000000	0.000000	-0.197784	0.000000	0.000000	0.000000
52	~526	D	0.000000	0.000000	-0.198589	0.000000	0.000000	0.000000
53	~558	D	0.000000	0.000000	-0.132454	0.000000	0.000000	0.000000
54	~566	D	0.000000	0.000000	-0.106234	0.000000	0.000000	0.000000
55	~574	D	0.000000	0.000000	-0.086364	0.000000	0.000000	0.000000
56	~590	D	0.000000	0.000000	-0.023026	0.000000	0.000000	0.000000
57	~1102	D	0.000000	0.000000	-0.014104	0.000000	0.000000	0.000000
58	~1222	D	0.000000	0.000000	-0.025321	0.000000	0.000000	0.000000
60	~1131	D	0.000000	0.000000	-0.041873	0.000000	0.000000	0.000000
61	~1140	D	0.000000	0.000000	-0.042533	0.000000	0.000000	0.000000
62	~1148	D	0.000000	0.000000	-0.042746	0.000000	0.000000	0.000000
63	~1157	D	0.000000	0.000000	-0.039425	0.000000	0.000000	0.000000
64	~1167	D	0.000000	0.000000	-0.027406	0.000000	0.000000	0.000000
65	~1197	D	0.000000	0.000000	-0.000777	0.000000	0.000000	0.000000
66	~1185	D	0.000000	0.000000	0.002506	0.000000	0.000000	0.000000
67	~1003	D	0.000000	0.000000	-0.140195	0.000000	0.000000	0.000000
68	~1041	D	0.000000	0.000000	-0.062106	0.000000	0.000000	0.000000
69	~489	D	0.000000	0.000000	-0.081800	0.000000	0.000000	0.000000
70	~497	D	0.000000	0.000000	-0.108236	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
71	~505	D	0.000000	0.000000	-0.130017	0.000000	0.000000	0.000000
72	~521	D	0.000000	0.000000	-0.145435	0.000000	0.000000	0.000000
73	~521	D	0.000000	0.000000	-0.145435	0.000000	0.000000	0.000000
74	~646	D	0.000000	0.000000	-0.138183	0.000000	0.000000	0.000000
75	~647	D	0.000000	0.000000	-0.090039	0.000000	0.000000	0.000000
76	~568	D	0.000000	0.000000	-0.073939	0.000000	0.000000	0.000000
77	~576	D	0.000000	0.000000	-0.063132	0.000000	0.000000	0.000000
78	~1094	D	0.000000	0.000000	-0.023414	0.000000	0.000000	0.000000
79	~1103	D	0.000000	0.000000	-0.016117	0.000000	0.000000	0.000000
80	~1232	D	0.000000	0.000000	-0.015664	0.000000	0.000000	0.000000
82	~1228	D	0.000000	0.000000	-0.011232	0.000000	0.000000	0.000000
83	194	D	0.000000	0.000000	-0.004127	0.000000	0.000000	0.000000
84	194	D	0.000000	0.000000	-0.004127	0.000000	0.000000	0.000000
85	~1229	D	0.000000	0.000000	-0.004716	0.000000	0.000000	0.000000
86	~1227	D	0.000000	0.000000	-0.004248	0.000000	0.000000	0.000000
87	128	D	0.000000	0.000000	-0.000540	0.000000	0.000000	0.000000
88	128	D	0.000000	0.000000	-0.000540	0.000000	0.000000	0.000000
89	~1086	D	0.000000	0.000000	-0.031019	0.000000	0.000000	0.000000
90	1287	D	0.000000	0.000000	-0.011277	0.000000	0.000000	0.000000
91	~1743	D	0.000000	0.000000	-0.034733	0.000000	0.000000	0.000000
92	~1779	D	0.000000	0.000000	-0.048281	0.000000	0.000000	0.000000
93	~1753	D	0.000000	0.000000	-0.052290	0.000000	0.000000	0.000000
94	~1753	D	0.000000	0.000000	-0.052290	0.000000	0.000000	0.000000
95	5116	D	0.000000	0.000000	-0.029756	0.000000	0.000000	0.000000
96	~939	D	0.000000	0.000000	-0.061609	0.000000	0.000000	0.000000
97	~698	D	0.000000	0.000000	-0.062273	0.000000	0.000000	0.000000
98	~721	D	0.000000	0.000000	-0.071645	0.000000	0.000000	0.000000
99	~731	D	0.000000	0.000000	-0.073662	0.000000	0.000000	0.000000
100	~941	D	0.000000	0.000000	-0.071559	0.000000	0.000000	0.000000
101	~754	D	0.000000	0.000000	-0.067672	0.000000	0.000000	0.000000
102	~766	D	0.000000	0.000000	-0.062588	0.000000	0.000000	0.000000
104	~778	D	0.000000	0.000000	-0.054276	0.000000	0.000000	0.000000
105	~801	D	0.000000	0.000000	-0.033664	0.000000	0.000000	0.000000
106	~801	D	0.000000	0.000000	-0.033664	0.000000	0.000000	0.000000
107	~876	D	0.000000	0.000000	-0.003714	0.000000	0.000000	0.000000
108	~934	D	0.000000	0.000000	-0.001349	0.000000	0.000000	0.000000
112	1357	D	0.000000	0.000000	0.001791	0.000000	0.000000	0.000000
113	1357	D	0.000000	0.000000	0.001791	0.000000	0.000000	0.000000
114	2379	D	0.000000	0.000000	-0.001122	0.000000	0.000000	0.000000
115	~1768	D	0.000000	0.000000	-0.001163	0.000000	0.000000	0.000000
116	2378	D	0.000000	0.000000	-0.001654	0.000000	0.000000	0.000000
117	~911	D	0.000000	0.000000	-0.002929	0.000000	0.000000	0.000000
118	~688	D	0.000000	0.000000	-0.059760	0.000000	0.000000	0.000000
119	~699	D	0.000000	0.000000	-0.074805	0.000000	0.000000	0.000000
120	~722	D	0.000000	0.000000	-0.083677	0.000000	0.000000	0.000000
121	~732	D	0.000000	0.000000	-0.084726	0.000000	0.000000	0.000000
123	~755	D	0.000000	0.000000	-0.075216	0.000000	0.000000	0.000000
124	~767	D	0.000000	0.000000	-0.068116	0.000000	0.000000	0.000000
126	~779	D	0.000000	0.000000	-0.057994	0.000000	0.000000	0.000000
127	~802	D	0.000000	0.000000	-0.035217	0.000000	0.000000	0.000000
128	~802	D	0.000000	0.000000	-0.035217	0.000000	0.000000	0.000000
129	~877	D	0.000000	0.000000	-0.003943	0.000000	0.000000	0.000000
130	220	D	0.000000	0.000000	-0.001624	0.000000	0.000000	0.000000
138	~658	D	0.000000	0.000000	-0.001386	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
139	~913	D	0.000000	0.000000	-0.003236	0.000000	0.000000	0.000000
140	~691	D	0.000000	0.000000	-0.068315	0.000000	0.000000	0.000000
141	~947	D	0.000000	0.000000	-0.085957	0.000000	0.000000	0.000000
142	~724	D	0.000000	0.000000	-0.088945	0.000000	0.000000	0.000000
143	~724	D	0.000000	0.000000	-0.088945	0.000000	0.000000	0.000000
144	~929	D	0.000000	0.000000	-0.036952	0.000000	0.000000	0.000000
145	~756	D	0.000000	0.000000	-0.075917	0.000000	0.000000	0.000000
146	~768	D	0.000000	0.000000	-0.068094	0.000000	0.000000	0.000000
148	~948	D	0.000000	0.000000	-0.057972	0.000000	0.000000	0.000000
149	~931	D	0.000000	0.000000	-0.035037	0.000000	0.000000	0.000000
150	~931	D	0.000000	0.000000	-0.035037	0.000000	0.000000	0.000000
151	~878	D	0.000000	0.000000	-0.004000	0.000000	0.000000	0.000000
152	~936	D	0.000000	0.000000	-0.001866	0.000000	0.000000	0.000000
160	~660	D	0.000000	0.000000	-0.000976	0.000000	0.000000	0.000000
161	~920	D	0.000000	0.000000	-0.002762	0.000000	0.000000	0.000000
162	~693	D	0.000000	0.000000	-0.050748	0.000000	0.000000	0.000000
163	~927	D	0.000000	0.000000	-0.058367	0.000000	0.000000	0.000000
164	~716	D	0.000000	0.000000	-0.058388	0.000000	0.000000	0.000000
165	~727	D	0.000000	0.000000	-0.057889	0.000000	0.000000	0.000000
166	~852	D	0.000000	0.000000	-0.028001	0.000000	0.000000	0.000000
167	~846	D	0.000000	0.000000	-0.026290	0.000000	0.000000	0.000000
168	~772	D	0.000000	0.000000	-0.024583	0.000000	0.000000	0.000000
170	~950	D	0.000000	0.000000	-0.023795	0.000000	0.000000	0.000000
171	~946	D	0.000000	0.000000	-0.013162	0.000000	0.000000	0.000000
172	~932	D	0.000000	0.000000	-0.011231	0.000000	0.000000	0.000000
173	~880	D	0.000000	0.000000	-0.001351	0.000000	0.000000	0.000000
174	229	D	0.000000	0.000000	-0.000917	0.000000	0.000000	0.000000
182	2393	D	0.000000	0.000000	-0.001168	0.000000	0.000000	0.000000
183	2512	D	0.000000	0.000000	-0.001877	0.000000	0.000000	0.000000
184	~694	D	0.000000	0.000000	-0.032674	0.000000	0.000000	0.000000
185	~893	D	0.000000	0.000000	-0.035285	0.000000	0.000000	0.000000
186	~843	D	0.000000	0.000000	-0.035266	0.000000	0.000000	0.000000
187	5117	D	0.000000	0.000000	-0.034957	0.000000	0.000000	0.000000
188	~1550	D	0.000000	0.000000	-0.011210	0.000000	0.000000	0.000000
189	3315	D	0.000000	0.000000	-0.007165	0.000000	0.000000	0.000000
190	~1512	D	0.000000	0.000000	-0.011298	0.000000	0.000000	0.000000
192	~785	D	0.000000	0.000000	-0.010068	0.000000	0.000000	0.000000
193	297	D	0.000000	0.000000	-0.002803	0.000000	0.000000	0.000000
194	~808	D	0.000000	0.000000	-0.002380	0.000000	0.000000	0.000000
195	~882	D	0.000000	0.000000	-0.000223	0.000000	0.000000	0.000000
196	2338	D	0.000000	0.000000	0.000323	0.000000	0.000000	0.000000
201	1592	D	0.000000	0.000000	0.001503	0.000000	0.000000	0.000000
202	~1349	D	0.000000	0.000000	-0.001612	0.000000	0.000000	0.000000
203	~1395	D	0.000000	0.000000	-0.003276	0.000000	0.000000	0.000000
204	2405	D	0.000000	0.000000	-0.006106	0.000000	0.000000	0.000000
205	~1915	D	0.000000	0.000000	-0.006324	0.000000	0.000000	0.000000
206	~683	D	0.000000	0.000000	-0.007479	0.000000	0.000000	0.000000
207	2394	D	0.000000	0.000000	-0.006328	0.000000	0.000000	0.000000
208	2409	D	0.000000	0.000000	-0.009745	0.000000	0.000000	0.000000
209	~739	D	0.000000	0.000000	-0.015638	0.000000	0.000000	0.000000
210	~1550	D	0.000000	0.000000	-0.011210	0.000000	0.000000	0.000000
211	~1551	D	0.000000	0.000000	-0.005155	0.000000	0.000000	0.000000
212	~774	D	0.000000	0.000000	-0.005525	0.000000	0.000000	0.000000
214	~1518	D	0.000000	0.000000	-0.006082	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
215	~840	D	0.000000	0.000000	-0.002512	0.000000	0.000000	0.000000
223	~1402	D	0.000000	0.000000	-0.026386	0.000000	0.000000	0.000000
224	~1256	D	0.000000	0.000000	-0.028848	0.000000	0.000000	0.000000
225	~1273	D	0.000000	0.000000	-0.036447	0.000000	0.000000	0.000000
226	~1291	D	0.000000	0.000000	-0.038367	0.000000	0.000000	0.000000
227	~1330	D	0.000000	0.000000	-0.032747	0.000000	0.000000	0.000000
230	5118	D	0.000000	0.000000	-0.018027	0.000000	0.000000	0.000000
231	5118	D	0.000000	0.000000	-0.018027	0.000000	0.000000	0.000000
232	~1547	D	0.000000	0.000000	-0.008913	0.000000	0.000000	0.000000
233	~1548	D	0.000000	0.000000	-0.008115	0.000000	0.000000	0.000000
234	~1514	D	0.000000	0.000000	-0.008270	0.000000	0.000000	0.000000
236	~1549	D	0.000000	0.000000	-0.010257	0.000000	0.000000	0.000000
237	~1522	D	0.000000	0.000000	-0.010298	0.000000	0.000000	0.000000
238	~2078	D	0.000000	0.000000	-0.000467	0.000000	0.000000	0.000000
239	2349	D	0.000000	0.000000	-0.001049	0.000000	0.000000	0.000000
240	~1902	D	0.000000	0.000000	-0.001029	0.000000	0.000000	0.000000
245	~1410	D	0.000000	0.000000	-0.028861	0.000000	0.000000	0.000000
246	~1257	D	0.000000	0.000000	-0.031568	0.000000	0.000000	0.000000
247	~1274	D	0.000000	0.000000	-0.039912	0.000000	0.000000	0.000000
248	~1292	D	0.000000	0.000000	-0.042081	0.000000	0.000000	0.000000
249	~1331	D	0.000000	0.000000	-0.036367	0.000000	0.000000	0.000000
250	~1566	D	0.000000	0.000000	-0.013297	0.000000	0.000000	0.000000
252	5118	D	0.000000	0.000000	-0.018027	0.000000	0.000000	0.000000
253	~1713	D	0.000000	0.000000	-0.028227	0.000000	0.000000	0.000000
254	~1706	D	0.000000	0.000000	-0.030450	0.000000	0.000000	0.000000
255	~1919	D	0.000000	0.000000	-0.000934	0.000000	0.000000	0.000000
256	~1919	D	0.000000	0.000000	-0.000934	0.000000	0.000000	0.000000
257	2390	D	0.000000	0.000000	-0.000429	0.000000	0.000000	0.000000
258	~1521	D	0.000000	0.000000	-0.000775	0.000000	0.000000	0.000000
259	407	D	0.000000	0.000000	-0.000609	0.000000	0.000000	0.000000
260	~2078	D	0.000000	0.000000	-0.000467	0.000000	0.000000	0.000000
261	~1901	D	0.000000	0.000000	0.000424	0.000000	0.000000	0.000000
262	2351	D	0.000000	0.000000	0.000444	0.000000	0.000000	0.000000
267	1597	D	0.000000	0.000000	-0.001795	0.000000	0.000000	0.000000
268	~1362	D	0.000000	0.000000	-0.002190	0.000000	0.000000	0.000000
269	~1397	D	0.000000	0.000000	0.005070	0.000000	0.000000	0.000000
270	674	D	0.000000	0.000000	-0.014556	0.000000	0.000000	0.000000
271	674	D	0.000000	0.000000	-0.014556	0.000000	0.000000	0.000000
272	~1570	D	0.000000	0.000000	-0.043736	0.000000	0.000000	0.000000
274	~1585	D	0.000000	0.000000	-0.009078	0.000000	0.000000	0.000000
275	~1713	D	0.000000	0.000000	-0.028227	0.000000	0.000000	0.000000
276	~1706	D	0.000000	0.000000	-0.030450	0.000000	0.000000	0.000000
277	~1899	D	0.000000	0.000000	-0.000652	0.000000	0.000000	0.000000
278	441	D	0.000000	0.000000	-0.000627	0.000000	0.000000	0.000000
279	~2112	D	0.000000	0.000000	-0.000504	0.000000	0.000000	0.000000
280	2369	D	0.000000	0.000000	-0.000520	0.000000	0.000000	0.000000
289	~1350	D	0.000000	0.000000	-0.000534	0.000000	0.000000	0.000000
290	2533	D	0.000000	0.000000	-0.000746	0.000000	0.000000	0.000000
291	~1354	D	0.000000	0.000000	-0.004765	0.000000	0.000000	0.000000
292	~1554	D	0.000000	0.000000	-0.028457	0.000000	0.000000	0.000000
293	~1554	D	0.000000	0.000000	-0.028457	0.000000	0.000000	0.000000
294	~1602	D	0.000000	0.000000	-0.054912	0.000000	0.000000	0.000000
296	2471	D	0.000000	0.000000	-0.007439	0.000000	0.000000	0.000000
297	~1712	D	0.000000	0.000000	-0.015730	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
298	~1707	D	0.000000	0.000000	-0.016816	0.000000	0.000000	0.000000
301	~2111	D	0.000000	0.000000	-0.000490	0.000000	0.000000	0.000000
302	~2111	D	0.000000	0.000000	-0.000490	0.000000	0.000000	0.000000
313	~1406	D	0.000000	0.000000	-0.044339	0.000000	0.000000	0.000000
314	~1412	D	0.000000	0.000000	-0.063587	0.000000	0.000000	0.000000
315	~1412	D	0.000000	0.000000	-0.063587	0.000000	0.000000	0.000000
316	~1564	D	0.000000	0.000000	-0.077647	0.000000	0.000000	0.000000
317	~1791	D	0.000000	0.000000	-0.037162	0.000000	0.000000	0.000000
318	976	D	0.000000	0.000000	-0.016713	0.000000	0.000000	0.000000
319	976	D	0.000000	0.000000	-0.016713	0.000000	0.000000	0.000000
320	~1707	D	0.000000	0.000000	-0.016816	0.000000	0.000000	0.000000
321	~2169	D	0.000000	0.000000	-0.000727	0.000000	0.000000	0.000000
322	488	D	0.000000	0.000000	-0.000547	0.000000	0.000000	0.000000
323	2371	D	0.000000	0.000000	-0.000400	0.000000	0.000000	0.000000
324	~1890	D	0.000000	0.000000	-0.000383	0.000000	0.000000	0.000000
335	~1408	D	0.000000	0.000000	-0.058150	0.000000	0.000000	0.000000
336	~1418	D	0.000000	0.000000	-0.070280	0.000000	0.000000	0.000000
337	~1418	D	0.000000	0.000000	-0.070280	0.000000	0.000000	0.000000
338	~1831	D	0.000000	0.000000	-0.086926	0.000000	0.000000	0.000000
339	~1792	D	0.000000	0.000000	-0.033791	0.000000	0.000000	0.000000
340	~1809	D	0.000000	0.000000	-0.016471	0.000000	0.000000	0.000000
341	~1807	D	0.000000	0.000000	-0.012384	0.000000	0.000000	0.000000
342	496	D	0.000000	0.000000	-0.001188	0.000000	0.000000	0.000000
343	~2169	D	0.000000	0.000000	-0.000727	0.000000	0.000000	0.000000
344	488	D	0.000000	0.000000	-0.000547	0.000000	0.000000	0.000000
345	2391	D	0.000000	0.000000	-0.000562	0.000000	0.000000	0.000000
346	~1883	D	0.000000	0.000000	-0.000520	0.000000	0.000000	0.000000
357	~1409	D	0.000000	0.000000	-0.059674	0.000000	0.000000	0.000000
358	~1413	D	0.000000	0.000000	-0.070400	0.000000	0.000000	0.000000
359	~1413	D	0.000000	0.000000	-0.070400	0.000000	0.000000	0.000000
360	~1840	D	0.000000	0.000000	-0.087466	0.000000	0.000000	0.000000
361	~1805	D	0.000000	0.000000	-0.030786	0.000000	0.000000	0.000000
362	507	D	0.000000	0.000000	-0.010794	0.000000	0.000000	0.000000
363	~1815	D	0.000000	0.000000	-0.006283	0.000000	0.000000	0.000000
364	~1841	D	0.000000	0.000000	-0.000818	0.000000	0.000000	0.000000
365	468	D	0.000000	0.000000	-0.001051	0.000000	0.000000	0.000000
366	468	D	0.000000	0.000000	-0.001051	0.000000	0.000000	0.000000
367	469	D	0.000000	0.000000	-0.000942	0.000000	0.000000	0.000000
368	~1845	D	0.000000	0.000000	-0.001097	0.000000	0.000000	0.000000
369	~1845	D	0.000000	0.000000	-0.001097	0.000000	0.000000	0.000000
370	10389	D	0.000000	0.000000	0.001877	0.000000	0.000000	0.000000
371	10389	D	0.000000	0.000000	0.001877	0.000000	0.000000	0.000000
372	~1614	D	0.000000	0.000000	0.003045	0.000000	0.000000	0.000000
374	2434	D	0.000000	0.000000	0.002216	0.000000	0.000000	0.000000
379	~1409	D	0.000000	0.000000	-0.059674	0.000000	0.000000	0.000000
380	766	D	0.000000	0.000000	-0.068101	0.000000	0.000000	0.000000
381	766	D	0.000000	0.000000	-0.068101	0.000000	0.000000	0.000000
382	~1840	D	0.000000	0.000000	-0.087466	0.000000	0.000000	0.000000
383	~1795	D	0.000000	0.000000	-0.035004	0.000000	0.000000	0.000000
384	975	D	0.000000	0.000000	-0.016078	0.000000	0.000000	0.000000
385	975	D	0.000000	0.000000	-0.016078	0.000000	0.000000	0.000000
386	~1440	D	0.000000	0.000000	-0.012782	0.000000	0.000000	0.000000
387	~1495	D	0.000000	0.000000	-0.028880	0.000000	0.000000	0.000000
388	~1495	D	0.000000	0.000000	-0.028880	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
389	469	D	0.000000	0.000000	-0.000942	0.000000	0.000000	0.000000
390	~1451	D	0.000000	0.000000	-0.041162	0.000000	0.000000	0.000000
391	~1496	D	0.000000	0.000000	-0.046960	0.000000	0.000000	0.000000
392	~1496	D	0.000000	0.000000	-0.046960	0.000000	0.000000	0.000000
393	~1461	D	0.000000	0.000000	-0.046501	0.000000	0.000000	0.000000
394	~1617	D	0.000000	0.000000	-0.058405	0.000000	0.000000	0.000000
395	~1662	D	0.000000	0.000000	-0.056720	0.000000	0.000000	0.000000
396	~1628	D	0.000000	0.000000	-0.050160	0.000000	0.000000	0.000000
401	1590	D	0.000000	0.000000	-0.001102	0.000000	0.000000	0.000000
402	~1386	D	0.000000	0.000000	-0.002630	0.000000	0.000000	0.000000
403	630	D	0.000000	0.000000	-0.002532	0.000000	0.000000	0.000000
404	~1722	D	0.000000	0.000000	-0.001722	0.000000	0.000000	0.000000
405	622	D	0.000000	0.000000	-0.001019	0.000000	0.000000	0.000000
406	~1428	D	0.000000	0.000000	-0.000847	0.000000	0.000000	0.000000
407	~1435	D	0.000000	0.000000	-0.001384	0.000000	0.000000	0.000000
408	~1441	D	0.000000	0.000000	-0.011391	0.000000	0.000000	0.000000
409	~1493	D	0.000000	0.000000	-0.025248	0.000000	0.000000	0.000000
410	~1493	D	0.000000	0.000000	-0.025248	0.000000	0.000000	0.000000
412	~1452	D	0.000000	0.000000	-0.035054	0.000000	0.000000	0.000000
413	~1494	D	0.000000	0.000000	-0.040126	0.000000	0.000000	0.000000
414	~1462	D	0.000000	0.000000	-0.040721	0.000000	0.000000	0.000000
415	~1462	D	0.000000	0.000000	-0.040721	0.000000	0.000000	0.000000
416	~1618	D	0.000000	0.000000	-0.036068	0.000000	0.000000	0.000000
417	~1634	D	0.000000	0.000000	-0.035598	0.000000	0.000000	0.000000
418	~1629	D	0.000000	0.000000	-0.034288	0.000000	0.000000	0.000000
1	2447	L	0.000000	0.000000	0.000736	0.000000	0.000000	0.000000
2	727	L	0.000000	0.000000	-0.000743	0.000000	0.000000	0.000000
3	~629	L	0.000000	0.000000	-0.000790	0.000000	0.000000	0.000000
4	~492	L	0.000000	0.000000	0.000893	0.000000	0.000000	0.000000
5	~500	L	0.000000	0.000000	0.000966	0.000000	0.000000	0.000000
6	~516	L	0.000000	0.000000	0.001068	0.000000	0.000000	0.000000
7	~601	L	0.000000	0.000000	0.001139	0.000000	0.000000	0.000000
8	~547	L	0.000000	0.000000	0.001316	0.000000	0.000000	0.000000
9	~599	L	0.000000	0.000000	0.001463	0.000000	0.000000	0.000000
10	~563	L	0.000000	0.000000	0.001188	0.000000	0.000000	0.000000
11	~571	L	0.000000	0.000000	0.000966	0.000000	0.000000	0.000000
12	~1213	L	0.000000	0.000000	-0.000079	0.000000	0.000000	0.000000
13	~1098	L	0.000000	0.000000	-0.001074	0.000000	0.000000	0.000000
14	~1108	L	0.000000	0.000000	-0.003190	0.000000	0.000000	0.000000
16	~1128	L	0.000000	0.000000	-0.006428	0.000000	0.000000	0.000000
17	~1137	L	0.000000	0.000000	-0.006592	0.000000	0.000000	0.000000
18	~1145	L	0.000000	0.000000	-0.006645	0.000000	0.000000	0.000000
19	~1154	L	0.000000	0.000000	-0.005958	0.000000	0.000000	0.000000
20	~1164	L	0.000000	0.000000	-0.003903	0.000000	0.000000	0.000000
21	~1217	L	0.000000	0.000000	-0.000453	0.000000	0.000000	0.000000
22	~1182	L	0.000000	0.000000	0.000276	0.000000	0.000000	0.000000
23	~1000	L	0.000000	0.000000	-0.017702	0.000000	0.000000	0.000000
24	~479	L	0.000000	0.000000	-0.012877	0.000000	0.000000	0.000000
25	~486	L	0.000000	0.000000	-0.021171	0.000000	0.000000	0.000000
26	~494	L	0.000000	0.000000	-0.029148	0.000000	0.000000	0.000000
27	~502	L	0.000000	0.000000	-0.036478	0.000000	0.000000	0.000000
28	~518	L	0.000000	0.000000	-0.046732	0.000000	0.000000	0.000000
29	~518	L	0.000000	0.000000	-0.046732	0.000000	0.000000	0.000000
30	~525	L	0.000000	0.000000	-0.047528	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
31	~557	L	0.000000	0.000000	-0.031878	0.000000	0.000000	0.000000
32	~651	L	0.000000	0.000000	-0.026252	0.000000	0.000000	0.000000
33	~648	L	0.000000	0.000000	-0.020970	0.000000	0.000000	0.000000
34	60	L	0.000000	0.000000	-0.001327	0.000000	0.000000	0.000000
35	~1099	L	0.000000	0.000000	-0.001132	0.000000	0.000000	0.000000
36	~1109	L	0.000000	0.000000	-0.004656	0.000000	0.000000	0.000000
38	~1129	L	0.000000	0.000000	-0.010006	0.000000	0.000000	0.000000
39	~1138	L	0.000000	0.000000	-0.010285	0.000000	0.000000	0.000000
40	~1146	L	0.000000	0.000000	-0.010389	0.000000	0.000000	0.000000
41	~1155	L	0.000000	0.000000	-0.009393	0.000000	0.000000	0.000000
42	~1165	L	0.000000	0.000000	-0.006297	0.000000	0.000000	0.000000
43	~1217	L	0.000000	0.000000	-0.000453	0.000000	0.000000	0.000000
44	~1183	L	0.000000	0.000000	0.000631	0.000000	0.000000	0.000000
45	~1002	L	0.000000	0.000000	-0.031593	0.000000	0.000000	0.000000
46	~1040	L	0.000000	0.000000	-0.015764	0.000000	0.000000	0.000000
47	~487	L	0.000000	0.000000	-0.025536	0.000000	0.000000	0.000000
48	~495	L	0.000000	0.000000	-0.035177	0.000000	0.000000	0.000000
49	~503	L	0.000000	0.000000	-0.043966	0.000000	0.000000	0.000000
50	~519	L	0.000000	0.000000	-0.055733	0.000000	0.000000	0.000000
51	~519	L	0.000000	0.000000	-0.055733	0.000000	0.000000	0.000000
52	~526	L	0.000000	0.000000	-0.056273	0.000000	0.000000	0.000000
53	~558	L	0.000000	0.000000	-0.037667	0.000000	0.000000	0.000000
54	~566	L	0.000000	0.000000	-0.030135	0.000000	0.000000	0.000000
55	~574	L	0.000000	0.000000	-0.024393	0.000000	0.000000	0.000000
56	~590	L	0.000000	0.000000	-0.004639	0.000000	0.000000	0.000000
57	~1102	L	0.000000	0.000000	-0.001685	0.000000	0.000000	0.000000
58	~1222	L	0.000000	0.000000	-0.004175	0.000000	0.000000	0.000000
60	~1131	L	0.000000	0.000000	-0.008492	0.000000	0.000000	0.000000
61	~1140	L	0.000000	0.000000	-0.008731	0.000000	0.000000	0.000000
62	~1148	L	0.000000	0.000000	-0.008855	0.000000	0.000000	0.000000
63	~1157	L	0.000000	0.000000	-0.008285	0.000000	0.000000	0.000000
64	~1167	L	0.000000	0.000000	-0.005766	0.000000	0.000000	0.000000
65	~1197	L	0.000000	0.000000	-0.000127	0.000000	0.000000	0.000000
66	~1185	L	0.000000	0.000000	0.000568	0.000000	0.000000	0.000000
67	~1003	L	0.000000	0.000000	-0.028721	0.000000	0.000000	0.000000
68	~1041	L	0.000000	0.000000	-0.013646	0.000000	0.000000	0.000000
69	~489	L	0.000000	0.000000	-0.021069	0.000000	0.000000	0.000000
70	~497	L	0.000000	0.000000	-0.029386	0.000000	0.000000	0.000000
71	~505	L	0.000000	0.000000	-0.036174	0.000000	0.000000	0.000000
72	~521	L	0.000000	0.000000	-0.041193	0.000000	0.000000	0.000000
73	~521	L	0.000000	0.000000	-0.041193	0.000000	0.000000	0.000000
74	~646	L	0.000000	0.000000	-0.039192	0.000000	0.000000	0.000000
75	~647	L	0.000000	0.000000	-0.025235	0.000000	0.000000	0.000000
76	~568	L	0.000000	0.000000	-0.020425	0.000000	0.000000	0.000000
77	~576	L	0.000000	0.000000	-0.017149	0.000000	0.000000	0.000000
78	~1094	L	0.000000	0.000000	-0.004698	0.000000	0.000000	0.000000
79	~1103	L	0.000000	0.000000	-0.002065	0.000000	0.000000	0.000000
80	~1232	L	0.000000	0.000000	-0.001988	0.000000	0.000000	0.000000
82	~1228	L	0.000000	0.000000	-0.001633	0.000000	0.000000	0.000000
83	194	L	0.000000	0.000000	-0.000676	0.000000	0.000000	0.000000
84	194	L	0.000000	0.000000	-0.000676	0.000000	0.000000	0.000000
85	~1229	L	0.000000	0.000000	-0.000803	0.000000	0.000000	0.000000
86	~1227	L	0.000000	0.000000	-0.000718	0.000000	0.000000	0.000000
87	128	L	0.000000	0.000000	-0.000062	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
88	2316	L	0.000000	0.000000	0.000114	0.000000	0.000000	0.000000
89	~1086	L	0.000000	0.000000	-0.004744	0.000000	0.000000	0.000000
90	1287	L	0.000000	0.000000	-0.002691	0.000000	0.000000	0.000000
91	~1743	L	0.000000	0.000000	-0.010083	0.000000	0.000000	0.000000
92	~1779	L	0.000000	0.000000	-0.014317	0.000000	0.000000	0.000000
93	~1753	L	0.000000	0.000000	-0.015589	0.000000	0.000000	0.000000
94	~1753	L	0.000000	0.000000	-0.015589	0.000000	0.000000	0.000000
95	5116	L	0.000000	0.000000	-0.008799	0.000000	0.000000	0.000000
96	~939	L	0.000000	0.000000	-0.017790	0.000000	0.000000	0.000000
97	~698	L	0.000000	0.000000	-0.017988	0.000000	0.000000	0.000000
98	~721	L	0.000000	0.000000	-0.020834	0.000000	0.000000	0.000000
99	~731	L	0.000000	0.000000	-0.021531	0.000000	0.000000	0.000000
100	~941	L	0.000000	0.000000	-0.021020	0.000000	0.000000	0.000000
101	~754	L	0.000000	0.000000	-0.019909	0.000000	0.000000	0.000000
102	~766	L	0.000000	0.000000	-0.018369	0.000000	0.000000	0.000000
104	~778	L	0.000000	0.000000	-0.015845	0.000000	0.000000	0.000000
105	~801	L	0.000000	0.000000	-0.009701	0.000000	0.000000	0.000000
106	~801	L	0.000000	0.000000	-0.009701	0.000000	0.000000	0.000000
107	~876	L	0.000000	0.000000	-0.000910	0.000000	0.000000	0.000000
108	~824	L	0.000000	0.000000	0.000464	0.000000	0.000000	0.000000
112	2373	L	0.000000	0.000000	-0.000033	0.000000	0.000000	0.000000
113	~1778	L	0.000000	0.000000	-0.000699	0.000000	0.000000	0.000000
114	~1767	L	0.000000	0.000000	-0.000607	0.000000	0.000000	0.000000
115	2379	L	0.000000	0.000000	-0.000575	0.000000	0.000000	0.000000
116	2378	L	0.000000	0.000000	-0.000339	0.000000	0.000000	0.000000
117	~911	L	0.000000	0.000000	-0.000660	0.000000	0.000000	0.000000
118	~688	L	0.000000	0.000000	-0.017110	0.000000	0.000000	0.000000
119	~699	L	0.000000	0.000000	-0.021558	0.000000	0.000000	0.000000
120	~722	L	0.000000	0.000000	-0.024260	0.000000	0.000000	0.000000
121	~732	L	0.000000	0.000000	-0.024667	0.000000	0.000000	0.000000
123	~755	L	0.000000	0.000000	-0.022003	0.000000	0.000000	0.000000
124	~767	L	0.000000	0.000000	-0.019888	0.000000	0.000000	0.000000
126	~779	L	0.000000	0.000000	-0.016857	0.000000	0.000000	0.000000
127	~802	L	0.000000	0.000000	-0.010116	0.000000	0.000000	0.000000
128	~802	L	0.000000	0.000000	-0.010116	0.000000	0.000000	0.000000
129	~877	L	0.000000	0.000000	-0.000965	0.000000	0.000000	0.000000
130	2332	L	0.000000	0.000000	0.000402	0.000000	0.000000	0.000000
138	2507	L	0.000000	0.000000	0.000344	0.000000	0.000000	0.000000
139	2509	L	0.000000	0.000000	-0.000592	0.000000	0.000000	0.000000
140	~691	L	0.000000	0.000000	-0.019489	0.000000	0.000000	0.000000
141	~947	L	0.000000	0.000000	-0.024755	0.000000	0.000000	0.000000
142	~724	L	0.000000	0.000000	-0.025709	0.000000	0.000000	0.000000
143	~724	L	0.000000	0.000000	-0.025709	0.000000	0.000000	0.000000
144	~929	L	0.000000	0.000000	-0.010689	0.000000	0.000000	0.000000
145	~756	L	0.000000	0.000000	-0.022159	0.000000	0.000000	0.000000
146	~768	L	0.000000	0.000000	-0.019838	0.000000	0.000000	0.000000
148	~948	L	0.000000	0.000000	-0.016833	0.000000	0.000000	0.000000
149	~931	L	0.000000	0.000000	-0.010055	0.000000	0.000000	0.000000
150	~931	L	0.000000	0.000000	-0.010055	0.000000	0.000000	0.000000
151	2330	L	0.000000	0.000000	-0.000979	0.000000	0.000000	0.000000
152	~833	L	0.000000	0.000000	0.000388	0.000000	0.000000	0.000000
160	2510	L	0.000000	0.000000	0.000399	0.000000	0.000000	0.000000
161	~920	L	0.000000	0.000000	-0.000361	0.000000	0.000000	0.000000
162	~693	L	0.000000	0.000000	-0.014310	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
163	~927	L	0.000000	0.000000	-0.016695	0.000000	0.000000	0.000000
164	~716	L	0.000000	0.000000	-0.016708	0.000000	0.000000	0.000000
165	~727	L	0.000000	0.000000	-0.016620	0.000000	0.000000	0.000000
166	~852	L	0.000000	0.000000	-0.008091	0.000000	0.000000	0.000000
167	~846	L	0.000000	0.000000	-0.007593	0.000000	0.000000	0.000000
168	~772	L	0.000000	0.000000	-0.007061	0.000000	0.000000	0.000000
170	~950	L	0.000000	0.000000	-0.006782	0.000000	0.000000	0.000000
171	~946	L	0.000000	0.000000	-0.003659	0.000000	0.000000	0.000000
172	~932	L	0.000000	0.000000	-0.003095	0.000000	0.000000	0.000000
173	~880	L	0.000000	0.000000	-0.000238	0.000000	0.000000	0.000000
174	~1905	L	0.000000	0.000000	0.000179	0.000000	0.000000	0.000000
182	~1913	L	0.000000	0.000000	0.000345	0.000000	0.000000	0.000000
183	~916	L	0.000000	0.000000	-0.000374	0.000000	0.000000	0.000000
184	~694	L	0.000000	0.000000	-0.009031	0.000000	0.000000	0.000000
185	~843	L	0.000000	0.000000	-0.009979	0.000000	0.000000	0.000000
186	~717	L	0.000000	0.000000	-0.009980	0.000000	0.000000	0.000000
187	5117	L	0.000000	0.000000	-0.009953	0.000000	0.000000	0.000000
188	~1550	L	0.000000	0.000000	-0.003189	0.000000	0.000000	0.000000
189	3315	L	0.000000	0.000000	-0.002060	0.000000	0.000000	0.000000
190	~1512	L	0.000000	0.000000	-0.003226	0.000000	0.000000	0.000000
192	~785	L	0.000000	0.000000	-0.002818	0.000000	0.000000	0.000000
193	297	L	0.000000	0.000000	-0.000676	0.000000	0.000000	0.000000
194	~808	L	0.000000	0.000000	-0.000552	0.000000	0.000000	0.000000
195	~958	L	0.000000	0.000000	0.000149	0.000000	0.000000	0.000000
196	2338	L	0.000000	0.000000	0.000207	0.000000	0.000000	0.000000
201	1592	L	0.000000	0.000000	0.000573	0.000000	0.000000	0.000000
202	~1349	L	0.000000	0.000000	-0.000448	0.000000	0.000000	0.000000
203	~1395	L	0.000000	0.000000	-0.001421	0.000000	0.000000	0.000000
204	2405	L	0.000000	0.000000	-0.003582	0.000000	0.000000	0.000000
205	~1915	L	0.000000	0.000000	-0.003715	0.000000	0.000000	0.000000
206	~683	L	0.000000	0.000000	-0.001662	0.000000	0.000000	0.000000
207	2394	L	0.000000	0.000000	-0.001541	0.000000	0.000000	0.000000
208	2409	L	0.000000	0.000000	-0.002505	0.000000	0.000000	0.000000
209	~739	L	0.000000	0.000000	-0.004294	0.000000	0.000000	0.000000
210	~1550	L	0.000000	0.000000	-0.003189	0.000000	0.000000	0.000000
211	~1551	L	0.000000	0.000000	-0.001510	0.000000	0.000000	0.000000
212	~774	L	0.000000	0.000000	-0.001612	0.000000	0.000000	0.000000
214	~1518	L	0.000000	0.000000	-0.001700	0.000000	0.000000	0.000000
215	~840	L	0.000000	0.000000	-0.000611	0.000000	0.000000	0.000000
223	~1402	L	0.000000	0.000000	-0.005951	0.000000	0.000000	0.000000
224	~1256	L	0.000000	0.000000	-0.007611	0.000000	0.000000	0.000000
225	~1273	L	0.000000	0.000000	-0.013430	0.000000	0.000000	0.000000
226	~1330	L	0.000000	0.000000	-0.021729	0.000000	0.000000	0.000000
227	~1330	L	0.000000	0.000000	-0.021729	0.000000	0.000000	0.000000
230	5118	L	0.000000	0.000000	-0.002897	0.000000	0.000000	0.000000
231	5118	L	0.000000	0.000000	-0.002897	0.000000	0.000000	0.000000
232	~1547	L	0.000000	0.000000	-0.002536	0.000000	0.000000	0.000000
233	~1548	L	0.000000	0.000000	-0.002481	0.000000	0.000000	0.000000
234	~1514	L	0.000000	0.000000	-0.002517	0.000000	0.000000	0.000000
236	~1549	L	0.000000	0.000000	-0.002897	0.000000	0.000000	0.000000
237	~1522	L	0.000000	0.000000	-0.002901	0.000000	0.000000	0.000000
238	~2078	L	0.000000	0.000000	-6.810E-006	0.000000	0.000000	0.000000
239	2349	L	0.000000	0.000000	-0.000204	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
240	~1902	L	0.000000	0.000000	-0.000200	0.000000	0.000000	0.000000
245	~1410	L	0.000000	0.000000	-0.006524	0.000000	0.000000	0.000000
246	~1257	L	0.000000	0.000000	-0.008372	0.000000	0.000000	0.000000
247	~1274	L	0.000000	0.000000	-0.014848	0.000000	0.000000	0.000000
248	~1331	L	0.000000	0.000000	-0.024265	0.000000	0.000000	0.000000
249	~1331	L	0.000000	0.000000	-0.024265	0.000000	0.000000	0.000000
250	~1557	L	0.000000	0.000000	-0.004255	0.000000	0.000000	0.000000
252	5118	L	0.000000	0.000000	-0.002897	0.000000	0.000000	0.000000
253	~1713	L	0.000000	0.000000	-0.004303	0.000000	0.000000	0.000000
254	~1706	L	0.000000	0.000000	-0.004623	0.000000	0.000000	0.000000
255	~1918	L	0.000000	0.000000	0.000206	0.000000	0.000000	0.000000
256	~1918	L	0.000000	0.000000	0.000206	0.000000	0.000000	0.000000
257	2390	L	0.000000	0.000000	0.000011	0.000000	0.000000	0.000000
258	~1912	L	0.000000	0.000000	0.000227	0.000000	0.000000	0.000000
259	2347	L	0.000000	0.000000	0.000231	0.000000	0.000000	0.000000
260	~2078	L	0.000000	0.000000	-6.810E-006	0.000000	0.000000	0.000000
261	~1901	L	0.000000	0.000000	0.000247	0.000000	0.000000	0.000000
262	2351	L	0.000000	0.000000	0.000250	0.000000	0.000000	0.000000
267	~1245	L	0.000000	0.000000	0.000549	0.000000	0.000000	0.000000
268	~1362	L	0.000000	0.000000	-0.000592	0.000000	0.000000	0.000000
269	~1396	L	0.000000	0.000000	-0.002054	0.000000	0.000000	0.000000
270	667	L	0.000000	0.000000	-0.003479	0.000000	0.000000	0.000000
271	~1591	L	0.000000	0.000000	-0.003562	0.000000	0.000000	0.000000
272	~1570	L	0.000000	0.000000	-0.011480	0.000000	0.000000	0.000000
274	~1585	L	0.000000	0.000000	-0.002111	0.000000	0.000000	0.000000
275	~1713	L	0.000000	0.000000	-0.004303	0.000000	0.000000	0.000000
276	~1706	L	0.000000	0.000000	-0.004623	0.000000	0.000000	0.000000
277	~1899	L	0.000000	0.000000	0.000048	0.000000	0.000000	0.000000
278	441	L	0.000000	0.000000	0.000043	0.000000	0.000000	0.000000
279	~1887	L	0.000000	0.000000	-0.000013	0.000000	0.000000	0.000000
280	2370	L	0.000000	0.000000	-0.000026	0.000000	0.000000	0.000000
289	~1350	L	0.000000	0.000000	-0.000066	0.000000	0.000000	0.000000
290	2533	L	0.000000	0.000000	-0.000100	0.000000	0.000000	0.000000
291	~1354	L	0.000000	0.000000	-0.000724	0.000000	0.000000	0.000000
292	~1554	L	0.000000	0.000000	-0.004148	0.000000	0.000000	0.000000
293	~1554	L	0.000000	0.000000	-0.004148	0.000000	0.000000	0.000000
294	~1602	L	0.000000	0.000000	-0.014116	0.000000	0.000000	0.000000
296	2471	L	0.000000	0.000000	-0.001918	0.000000	0.000000	0.000000
297	~1712	L	0.000000	0.000000	-0.002168	0.000000	0.000000	0.000000
298	~1707	L	0.000000	0.000000	-0.002313	0.000000	0.000000	0.000000
301	~2111	L	0.000000	0.000000	4.539E-006	0.000000	0.000000	0.000000
302	~2111	L	0.000000	0.000000	4.539E-006	0.000000	0.000000	0.000000
313	~1406	L	0.000000	0.000000	-0.007220	0.000000	0.000000	0.000000
314	~1412	L	0.000000	0.000000	-0.009359	0.000000	0.000000	0.000000
315	~1412	L	0.000000	0.000000	-0.009359	0.000000	0.000000	0.000000
316	982	L	0.000000	0.000000	-0.019000	0.000000	0.000000	0.000000
317	~1791	L	0.000000	0.000000	-0.010056	0.000000	0.000000	0.000000
318	976	L	0.000000	0.000000	-0.004605	0.000000	0.000000	0.000000
319	976	L	0.000000	0.000000	-0.004605	0.000000	0.000000	0.000000
320	~1707	L	0.000000	0.000000	-0.002313	0.000000	0.000000	0.000000
321	~2169	L	0.000000	0.000000	-0.000017	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
322	488	L	0.000000	0.000000	-3.657E-006	0.000000	0.000000	0.000000
323	~1884	L	0.000000	0.000000	0.000087	0.000000	0.000000	0.000000
324	2399	L	0.000000	0.000000	0.000091	0.000000	0.000000	0.000000
335	~1408	L	0.000000	0.000000	-0.009439	0.000000	0.000000	0.000000
336	~1418	L	0.000000	0.000000	-0.010226	0.000000	0.000000	0.000000
337	~1418	L	0.000000	0.000000	-0.010226	0.000000	0.000000	0.000000
338	~1831	L	0.000000	0.000000	-0.021482	0.000000	0.000000	0.000000
339	~1792	L	0.000000	0.000000	-0.008964	0.000000	0.000000	0.000000
340	~1809	L	0.000000	0.000000	-0.004479	0.000000	0.000000	0.000000
341	~1807	L	0.000000	0.000000	-0.003396	0.000000	0.000000	0.000000
342	~1860	L	0.000000	0.000000	0.000176	0.000000	0.000000	0.000000
343	~2169	L	0.000000	0.000000	-0.000017	0.000000	0.000000	0.000000
344	488	L	0.000000	0.000000	-3.657E-006	0.000000	0.000000	0.000000
345	2391	L	0.000000	0.000000	-0.000049	0.000000	0.000000	0.000000
346	~1883	L	0.000000	0.000000	-0.000044	0.000000	0.000000	0.000000
357	~1409	L	0.000000	0.000000	-0.009650	0.000000	0.000000	0.000000
358	~1413	L	0.000000	0.000000	-0.010210	0.000000	0.000000	0.000000
359	~1413	L	0.000000	0.000000	-0.010210	0.000000	0.000000	0.000000
360	~1831	L	0.000000	0.000000	-0.021482	0.000000	0.000000	0.000000
361	~1805	L	0.000000	0.000000	-0.007998	0.000000	0.000000	0.000000
362	507	L	0.000000	0.000000	-0.002843	0.000000	0.000000	0.000000
363	~1815	L	0.000000	0.000000	-0.001672	0.000000	0.000000	0.000000
364	~1841	L	0.000000	0.000000	-0.000087	0.000000	0.000000	0.000000
365	468	L	0.000000	0.000000	-0.000141	0.000000	0.000000	0.000000
366	468	L	0.000000	0.000000	-0.000141	0.000000	0.000000	0.000000
367	469	L	0.000000	0.000000	-0.000111	0.000000	0.000000	0.000000
368	~1845	L	0.000000	0.000000	-0.000148	0.000000	0.000000	0.000000
369	~1845	L	0.000000	0.000000	-0.000148	0.000000	0.000000	0.000000
370	10389	L	0.000000	0.000000	0.000508	0.000000	0.000000	0.000000
371	10389	L	0.000000	0.000000	0.000508	0.000000	0.000000	0.000000
372	~1614	L	0.000000	0.000000	0.000787	0.000000	0.000000	0.000000
374	2434	L	0.000000	0.000000	0.000533	0.000000	0.000000	0.000000
379	~1409	L	0.000000	0.000000	-0.009650	0.000000	0.000000	0.000000
380	~1323	L	0.000000	0.000000	-0.010025	0.000000	0.000000	0.000000
381	766	L	0.000000	0.000000	-0.009833	0.000000	0.000000	0.000000
382	~1840	L	0.000000	0.000000	-0.021431	0.000000	0.000000	0.000000
383	~1795	L	0.000000	0.000000	-0.009025	0.000000	0.000000	0.000000
384	975	L	0.000000	0.000000	-0.004239	0.000000	0.000000	0.000000
385	975	L	0.000000	0.000000	-0.004239	0.000000	0.000000	0.000000
386	~1440	L	0.000000	0.000000	-0.003386	0.000000	0.000000	0.000000
387	~1495	L	0.000000	0.000000	-0.007601	0.000000	0.000000	0.000000
388	~1495	L	0.000000	0.000000	-0.007601	0.000000	0.000000	0.000000
389	469	L	0.000000	0.000000	-0.000111	0.000000	0.000000	0.000000
390	~1451	L	0.000000	0.000000	-0.010558	0.000000	0.000000	0.000000
391	~1457	L	0.000000	0.000000	-0.011371	0.000000	0.000000	0.000000
392	~1496	L	0.000000	0.000000	-0.011298	0.000000	0.000000	0.000000
393	~1461	L	0.000000	0.000000	-0.010496	0.000000	0.000000	0.000000
394	~1617	L	0.000000	0.000000	-0.014150	0.000000	0.000000	0.000000
395	~1662	L	0.000000	0.000000	-0.013372	0.000000	0.000000	0.000000
396	~1628	L	0.000000	0.000000	-0.010843	0.000000	0.000000	0.000000
401	2526	L	0.000000	0.000000	0.000159	0.000000	0.000000	0.000000
402	~1386	L	0.000000	0.000000	-0.000314	0.000000	0.000000	0.000000
403	630	L	0.000000	0.000000	-0.000303	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 1 of 2

Panel	Node	OutputCase	Ux	Uy	Uz	Rx	Ry	Rz
			in	in	in	Radians	Radians	Radians
404	~1722	L	0.000000	0.000000	-0.000223	0.000000	0.000000	0.000000
405	622	L	0.000000	0.000000	-0.000134	0.000000	0.000000	0.000000
406	599	L	0.000000	0.000000	-0.000076	0.000000	0.000000	0.000000
407	~1435	L	0.000000	0.000000	-0.000291	0.000000	0.000000	0.000000
408	~1441	L	0.000000	0.000000	-0.003001	0.000000	0.000000	0.000000
409	~1493	L	0.000000	0.000000	-0.006632	0.000000	0.000000	0.000000
410	~1493	L	0.000000	0.000000	-0.006632	0.000000	0.000000	0.000000
412	~1452	L	0.000000	0.000000	-0.008955	0.000000	0.000000	0.000000
413	~1458	L	0.000000	0.000000	-0.009636	0.000000	0.000000	0.000000
414	~1494	L	0.000000	0.000000	-0.009597	0.000000	0.000000	0.000000
415	~1462	L	0.000000	0.000000	-0.009176	0.000000	0.000000	0.000000
416	~1618	L	0.000000	0.000000	-0.008693	0.000000	0.000000	0.000000
417	~1634	L	0.000000	0.000000	-0.008330	0.000000	0.000000	0.000000
418	~1629	L	0.000000	0.000000	-0.007265	0.000000	0.000000	0.000000

Table 36: Nodal Displacements - Summary, Part 2 of 2

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
1	2447	D	33.882	239.240
2	727	D	41.880	278.360
3	~629	D	41.880	284.266
4	~492	D	37.943	346.812
5	~500	D	37.943	376.649
6	~516	D	37.943	445.760
7	~601	D	37.943	481.844
8	~547	D	37.943	552.129
9	~599	D	37.943	593.238
10	~563	D	37.943	615.779
11	~571	D	37.943	632.720
12	~1213	D	37.943	711.859
13	~1098	D	41.880	744.285
14	~1108	D	41.880	782.617
16	~1128	D	41.880	859.280
17	~1137	D	41.880	871.280
18	~1145	D	41.880	883.280
19	~1154	D	41.880	923.960
20	~1164	D	41.880	964.640
21	~1217	D	41.880	1011.415
22	~1182	D	41.880	1023.226
23	~1000	D	97.052	108.057
24	~479	D	128.014	278.360
25	~486	D	128.014	316.976
26	~494	D	128.014	346.812
27	~502	D	128.014	376.649
28	~518	D	128.014	445.760
29	~518	D	128.014	445.760
30	~525	D	128.014	484.154
31	~557	D	128.014	591.674
32	~651	D	133.920	615.779
33	~648	D	133.920	632.720
34	60	D	133.920	705.954

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
35	~1195	D	133.920	744.285
36	~1109	D	84.947	782.617
38	~1129	D	84.947	859.280
39	~1138	D	84.947	871.280
40	~1146	D	84.947	883.280
41	~1155	D	84.947	923.960
42	~1165	D	84.947	964.640
43	~1217	D	41.880	1011.415
44	~1183	D	84.947	1023.226
45	~1002	D	191.400	108.057
46	~1040	D	191.400	278.360
47	~487	D	170.596	316.976
48	~495	D	170.596	346.812
49	~503	D	170.596	376.649
50	~519	D	170.596	445.760
51	~519	D	170.596	445.760
52	~526	D	170.596	484.154
53	~558	D	170.596	591.674
54	~566	D	170.596	615.779
55	~574	D	170.596	632.720
56	~590	D	213.177	705.954
57	~1102	D	179.794	744.285
58	~1222	D	133.920	782.617
60	~1131	D	139.826	859.280
61	~1140	D	139.826	871.280
62	~1148	D	139.826	883.280
63	~1157	D	139.826	923.960
64	~1167	D	139.826	964.640
65	~1197	D	179.794	1017.320
66	~1185	D	139.826	1023.226
67	~1003	D	238.574	108.057
68	~1041	D	238.574	278.360
69	~489	D	255.758	316.976
70	~497	D	255.758	346.812
71	~505	D	255.758	376.649
72	~521	D	255.758	445.760
73	~521	D	255.758	445.760
74	~646	D	255.758	482.154
75	~647	D	255.758	589.674
76	~568	D	255.758	615.779
77	~576	D	255.758	632.720
78	~1094	D	219.762	705.954
79	~1103	D	219.762	744.285
80	~1232	D	215.760	782.617
82	~1228	D	215.760	820.949
83	194	D	215.760	871.280
84	194	D	215.760	871.280
85	~1229	D	215.760	923.960
86	~1227	D	215.760	964.640
87	128	D	215.760	1017.320
88	128	D	215.760	1017.320
89	~1086	D	340.920	108.057
90	1287	D	340.920	278.360
91	~1743	D	340.920	327.038

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
92	~1779	D	340.920	357.622
93	~1753	D	340.920	386.840
94	~1753	D	340.920	386.840
95	5116	D	340.920	445.760
96	~939	D	442.103	589.674
97	~698	D	442.103	591.674
98	~721	D	442.103	632.720
99	~731	D	442.103	666.384
100	~941	D	442.103	705.954
101	~754	D	442.103	745.240
102	~766	D	442.103	782.378
104	~778	D	442.103	819.515
105	~801	D	442.103	871.280
106	~801	D	442.103	871.280
107	~876	D	442.103	922.543
108	~934	D	442.103	926.480
112	1357	D	450.042	278.360
113	1357	D	450.042	278.360
114	2379	D	446.040	357.622
115	~1768	D	446.040	363.527
116	2378	D	446.040	445.760
117	~911	D	464.974	451.666
118	~688	D	464.974	555.834
119	~699	D	464.974	591.674
120	~722	D	464.974	632.720
121	~732	D	464.974	666.384
123	~755	D	464.974	745.240
124	~767	D	464.974	782.378
126	~779	D	464.974	819.515
127	~802	D	464.974	871.280
128	~802	D	464.974	871.280
129	~877	D	470.880	922.543
130	220	D	470.880	926.480
138	~658	D	505.578	445.760
139	~913	D	548.974	449.697
140	~691	D	505.578	555.834
141	~947	D	505.578	612.440
142	~724	D	493.767	632.720
143	~724	D	493.767	632.720
144	~929	D	548.974	743.240
145	~756	D	476.786	745.240
146	~768	D	476.786	782.378
148	~948	D	470.880	819.515
149	~931	D	470.880	871.280
150	~931	D	470.880	871.280
151	~878	D	493.767	922.543
152	~936	D	505.578	926.480
160	~660	D	560.786	445.760
161	~920	D	560.786	449.697
162	~693	D	560.786	555.834
163	~927	D	560.786	612.440
164	~716	D	560.786	614.440
165	~727	D	560.786	632.720
166	~852	D	560.786	736.220

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
167	~846	D	560.786	743.240
168	~772	D	560.786	782.378
170	~950	D	554.880	819.515
171	~946	D	554.880	862.558
172	~932	D	554.880	871.280
173	~880	D	554.880	922.543
174	229	D	554.880	926.480
182	2393	D	618.000	445.760
183	2512	D	592.826	449.697
184	~694	D	586.920	555.834
185	~893	D	586.920	610.440
186	~843	D	586.920	612.440
187	5117	D	586.920	632.720
188	~1550	D	618.000	705.954
189	3315	D	586.920	743.240
190	~1512	D	586.920	783.695
192	~785	D	586.920	819.515
193	297	D	586.920	862.558
194	~808	D	586.920	871.280
195	~882	D	586.920	920.575
196	2338	D	590.857	932.386
201	1592	D	630.148	334.446
202	~1349	D	638.022	350.194
203	~1395	D	638.022	378.966
204	2405	D	638.022	445.760
205	~1915	D	638.022	449.697
206	~683	D	618.000	519.994
207	2394	D	618.000	612.440
208	2409	D	618.000	632.720
209	~739	D	618.000	666.384
210	~1550	D	618.000	705.954
211	~1551	D	618.000	778.576
212	~774	D	618.000	782.378
214	~1518	D	618.000	825.095
215	~840	D	618.000	862.558
223	~1402	D	730.293	342.320
224	~1256	D	730.293	350.194
225	~1273	D	730.293	378.966
226	~1291	D	730.293	394.714
227	~1330	D	730.293	445.760
230	5118	D	758.040	632.720
231	5118	D	758.040	632.720
232	~1547	D	664.680	705.954
233	~1548	D	664.680	779.758
234	~1514	D	664.680	783.695
236	~1549	D	664.680	862.558
237	~1522	D	664.680	866.495
238	~2078	D	758.040	894.519
239	2349	D	752.134	922.543
240	~1902	D	752.134	926.480
245	~1410	D	772.491	342.320
246	~1257	D	772.491	350.194
247	~1274	D	772.491	378.966
248	~1292	D	772.491	394.714

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
249	~1331	D	772.491	445.760
250	~1566	D	906.960	539.150
252	5118	D	758.040	632.720
253	~1713	D	906.960	705.954
254	~1706	D	906.960	711.859
255	~1919	D	855.351	779.758
256	~1919	D	855.351	779.758
257	2390	D	903.023	812.373
258	~1521	D	758.040	825.095
259	407	D	758.040	862.558
260	~2078	D	758.040	894.519
261	~1901	D	763.946	926.480
262	2351	D	763.946	930.417
267	1597	D	906.960	342.320
268	~1362	D	906.960	350.194
269	~1397	D	979.470	386.840
270	674	D	1009.030	445.760
271	674	D	1009.030	445.760
272	~1570	D	1009.030	539.150
274	~1585	D	980.871	632.720
275	~1713	D	906.960	705.954
276	~1706	D	906.960	711.859
277	~1899	D	906.960	775.821
278	441	D	906.960	779.758
279	~2112	D	1003.649	818.278
280	2369	D	971.906	822.215
289	~1350	D	1021.976	342.320
290	2533	D	1021.976	350.194
291	~1354	D	1021.976	386.840
292	~1554	D	1039.960	445.760
293	~1554	D	1039.960	445.760
294	~1602	D	1041.960	539.150
296	2471	D	1039.960	630.720
297	~1712	D	1039.960	705.954
298	~1707	D	1041.960	711.859
301	~2111	D	1035.391	818.278
302	~2111	D	1035.391	818.278
313	~1406	D	1107.120	386.840
314	~1412	D	1123.569	445.760
315	~1412	D	1123.569	445.760
316	~1564	D	1107.120	492.455
317	~1791	D	1107.078	602.864
318	976	D	1107.120	632.720
319	976	D	1107.120	632.720
320	~1707	D	1041.960	711.859
321	~2169	D	1132.080	745.809
322	488	D	1132.080	779.758
323	2371	D	1067.134	814.341
324	~1890	D	1067.134	818.278
335	~1408	D	1153.380	386.840
336	~1418	D	1164.386	445.760
337	~1418	D	1164.386	445.760
338	~1831	D	1172.400	539.150
339	~1792	D	1138.739	602.864

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
340	~1809	D	1138.739	625.700
341	~1807	D	1138.739	632.720
342	496	D	1132.080	705.954
343	~2169	D	1132.080	745.809
344	488	D	1132.080	779.758
345	2391	D	1136.017	812.373
346	~1883	D	1136.017	818.278
357	~1409	D	1199.640	386.840
358	~1413	D	1175.704	445.760
359	~1413	D	1175.704	445.760
360	~1840	D	1199.640	539.164
361	~1805	D	1172.400	602.864
362	507	D	1179.420	625.700
363	~1815	D	1179.420	632.720
364	~1841	D	1199.640	742.856
365	468	D	1199.640	779.758
366	468	D	1199.640	779.758
367	469	D	1199.640	818.278
368	~1845	D	1199.640	862.599
369	~1845	D	1199.640	862.599
370	10389	D	1195.640	906.920
371	10389	D	1195.640	906.920
372	~1614	D	1195.640	972.387
374	2434	D	1195.703	1037.215
379	~1409	D	1199.640	386.840
380	766	D	1199.640	445.760
381	766	D	1199.640	445.760
382	~1840	D	1199.640	539.164
383	~1795	D	1229.831	602.864
384	975	D	1229.760	632.720
385	975	D	1229.760	632.720
386	~1440	D	1287.240	739.903
387	~1495	D	1287.240	779.758
388	~1495	D	1287.240	779.758
389	469	D	1199.640	818.278
390	~1451	D	1287.240	819.614
391	~1496	D	1287.240	871.280
392	~1496	D	1287.240	871.280
393	~1461	D	1287.240	906.920
394	~1617	D	1314.680	972.387
395	~1662	D	1314.680	1007.120
396	~1628	D	1314.680	1043.120
401	1590	D	1331.160	386.840
402	~1386	D	1331.160	439.855
403	630	D	1331.160	445.760
404	~1722	D	1331.160	492.462
405	622	D	1331.160	607.867
406	~1428	D	1365.380	632.720
407	~1435	D	1331.160	705.954
408	~1441	D	1331.160	739.903
409	~1493	D	1331.160	779.758
410	~1493	D	1331.160	779.758
412	~1452	D	1331.160	819.614
413	~1494	D	1331.160	871.280

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
414	~1462	D	1331.160	906.920
415	~1462	D	1331.160	906.920
416	~1618	D	1357.140	972.387
417	~1634	D	1357.140	1007.120
418	~1629	D	1357.140	1043.120
1	2447	L	33.882	239.240
2	727	L	41.880	278.360
3	~629	L	41.880	284.266
4	~492	L	37.943	346.812
5	~500	L	37.943	376.649
6	~516	L	37.943	445.760
7	~601	L	37.943	481.844
8	~547	L	37.943	552.129
9	~599	L	37.943	593.238
10	~563	L	37.943	615.779
11	~571	L	37.943	632.720
12	~1213	L	37.943	711.859
13	~1098	L	41.880	744.285
14	~1108	L	41.880	782.617
16	~1128	L	41.880	859.280
17	~1137	L	41.880	871.280
18	~1145	L	41.880	883.280
19	~1154	L	41.880	923.960
20	~1164	L	41.880	964.640
21	~1217	L	41.880	1011.415
22	~1182	L	41.880	1023.226
23	~1000	L	97.052	108.057
24	~479	L	128.014	278.360
25	~486	L	128.014	316.976
26	~494	L	128.014	346.812
27	~502	L	128.014	376.649
28	~518	L	128.014	445.760
29	~518	L	128.014	445.760
30	~525	L	128.014	484.154
31	~557	L	128.014	591.674
32	~651	L	133.920	615.779
33	~648	L	133.920	632.720
34	60	L	133.920	705.954
35	~1099	L	84.947	744.285
36	~1109	L	84.947	782.617
38	~1129	L	84.947	859.280
39	~1138	L	84.947	871.280
40	~1146	L	84.947	883.280
41	~1155	L	84.947	923.960
42	~1165	L	84.947	964.640
43	~1217	L	41.880	1011.415
44	~1183	L	84.947	1023.226
45	~1002	L	191.400	108.057
46	~1040	L	191.400	278.360
47	~487	L	170.596	316.976
48	~495	L	170.596	346.812
49	~503	L	170.596	376.649
50	~519	L	170.596	445.760
51	~519	L	170.596	445.760

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
52	~526	L	170.596	484.154
53	~558	L	170.596	591.674
54	~566	L	170.596	615.779
55	~574	L	170.596	632.720
56	~590	L	213.177	705.954
57	~1102	L	179.794	744.285
58	~1222	L	133.920	782.617
60	~1131	L	139.826	859.280
61	~1140	L	139.826	871.280
62	~1148	L	139.826	883.280
63	~1157	L	139.826	923.960
64	~1167	L	139.826	964.640
65	~1197	L	179.794	1017.320
66	~1185	L	139.826	1023.226
67	~1003	L	238.574	108.057
68	~1041	L	238.574	278.360
69	~489	L	255.758	316.976
70	~497	L	255.758	346.812
71	~505	L	255.758	376.649
72	~521	L	255.758	445.760
73	~521	L	255.758	445.760
74	~646	L	255.758	482.154
75	~647	L	255.758	589.674
76	~568	L	255.758	615.779
77	~576	L	255.758	632.720
78	~1094	L	219.762	705.954
79	~1103	L	219.762	744.285
80	~1232	L	215.760	782.617
82	~1228	L	215.760	820.949
83	194	L	215.760	871.280
84	194	L	215.760	871.280
85	~1229	L	215.760	923.960
86	~1227	L	215.760	964.640
87	128	L	215.760	1017.320
88	2316	L	219.762	1029.320
89	~1086	L	340.920	108.057
90	1287	L	340.920	278.360
91	~1743	L	340.920	327.038
92	~1779	L	340.920	357.622
93	~1753	L	340.920	386.840
94	~1753	L	340.920	386.840
95	5116	L	340.920	445.760
96	~939	L	442.103	589.674
97	~698	L	442.103	591.674
98	~721	L	442.103	632.720
99	~731	L	442.103	666.384
100	~941	L	442.103	705.954
101	~754	L	442.103	745.240
102	~766	L	442.103	782.378
104	~778	L	442.103	819.515
105	~801	L	442.103	871.280
106	~801	L	442.103	871.280
107	~876	L	442.103	922.543
108	~824	L	442.103	930.417

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
112	2373	L	446.040	278.360
113	~1778	L	446.040	327.038
114	~1767	L	446.040	351.716
115	2379	L	446.040	357.622
116	2378	L	446.040	445.760
117	~911	L	464.974	451.666
118	~688	L	464.974	555.834
119	~699	L	464.974	591.674
120	~722	L	464.974	632.720
121	~732	L	464.974	666.384
123	~755	L	464.974	745.240
124	~767	L	464.974	782.378
126	~779	L	464.974	819.515
127	~802	L	464.974	871.280
128	~802	L	464.974	871.280
129	~877	L	470.880	922.543
130	2332	L	464.974	930.417
138	2507	L	505.578	441.823
139	2509	L	493.767	449.697
140	~691	L	505.578	555.834
141	~947	L	505.578	612.440
142	~724	L	493.767	632.720
143	~724	L	493.767	632.720
144	~929	L	548.974	743.240
145	~756	L	476.786	745.240
146	~768	L	476.786	782.378
148	~948	L	470.880	819.515
149	~931	L	470.880	871.280
150	~931	L	470.880	871.280
151	2330	L	476.786	922.543
152	~833	L	470.880	930.417
160	2510	L	581.014	441.823
161	~920	L	560.786	449.697
162	~693	L	560.786	555.834
163	~927	L	560.786	612.440
164	~716	L	560.786	614.440
165	~727	L	560.786	632.720
166	~852	L	560.786	736.220
167	~846	L	560.786	743.240
168	~772	L	560.786	782.378
170	~950	L	554.880	819.515
171	~946	L	554.880	862.558
172	~932	L	554.880	871.280
173	~880	L	554.880	922.543
174	~1905	L	586.920	932.386
182	~1913	L	586.920	441.823
183	~916	L	618.000	449.697
184	~694	L	586.920	555.834
185	~843	L	586.920	612.440
186	~717	L	586.920	614.440
187	5117	L	586.920	632.720
188	~1550	L	618.000	705.954
189	3315	L	586.920	743.240
190	~1512	L	586.920	783.695

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
192	~785	L	586.920	819.515
193	297	L	586.920	862.558
194	~808	L	586.920	871.280
195	~958	L	590.857	926.480
196	2338	L	590.857	932.386
201	1592	L	630.148	334.446
202	~1349	L	638.022	350.194
203	~1395	L	638.022	378.966
204	2405	L	638.022	445.760
205	~1915	L	638.022	449.697
206	~683	L	618.000	519.994
207	2394	L	618.000	612.440
208	2409	L	618.000	632.720
209	~739	L	618.000	666.384
210	~1550	L	618.000	705.954
211	~1551	L	618.000	778.576
212	~774	L	618.000	782.378
214	~1518	L	618.000	825.095
215	~840	L	618.000	862.558
223	~1402	L	730.293	342.320
224	~1256	L	730.293	350.194
225	~1273	L	730.293	378.966
226	~1330	L	730.293	445.760
227	~1330	L	730.293	445.760
230	5118	L	758.040	632.720
231	5118	L	758.040	632.720
232	~1547	L	664.680	705.954
233	~1548	L	664.680	779.758
234	~1514	L	664.680	783.695
236	~1549	L	664.680	862.558
237	~1522	L	664.680	866.495
238	~2078	L	758.040	894.519
239	2349	L	752.134	922.543
240	~1902	L	752.134	926.480
245	~1410	L	772.491	342.320
246	~1257	L	772.491	350.194
247	~1274	L	772.491	378.966
248	~1331	L	772.491	445.760
249	~1331	L	772.491	445.760
250	~1557	L	906.960	492.455
252	5118	L	758.040	632.720
253	~1713	L	906.960	705.954
254	~1706	L	906.960	711.859
255	~1918	L	809.649	779.758
256	~1918	L	809.649	779.758
257	2390	L	903.023	812.373
258	~1912	L	763.946	862.558
259	2347	L	763.946	866.495
260	~2078	L	758.040	894.519
261	~1901	L	763.946	926.480
262	2351	L	763.946	930.417
267	~1245	L	952.712	334.446
268	~1362	L	906.960	350.194
269	~1396	L	906.960	378.966

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
270	667	L	906.960	445.760
271	~1591	L	906.960	449.762
272	~1570	L	1009.030	539.150
274	~1585	L	980.871	632.720
275	~1713	L	906.960	705.954
276	~1706	L	906.960	711.859
277	~1899	L	906.960	775.821
278	441	L	906.960	779.758
279	~1887	L	910.897	818.278
280	2370	L	960.094	822.215
289	~1350	L	1021.976	342.320
290	2533	L	1021.976	350.194
291	~1354	L	1021.976	386.840
292	~1554	L	1039.960	445.760
293	~1554	L	1039.960	445.760
294	~1602	L	1041.960	539.150
296	2471	L	1039.960	630.720
297	~1712	L	1039.960	705.954
298	~1707	L	1041.960	711.859
301	~2111	L	1035.391	818.278
302	~2111	L	1035.391	818.278
313	~1406	L	1107.120	386.840
314	~1412	L	1123.569	445.760
315	~1412	L	1123.569	445.760
316	982	L	1107.078	539.150
317	~1791	L	1107.078	602.864
318	976	L	1107.120	632.720
319	976	L	1107.120	632.720
320	~1707	L	1041.960	711.859
321	~2169	L	1132.080	745.809
322	488	L	1132.080	779.758
323	~1884	L	1128.143	818.278
324	2399	L	1128.143	824.184
335	~1408	L	1153.380	386.840
336	~1418	L	1164.386	445.760
337	~1418	L	1164.386	445.760
338	~1831	L	1172.400	539.150
339	~1792	L	1138.739	602.864
340	~1809	L	1138.739	625.700
341	~1807	L	1138.739	632.720
342	~1860	L	1162.907	711.859
343	~2169	L	1132.080	745.809
344	488	L	1132.080	779.758
345	2391	L	1136.017	812.373
346	~1883	L	1136.017	818.278
357	~1409	L	1199.640	386.840
358	~1413	L	1175.704	445.760
359	~1413	L	1175.704	445.760
360	~1831	L	1172.400	539.150
361	~1805	L	1172.400	602.864
362	507	L	1179.420	625.700
363	~1815	L	1179.420	632.720
364	~1841	L	1199.640	742.856
365	468	L	1199.640	779.758

Table 36: Nodal Displacements - Summary, Part 2 of 2

Panel	Node	OutputCase	GlobalX	GlobalY
			in	in
366	468	L	1199.640	779.758
367	469	L	1199.640	818.278
368	~1845	L	1199.640	862.599
369	~1845	L	1199.640	862.599
370	10389	L	1195.640	906.920
371	10389	L	1195.640	906.920
372	~1614	L	1195.640	972.387
374	2434	L	1195.703	1037.215
379	~1409	L	1199.640	386.840
380	~1323	L	1199.640	420.237
381	766	L	1199.640	445.760
382	~1840	L	1199.640	539.164
383	~1795	L	1229.831	602.864
384	975	L	1229.760	632.720
385	975	L	1229.760	632.720
386	~1440	L	1287.240	739.903
387	~1495	L	1287.240	779.758
388	~1495	L	1287.240	779.758
389	469	L	1199.640	818.278
390	~1451	L	1287.240	819.614
391	~1457	L	1287.240	865.375
392	~1496	L	1287.240	871.280
393	~1461	L	1287.240	906.920
394	~1617	L	1314.680	972.387
395	~1662	L	1314.680	1007.120
396	~1628	L	1314.680	1043.120
401	2526	L	1334.160	383.840
402	~1386	L	1331.160	439.855
403	630	L	1331.160	445.760
404	~1722	L	1331.160	492.462
405	622	L	1331.160	607.867
406	599	L	1395.600	632.720
407	~1435	L	1331.160	705.954
408	~1441	L	1331.160	739.903
409	~1493	L	1331.160	779.758
410	~1493	L	1331.160	779.758
412	~1452	L	1331.160	819.614
413	~1458	L	1331.160	865.375
414	~1494	L	1331.160	871.280
415	~1462	L	1331.160	906.920
416	~1618	L	1357.140	972.387
417	~1634	L	1357.140	1007.120
418	~1629	L	1357.140	1043.120

Design

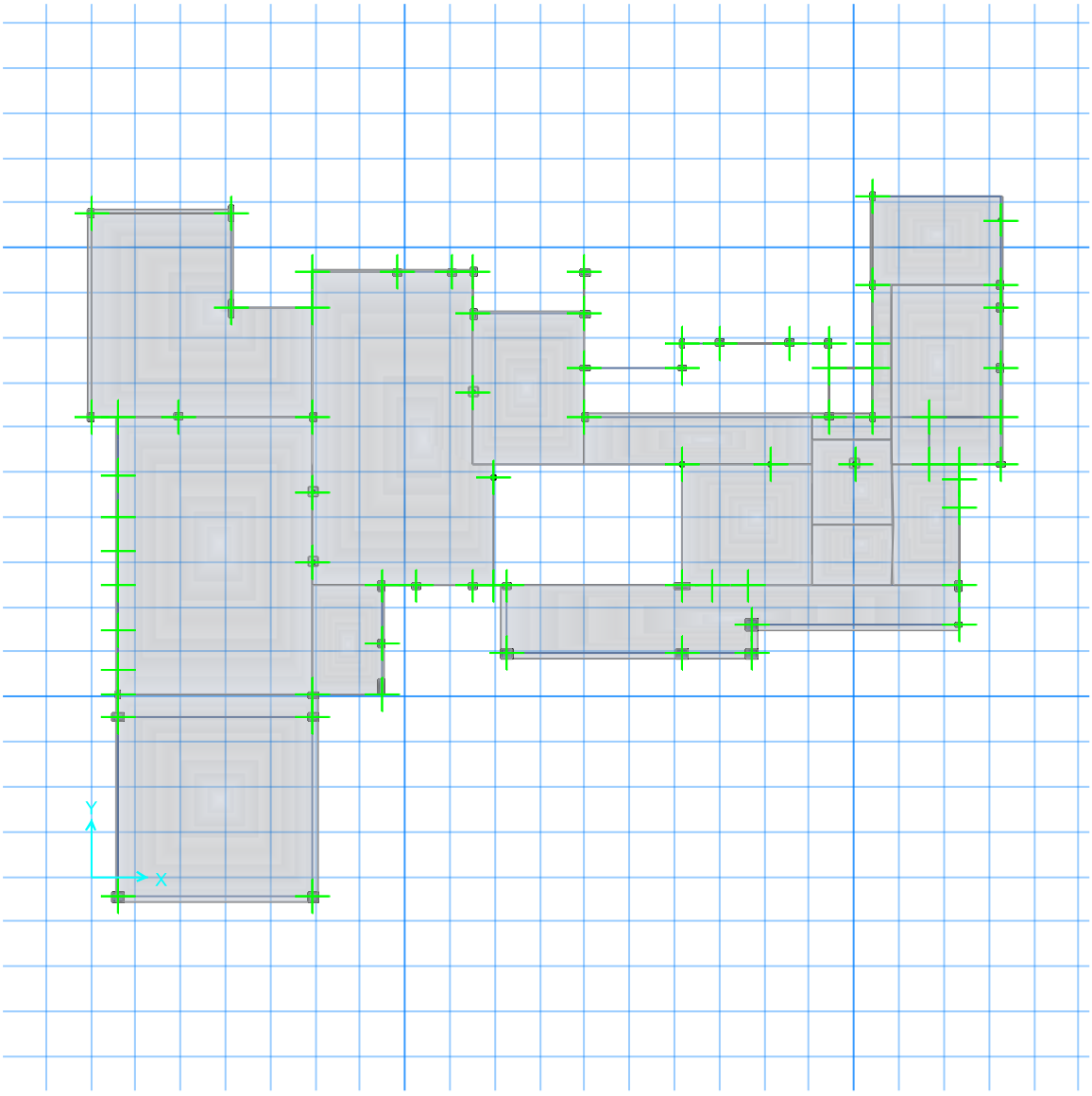


Figure 3: Finite element model

6. Design summary

This section provides design information for beams, strips, and punching checks.

6.1. Preferences

Table 37: Design Preferences 01 - Resistance Factors

Table 37: Design Preferences 01 - Resistance Factors

PhiTen	PhiComp	PhiShear
0.900000	0.650000	0.750000

Table 38: Design Preferences 02 - Rebar Cover - Slabs

Table 38: Design Preferences 02 - Rebar Cover - Slabs

CoverTop	CoverBot	BarSize	InnerLayer	PTCGSTop	PTCGSBotExt	PTCGSBotInt	SlabType
in	in			in	in	in	
1.0000	1.0000	#4	B	0.9843	1.5748	0.9843	Two Way

Table 39: Design Preferences 03 - Rebar Cover - Beams

Table 39: Design Preferences 03 - Rebar Cover - Beams

CoverTop	CoverBot	BarSizeF	BarSizeS	PTCGSTop	PTCGSBot
in	in			in	in
1.5748	1.5748	#5	#3	1.9685	1.9685

Table 40: Design Preferences 04 - Prestress Data

Table 40: Design Preferences 04 - Prestress Data

UserStress	InitConcRat	LLFraction
No	0.800000	0.500000

6.2. Overwrites

Table 41: Slab Design Overwrites 01 - Strip Based

Table 41: Slab Design Overwrites 01 - Strip Based

Strip	Layer	DesignType	RLLF	IgnorePT	RebarMat
CSA1	A	Column	1.000000	No	CSA-G30.18Gr400-fy=60ksi
CSA2	A	Column	1.000000	No	CSA-G30.18Gr400-fy=60ksi

Table 41: Slab Design Overwrites 01 - Strip Based

Strip	Layer	DesignType	RLLF	IgnorePT	RebarMat
MSA1	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA2	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA3	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA4	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA5	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA6	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA7	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA8	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA9	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB1	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB2	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB3	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB4	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB5	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB6	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB7	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB8	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB9	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA10	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA11	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA12	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA13	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSA14	A	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB10	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB11	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB12	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB13	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB14	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB15	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB16	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi

Table 41: Slab Design Overwrites 01 - Strip Based

Strip	Layer	DesignType	RLLF	IgnorePT	RebarMat
MSB17	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB18	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB19	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB20	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB21	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB22	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi
MSB23	B	Middle	1.000000	No	CSA-G30.18Gr400-fy=60ksi

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 1 of 2

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 1 of 2

Area	RebarMat
2	CSA-G30.18Gr400-fy=60ksi
143	CSA-G30.18Gr400-fy=60ksi
213	CSA-G30.18Gr400-fy=60ksi
214	CSA-G30.18Gr400-fy=60ksi
215	CSA-G30.18Gr400-fy=60ksi
262	CSA-G30.18Gr400-fy=60ksi
266	CSA-G30.18Gr400-fy=60ksi
267	CSA-G30.18Gr400-fy=60ksi
270	CSA-G30.18Gr400-fy=60ksi
273	CSA-G30.18Gr400-fy=60ksi
276	CSA-G30.18Gr400-fy=60ksi
440	CSA-G30.18Gr400-fy=60ksi
549	CSA-G30.18Gr400-fy=60ksi
550	CSA-G30.18Gr400-fy=60ksi
598	CSA-G30.18Gr400-fy=60ksi
599	CSA-G30.18Gr400-fy=60ksi
600	CSA-G30.18Gr400-fy=60ksi
601	CSA-G30.18Gr400-fy=60ksi
602	CSA-G30.18Gr400-fy=60ksi
603	CSA-G30.18Gr400-fy=60ksi
604	CSA-G30.18Gr400-fy=60ksi
605	CSA-G30.18Gr400-fy=60ksi
606	CSA-G30.18Gr400-fy=60ksi
607	CSA-G30.18Gr400-fy=60ksi
608	CSA-G30.18Gr400-fy=60ksi
609	CSA-G30.18Gr400-fy=60ksi
610	CSA-G30.18Gr400-fy=60ksi
611	CSA-G30.18Gr400-fy=60ksi
612	CSA-G30.18Gr400-fy=60ksi
613	CSA-G30.18Gr400-fy=60ksi
614	CSA-G30.18Gr400-fy=60ksi
615	CSA-G30.18Gr400-fy=60ksi
616	CSA-G30.18Gr400-fy=60ksi

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 1 of 2

Area	RebarMat
617	CSA-G30.18Gr400-fy=60ksi
618	CSA-G30.18Gr400-fy=60ksi
619	CSA-G30.18Gr400-fy=60ksi
620	CSA-G30.18Gr400-fy=60ksi
621	CSA-G30.18Gr400-fy=60ksi
622	CSA-G30.18Gr400-fy=60ksi
623	CSA-G30.18Gr400-fy=60ksi
624	CSA-G30.18Gr400-fy=60ksi
625	CSA-G30.18Gr400-fy=60ksi
626	CSA-G30.18Gr400-fy=60ksi
627	CSA-G30.18Gr400-fy=60ksi
629	CSA-G30.18Gr400-fy=60ksi
632	CSA-G30.18Gr400-fy=60ksi
635	CSA-G30.18Gr400-fy=60ksi
638	CSA-G30.18Gr400-fy=60ksi
640	CSA-G30.18Gr400-fy=60ksi
641	CSA-G30.18Gr400-fy=60ksi
642	CSA-G30.18Gr400-fy=60ksi
643	CSA-G30.18Gr400-fy=60ksi
644	CSA-G30.18Gr400-fy=60ksi
645	CSA-G30.18Gr400-fy=60ksi
646	CSA-G30.18Gr400-fy=60ksi
647	CSA-G30.18Gr400-fy=60ksi
648	CSA-G30.18Gr400-fy=60ksi
649	CSA-G30.18Gr400-fy=60ksi
650	CSA-G30.18Gr400-fy=60ksi
651	CSA-G30.18Gr400-fy=60ksi
652	CSA-G30.18Gr400-fy=60ksi
653	CSA-G30.18Gr400-fy=60ksi
654	CSA-G30.18Gr400-fy=60ksi
655	CSA-G30.18Gr400-fy=60ksi
656	CSA-G30.18Gr400-fy=60ksi
657	CSA-G30.18Gr400-fy=60ksi
658	CSA-G30.18Gr400-fy=60ksi
659	CSA-G30.18Gr400-fy=60ksi
1341	CSA-G30.18Gr400-fy=60ksi
1344	CSA-G30.18Gr400-fy=60ksi
1346	CSA-G30.18Gr400-fy=60ksi
1347	CSA-G30.18Gr400-fy=60ksi
1348	CSA-G30.18Gr400-fy=60ksi
1350	CSA-G30.18Gr400-fy=60ksi

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 2 of 2

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 2 of 2

Area	RLLF	Design	IgnorePT
2	1.000000	Yes	No
143	1.000000	Yes	No
213	1.000000	Yes	No
214	1.000000	Yes	No
215	1.000000	Yes	No

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 2 of 2

Area	RLLF	Design	IgnorePT
262	1.000000	Yes	No
266	1.000000	Yes	No
267	1.000000	Yes	No
270	1.000000	Yes	No
273	1.000000	Yes	No
276	1.000000	Yes	No
440	1.000000	Yes	No
549	1.000000	Yes	No
550	1.000000	Yes	No
598	1.000000	Yes	No
599	1.000000	Yes	No
600	1.000000	Yes	No
601	1.000000	Yes	No
602	1.000000	Yes	No
603	1.000000	Yes	No
604	1.000000	Yes	No
605	1.000000	Yes	No
606	1.000000	Yes	No
607	1.000000	Yes	No
608	1.000000	Yes	No
609	1.000000	Yes	No
610	1.000000	Yes	No
611	1.000000	Yes	No
612	1.000000	Yes	No
613	1.000000	Yes	No
614	1.000000	Yes	No
615	1.000000	Yes	No
616	1.000000	Yes	No
617	1.000000	Yes	No
618	1.000000	Yes	No
619	1.000000	Yes	No
620	1.000000	Yes	No
621	1.000000	Yes	No
622	1.000000	Yes	No
623	1.000000	Yes	No
624	1.000000	Yes	No
625	1.000000	Yes	No
626	1.000000	Yes	No
627	1.000000	Yes	No
629	1.000000	Yes	No
632	1.000000	Yes	No
635	1.000000	Yes	No
638	1.000000	Yes	No
640	1.000000	Yes	No
641	1.000000	Yes	No
642	1.000000	Yes	No
643	1.000000	Yes	No
644	1.000000	Yes	No
645	1.000000	Yes	No
646	1.000000	Yes	No
647	1.000000	Yes	No
648	1.000000	Yes	No
649	1.000000	Yes	No

Table 42: Slab Design Overwrites 02 - Finite Element Based, Part 2 of 2

Area	RLLF	Design	IgnorePT
650	1.000000	Yes	No
651	1.000000	Yes	No
652	1.000000	Yes	No
653	1.000000	Yes	No
654	1.000000	Yes	No
655	1.000000	Yes	No
656	1.000000	Yes	No
657	1.000000	Yes	No
658	1.000000	Yes	No
659	1.000000	Yes	No
1341	1.000000	Yes	No
1344	1.000000	Yes	No
1346	1.000000	Yes	No
1347	1.000000	Yes	No
1348	1.000000	Yes	No
1350	1.000000	Yes	No

Table 43: Beam Design Overwrites

Table 43: Beam Design Overwrites

Line	CoverType	RLLF	IgnorePT
1	Section	1.000000	No
3	Section	1.000000	No
4	Section	1.000000	No
5	Section	1.000000	No
23	Section	1.000000	No
24	Section	1.000000	No
25	Section	1.000000	No
26	Section	1.000000	No
27	Section	1.000000	No
28	Section	1.000000	No
42	Section	1.000000	No
43	Section	1.000000	No
44	Section	1.000000	No
45	Section	1.000000	No
46	Section	1.000000	No
47	Section	1.000000	No
50	Section	1.000000	No
51	Section	1.000000	No
57	Section	1.000000	No
58	Section	1.000000	No
61	Section	1.000000	No
62	Section	1.000000	No
63	Section	1.000000	No
64	Section	1.000000	No
65	Section	1.000000	No
66	Section	1.000000	No
67	Section	1.000000	No
68	Section	1.000000	No
69	Section	1.000000	No
70	Section	1.000000	No

Table 43: Beam Design Overwrites

Line	CoverType	RLLF	IgnorePT
71	Section	1.000000	No
72	Section	1.000000	No
74	Section	1.000000	No
75	Section	1.000000	No
76	Section	1.000000	No
84	Section	1.000000	No
85	Section	1.000000	No
86	Section	1.000000	No
87	Section	1.000000	No
88	Section	1.000000	No
89	Section	1.000000	No
90	Section	1.000000	No
91	Section	1.000000	No
92	Section	1.000000	No
93	Section	1.000000	No
94	Section	1.000000	No
95	Section	1.000000	No
96	Section	1.000000	No
97	Section	1.000000	No
98	Section	1.000000	No
99	Section	1.000000	No
100	Section	1.000000	No
101	Section	1.000000	No
102	Section	1.000000	No
105	Section	1.000000	No
111	Section	1.000000	No
113	Section	1.000000	No
114	Section	1.000000	No
117	Section	1.000000	No
119	Section	1.000000	No
120	Section	1.000000	No
121	Section	1.000000	No
124	Section	1.000000	No

Table 44: Punching Shear Design Overwrites 01 - General

Table 44: Punching Shear Design Overwrites 01 - General

Point	Check	LocType	EffDepth	ReinfType
23	Program Determined	Auto	Auto	None
24	Program Determined	Auto	Auto	None
60	Program Determined	Auto	Auto	None
67	Program Determined	Auto	Auto	None
128	Program Determined	Auto	Auto	None
194	Program Determined	Auto	Auto	None
220	Program Determined	Auto	Auto	None

Table 44: Punching Shear Design Overwrites 01 - General

Point	Check	LocType	EffDepth	ReinfType
228	Program Determined	Auto	Auto	None
229	Program Determined	Auto	Auto	None
297	Program Determined	Auto	Auto	None
398	Program Determined	Auto	Auto	None
399	Program Determined	Auto	Auto	None
407	Program Determined	Auto	Auto	None
413	Program Determined	Auto	Auto	None
424	Program Determined	Auto	Auto	None
431	Program Determined	Auto	Auto	None
441	Program Determined	Auto	Auto	None
446	Program Determined	Auto	Auto	None
447	Program Determined	Auto	Auto	None
448	Program Determined	Auto	Auto	None
449	Program Determined	Auto	Auto	None
477	Program Determined	Auto	Auto	None
482	Program Determined	Auto	Auto	None
496	Program Determined	Auto	Auto	None
529	Program Determined	Auto	Auto	None
535	Program Determined	Auto	Auto	None
541	Program Determined	Auto	Auto	None
580	Program Determined	Auto	Auto	None
586	Program Determined	Auto	Auto	None
599	Program Determined	Auto	Auto	None
630	Program Determined	Auto	Auto	None
667	Program Determined	Auto	Auto	None
691	Program Determined	Auto	Auto	None
697	Program Determined	Auto	Auto	None
703	Program Determined	Auto	Auto	None
709	Program Determined	Auto	Auto	None
727	Program Determined	Auto	Auto	None

Table 44: Punching Shear Design Overwrites 01 - General

Point	Check	LocType	EffDepth	ReinfType
766	Program Determined	Auto	Auto	None
767	Program Determined	Auto	Auto	None
1236	Program Determined	Auto	Auto	None
1287	Program Determined	Auto	Auto	None
1590	Program Determined	Auto	Auto	None
1597	Program Determined	Auto	Auto	None
2373	Program Determined	Auto	Auto	None
2378	Program Determined	Auto	Auto	None
2379	Program Determined	Auto	Auto	None
2387	Program Determined	Auto	Auto	None
2394	Program Determined	Auto	Auto	None
2397	Program Determined	Auto	Auto	None
2405	Program Determined	Auto	Auto	None
2410	Program Determined	Auto	Auto	None
2411	Program Determined	Auto	Auto	None
2412	Program Determined	Auto	Auto	None
3302	Program Determined	Auto	Auto	None
3303	Program Determined	Auto	Auto	None
3315	Program Determined	Auto	Auto	None

6.3. Slab design

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
CSA1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-178680.26	0.52
CSA1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-509.54	0.00
CSA1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-649866.76	1.96
CSA1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-660955.01	2.00

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
CSA1	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-2550.18	2.329E-002
CSA1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-56553.12	0.16
CSA2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-18248.21	5.759E-002
CSA2	Span 1	Middle		0.00	0.00
CSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-676077.80	2.05
CSA2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-746469.61	2.28
CSA2	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-109077.95	0.36
CSA2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-95336.09	0.31
MSA1	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-57765.73	0.20
MSA1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-255418.11	0.74
MSA1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-170776.58	0.50
MSA2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-65486.88	0.22
MSA2	Span 1	Middle		0.00	0.00
MSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-53435.29	0.17
MSA3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-167340.89	0.48
MSA3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-344536.41	1.02
MSA3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-358977.52	1.05
MSA3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-358977.52	1.05
MSA3	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-17158.29	6.528E-002
MSA3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-72122.11	0.25
MSA3	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-104360.73	0.34
MSA3	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-34613.70	0.11
MSA3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-132712.61	0.39
MSA3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-189877.68	0.55
MSA3	Span 4	Middle		0.00	0.00
MSA3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-283768.05	0.82
MSA3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-283768.05	0.82
MSA3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-38166.61	0.11
MSA3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-257322.24	0.74
MSA3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-268243.17	0.78

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSA3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-64876.57	0.18
MSA3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	-236713.01	0.68
MSA3	Span 7	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-236713.01	0.68
MSA3	Span 7	Middle		0.00	0.00
MSA3	Span 7	End	Comb=1.2 D + 1.6 L +1.6 Lr	-70250.58	0.20
MSA4	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-65214.96	0.19
MSA4	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-463516.63	1.38
MSA4	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-91975.40	0.26
MSA5	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-71655.77	0.21
MSA5	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-169279.14	0.50
MSA5	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-143906.07	0.44
MSA6	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-24629.59	7.710E-002
MSA6	Cantilever Start	Middle		0.00	0.00
MSA6	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-36179.53	0.10
MSA7	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-106986.39	0.32
MSA7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-373947.48	1.09
MSA7	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-239279.24	0.70
MSA7	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-239279.24	0.70
MSA7	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-79666.06	0.25
MSA7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-79949.42	0.25
MSA8	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-272918.74	0.81
MSA8	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-199947.86	0.58
MSA8	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-106691.98	0.31
MSA8	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-81013.70	0.23
MSA8	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-47423.05	0.14
MSA8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-5682.54	2.356E-002
MSA9	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-19081.74	7.043E-002
MSA9	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-55063.10	0.17
MSA9	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-221372.27	0.64
MSA9	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-221372.27	0.64

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSA9	Span 1	Middle		0.00	0.00
MSA9	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-313015.96	0.92
MSA9	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-313015.96	0.92
MSA9	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-117862.87	0.35
MSA9	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-96538.07	0.32
MSB1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-92818.85	0.29
MSB1	Span 1	Middle		0.00	0.00
MSB1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-266606.85	0.84
MSB1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-266606.85	0.84
MSB1	Span 2	Middle		0.00	0.00
MSB1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-293222.43	0.92
MSB1	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-293222.43	0.92
MSB1	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-3671.00	0.00
MSB1	Span 3	End		0.00	0.00
MSB1	Span 4	Start		0.00	0.00
MSB1	Span 4	Middle		0.00	0.00
MSB1	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-3122.35	0.00
MSB1	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-3122.35	0.00
MSB1	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-155088.48	0.48
MSB1	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-508254.03	1.65
MSB1	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-510281.83	1.67
MSB1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-2121.55	3.309E-002
MSB1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-22776.00	0.11
MSB2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-34259.04	0.11
MSB2	Span 1	Middle		0.00	0.00
MSB2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-288291.35	0.91
MSB2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-288291.35	0.91
MSB2	Span 2	Middle		0.00	0.00
MSB2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-320491.58	1.01
MSB2	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-320491.58	1.01
MSB2	Span 3	Middle		0.00	0.00
MSB2	Span 3	End		0.00	0.00
MSB2	Span 4	Start		0.00	0.00
MSB2	Span 4	Middle		0.00	0.00
MSB2	Span 4	End		0.00	0.00

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB2	Span 5	Start		0.00	0.00
MSB2	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-114796.74	0.36
MSB2	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-326870.59	1.04
MSB2	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-326870.59	1.04
MSB2	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-100270.11	0.38
MSB2	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-110142.40	0.36
MSB3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-55455.10	0.17
MSB3	Span 1	Middle		0.00	0.00
MSB3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-255846.43	0.80
MSB3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-255846.43	0.80
MSB3	Span 2	Middle		0.00	0.00
MSB3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-341998.17	1.08
MSB3	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-341998.17	1.08
MSB3	Span 3	Middle		0.00	0.00
MSB3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-6057.97	1.857E-002
MSB3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-6057.97	1.857E-002
MSB3	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-2114.42	0.00
MSB3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-16761.31	5.402E-002
MSB3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-16761.31	5.402E-002
MSB3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-126745.95	0.40
MSB3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-217711.26	0.69
MSB3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-217711.26	0.69
MSB3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-47387.44	0.20
MSB3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	-39720.35	0.19
MSB4	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-276598.43	0.87
MSB4	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-85244.35	0.27
MSB4	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-92680.28	0.32
MSB5	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-55808.85	0.17
MSB5	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-7875.61	3.127E-002
MSB5	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-324784.58	1.03
MSB5	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-324784.58	1.03

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB5	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-78690.32	0.25
MSB5	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-56422.78	0.19
MSB5	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-56422.78	0.19
MSB5	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-62367.76	0.19
MSB5	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-76607.10	0.27
MSB6	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-312329.35	0.98
MSB6	Span 1	Middle		0.00	0.00
MSB6	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-326819.03	1.03
MSB6	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-427871.04	0.46
MSB6	Span 2	Middle		0.00	0.00
MSB6	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-426959.68	0.46
MSB6	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-381771.81	1.21
MSB6	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-58114.54	0.18
MSB6	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-229488.41	0.72
MSB6	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-327215.61	0.36
MSB6	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-320701.60	0.35
MSB6	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-284229.28	0.31
MSB6	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-236951.95	0.75
MSB6	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-19800.67	6.505E-002
MSB6	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-280562.17	0.88
MSB7	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-161643.94	0.53
MSB7	Cantilever Start	Middle		0.00	0.00
MSB7	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-111670.18	0.36
MSB7	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-111670.18	0.36
MSB7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-9528.98	5.563E-002
MSB7	Span 1	End		0.00	0.00
MSB7	Span 2	Start		0.00	0.00
MSB7	Span 2	Middle		0.00	0.00
MSB7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-72501.99	0.23
MSB8	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-42253.32	0.15
MSB8	Span 1	Middle		0.00	0.00
MSB8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-22184.01	9.026E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB8	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-22184.01	9.026E-002
MSB8	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-490.58	2.145E-002
MSB8	Span 2	End		0.00	0.00
MSB8	Span 3	Start		0.00	0.00
MSB8	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-54229.66	0.17
MSB8	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-97227.29	0.31
MSB9	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-8344.65	4.584E-002
MSB9	Span 1	Middle		0.00	0.00
MSB9	Span 1	End	DCONU1	-1785.75	1.948E-002
MSA10	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-67429.56	0.20
MSA10	Span 1	Middle		0.00	0.00
MSA10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-65292.23	0.19
MSA11	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-134179.08	0.39
MSA11	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-15235.83	4.328E-002
MSA11	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-26449.14	7.519E-002
MSA12	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-25787.02	8.486E-002
MSA12	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-5878.13	3.567E-002
MSA12	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-45825.82	0.16
MSA13	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-35230.77	0.16
MSA13	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-34420.62	0.18
MSA13	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-60583.73	0.29
MSA13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-34216.59	0.15
MSA13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-173.07	0.00
MSA13	Span 2	End	DCONU1	-6832.86	2.758E-002
MSA14	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-31170.88	0.13
MSA14	Cantilever Start	Middle		0.00	0.00
MSA14	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-18457.60	7.463E-002
MSA14	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-18457.60	7.463E-002
MSA14	Cantilever End	Middle		0.00	0.00
MSA14	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-11704.09	4.816E-002
MSB10	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-95134.28	0.33
MSB10	Cantilever Start	Middle		0.00	0.00

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB10	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-69715.58	0.24
MSB10	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-69715.58	0.24
MSB10	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-35896.35	0.11
MSB10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-7741.49	2.374E-002
MSB10	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-7741.49	2.374E-002
MSB10	Span 2	Middle		0.00	0.00
MSB10	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-32383.63	9.953E-002
MSB10	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-32383.63	9.953E-002
MSB10	Span 3	Middle		0.00	0.00
MSB10	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-114894.64	0.36
MSB10	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-114894.64	0.36
MSB10	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-52763.53	0.16
MSB10	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-332389.63	1.06
MSB10	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-332389.63	1.06
MSB10	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-9190.78	2.819E-002
MSB10	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	-70294.39	0.22
MSB11	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-25890.52	8.853E-002
MSB11	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-22924.28	7.157E-002
MSB11	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	-3655.76	1.677E-002
MSB12	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-50894.44	0.18
MSB12	Span 1	Middle	DCONU1	-29579.36	0.12
MSB12	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-52028.09	0.18
MSB13	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-88531.97	0.32
MSB13	Span 1	Middle	DCONU1	-10830.38	5.545E-002
MSB13	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-180225.95	0.57
MSB13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-180225.95	0.57
MSB13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-79593.07	0.25
MSB13	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-10438.28	3.202E-002
MSB14	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-8584.23	5.742E-002
MSB14	Span 1	Middle	DCONU1	-2428.95	2.611E-002
MSB14	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-333886.01	1.06

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB14	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-307925.59	0.98
MSB14	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-88125.14	0.27
MSB14	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-25623.51	7.869E-002
MSB15	Span 1	Start	DCONU1	-7059.06	4.312E-002
MSB15	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-1333.39	1.599E-002
MSB15	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-282445.68	0.89
MSB15	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-282445.68	0.89
MSB15	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-133224.09	0.41
MSB15	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-44525.98	0.14
MSB16	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-165.69	1.835E-002
MSB16	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-105321.83	0.33
MSB16	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-341941.55	1.11
MSB16	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-341941.55	1.11
MSB16	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-84507.73	0.27
MSB16	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-2485.81	0.00
MSB17	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-65897.29	0.21
MSB17	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-214495.60	0.71
MSB17	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-90279.48	0.28
MSB18	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-26040.98	0.12
MSB18	Cantilever Start	Middle		0.00	0.00
MSB18	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	-56808.76	0.20
MSB18	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-56808.76	0.20
MSB18	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-56319.73	0.22
MSB18	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-275294.34	0.90
MSB18	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-275294.34	0.90
MSB18	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-38466.64	0.14
MSB18	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-195527.63	0.62
MSB18	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-195527.63	0.62
MSB18	Span 3	Middle	DCONU1	-482.62	0.00
MSB18	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-5821.32	2.245E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Strip	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
MSB19	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-20253.23	7.969E-002
MSB19	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-34844.41	0.14
MSB19	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-50152.45	0.20
MSB19	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-50152.45	0.20
MSB19	Span 2	Middle		0.00	0.00
MSB19	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	-97590.24	0.35
MSB19	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-97590.24	0.35
MSB19	Span 3	Middle		0.00	0.00
MSB19	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	-178343.97	0.59
MSB19	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-178343.97	0.59
MSB19	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-183528.32	0.59
MSB19	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	-4343.28	1.615E-002
MSB20	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-8857.59	4.301E-002
MSB20	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-128525.43	0.40
MSB20	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-7157.23	2.537E-002
MSB21	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-7576.60	4.912E-002
MSB21	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-43144.64	0.20
MSB21	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-110965.37	0.51
MSB22	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-3034.49	1.534E-002
MSB22	Span 1	Middle		0.00	0.00
MSB22	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1016.67	1.086E-002
MSB23	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-2730.62	1.414E-002
MSB23	Span 1	Middle		0.00	0.00
MSB23	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1988.60	1.047E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
CSA1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	255799.96	0.74
CSA1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	303515.32	0.88
CSA1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	50242.60	0.16

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
CSA1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	149884.19	0.17
CSA1	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	219321.06	0.64
CSA1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	132750.58	0.38
CSA2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	233529.96	0.68
CSA2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	379727.06	1.11
CSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	341608.65	0.37
CSA2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	351834.74	0.38
CSA2	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	92208.99	0.35
CSA2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	104955.72	0.38
MSA1	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	139874.17	0.43
MSA1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	197755.20	0.57
MSA1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	68419.52	0.20
MSA2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	129992.31	0.41
MSA2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	140740.62	0.44
MSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	75835.82	0.25
MSA3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	213751.48	0.62
MSA3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	224089.06	0.65
MSA3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	250383.47	0.72
MSA3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	141641.08	0.41
MSA3	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	37965.72	0.13
MSA3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	44474.79	0.17
MSA3	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	44474.79	0.17
MSA3	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	11604.50	5.941E-002
MSA3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	53819.17	0.15
MSA3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	53819.17	0.15
MSA3	Span 4	Middle		0.00	0.00
MSA3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	6390.80	2.031E-002
MSA3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	29435.17	8.370E-002
MSA3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	99830.11	0.29
MSA3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	14455.87	4.108E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSA3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	14455.87	4.108E-002
MSA3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	137706.51	0.39
MSA3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	19572.46	5.562E-002
MSA3	Span 7	Start	Comb=1.2 D + 1.6 L +1.6 Lr	19572.46	5.562E-002
MSA3	Span 7	Middle		0.00	0.00
MSA3	Span 7	End	Comb=1.2 D + 1.6 L +1.6 Lr	48288.73	0.14
MSA4	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	370559.38	1.09
MSA4	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	319509.02	0.94
MSA4	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	169454.69	0.52
MSA5	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	343586.48	1.01
MSA5	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	328542.43	0.96
MSA5	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	41268.15	0.13
MSA6	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	148140.97	0.43
MSA6	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	222405.48	0.64
MSA6	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	124633.73	0.36
MSA7	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	104898.69	0.32
MSA7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	164402.22	0.47
MSA7	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	245899.58	0.71
MSA7	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	17339.33	6.816E-002
MSA7	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	62661.08	0.20
MSA7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	45804.52	0.14
MSA8	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	145120.66	0.42
MSA8	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	251599.25	0.73
MSA8	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	65637.95	0.19
MSA8	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	62992.20	0.18
MSA8	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	11569.27	4.377E-002
MSA8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	12885.00	4.402E-002
MSA9	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	22841.29	8.112E-002
MSA9	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	24121.82	8.606E-002
MSA9	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	35613.83	0.10

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSA9	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	35613.83	0.10
MSA9	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	251392.72	0.73
MSA9	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	90744.83	0.26
MSA9	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	18370.50	6.435E-002
MSA9	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	50626.56	0.16
MSA9	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	36246.42	0.13
MSB1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	172336.37	0.54
MSB1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	206567.25	0.65
MSB1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	64022.87	0.20
MSB1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB1	Span 2	Middle		0.00	0.00
MSB1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB1	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	58238.92	0.18
MSB1	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	200977.48	0.63
MSB1	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	194116.79	0.61
MSB1	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	180690.43	0.56
MSB1	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	205105.51	0.64
MSB1	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	191888.02	0.60
MSB1	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	191888.02	0.60
MSB1	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	135279.15	0.42
MSB1	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	1216.63	1.739E-002
MSB1	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	17793.35	8.107E-002
MSB1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	119716.38	0.41
MSB1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	114882.64	0.39
MSB2	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	149343.63	0.46
MSB2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	215908.71	0.67
MSB2	Span 1	End	DCONU1	34138.88	0.10
MSB2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB2	Span 2	Middle		0.00	0.00
MSB2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB2	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	8204.22	2.516E-002
MSB2	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	212805.86	0.66
MSB2	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	228937.33	0.72
MSB2	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	207074.55	0.65
MSB2	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	178621.60	0.56
MSB2	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	161197.38	0.50
MSB2	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	161197.38	0.50
MSB2	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	99504.24	0.31
MSB2	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	3.329E-002
MSB2	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	17142.58	0.10
MSB2	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	105292.75	0.38
MSB2	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	105077.73	0.35
MSB3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	160777.28	0.50
MSB3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	211619.53	0.66
MSB3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	49095.46	0.15
MSB3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB3	Span 2	Middle		0.00	0.00
MSB3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB3	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	19928.94	6.118E-002
MSB3	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	258082.53	0.81
MSB3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	226111.73	0.71
MSB3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	148652.52	0.46
MSB3	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	151305.26	0.47
MSB3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	154147.94	0.48
MSB3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	154147.94	0.48
MSB3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	136211.38	0.43
MSB3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	3.166E-002
MSB3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3842.47	6.131E-002
MSB3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	33167.99	0.17
MSB3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	33167.99	0.17

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB4	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	187801.84	0.59
MSB4	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	97705.15	0.33
MSB4	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	113730.16	0.38
MSB5	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	78489.40	0.24
MSB5	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	111176.63	0.34
MSB5	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	40473.82	0.12
MSB5	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	786.93	0.00
MSB5	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	63231.24	0.21
MSB5	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	25165.15	8.923E-002
MSB5	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	79060.51	0.25
MSB5	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	97362.94	0.31
MSB5	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	119305.85	0.39
MSB6	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	78824.76	0.24
MSB6	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	188237.67	0.59
MSB6	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	67016.54	0.21
MSB6	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB6	Span 2	Middle		0.00	0.00
MSB6	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB6	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB6	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	22951.16	7.047E-002
MSB6	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	6728.76	2.538E-002
MSB6	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB6	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	5468.70	0.00
MSB6	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	6004.72	0.00
MSB6	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	14648.20	5.629E-002
MSB6	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	91886.05	0.29
MSB6	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	6404.31	3.010E-002
MSB7	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	29699.62	0.12
MSB7	Cantilever Start	Middle		0.00	0.00
MSB7	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	16619.99	6.307E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB7	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	16619.99	6.307E-002
MSB7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	104264.16	0.35
MSB7	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	124868.64	0.40
MSB7	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	128923.72	0.41
MSB7	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	102660.77	0.32
MSB7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	84608.26	0.27
MSB8	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	35022.85	0.13
MSB8	Span 1	Middle		0.00	0.00
MSB8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	34173.25	0.13
MSB8	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	34173.25	0.13
MSB8	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	150349.43	0.48
MSB8	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	146150.13	0.46
MSB8	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	146150.13	0.46
MSB8	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	113868.52	0.35
MSB8	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	88129.78	0.28
MSB9	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	623.69	1.058E-002
MSB9	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	54108.89	0.26
MSB9	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2598.58	2.280E-002
MSA10	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	83337.01	0.25
MSA10	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	141742.75	0.41
MSA10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	108509.69	0.31
MSA11	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	90577.64	0.26
MSA11	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	94018.46	0.27
MSA11	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	121277.19	0.35
MSA12	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	40746.85	0.14
MSA12	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	111106.65	0.32
MSA12	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	258594.86	0.75
MSA13	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	47993.03	0.22
MSA13	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	47993.03	0.22
MSA13	Span 1	End	DCONU1	1797.58	4.699E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSA13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	23650.12	9.570E-002
MSA13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	38957.27	0.16
MSA13	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	19695.36	7.965E-002
MSA14	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	55462.97	0.23
MSA14	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	94703.63	0.39
MSA14	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	64833.39	0.26
MSA14	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5155.00	2.140E-002
MSA14	Cantilever End	Middle		0.00	0.00
MSA14	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	9033.14	3.736E-002
MSB10	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	98602.73	0.34
MSB10	Cantilever Start	Middle		0.00	0.00
MSB10	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	123740.52	0.41
MSB10	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	123740.52	0.41
MSB10	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	141769.99	0.44
MSB10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	59538.99	0.18
MSB10	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	59538.99	0.18
MSB10	Span 2	Middle		0.00	0.00
MSB10	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	42193.09	0.13
MSB10	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	42193.09	0.13
MSB10	Span 3	Middle		0.00	0.00
MSB10	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	59065.30	0.18
MSB10	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	59065.30	0.18
MSB10	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	120563.97	0.37
MSB10	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB10	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB10	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	80171.14	0.25
MSB10	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	1981.43	0.00
MSB11	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	75819.46	0.24
MSB11	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	11662.23	3.692E-002
MSB11	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	29167.73	0.10

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB12	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	42464.62	0.15
MSB12	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	62096.24	0.21
MSB12	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	28479.77	0.11
MSB13	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	129813.90	0.47
MSB13	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	199928.03	0.65
MSB13	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	92847.18	0.31
MSB13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	40574.19	0.13
MSB13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB13	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	17887.02	5.490E-002
MSB14	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	153186.20	0.54
MSB14	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	209535.66	0.68
MSB14	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	36704.24	0.12
MSB14	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	36704.24	0.12
MSB14	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	748.76	0.00
MSB14	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	33953.48	0.10
MSB15	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	173306.42	0.58
MSB15	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	222592.61	0.73
MSB15	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	65513.32	0.22
MSB15	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	252.08	0.00
MSB15	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	0.00
MSB15	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	30455.52	9.411E-002
MSB16	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	159739.53	0.52
MSB16	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	139472.65	0.46
MSB16	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	15292.82	7.050E-002
MSB16	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	11305.24	5.734E-002
MSB16	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	18798.53	7.051E-002
MSB16	Span 2	End	DCONU1	2460.30	0.00
MSB17	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	228084.97	0.74
MSB17	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	96110.46	0.34
MSB17	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	116685.73	0.37

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB18	Cantilever Start	Start	Comb=1.2 D + 1.6 L +1.6 Lr	11233.12	5.609E-002
MSB18	Cantilever Start	Middle		0.00	0.00
MSB18	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	60731.97	0.21
MSB18	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	114137.88	0.39
MSB18	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	159999.00	0.54
MSB18	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	114341.93	0.40
MSB18	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	11272.52	7.349E-002
MSB18	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	97084.11	0.32
MSB18	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	358.68	1.898E-002
MSB18	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	27446.89	8.818E-002
MSB18	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	97124.44	0.30
MSB18	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	88657.76	0.28
MSB19	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	48322.94	0.17
MSB19	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	52036.27	0.21
MSB19	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	115918.31	0.40
MSB19	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	115918.31	0.40
MSB19	Span 2	Middle		0.00	0.00
MSB19	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	95105.52	0.34
MSB19	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	95105.52	0.34
MSB19	Span 3	Middle		0.00	0.00
MSB19	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	41038.73	0.16
MSB19	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	41038.73	0.16
MSB19	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	90346.00	0.28
MSB19	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	70608.89	0.22
MSB20	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	11173.12	4.121E-002
MSB20	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	85277.01	0.27
MSB20	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	27212.54	8.861E-002
MSB21	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	14504.20	8.046E-002
MSB21	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	7653.08	4.910E-002
MSB21	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	6734.05	3.176E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Strip	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
MSB22	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3291.00	1.650E-002
MSB22	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	25134.40	0.12
MSB22	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4877.86	2.827E-002
MSB23	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	8273.87	3.918E-002
MSB23	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	49957.69	0.23
MSB23	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4995.82	2.404E-002

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
CSA1	Span 1	Start		0.00	0.00	OK	A
CSA1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2111.53	0.00	OK	A
CSA1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5583.18	0.00	OK	A
CSA1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1560.37	0.00	OK	A
CSA1	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4058.65	0.00	OK	A
CSA1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	3653.82	0.00	OK	A
CSA2	Span 1	Start		0.00	0.00	OK	A
CSA2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3203.79	0.00	OK	A
CSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	6806.22	0.00	OK	A
CSA2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1692.04	0.00	OK	A
CSA2	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	554.70	0.00	OK	A
CSA2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	4519.78	0.00	OK	A
MSA1	Cantilever End	Start		0.00	0.00	OK	A
MSA1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1607.28	0.00	OK	A
MSA1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	3025.63	0.00	OK	A
MSA2	Span 1	Start		0.00	0.00	OK	A
MSA2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	377.52	0.00	OK	A
MSA2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2050.74	0.00	OK	A
MSA3	Span 1	Start		0.00	0.00	OK	A
MSA3	Span 1	Middle	DCONU1	239.11	0.00	OK	A
MSA3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	372.35	0.00	OK	A
MSA3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	7438.84	0.00	OK	A

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSA3	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	13.94	0.00	OK	A
MSA3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	441.29	0.00	OK	A
MSA3	Span 3	Start	DCONU1	366.17	0.00	OK	A
MSA3	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	437.76	0.00	OK	A
MSA3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	2965.56	0.00	OK	A
MSA3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1381.61	0.00	OK	A
MSA3	Span 4	Middle		0.00	0.00	OK	A
MSA3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	13106.55	0.00	OK	A
MSA3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	13106.55	0.00	OK	A
MSA3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3774.89	0.00	OK	A
MSA3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	9327.16	0.00	OK	A
MSA3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4633.54	0.00	OK	A
MSA3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4508.42	0.00	OK	A
MSA3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	7783.52	0.00	OK	A
MSA3	Span 7	Start	Comb=1.2 D + 1.6 L +1.6 Lr	7783.52	0.00	OK	A
MSA3	Span 7	Middle		0.00	0.00	OK	A
MSA3	Span 7	End	Comb=1.2 D + 1.6 L +1.6 Lr	2811.36	0.00	OK	A
MSA4	Cantilever Start	Start		0.00	0.00	OK	A
MSA4	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1273.46	0.00	OK	A
MSA4	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	1282.16	0.00	OK	A
MSA5	Cantilever Start	Start		0.00	0.00	OK	A
MSA5	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	747.25	0.00	OK	A
MSA5	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	438.32	0.00	OK	A
MSA6	Cantilever Start	Start		0.00	0.00	OK	A
MSA6	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1532.38	0.00	OK	A
MSA6	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	3377.07	0.00	OK	A
MSA7	Span 1	Start		0.00	0.00	OK	A
MSA7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	700.92	0.00	OK	A
MSA7	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1260.27	0.00	OK	A
MSA7	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2271.44	0.00	OK	A
MSA7	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3408.26	0.00	OK	A

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSA7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	3556.14	0.00	OK	A
MSA8	Cantilever Start	Start		0.00	0.00	OK	A
MSA8	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	6421.07	0.00	OK	A
MSA8	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	177.93	0.00	OK	A
MSA8	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	999.40	0.00	OK	A
MSA8	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	45.23	0.00	OK	A
MSA8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	350.84	0.00	OK	A
MSA9	Cantilever Start	Start		0.00	0.00	OK	A
MSA9	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2254.74	0.00	OK	A
MSA9	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	6452.73	0.00	OK	A
MSA9	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6452.73	0.00	OK	A
MSA9	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3778.25	0.00	OK	A
MSA9	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4653.05	0.00	OK	A
MSA9	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5681.43	0.00	OK	A
MSA9	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3548.13	0.00	OK	A
MSA9	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	4523.66	0.00	OK	A
MSB1	Span 1	Start		0.00	0.00	OK	B
MSB1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1494.87	0.00	OK	B
MSB1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5265.45	0.00	OK	B
MSB1	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	7800.31	0.00	OK	B
MSB1	Span 2	Middle		0.00	0.00	OK	B
MSB1	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	981.89	0.00	OK	B
MSB1	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	981.89	0.00	OK	B
MSB1	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2493.78	0.00	OK	B
MSB1	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	637.42	0.00	OK	B
MSB1	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	96.85	0.00	OK	B
MSB1	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	419.50	0.00	OK	B
MSB1	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	2285.66	0.00	OK	B
MSB1	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2285.66	0.00	OK	B
MSB1	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3094.60	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB1	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	11887.50	0.00	OK	B
MSB1	Cantilever End	Start	DCONU1	116.48	0.00	OK	B
MSB1	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1309.37	0.00	OK	B
MSB1	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	584.01	0.00	OK	B
MSB2	Span 1	Start		0.00	0.00	OK	B
MSB2	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1737.26	0.00	OK	B
MSB2	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	6196.09	0.00	OK	B
MSB2	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	8292.13	0.00	OK	B
MSB2	Span 2	Middle		0.00	0.00	OK	B
MSB2	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	972.90	0.00	OK	B
MSB2	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	972.90	0.00	OK	B
MSB2	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3098.23	0.00	OK	B
MSB2	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	505.61	0.00	OK	B
MSB2	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	212.02	0.00	OK	B
MSB2	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	671.06	0.00	OK	B
MSB2	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	2256.46	0.00	OK	B
MSB2	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2256.46	0.00	OK	B
MSB2	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3075.89	0.00	OK	B
MSB2	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	6745.22	0.00	OK	B
MSB2	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6745.22	0.00	OK	B
MSB2	Cantilever End	Middle	DCONU1	119.34	0.00	OK	B
MSB2	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	2719.04	0.00	OK	B
MSB3	Span 1	Start		0.00	0.00	OK	B
MSB3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1619.42	0.00	OK	B
MSB3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5602.07	0.00	OK	B
MSB3	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	7931.82	0.00	OK	B
MSB3	Span 2	Middle		0.00	0.00	OK	B
MSB3	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	2796.97	0.00	OK	B
MSB3	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2796.97	0.00	OK	B
MSB3	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2784.33	0.00	OK	B
MSB3	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	903.43	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB3	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	781.31	0.00	OK	B
MSB3	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	245.39	0.00	OK	B
MSB3	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	1150.15	0.00	OK	B
MSB3	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1150.15	0.00	OK	B
MSB3	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1221.85	0.00	OK	B
MSB3	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	3919.31	0.00	OK	B
MSB3	Span 6	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3919.31	0.00	OK	B
MSB3	Span 6	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1343.72	0.00	OK	B
MSB3	Span 6	End	Comb=1.2 D + 1.6 L +1.6 Lr	2855.61	0.00	OK	B
MSB4	Span 1	Start		0.00	0.00	OK	B
MSB4	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3183.73	0.00	OK	B
MSB4	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	909.17	0.00	OK	B
MSB5	Span 1	Start		0.00	0.00	OK	B
MSB5	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	549.08	0.00	OK	B
MSB5	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	3910.59	0.00	OK	B
MSB5	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3756.28	0.00	OK	B
MSB5	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4574.16	0.00	OK	B
MSB5	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	3479.84	0.00	OK	B
MSB5	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3479.84	0.00	OK	B
MSB5	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2140.87	0.00	OK	B
MSB5	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	251.18	0.00	OK	B
MSB6	Span 1	Start		0.00	0.00	OK	B
MSB6	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	769.91	0.00	OK	B
MSB6	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4852.35	0.00	OK	B
MSB6	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	12830.62	0.00	OK	B
MSB6	Span 2	Middle		0.00	0.00	OK	B
MSB6	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	64483.21	0.75	OK	B
MSB6	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	64483.21	0.75	OK	B
MSB6	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	130.72	0.00	OK	B
MSB6	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	6941.25	0.00	OK	B
MSB6	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	16796.50	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB6	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	24514.75	0.14	OK	B
MSB6	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	24513.44	0.00	OK	B
MSB6	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	24513.44	0.00	OK	B
MSB6	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	7158.85	0.00	OK	B
MSB6	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	9116.65	0.00	OK	B
MSB7	Cantilever Start	Start		0.00	0.00	OK	B
MSB7	Cantilever Start	Middle		0.00	0.00	OK	B
MSB7	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	5382.00	0.00	OK	B
MSB7	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5382.00	0.00	OK	B
MSB7	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3269.58	0.00	OK	B
MSB7	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	336.88	0.00	OK	B
MSB7	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	336.88	0.00	OK	B
MSB7	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	614.03	0.00	OK	B
MSB7	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	2217.81	0.00	OK	B
MSB8	Span 1	Start		0.00	0.00	OK	B
MSB8	Span 1	Middle		0.00	0.00	OK	B
MSB8	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4352.87	0.00	OK	B
MSB8	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4352.87	0.00	OK	B
MSB8	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2670.13	0.00	OK	B
MSB8	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	122.06	0.00	OK	B
MSB8	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	122.06	0.00	OK	B
MSB8	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1676.95	0.00	OK	B
MSB8	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	1092.87	0.00	OK	B
MSB9	Span 1	Start		0.00	0.00	OK	B
MSB9	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1550.12	0.00	OK	B
MSB9	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1845.40	0.00	OK	B
MSA10	Span 1	Start		0.00	0.00	OK	A
MSA10	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1328.01	0.00	OK	A
MSA10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2257.11	0.00	OK	A
MSA11	Span 1	Start		0.00	0.00	OK	A
MSA11	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	836.23	0.00	OK	A
MSA11	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	67.61	0.00	OK	A

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSA12	Cantilever End	Start		0.00	0.00	OK	A
MSA12	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	591.98	0.00	OK	A
MSA12	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	4072.66	0.00	OK	A
MSA13	Span 1	Start		0.00	0.00	OK	A
MSA13	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	88.94	0.00	OK	A
MSA13	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	634.69	0.00	OK	A
MSA13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	506.53	0.00	OK	A
MSA13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	384.93	0.00	OK	A
MSA13	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	510.22	0.00	OK	A
MSA14	Cantilever Start	Start		0.00	0.00	OK	A
MSA14	Cantilever Start	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	100.60	0.00	OK	A
MSA14	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	937.42	0.00	OK	A
MSA14	Cantilever End	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1237.37	0.00	OK	A
MSA14	Cantilever End	Middle		0.00	0.00	OK	A
MSA14	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	1261.81	0.00	OK	A
MSB10	Cantilever Start	Start		0.00	0.00	OK	B
MSB10	Cantilever Start	Middle		0.00	0.00	OK	B
MSB10	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	4286.53	0.00	OK	B
MSB10	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4286.53	0.00	OK	B
MSB10	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1010.29	0.00	OK	B
MSB10	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5471.84	0.00	OK	B
MSB10	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5471.84	0.00	OK	B
MSB10	Span 2	Middle		0.00	0.00	OK	B
MSB10	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	3415.41	0.00	OK	B
MSB10	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3415.41	0.00	OK	B
MSB10	Span 3	Middle		0.00	0.00	OK	B
MSB10	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	1470.00	0.00	OK	B
MSB10	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1470.00	0.00	OK	B
MSB10	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	577.80	0.00	OK	B
MSB10	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	19147.20	0.00	OK	B
MSB10	Span 5	Start	Comb=1.2 D + 1.6 L +1.6 Lr	19147.20	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB10	Span 5	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4375.51	0.00	OK	B
MSB10	Span 5	End	Comb=1.2 D + 1.6 L +1.6 Lr	965.77	0.00	OK	B
MSB11	Cantilever End	Start		0.00	0.00	OK	B
MSB11	Cantilever End	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	29.78	0.00	OK	B
MSB11	Cantilever End	End	Comb=1.2 D + 1.6 L +1.6 Lr	532.87	0.00	OK	B
MSB12	Span 1	Start		0.00	0.00	OK	B
MSB12	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	798.50	0.00	OK	B
MSB12	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	959.33	0.00	OK	B
MSB13	Span 1	Start		0.00	0.00	OK	B
MSB13	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4752.42	0.00	OK	B
MSB13	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2533.48	0.00	OK	B
MSB13	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3360.31	0.00	OK	B
MSB13	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4215.26	0.00	OK	B
MSB13	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	1963.69	0.00	OK	B
MSB14	Span 1	Start		0.00	0.00	OK	B
MSB14	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	639.89	0.00	OK	B
MSB14	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	8232.78	0.00	OK	B
MSB14	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	324.73	0.00	OK	B
MSB14	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1983.62	0.00	OK	B
MSB14	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	1983.62	0.00	OK	B
MSB15	Span 1	Start		0.00	0.00	OK	B
MSB15	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3632.78	0.00	OK	B
MSB15	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	3657.27	0.00	OK	B
MSB15	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6030.09	0.00	OK	B
MSB15	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4619.70	0.00	OK	B
MSB15	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	1750.56	0.00	OK	B
MSB16	Span 1	Start		0.00	0.00	OK	B
MSB16	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1233.36	0.00	OK	B
MSB16	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	9084.03	0.00	OK	B
MSB16	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	15190.69	0.00	OK	B
MSB16	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	6628.36	0.00	OK	B
MSB16	Span 2	End	DCONU1	462.95	0.00	OK	B
MSB17	Span 1	Start		0.00	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB17	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3839.59	0.00	OK	B
MSB17	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2486.39	0.00	OK	B
MSB18	Cantilever Start	Start		0.00	0.00	OK	B
MSB18	Cantilever Start	Middle		0.00	0.00	OK	B
MSB18	Cantilever Start	End	Comb=1.2 D + 1.6 L +1.6 Lr	3555.04	0.00	OK	B
MSB18	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3555.04	0.00	OK	B
MSB18	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2171.31	0.00	OK	B
MSB18	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2797.46	0.00	OK	B
MSB18	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	8922.71	0.00	OK	B
MSB18	Span 2	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	8566.62	0.00	OK	B
MSB18	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	6277.46	0.00	OK	B
MSB18	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6277.46	0.00	OK	B
MSB18	Span 3	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1290.58	0.00	OK	B
MSB18	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	1531.65	0.00	OK	B
MSB19	Span 1	Start		0.00	0.00	OK	B
MSB19	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	648.44	0.00	OK	B
MSB19	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	533.53	0.00	OK	B
MSB19	Span 2	Start	Comb=1.2 D + 1.6 L +1.6 Lr	533.53	0.00	OK	B
MSB19	Span 2	Middle		0.00	0.00	OK	B
MSB19	Span 2	End	Comb=1.2 D + 1.6 L +1.6 Lr	1134.40	0.00	OK	B
MSB19	Span 3	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1134.40	0.00	OK	B
MSB19	Span 3	Middle		0.00	0.00	OK	B
MSB19	Span 3	End	Comb=1.2 D + 1.6 L +1.6 Lr	5013.49	0.00	OK	B
MSB19	Span 4	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5013.49	0.00	OK	B
MSB19	Span 4	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	5170.30	0.00	OK	B
MSB19	Span 4	End	Comb=1.2 D + 1.6 L +1.6 Lr	1235.92	0.00	OK	B
MSB20	Span 1	Start		0.00	0.00	OK	B
MSB20	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2094.86	0.00	OK	B
MSB20	Span 1	End	DCONU1	56.61	0.00	OK	B
MSB21	Span 1	Start		0.00	0.00	OK	B
MSB21	Span 1	Middle	DCONU1	442.00	0.00	OK	B
MSB21	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	12438.93	0.00	OK	B
MSB22	Span 1	Start		0.00	0.00	OK	B

Table 45: Concrete Slab Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Strip	SpanID	Location	VCombo	VForce	VArea	Status	Layer
				lb	in2/ft		
MSB22	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	558.57	0.00	OK	B
MSB22	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	876.56	0.00	OK	B
MSB23	Span 1	Start		0.00	0.00	OK	B
MSB23	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	527.03	0.00	OK	B
MSB23	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1059.12	0.00	OK	B

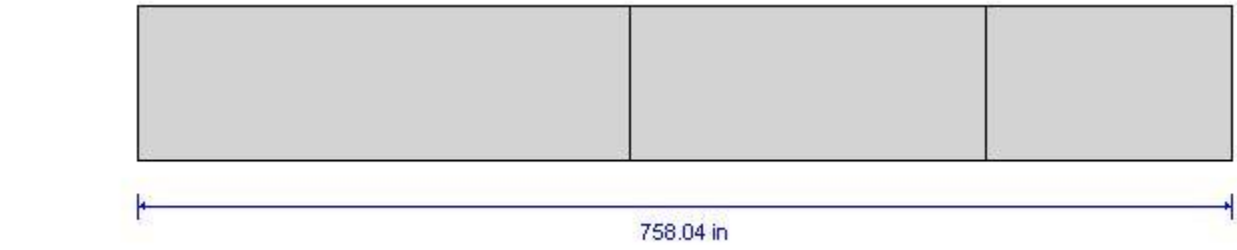
ACI 318-14 Concrete Strip Design

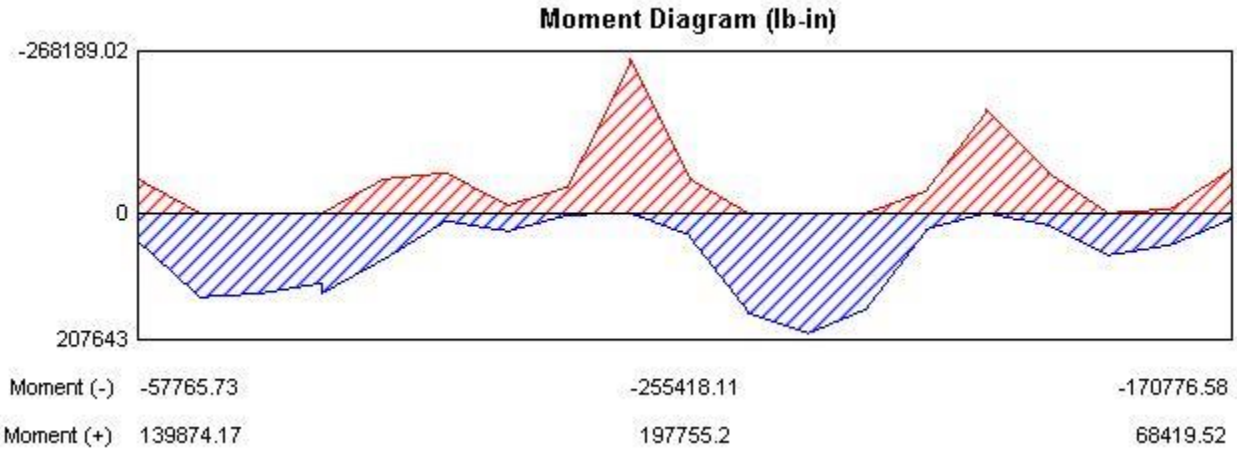
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA1
Length = 758.04 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

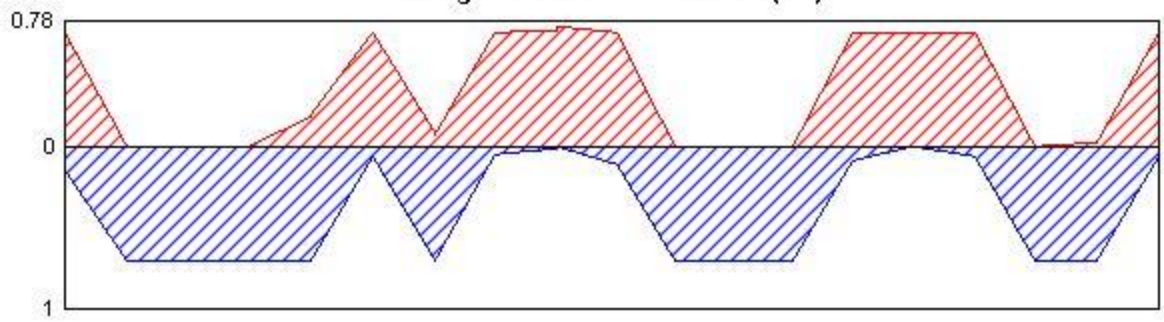
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





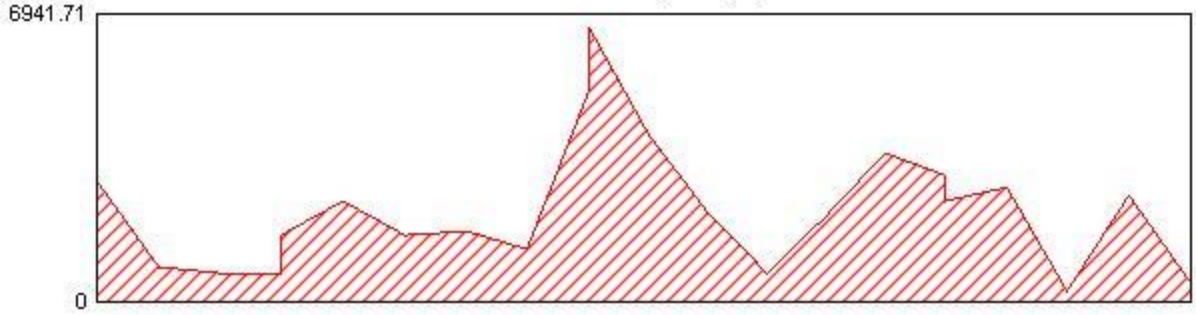
Moment (-)	-57765.73	-255418.11	-170776.58
Moment (+)	139874.17	197755.2	68419.52

Longitudinal Reinforcement (in2)



As (top)	0.2	0.74	0.5
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.43	0.57	0.2
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	1607.28	3025.63

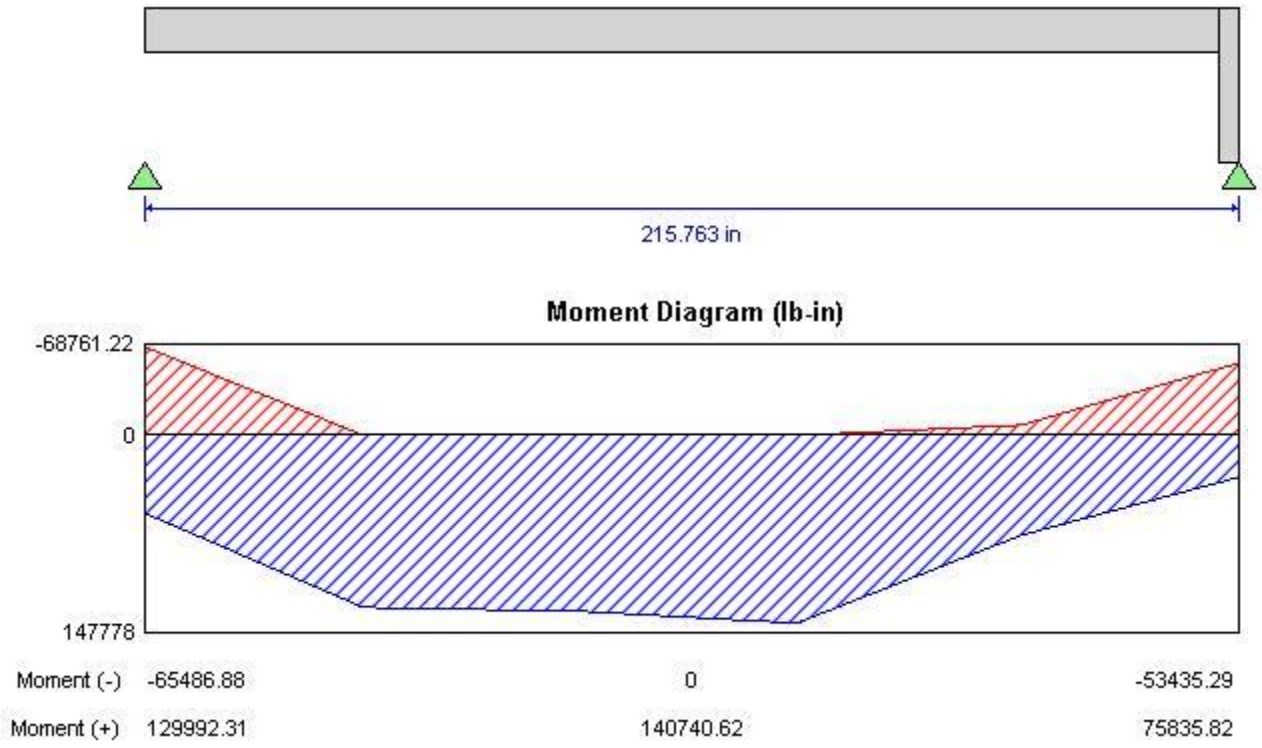
ACI 318-14 Concrete Strip Design

Geometric Properties

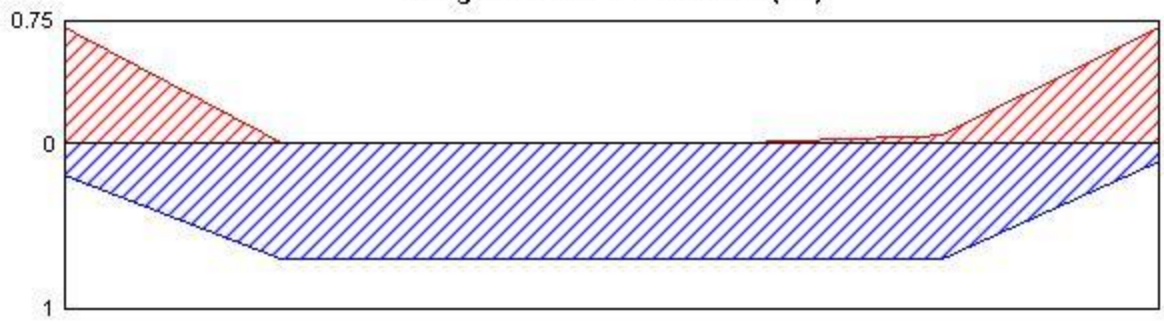
Combination = Overall Envelope
Strip Label = MSA2
Length = 215.763 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2

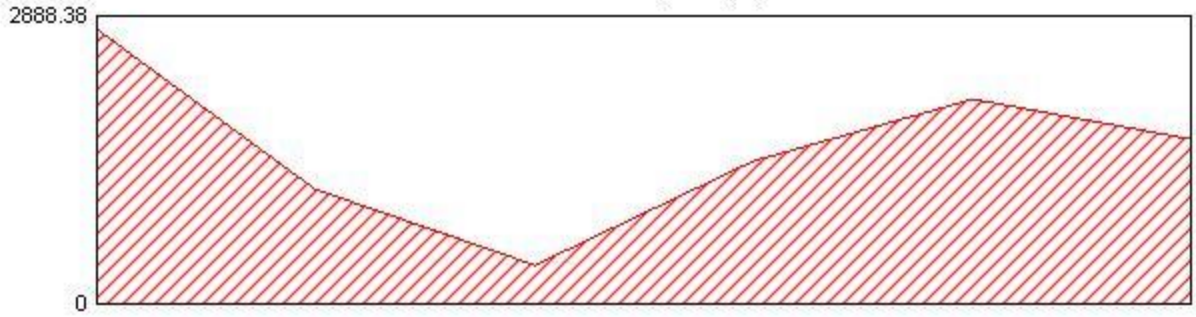


Longitudinal Reinforcement (in2)



As (top)	0.22	0	0.17
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.41	0.44	0.25
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	377.52	2050.74

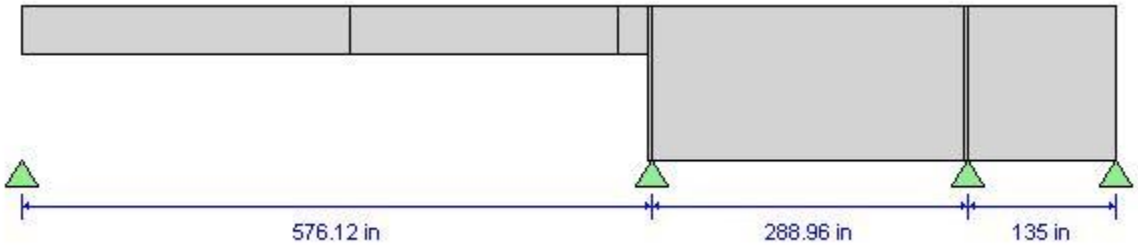
ACI 318-14 Concrete Strip Design

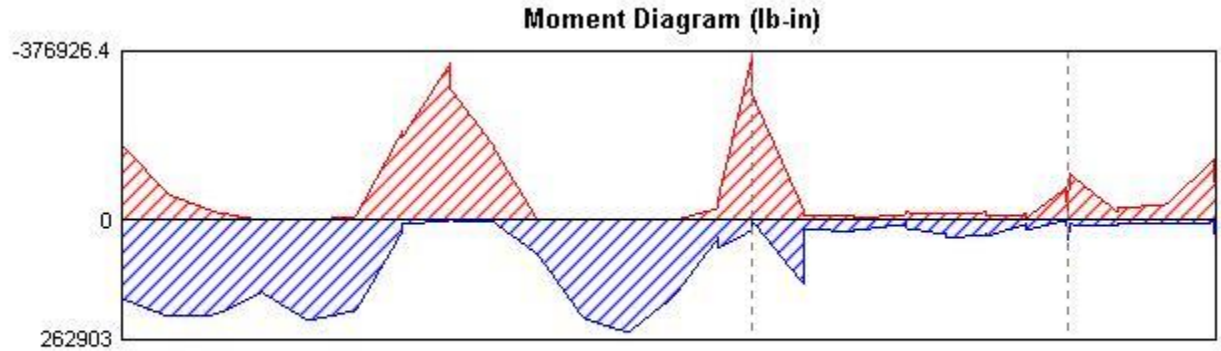
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA3
Length = 1289.28 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

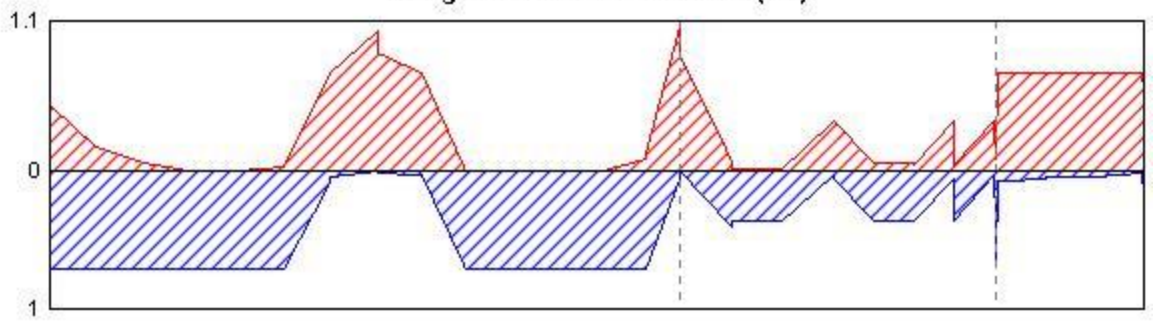
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





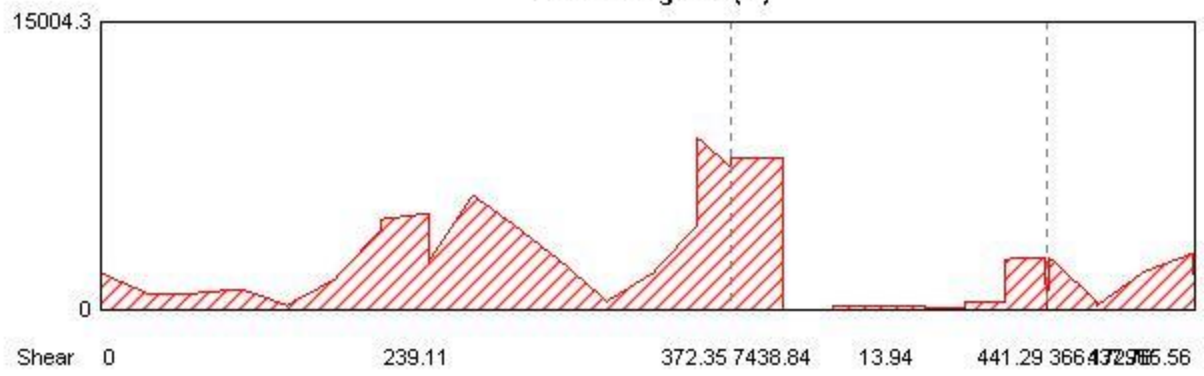
Moment (-)	-167340.89	-344536.41	-358977.52	-358977.52	17158.29	-72122.11	-104[illegible]2.61
Moment (+)	213751.48	224089.06	250383.47	141641.08	37965.72	44474.79	444[illegible]9.17

Longitudinal Reinforcement (in2)

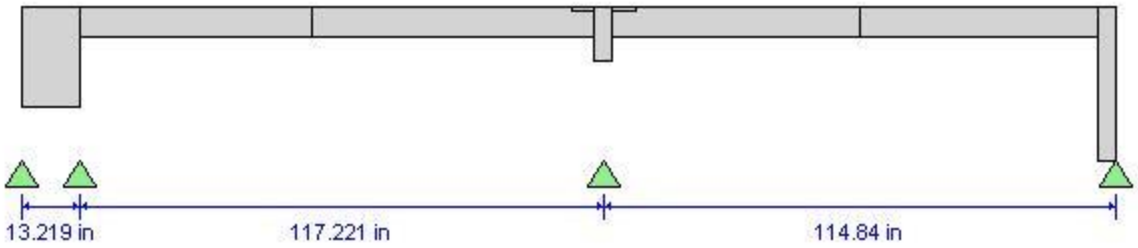


As (top)	0.48	1.02	1.05	1.05	6.528E-002	0.25	0.34	0.11	0.39
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr						
As (bot)	0.62	0.65	0.72	0.41	0.13	0.17	0.5	7.941E-002	0.25
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr						

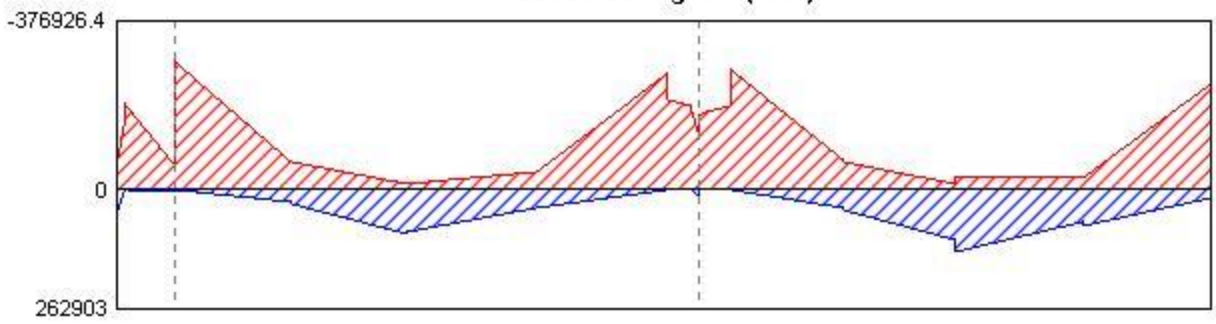
Shear Diagram (lb)



Shear	0	239.11	372.35	7438.84	13.94	441.29	366	437	5.56

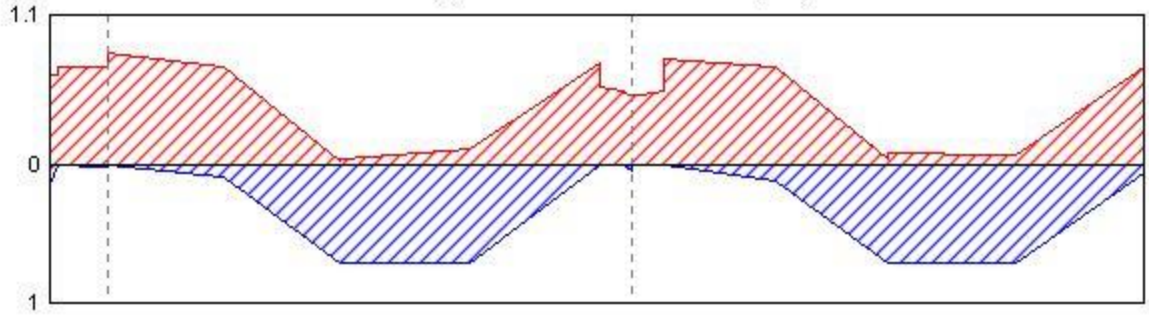


Moment Diagram (lb-in)



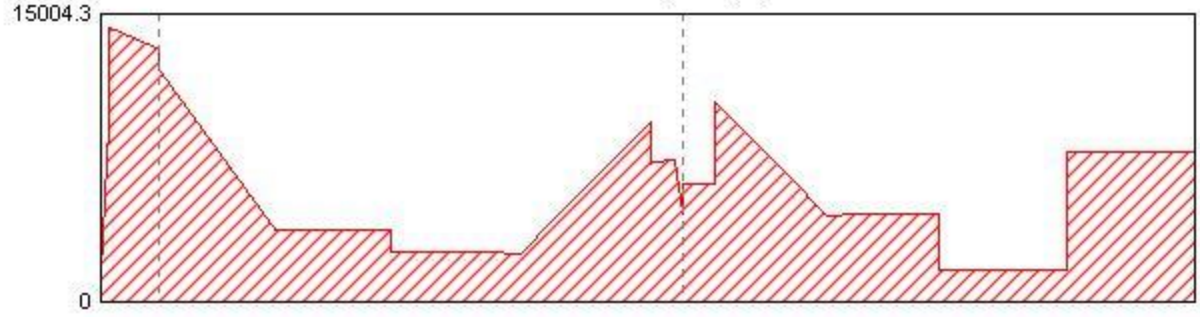
Moment (-)	[illegible]	-38166.61	-257322.24	-268243.17	-64876.57	-236713.01
Moment (+)	[illegible]	99830.11	14455.87	14455.87	137706.51	19572.46

Longitudinal Reinforcement (in2)

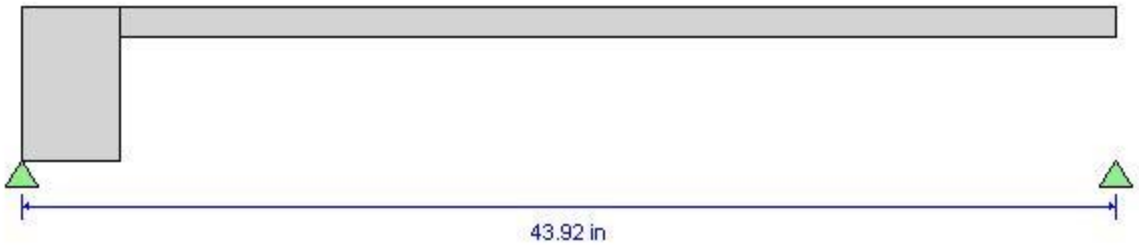


As (top)	[illegible]	0.11	0.74	0.78	0.18	0.68
	1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr
As (bot)	[illegible]	8.37E-002	0.29	4.108E-002	4.108E-002	0.39	5.562E-002
	1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr	Comb=1.2 D + 1.6 L + 1.6 Lr

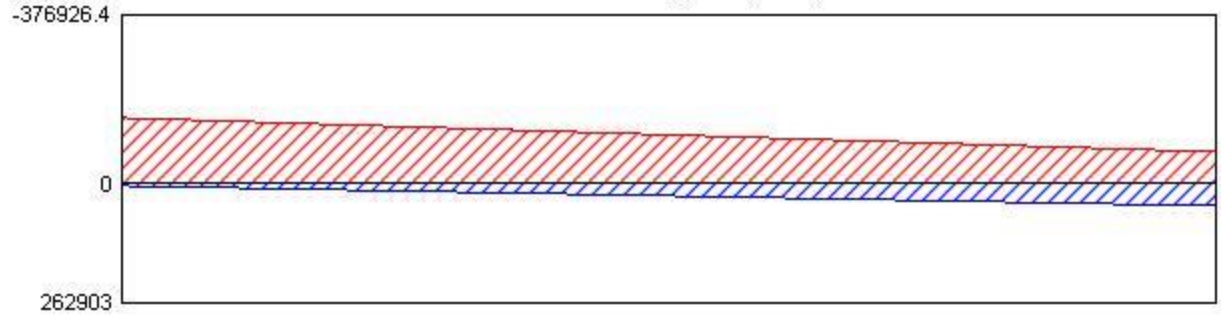
Shear Diagram (lb)



Shear	[illegible]	3774.89	9327.16	4633.54	4508.42	7783.52

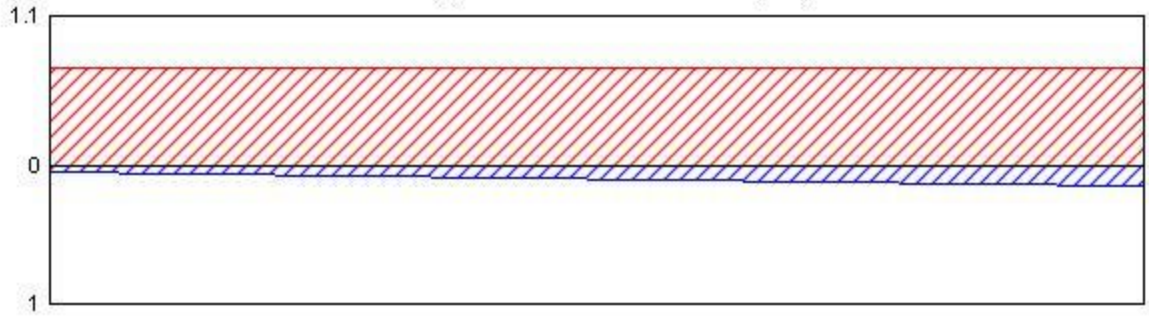


Moment Diagram (lb-in)

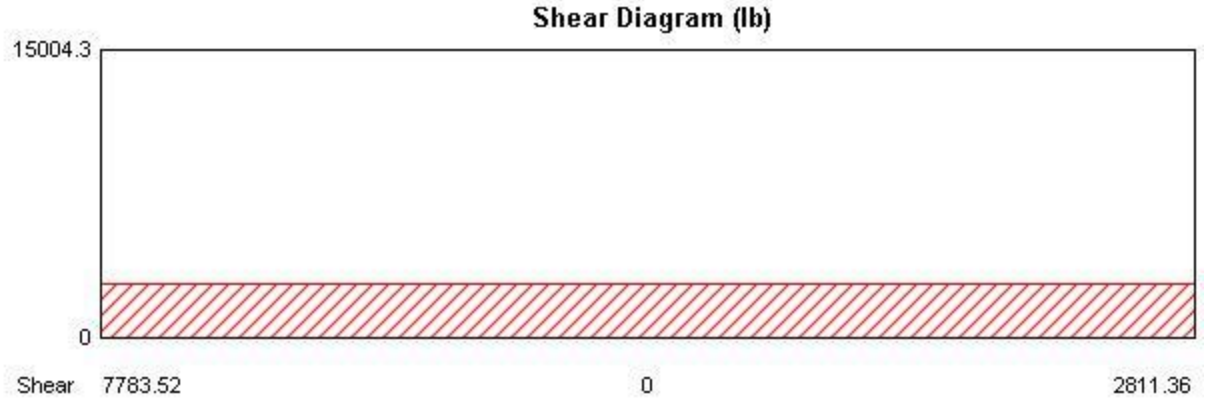


Moment (-)	-236713.01	0	-70250.58
Moment (+)	19572.46	0	48288.73

Longitudinal Reinforcement (in2)



As (top)	0.68	0	0.2
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	5.562E-002	0	0.14
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr


Shear Diagram (lb)
15004.3
0
Shear
7783.52
0
2811.36

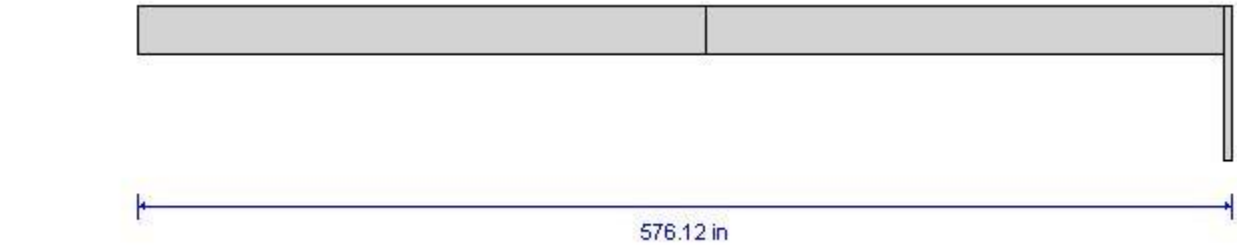
ACI 318-14 Concrete Strip Design

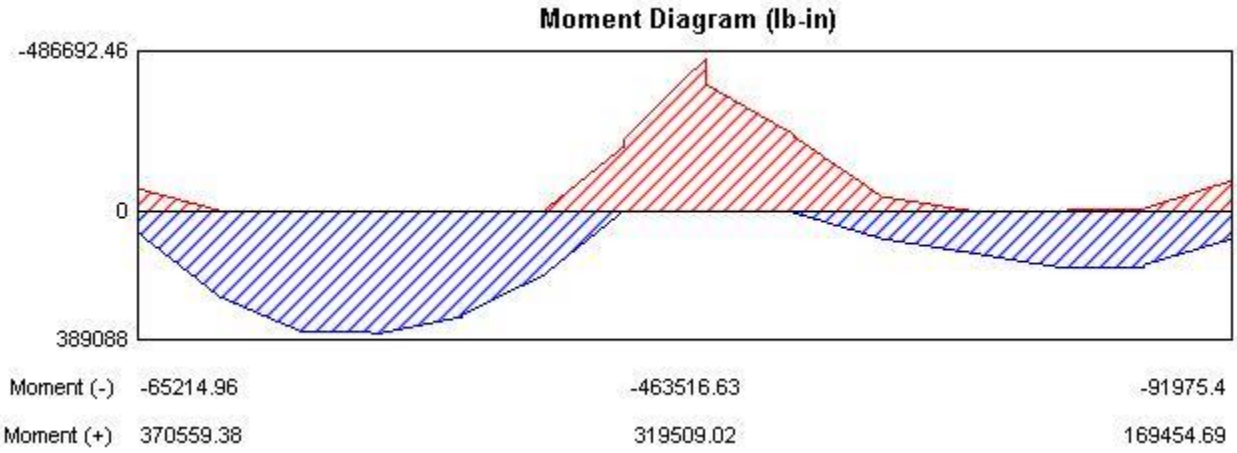
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA4
Length = 576.12 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

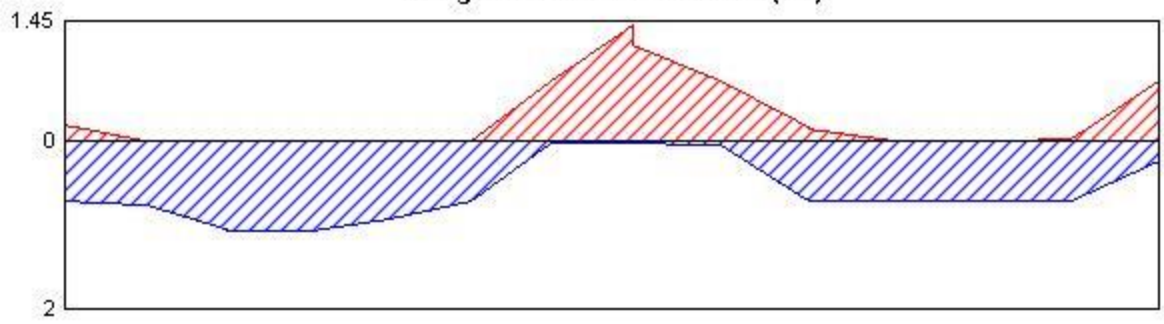
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





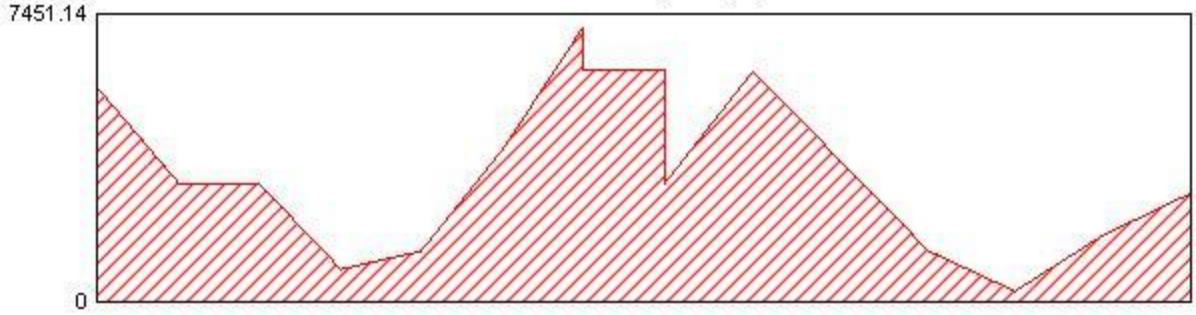
Moment (-)	-65214.96	-463516.63	-91975.4
Moment (+)	370559.38	319509.02	169454.69

Longitudinal Reinforcement (in2)



As (top)	0.19	1.38	0.26
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	1.09	0.94	0.52
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	1273.46	1282.16

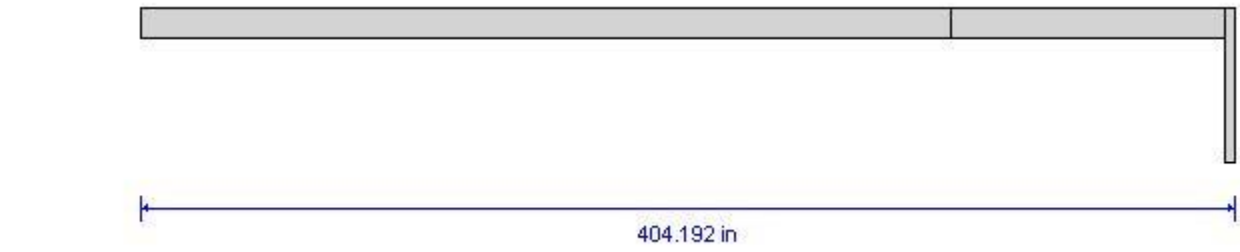
ACI 318-14 Concrete Strip Design

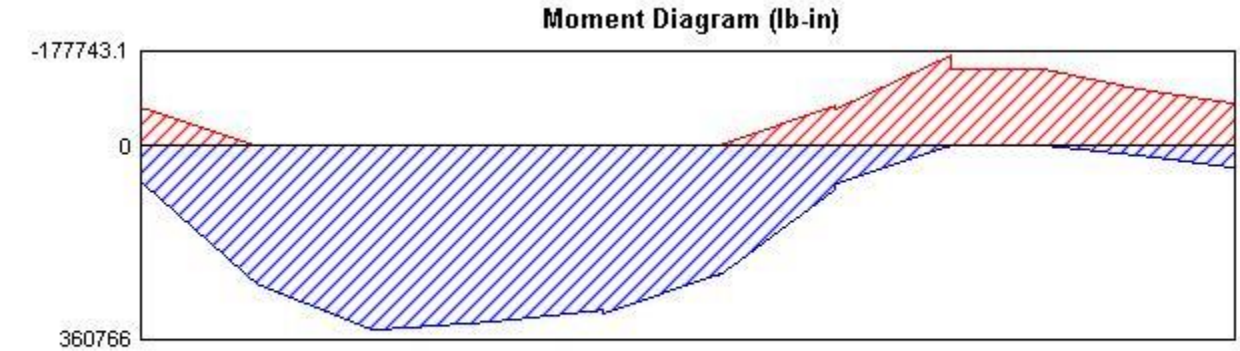
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA5
Length = 404.192 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

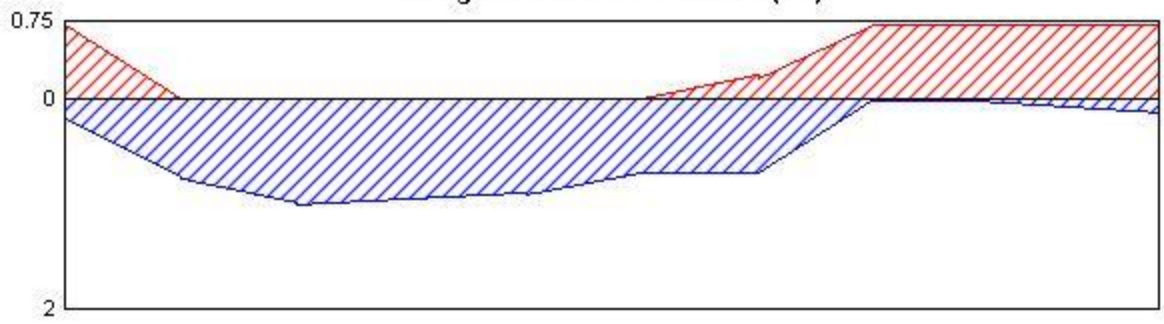
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





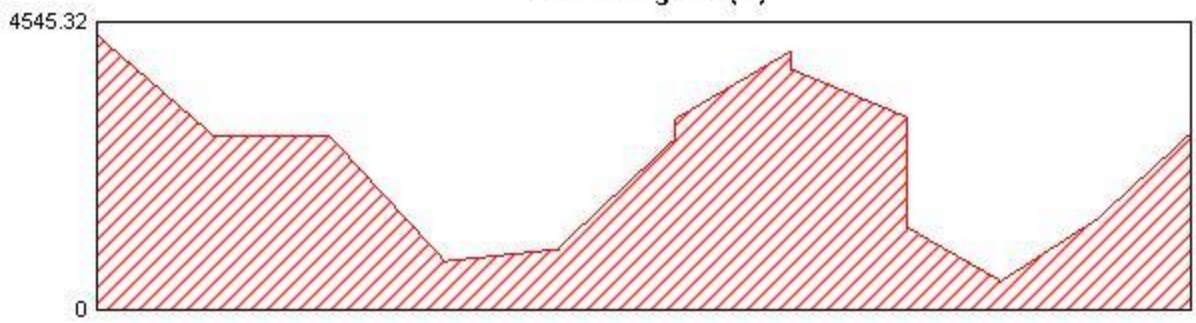
Moment (-)	-71655.77	-169279.14	-143906.07
Moment (+)	343586.48	328542.43	41268.15

Longitudinal Reinforcement (in2)



As (top)	0.21	0.5	0.44
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	1.01	0.96	0.13
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	747.25	438.32

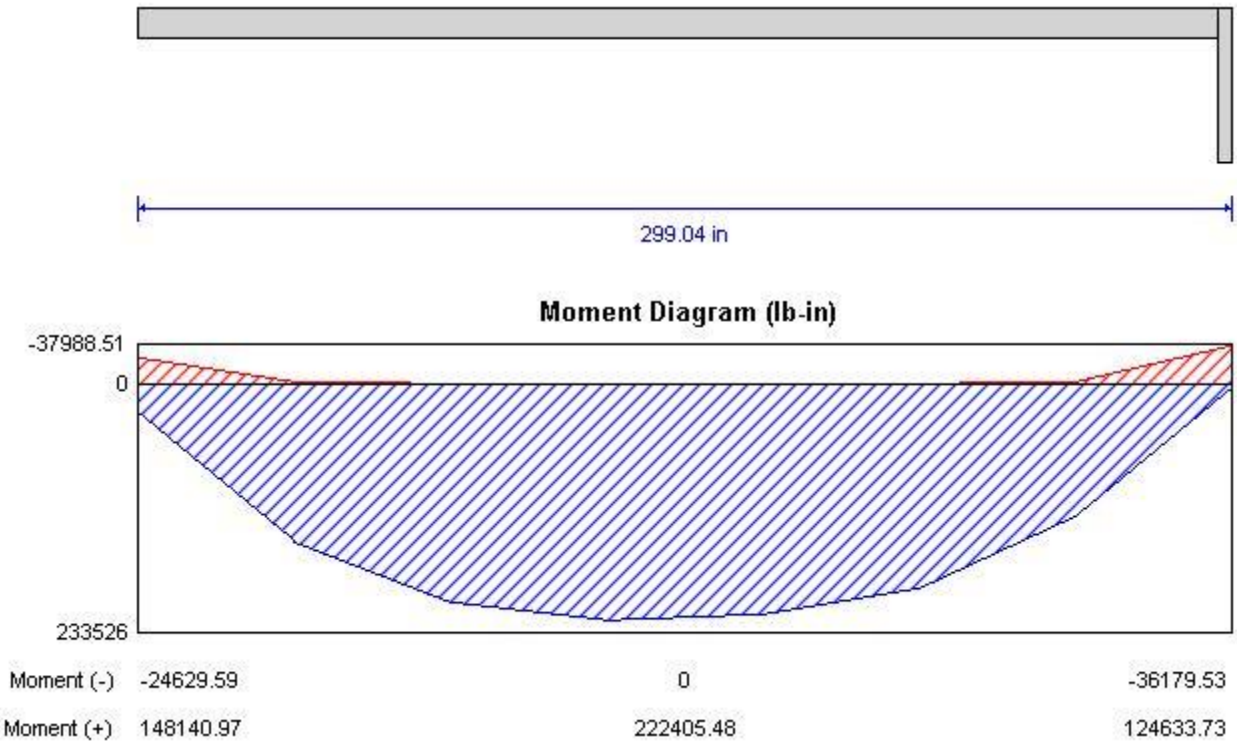
ACI 318-14 Concrete Strip Design

Geometric Properties

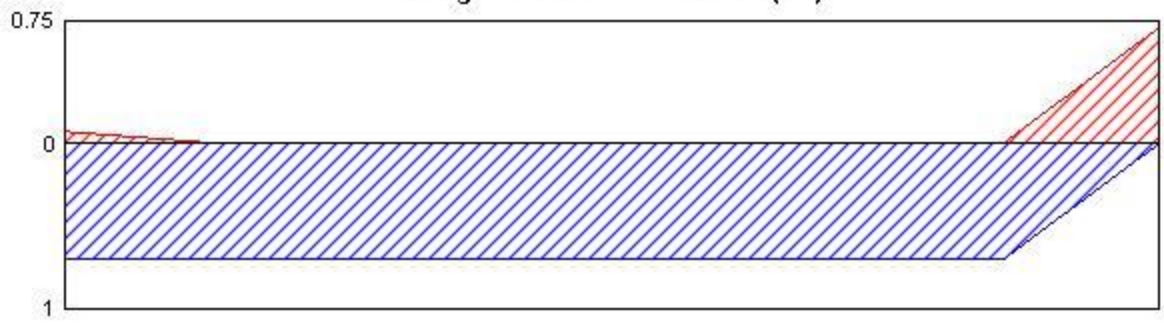
Combination = Overall Envelope
Strip Label = MSA6
Length = 299.04 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2

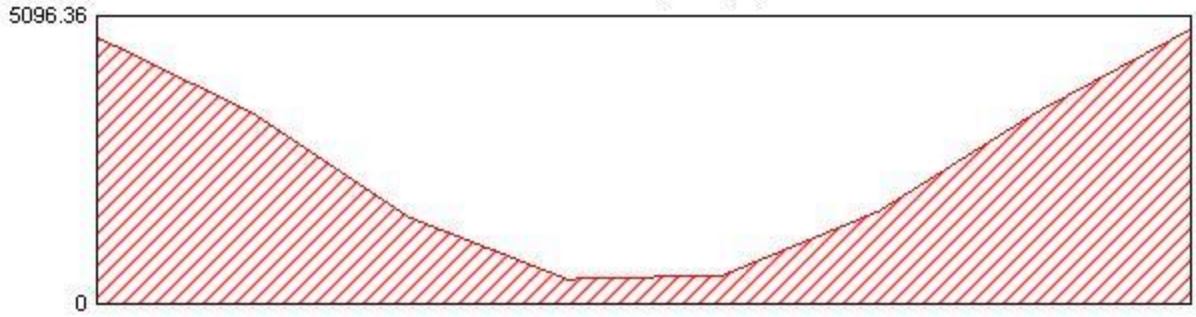


Longitudinal Reinforcement (in2)



As (top)	7.71E-002	0	0.1
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.43	0.64	0.36
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	1532.38	3377.07

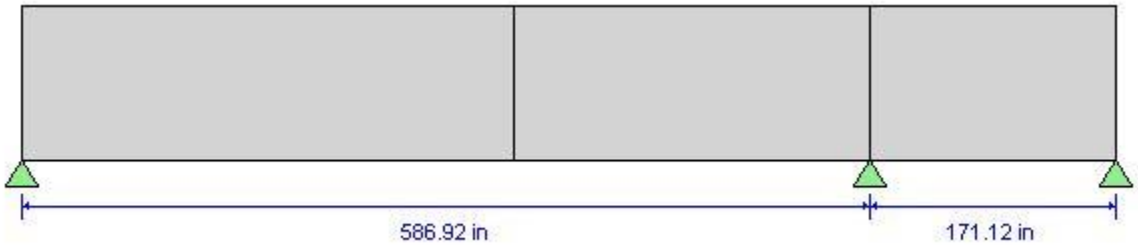
ACI 318-14 Concrete Strip Design

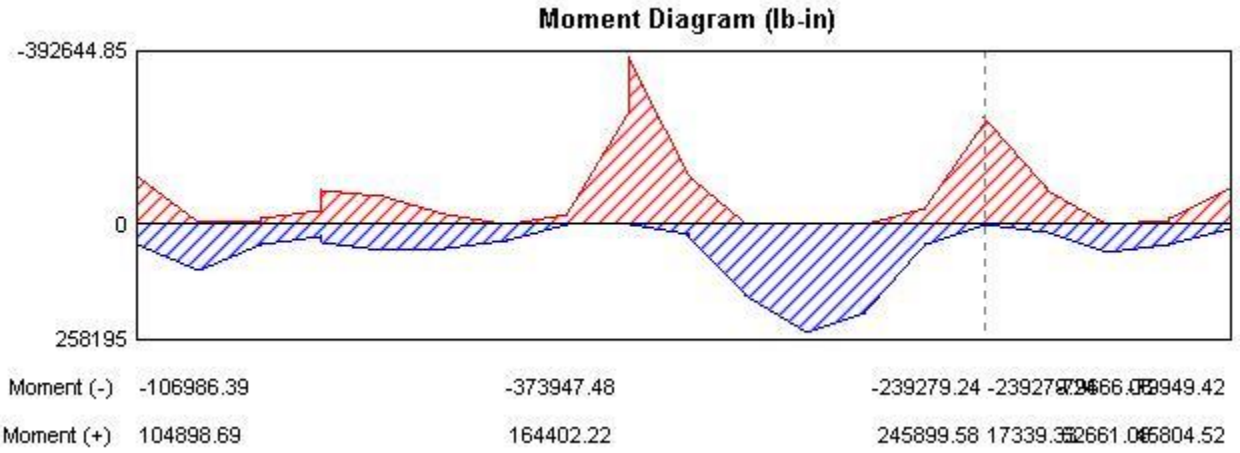
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA7
Length = 758.04 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

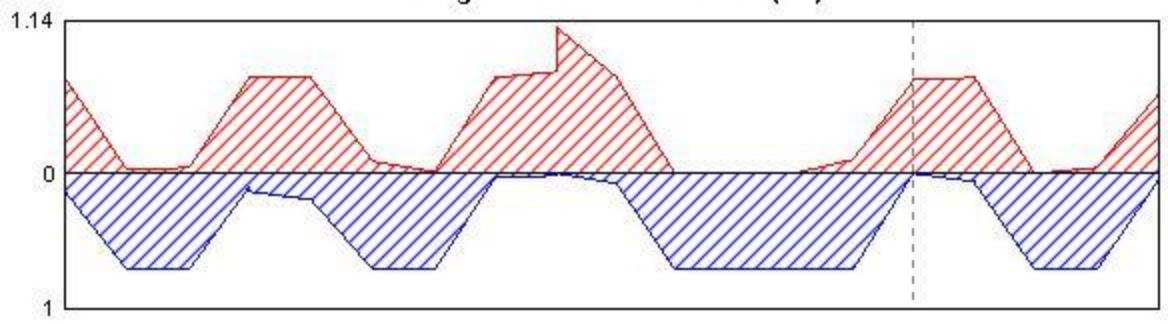
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





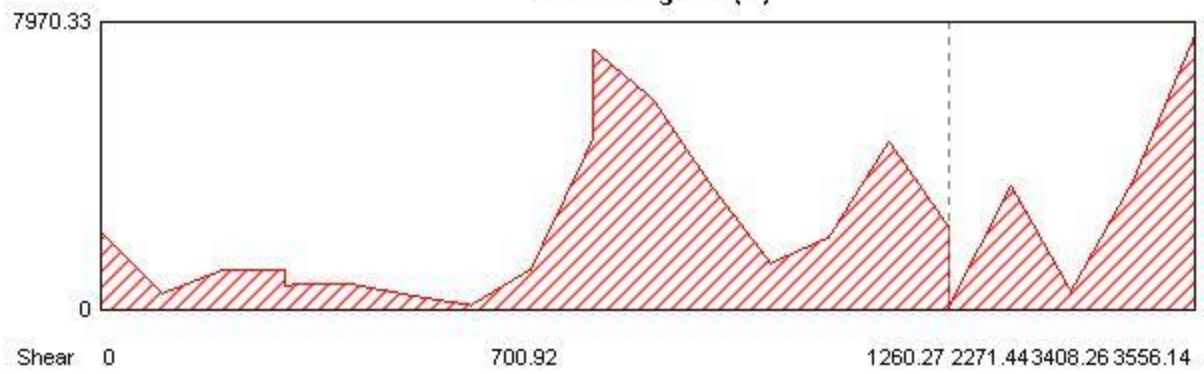
Moment (-)	-106986.39	-373947.48	-239279.24 -239278	66.0 949.42
Moment (+)	104898.69	164402.22	245899.58 17339.3	2661.0 65804.52

Longitudinal Reinforcement (in2)



As (top)	0.32	1.09	0.7	0.7	0.25	0.25
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 L			
As (bot)	0.32	0.47	0.71	6.816E-00	2.2	0.14
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 L			

Shear Diagram (lb)



Shear	0	700.92	1260.27	2271.44	3408.26	3556.14

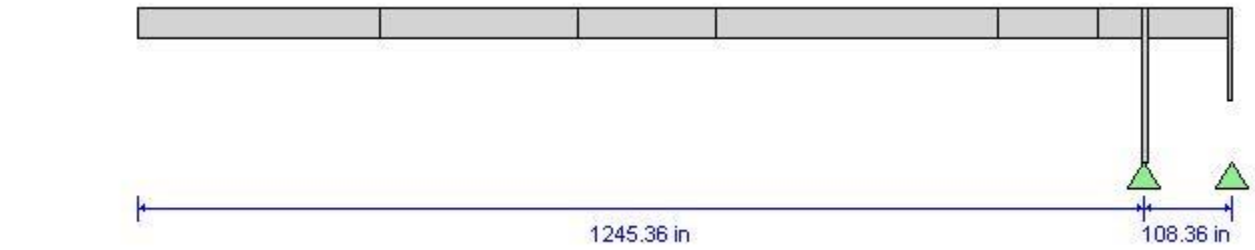
ACI 318-14 Concrete Strip Design

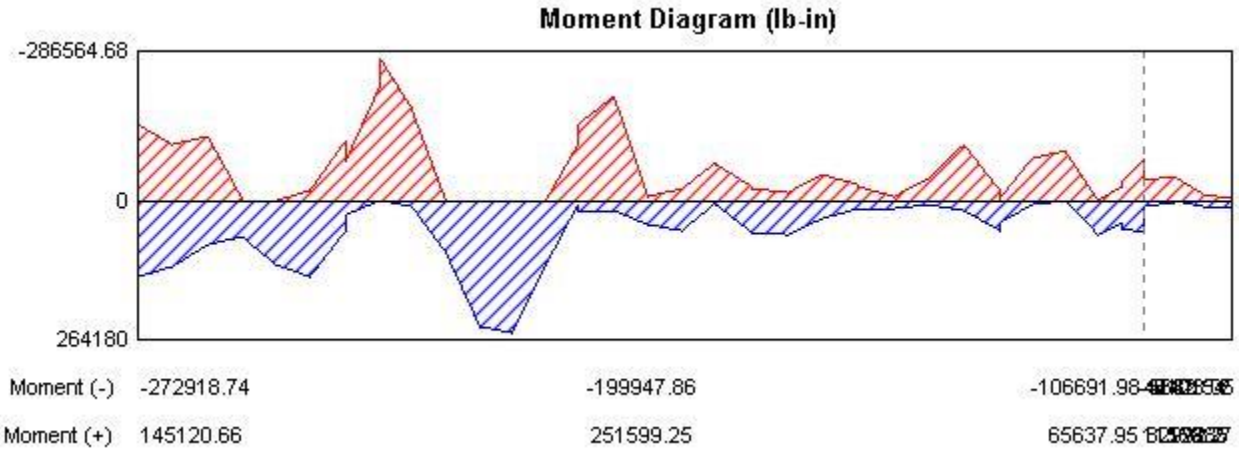
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA8
Length = 1353.72 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

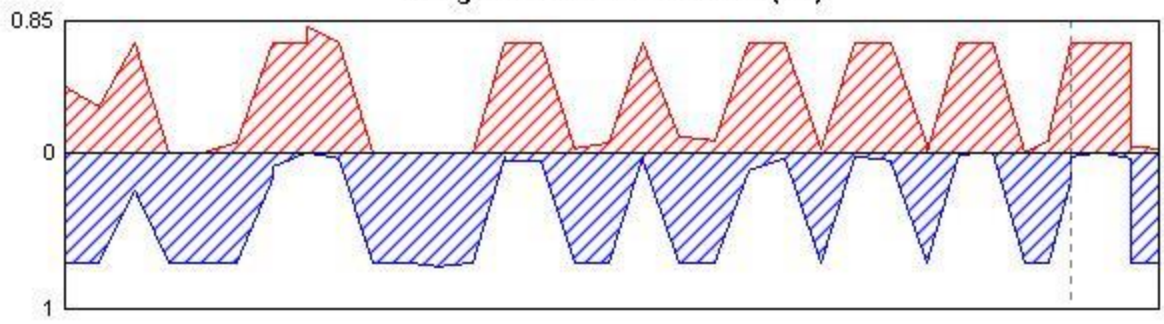
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2



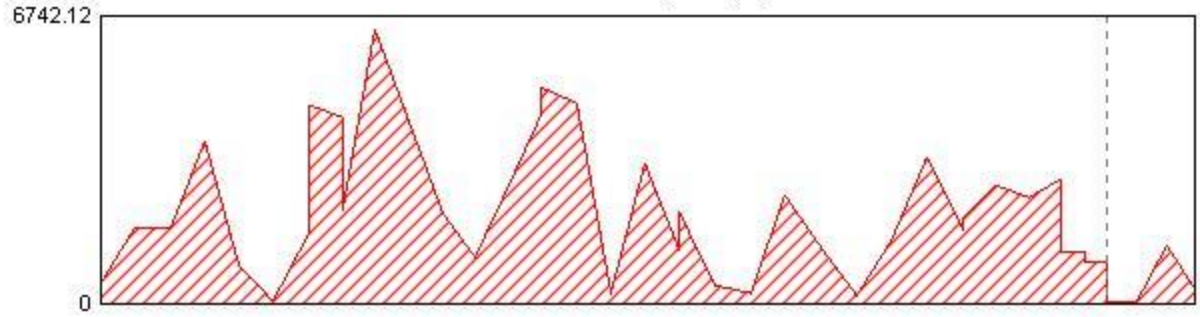


Longitudinal Reinforcement (in2)



As (top)	0.81	0.58	0.3
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2
As (bot)	0.42	0.73	0.
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2

Shear Diagram (lb)



Shear	0	6421.07	177.93

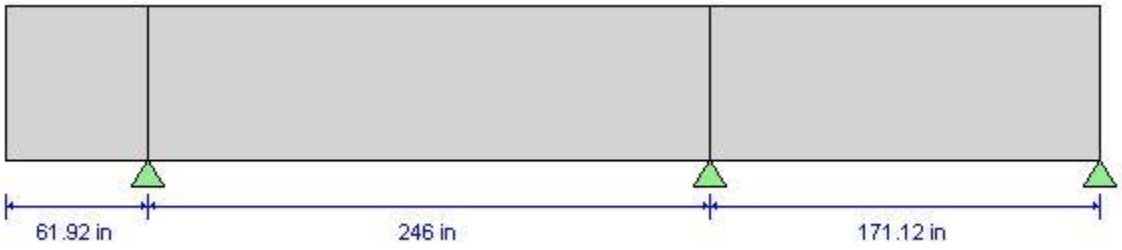
ACI 318-14 Concrete Strip Design

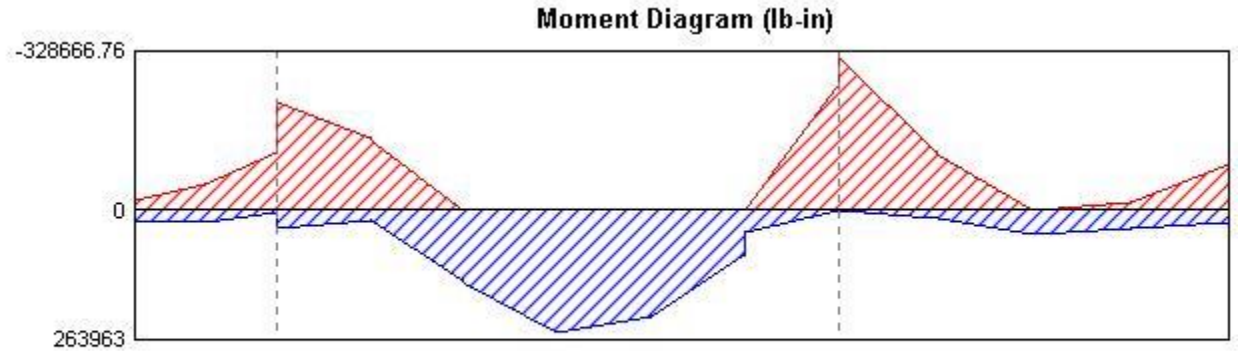
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA9
Length = 479.04 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

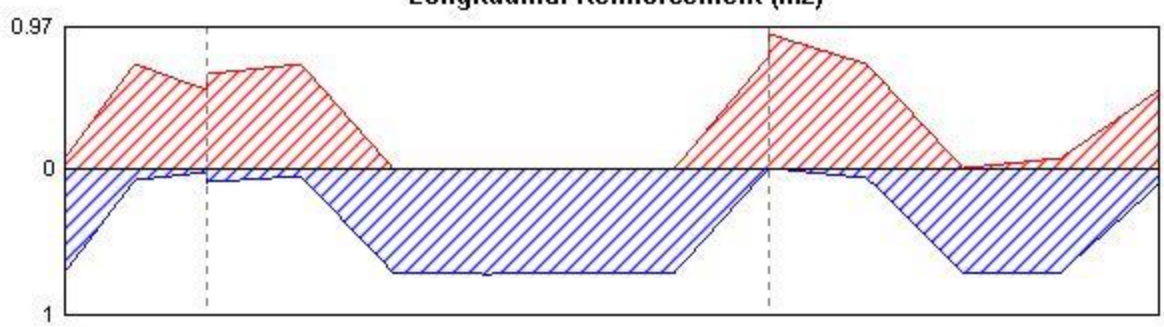
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





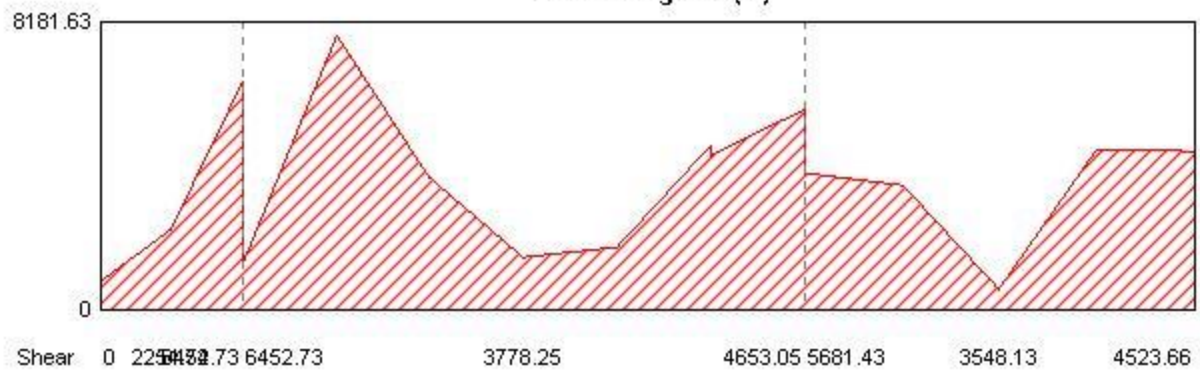
Moment (-)	-19[illegible].27	-221372.27	0	-313015.96	-313015.96	-117862.87	-96538.07
Moment (+)	22[illegible]83	35613.83	251392.72	90744.83	18370.5	50626.56	36246.42

Longitudinal Reinforcement (in2)



As (top)	7.048E-002	0.64	0	0.92	0.92	0.35	0.32
Comb	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr		
As (bot)	8.160E-002	0.1	0.73	0.26	6.435E-002	0.16	0.13
Comb	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	0	2254.73	6452.73	3778.25	4653.05	5681.43	3548.13	4523.66

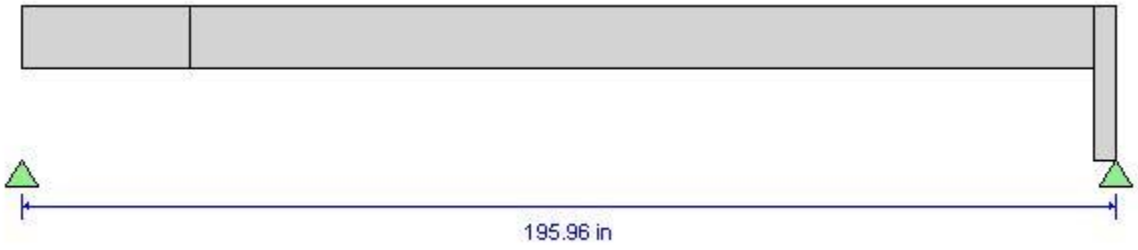
ACI 318-14 Concrete Strip Design

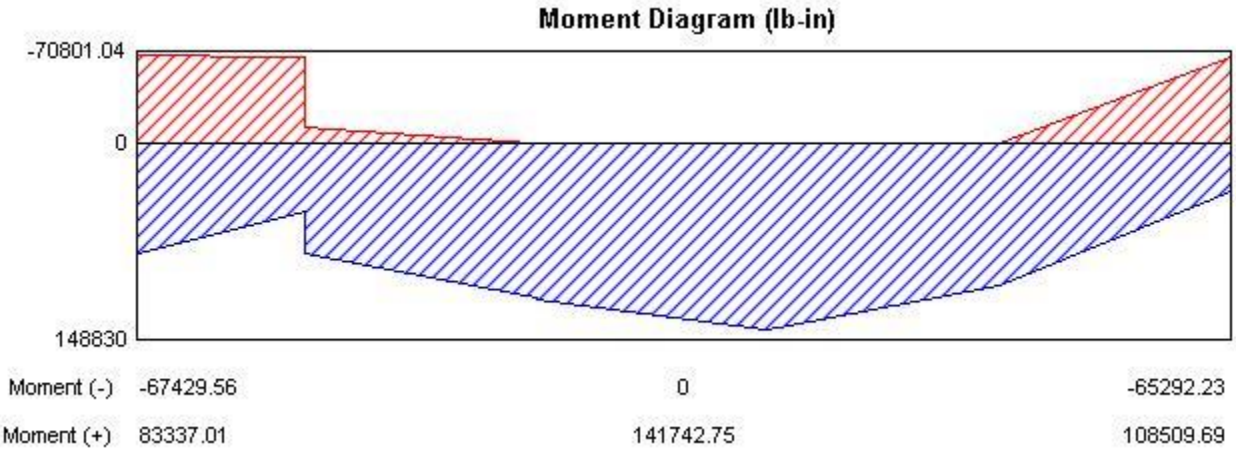
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA10
Length = 195.96 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

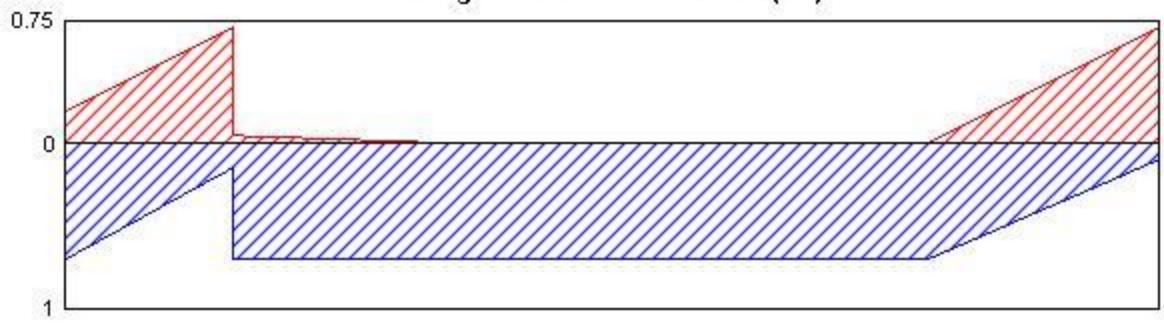
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





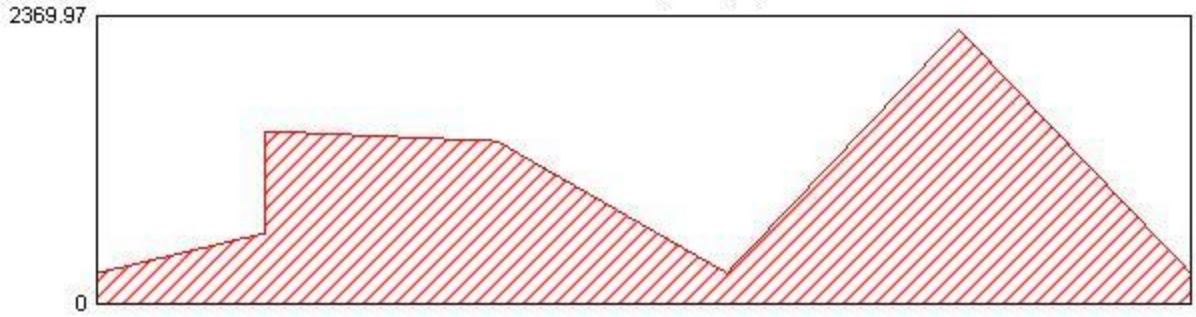
Moment (-)	-67429.56	0	-65292.23
Moment (+)	83337.01	141742.75	108509.69

Longitudinal Reinforcement (in2)



As (top)	0.2	0	0.19
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.25	0.41	0.31
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	1328.01	2257.11

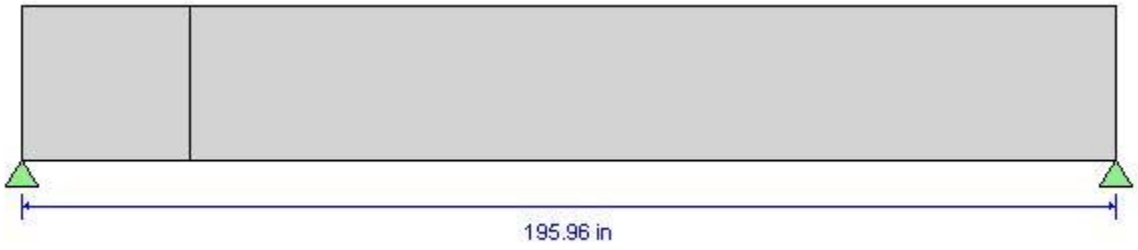
ACI 318-14 Concrete Strip Design

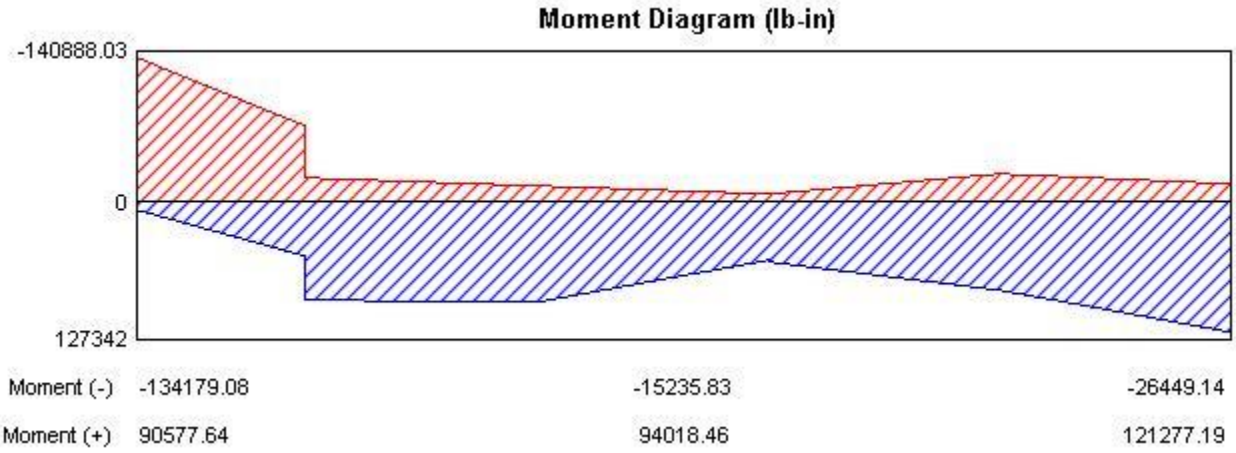
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA11
Length = 195.96 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

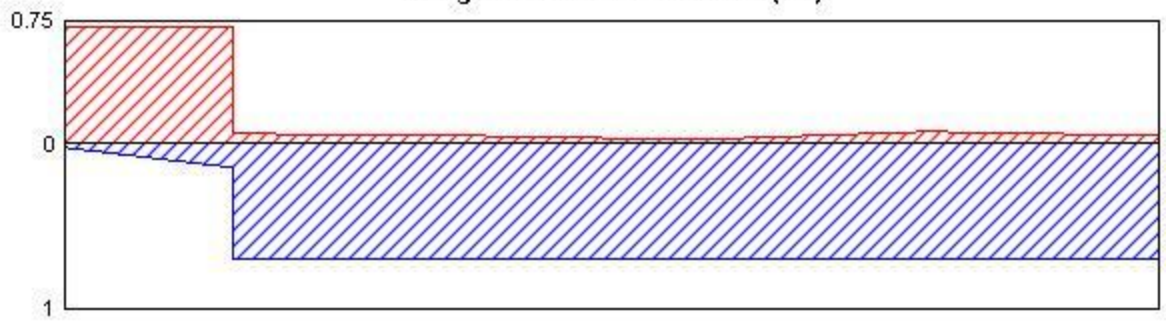
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





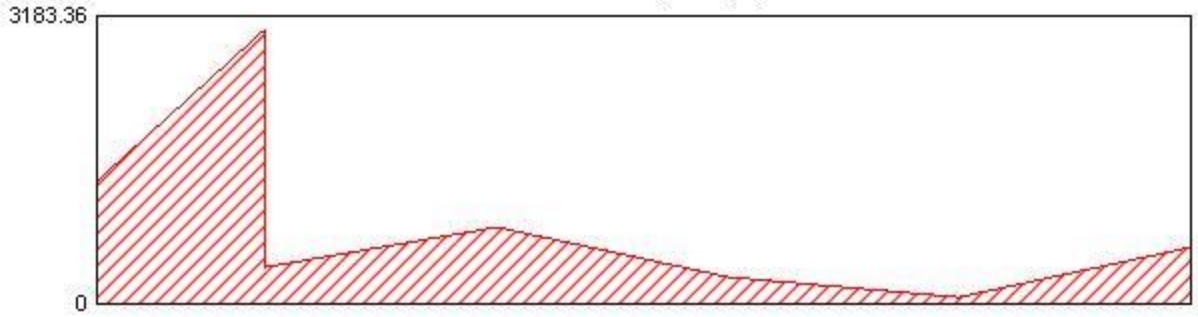
Moment (-)	-134179.08	-15235.83	-26449.14
Moment (+)	90577.64	94018.46	121277.19

Longitudinal Reinforcement (in2)



As (top)	0.39	4.328E-002	7.519E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.26	0.27	0.35
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	836.23	67.61

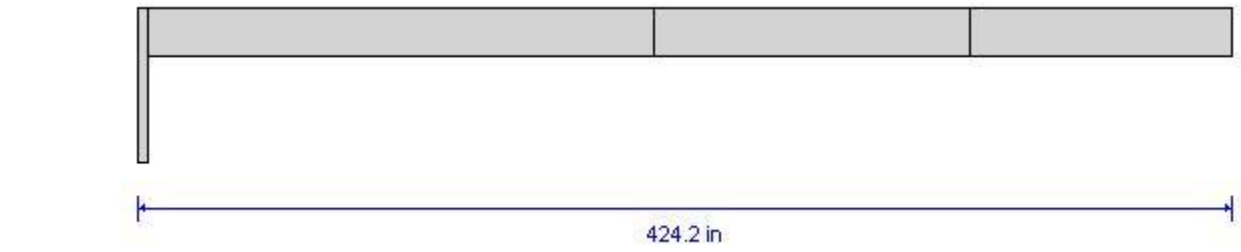
ACI 318-14 Concrete Strip Design

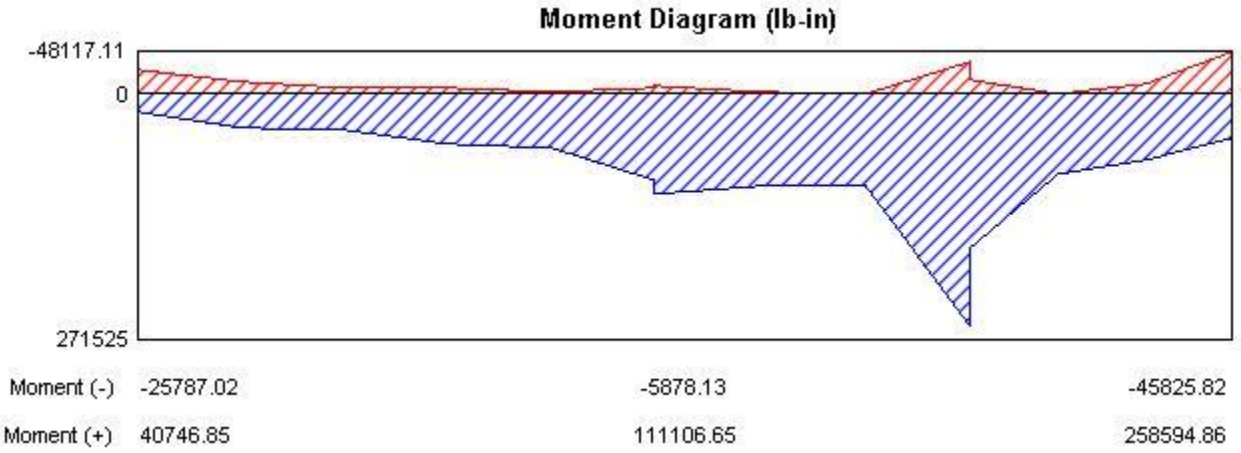
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA12
Length = 424.2 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

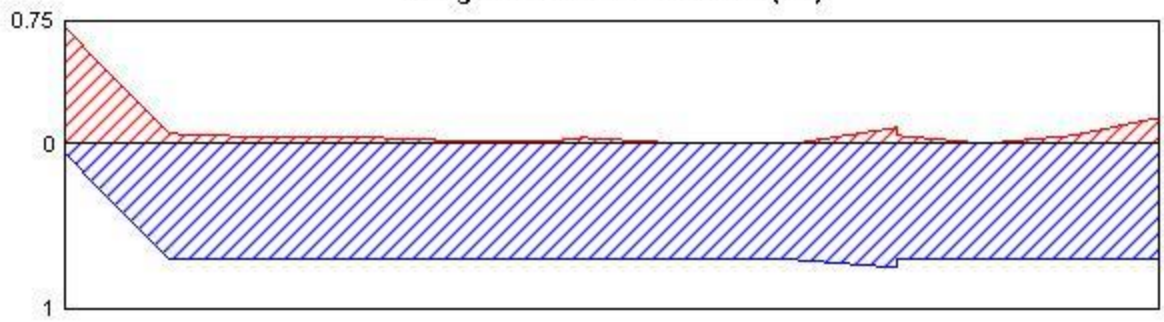
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





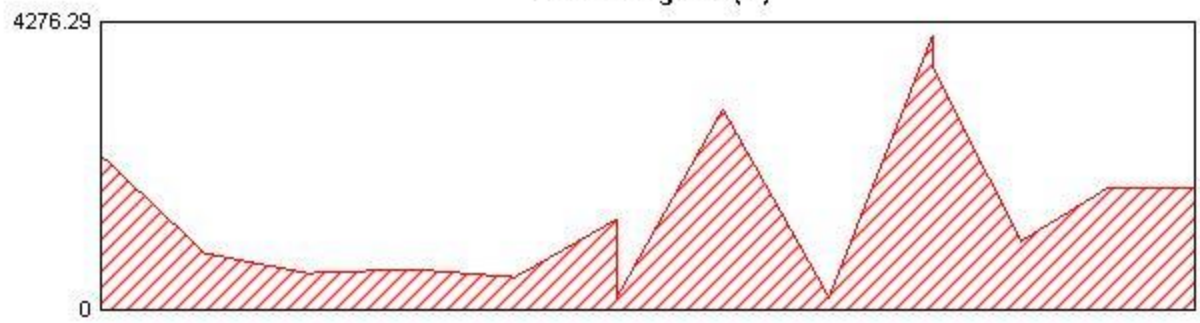
Moment (-)	-25787.02	-5878.13	-45825.82
Moment (+)	40746.85	111106.65	258594.86

Longitudinal Reinforcement (in2)



As (top)	8.486E-002	3.567E-002	0.16
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.14	0.32	0.75
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	591.98	4072.66

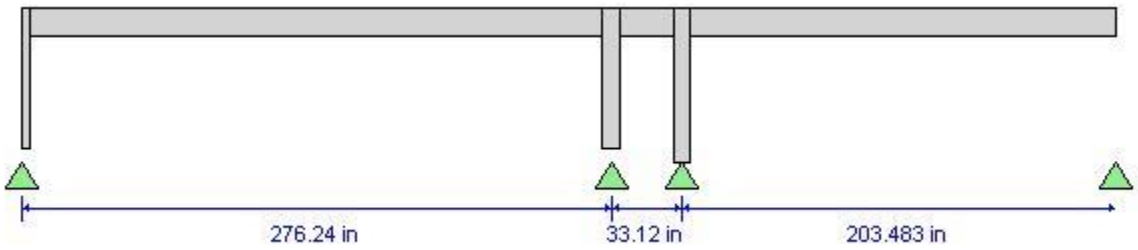
ACI 318-14 Concrete Strip Design

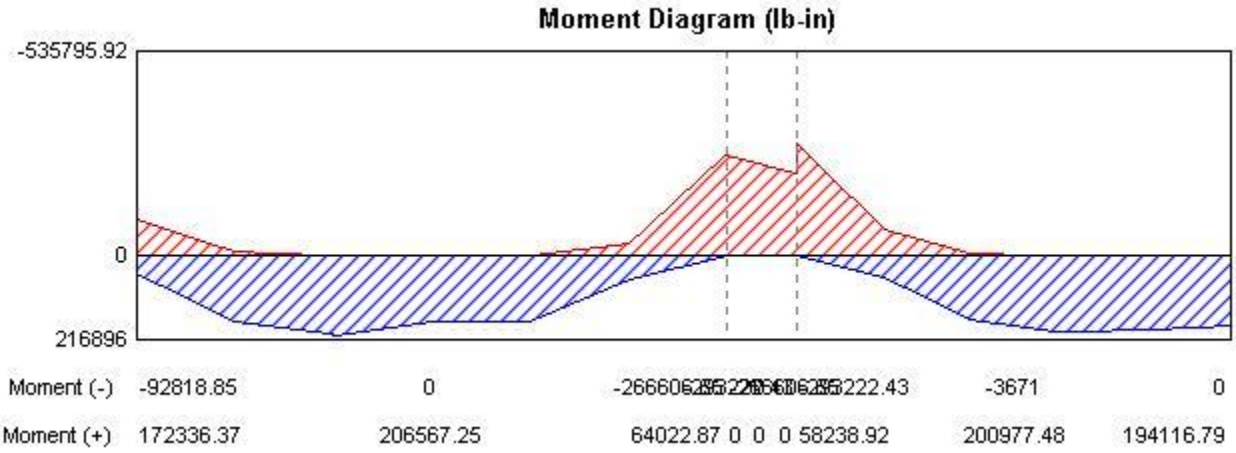
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB1
Length = 1038.12 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

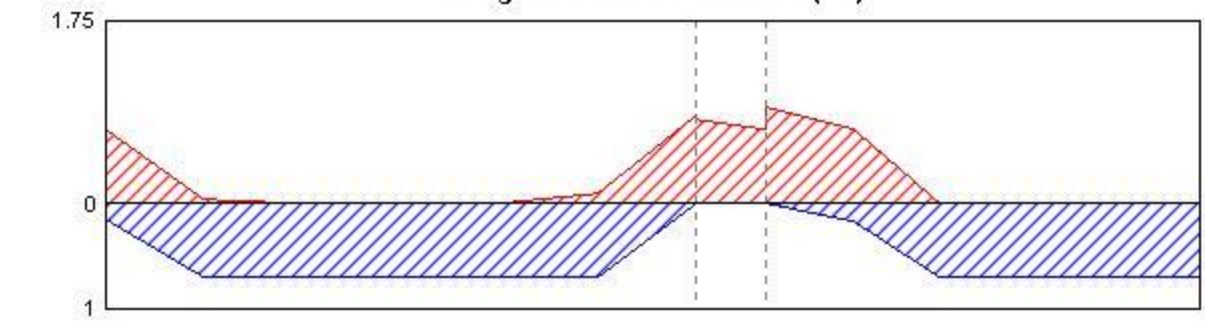
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





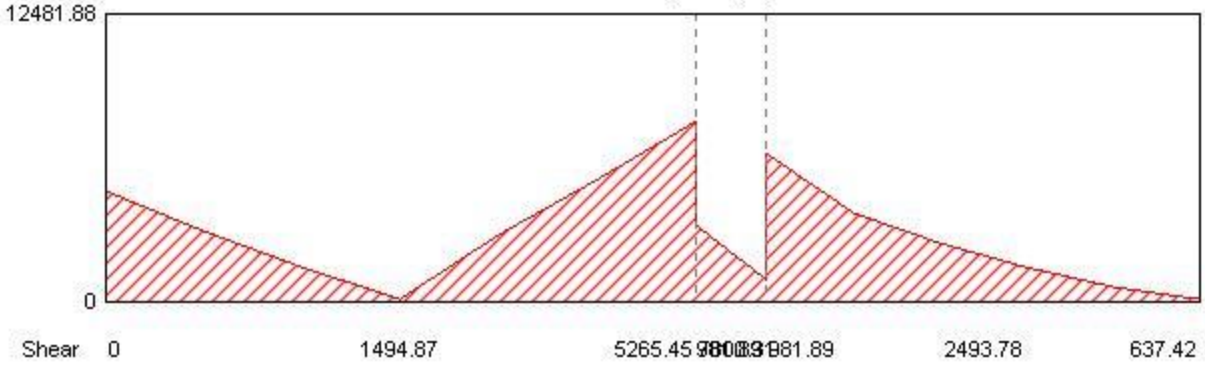
Moment (-)	-92818.85	0	-266606[illegible]222.43	-3671	0	
Moment (+)	172336.37	206567.25	64022.87 0 0 0 58238.92	200977.48	194116.79	

Longitudinal Reinforcement (in2)

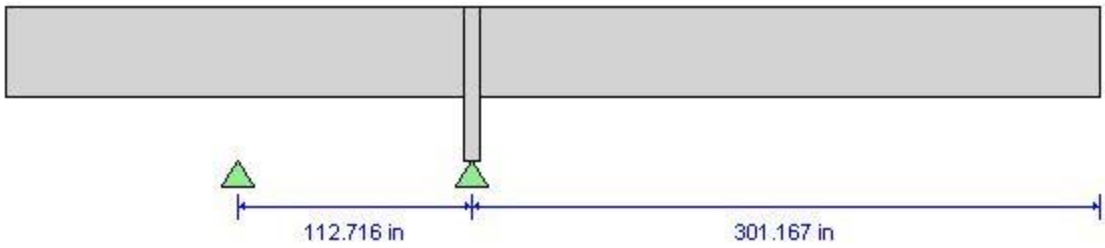


As (top)	0.29	0	0.84	0.92	0	0	
Combo	Comb=1.2 D + 1.6 L +1.6 Lr						
As (bot)	0.54	0.65	0.2	0	0	0.18	0.63	0.61
Combo	Comb=1.2 D + 1.6						

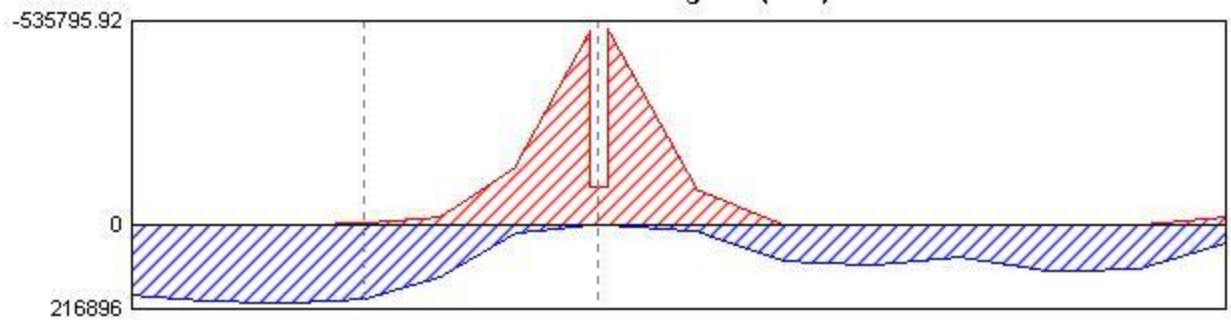
Shear Diagram (lb)



Shear	0	1494.87	5265.45	981.89	2493.78	637.42

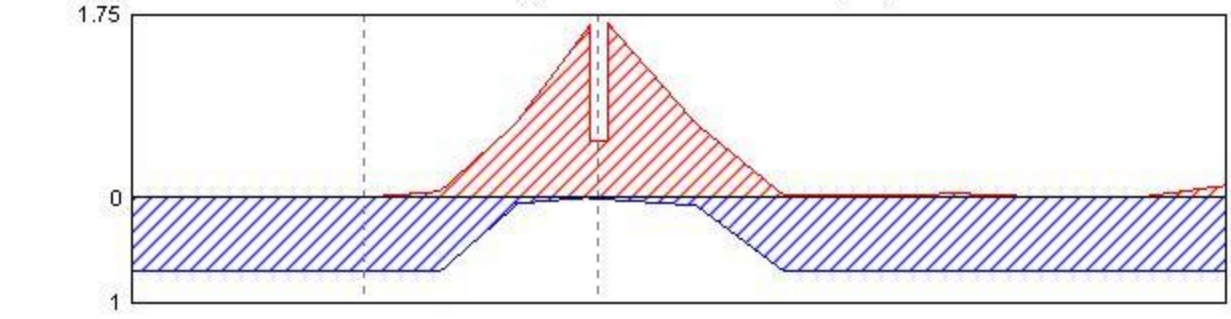


Moment Diagram (lb-in)



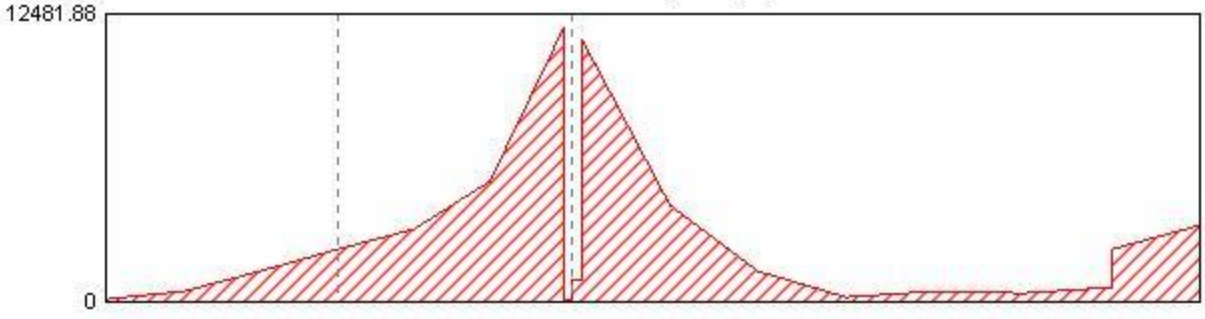
Moment (-)	0	0	-3122.35	-3122.35	50	254.03	-510281.83	-2121.55	-22776
Moment (+)	180690	105	1888.02	191888	279.15	216.63	17793.35	119716.38	114882.64

Longitudinal Reinforcement (in2)



As (top)	0	0	0	0	0.48	1.65	1.67	3.309E-002	0.11
Comb	Comb=1.2 D + 1.6 L +1.6						Comb=1.2 D + 1.6 L	Comb=1.2 D + 1.6 L	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.56	0.64	0.6	0.6	0.4	2739E-002	8.107E-002	0.41	0.39
Comb							Comb=1.2 D + 1.6 L	Comb=1.2 D + 1.6 L	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	96.85	419.5	2285.66	2285.66	3094.61	1887.5	116.48	1309.37	584.01

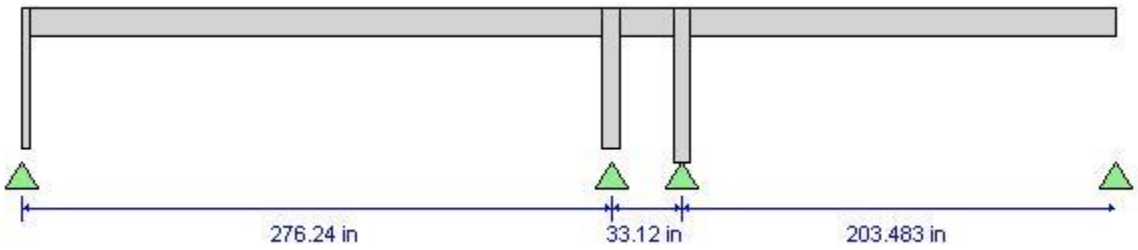
ACI 318-14 Concrete Strip Design

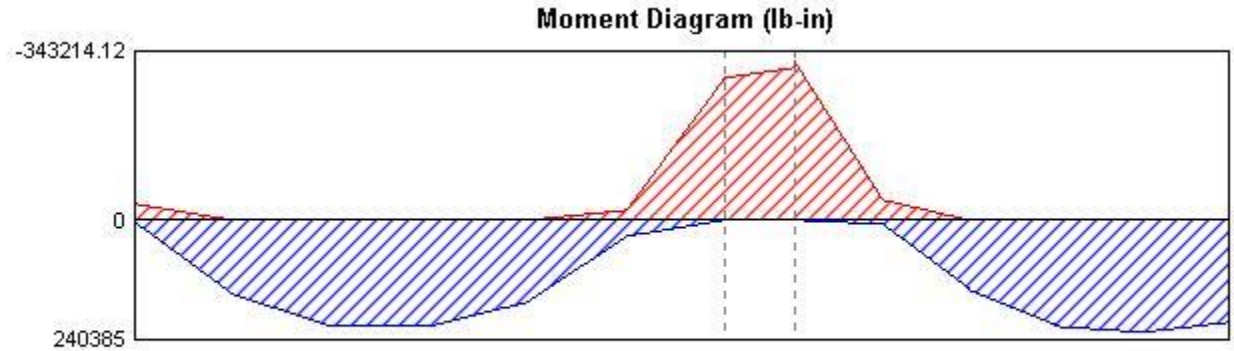
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB2
Length = 1048.32 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

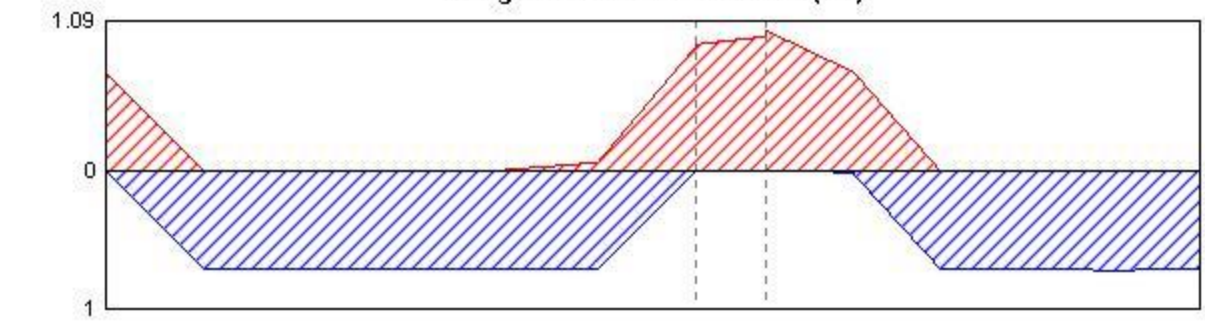
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





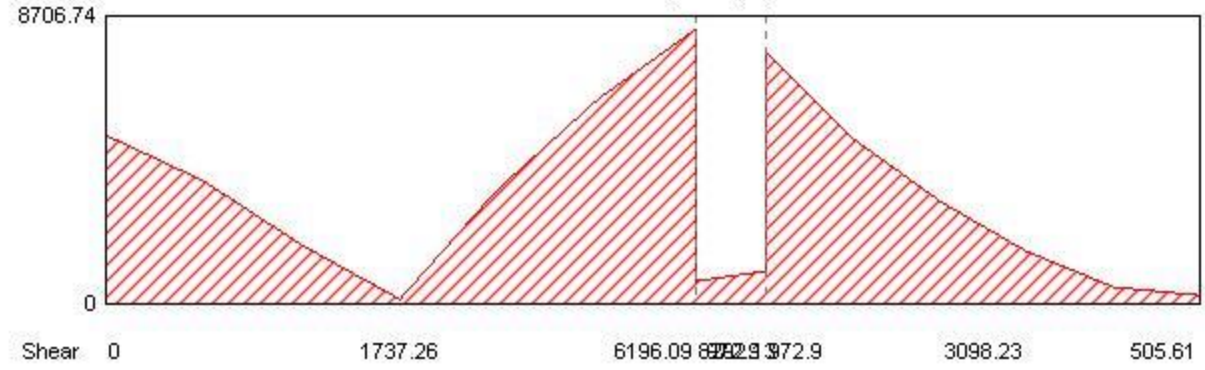
Moment (-)	-34259.04	0	-288291335[illegible]491.58	0	0
Moment (+)	149343.63	215908.71	34138.88 0 0 0 8204.22	212805.86	228937.33

Longitudinal Reinforcement (in2)

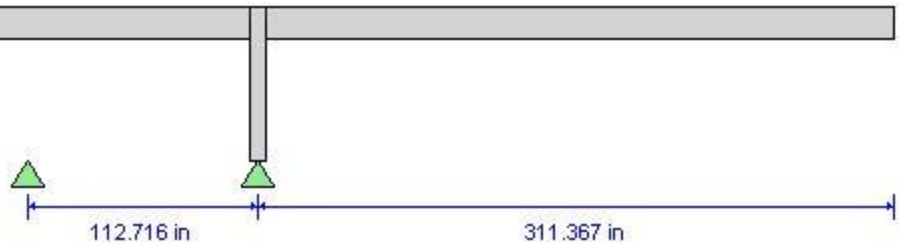


As (top)	0.11		0			0.91	0.91	1.01	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr									
As (bot)	0.46		0.67			0.1	0	0	2.516E-002	0.66	0.72
Combo	Comb=1.2 D + 1.6 L +1.6 Lr									

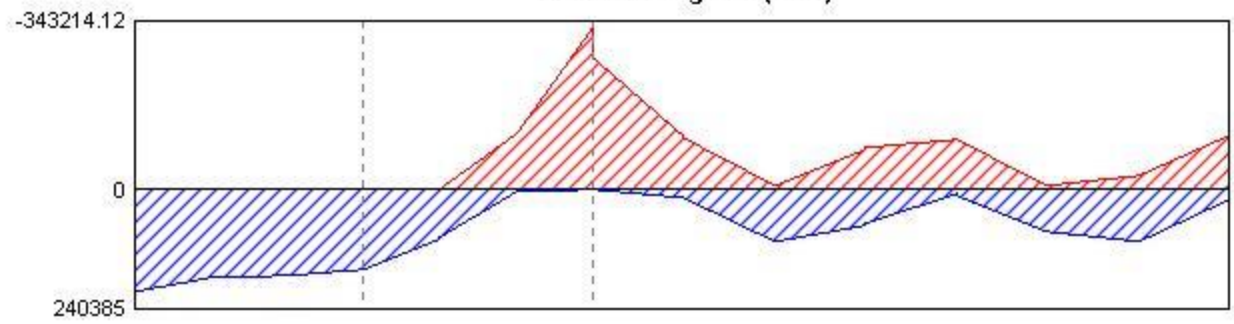
Shear Diagram (lb)



Shear	0	1737.26	6196.09		3098.23	505.61

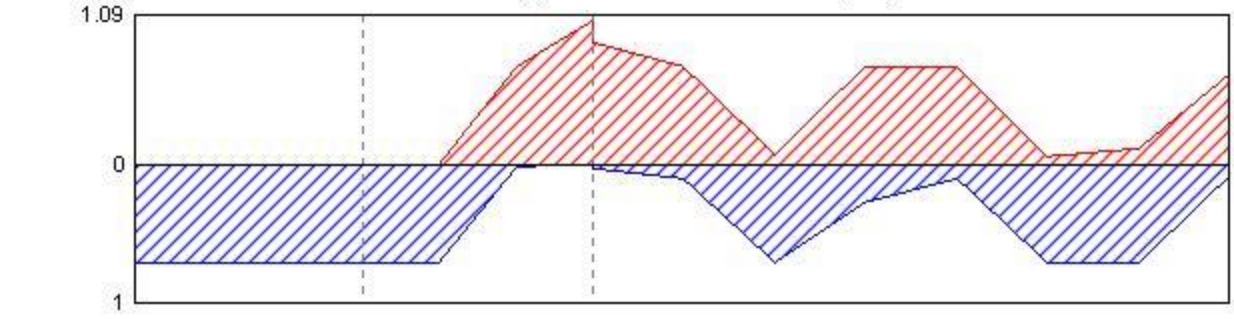


Moment Diagram (lb-in)



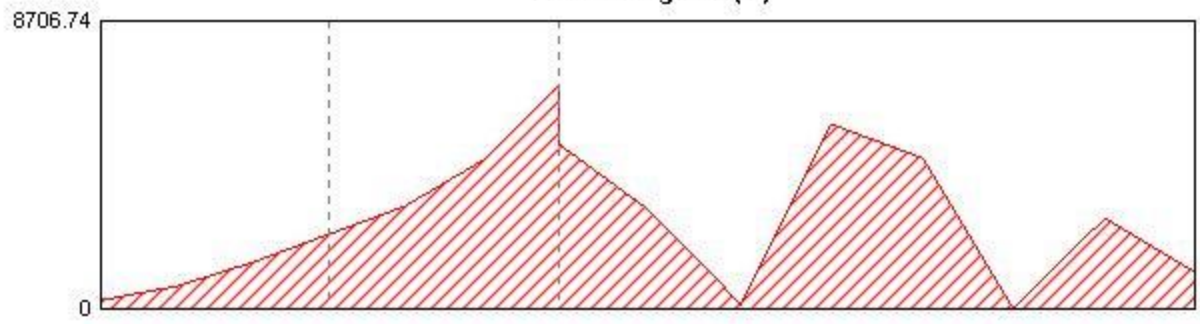
Moment (-)	0	0	0	0	-114798.25	70.59	-326870.59	-100270.11	-110142.4
Moment (+)	207074.38	66216.66	197.38	161197.38	99604.24	0	17142.58	105292.75	105077.73

Longitudinal Reinforcement (in2)



As (top)	0	0	0	0	0.36	1.04	1.04	0.38	0.36
Combo			Comb=1.2 D + 1.6 L +1.6 Lr				Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	
As (bot)	0.65	0.56	0.5	0.5	0.3	1.329E-002	0.1	0.38	0.35
Combo							Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)



Shear	212.02	671.06	2256.46	2256.46	8075.89	6745.22	6745.22	119.34	2719.04

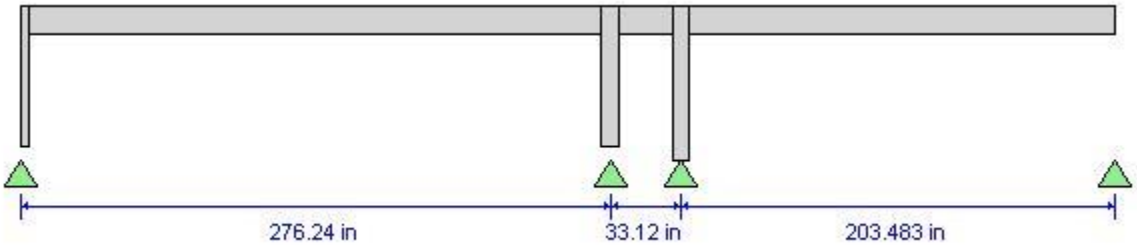
ACI 318-14 Concrete Strip Design

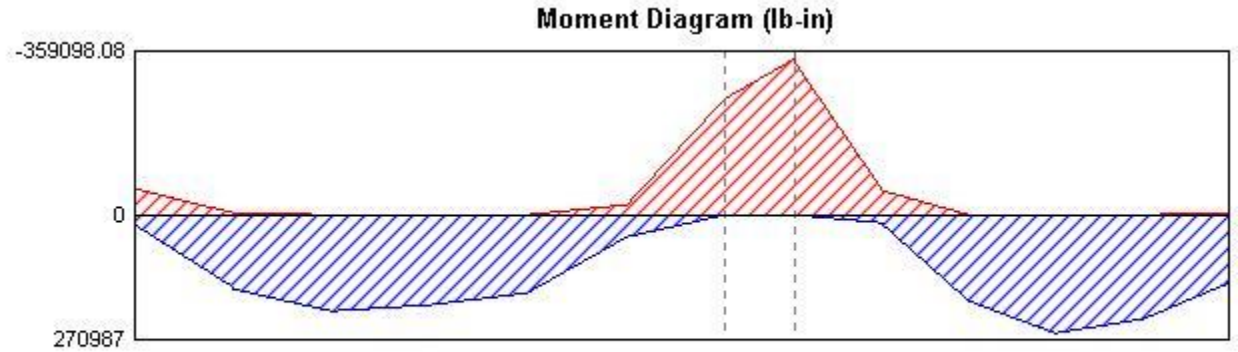
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB3
Length = 902.28 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

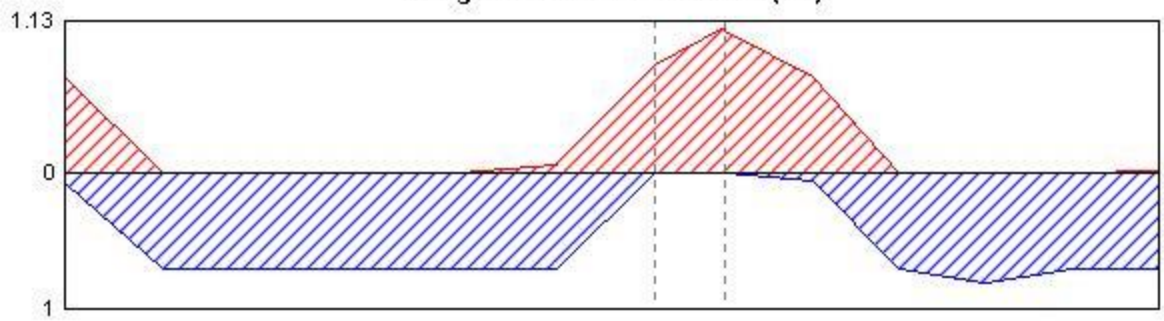
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





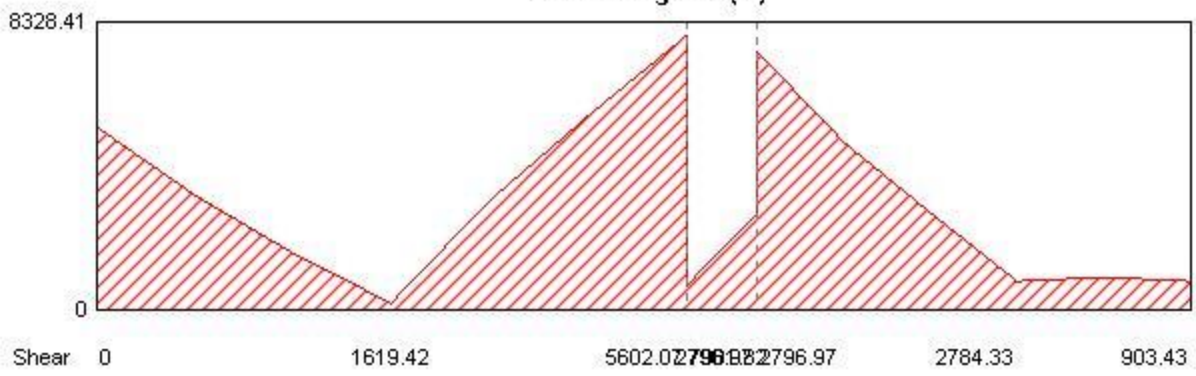
Moment (-)	-55455.1	0	-255846343 998.17	0	-6057.97	
Moment (+)	160777.28	211619.53	49095.46 0 0 0 19928.94	258082.53	226111.73	

Longitudinal Reinforcement (in2)

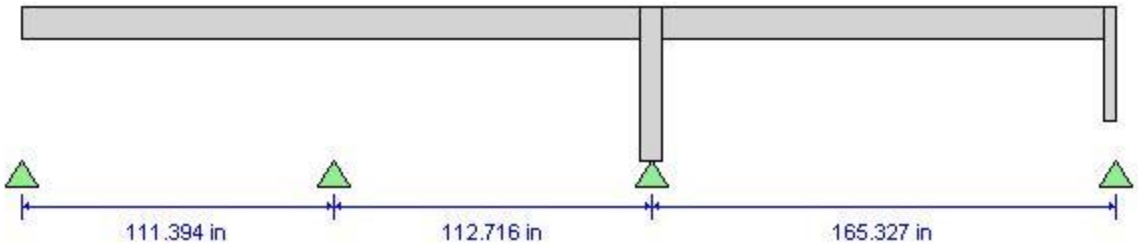


As (top)	0.17		0			0.8	0.8	1.08	1.08	0	1.857E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr										Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.5		0.66			0.15	0	0	0	6.118E-002	0.81	0.71
Combo	Comb=1.2 D + 1.6										L +1.6 Lr

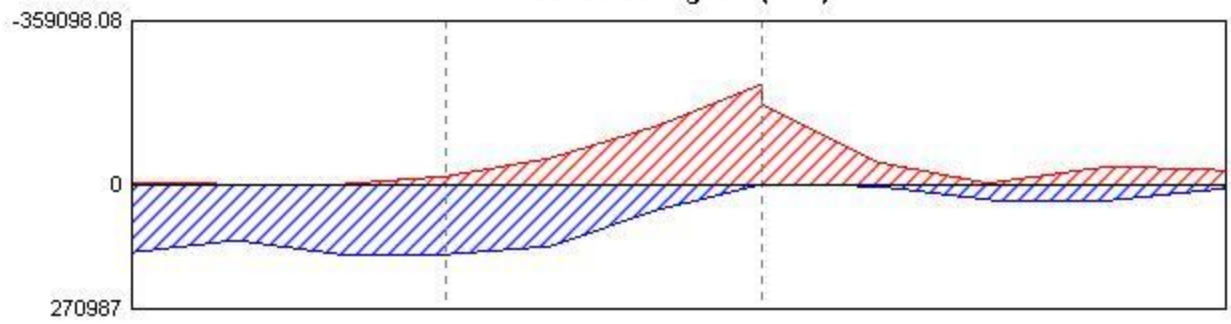
Shear Diagram (lb)



Shear	0	1619.42	5602.07	2796.97	2796.97	2784.33	903.43

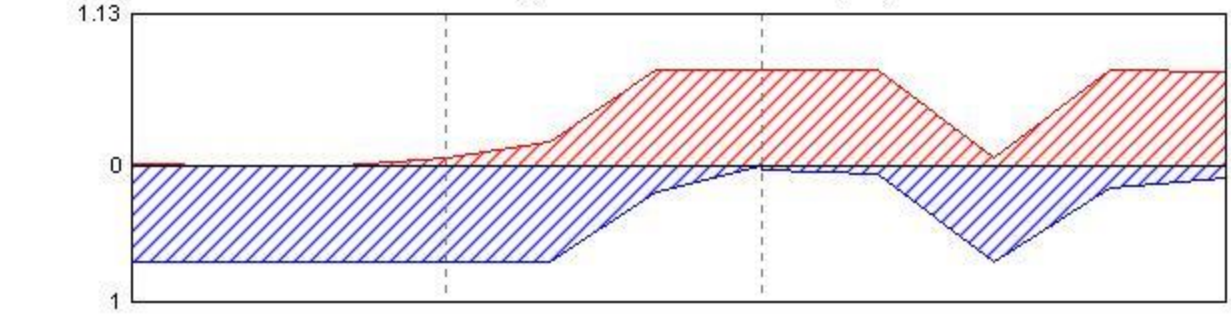


Moment Diagram (lb-in)



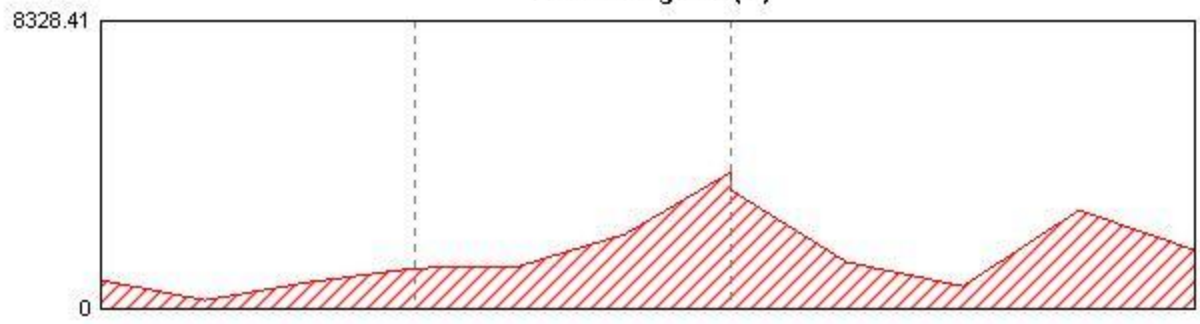
Moment (-)	-6057.97	-2114.42	-16761.31	-16761.31	-126745.9	-217711.26	-217711.26	-47387.44	-39720.35
Moment (+)	148652.5	2151305.26	154147.94	154147.94	136211.38	0	3842.47	33167.99	33167.99

Longitudinal Reinforcement (in2)



As (top)	1.857E-002	0	5.402E-002	5.402E-002	0.4	0.69	0.69	0.2	0.19
Combo	Co[illegible] Lr						Co[illegible] Lr	Comb=1.2	[illegible] L +1.6 Lr
As (bot)	0.46	0.47	0.48	0.48	0.43	3.166E-002	6.131E-002	0.17	0.17
Combo	Co[illegible] Lr						Co[illegible] Lr	Comb=1.2	[illegible] L +1.6 Lr

Shear Diagram (lb)



Shear	781.31	245.39	1150.15	1150.15	1221.85	3919.31	3919.31	1343.72	2855.61

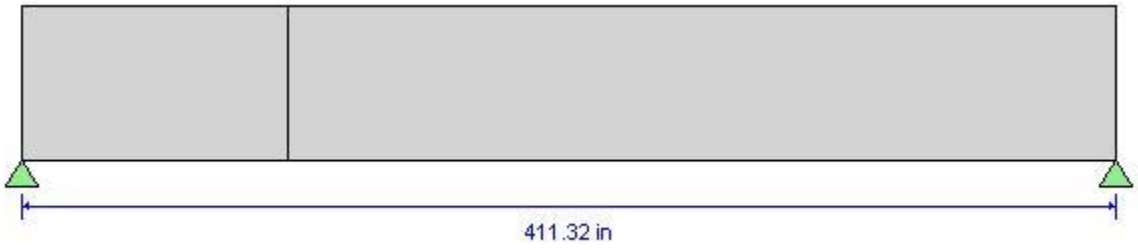
ACI 318-14 Concrete Strip Design

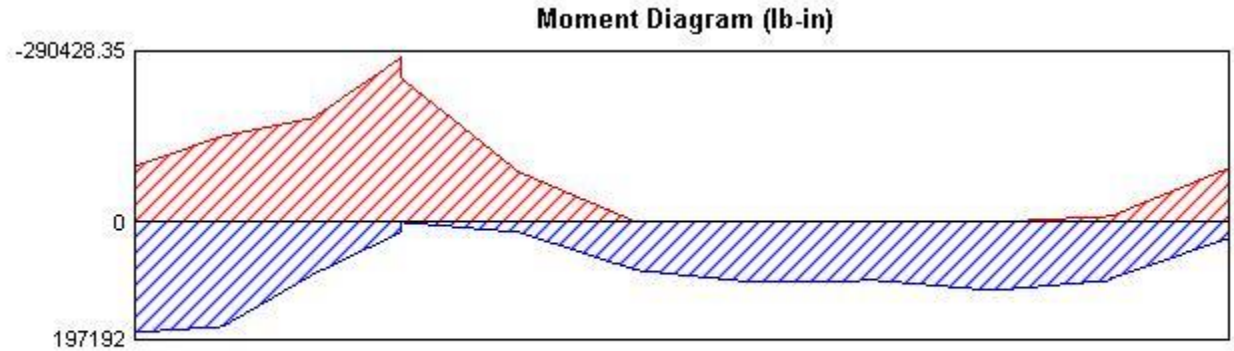
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB4
Length = 411.32 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

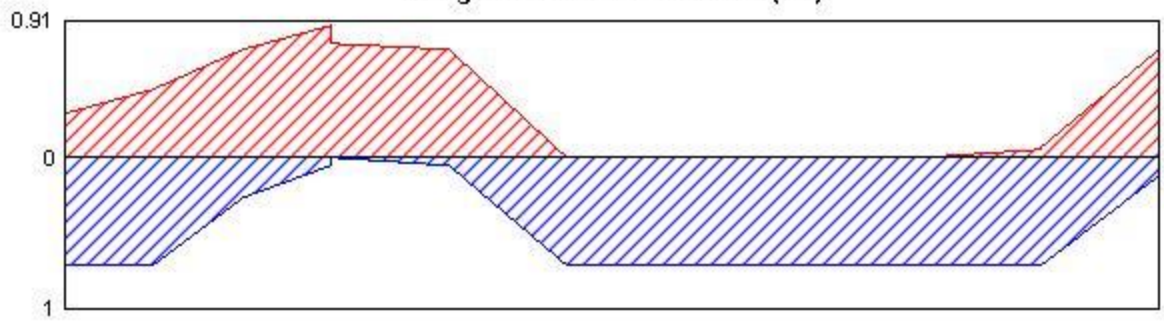
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





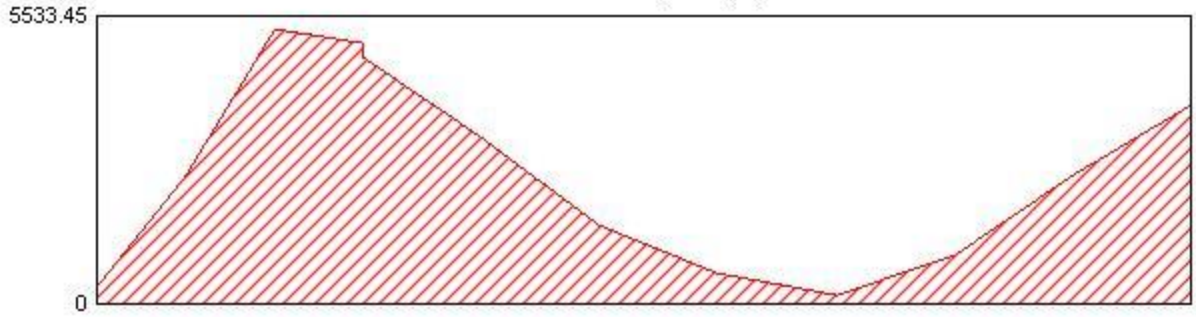
Moment (-)	-276598.43	-85244.35	-92680.28
Moment (+)	187801.84	97705.15	113730.16

Longitudinal Reinforcement (in2)



As (top)	0.87	0.27	0.32
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.59	0.33	0.38
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	3183.73	909.17

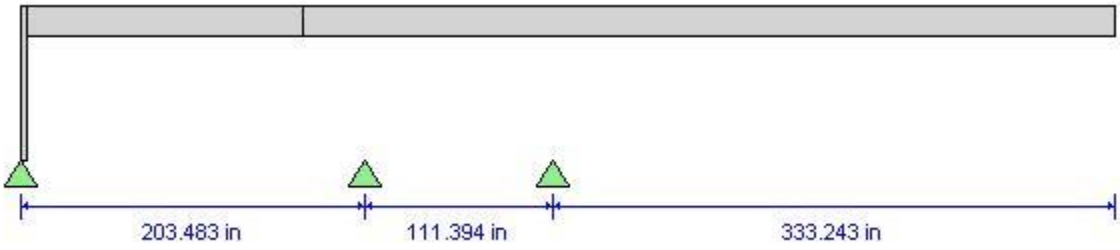
ACI 318-14 Concrete Strip Design

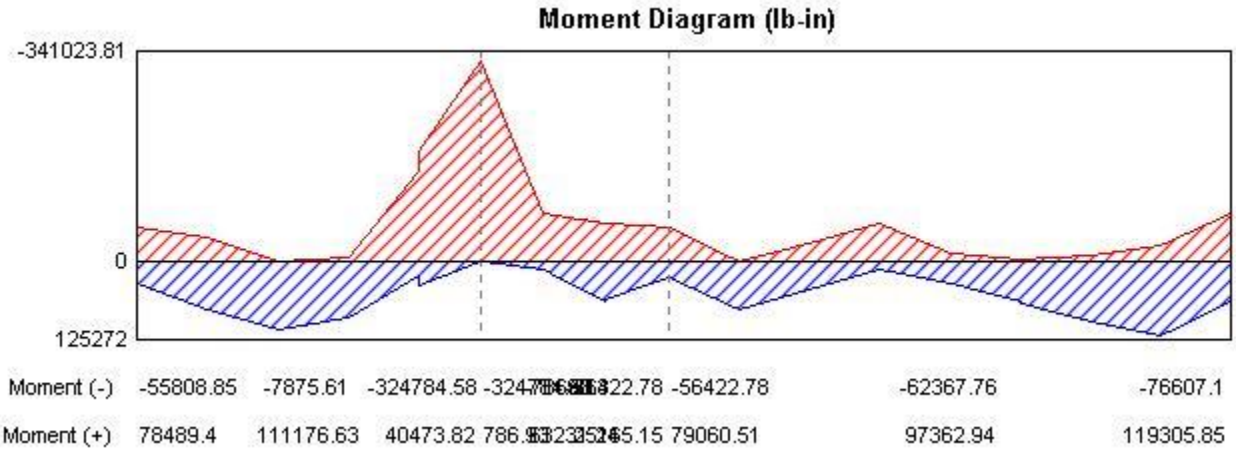
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB5
Length = 648.12 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

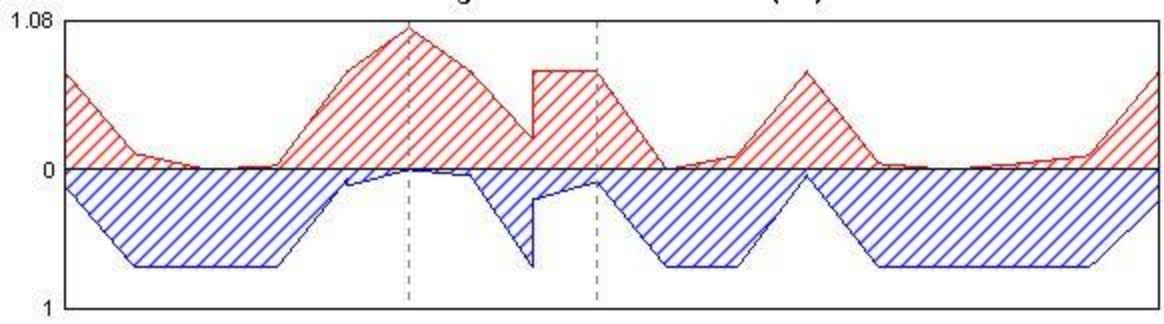
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2



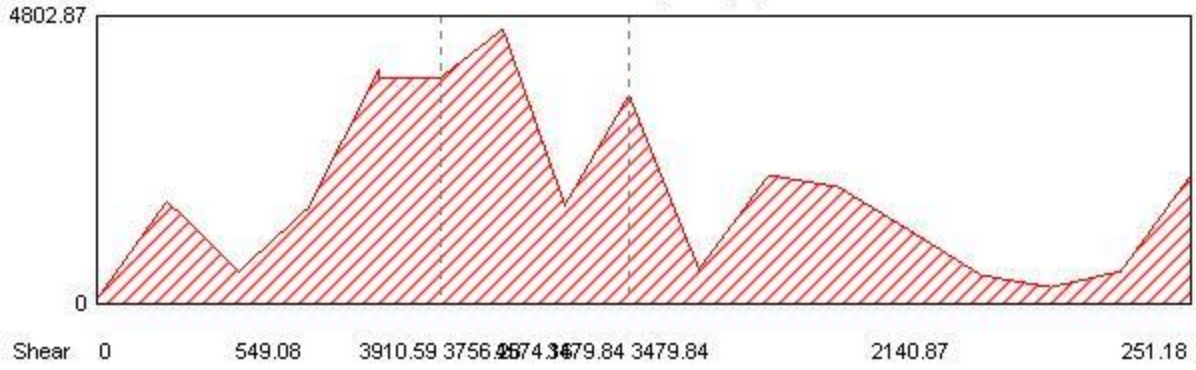


Longitudinal Reinforcement (in2)



As (top)	0.17	3.127E-002	1.03	1.03	0.25	0.19	0.19	0.19	0.27
Combo	Com							.2 D +	+ 1.6 L +1.6 Lr
As (bot)	0.24	0.34	0.12	0	8.2	23E-002	0.25	0.31	0.39
Combo	Com							.2 D +	+ 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	549.08	3910.59	3756.4	574.3	3479.84	3479.84	2140.87	251.18

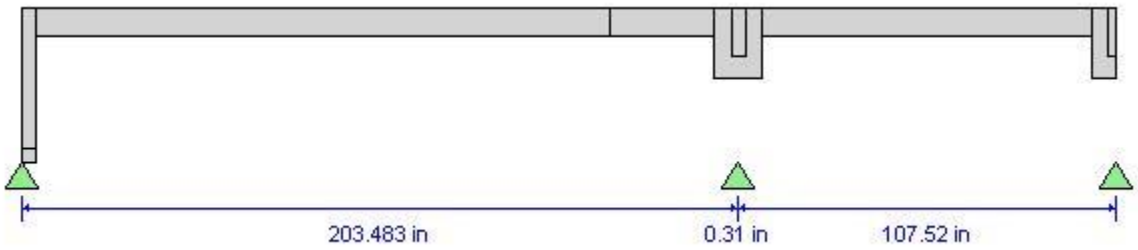
ACI 318-14 Concrete Strip Design

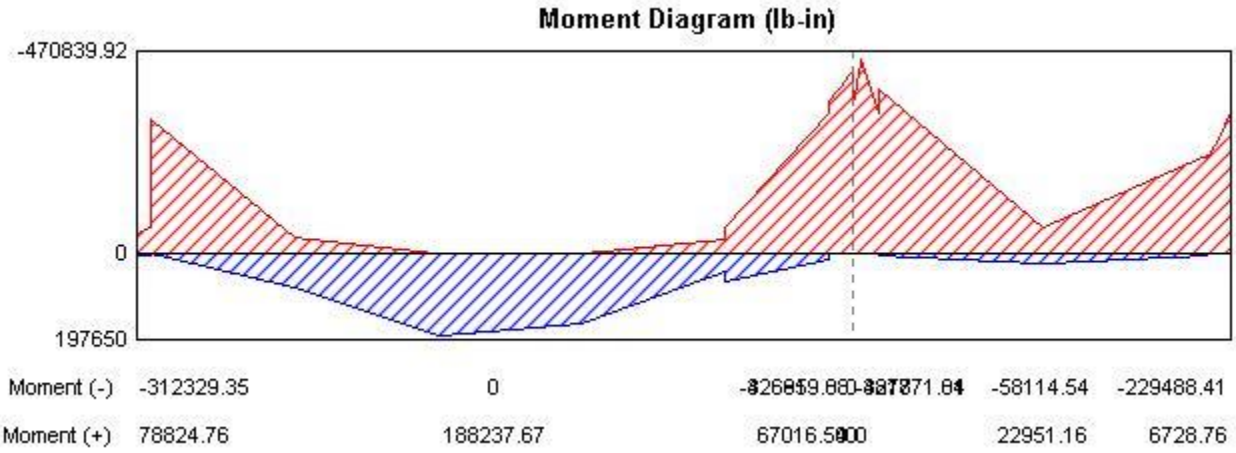
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB6
Length = 427.593 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

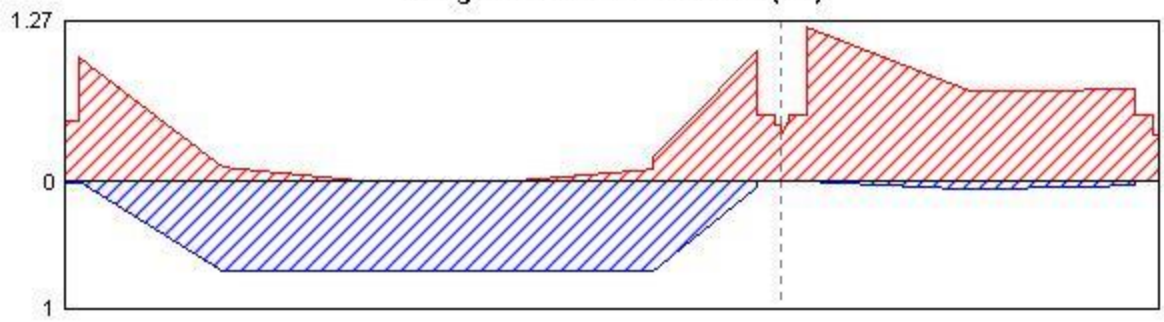
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





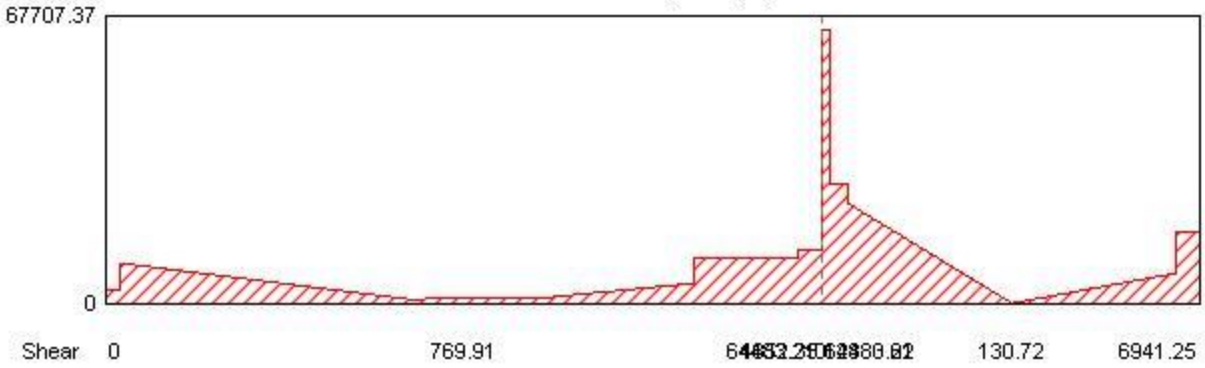
Moment (-)	-312329.35	0	-[illegible]	-[illegible]	-58114.54	-229488.41
Moment (+)	78824.76	188237.67	67016.59[illegible]		22951.16	6728.76

Longitudinal Reinforcement (in2)



As (top)	0.98	0	0.9600.26	0.18	0.72	
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr	Comb[illegible]1.6 Lr		
As (bot)	0.24	0.59	0.2000	7.047E-002	2.538E-002	
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb[illegible]1.6 Lr		

Shear Diagram (lb)

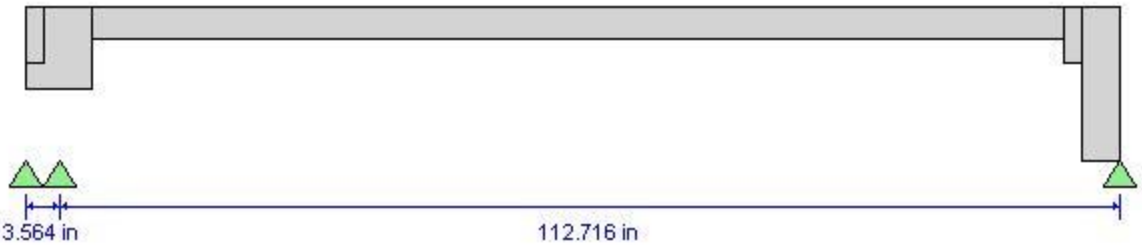


Shear	0	769.91	64453.26[illegible]80.82	130.72	6941.25

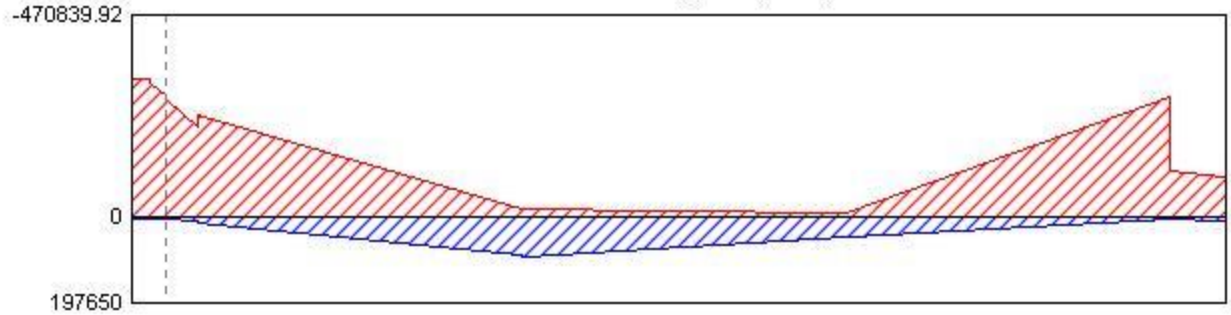
Transverse Reinforcement (in2/ft)



Asv/s reqd	0	0	0.7800.75	0	0
Combo			Comb=1.2 D +[illegible]1.6 Lr		

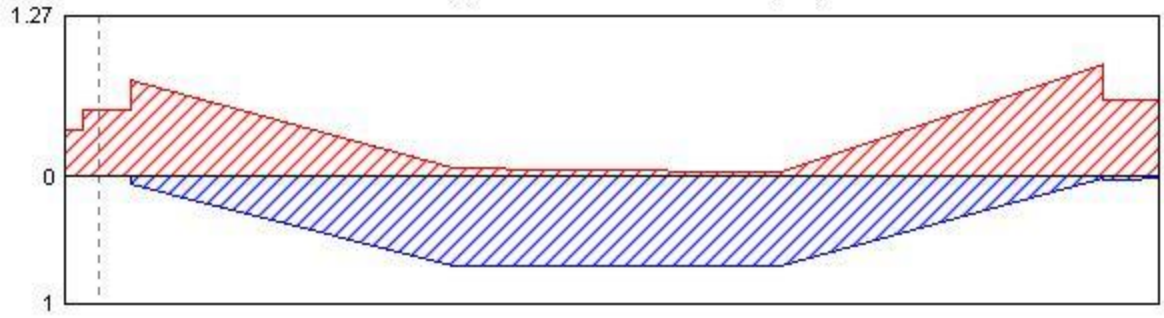


Moment Diagram (lb-in)

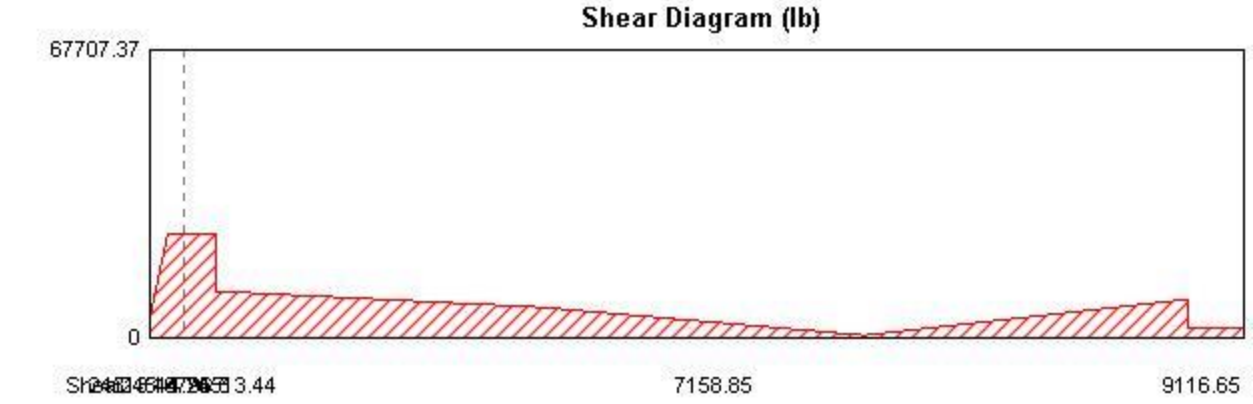


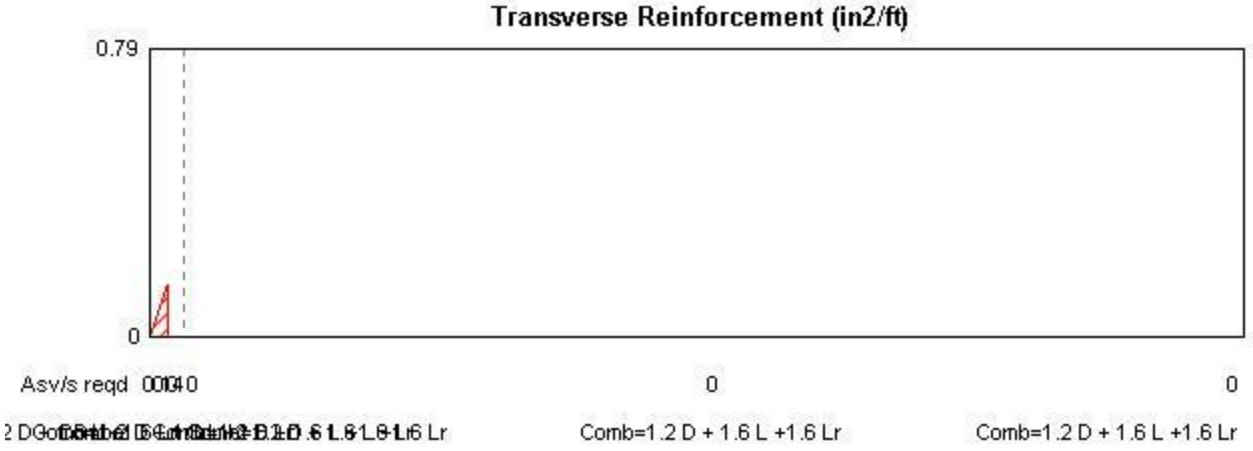
Mome[illegible].95	-19800.67	-280562.17
Moment (6[illegible].74648.2	91886.05	6404.31

Longitudinal Reinforcement (in2)



As (top)	[illegible]0.75	6.505E-002	0.88
2 DGo[illegible]6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	5.629E-002	0.29	3.01E-002
2 DGo[illegible]6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Shear Diagram (lb)
67707.37
0
7158.85
9116.65


Transverse Reinforcement (in2/ft)
0.79
0
Asv/s reqd
0
0
Comb=1.2 D + 1.6 L +1.6 Lr
Comb=1.2 D + 1.6 L +1.6 Lr

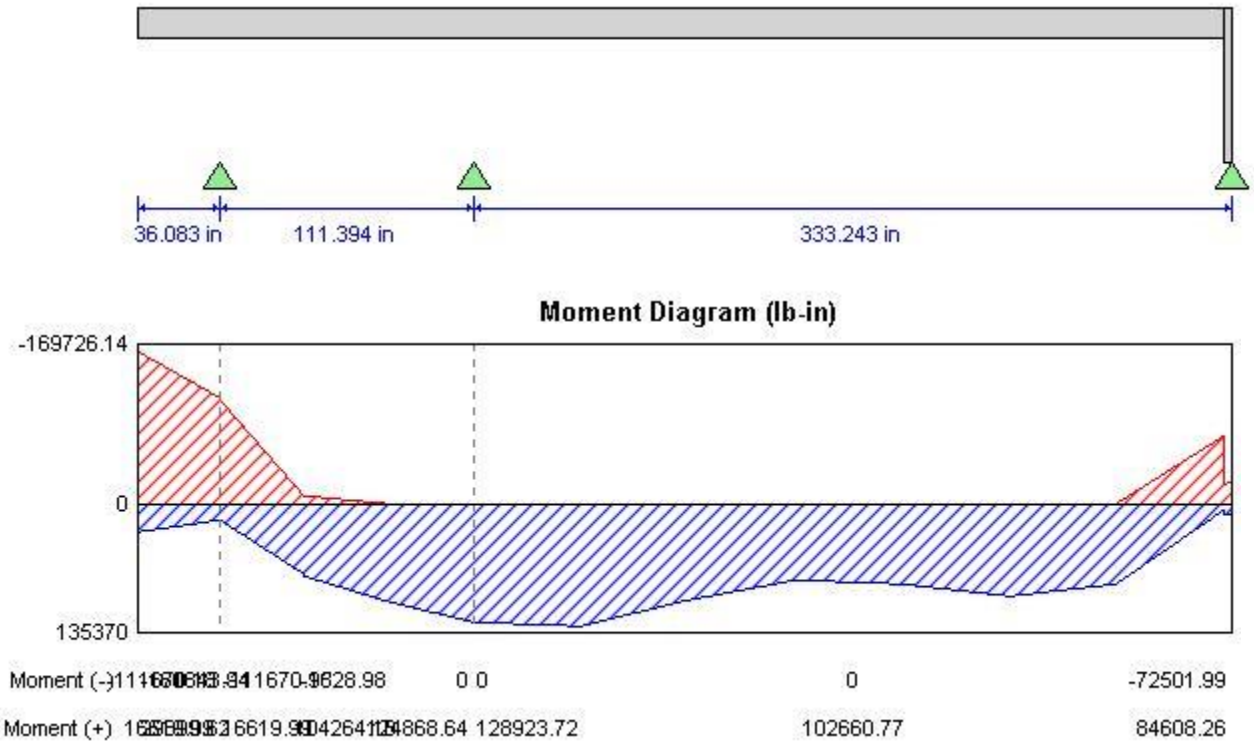
ACI 318-14 Concrete Strip Design

Geometric Properties

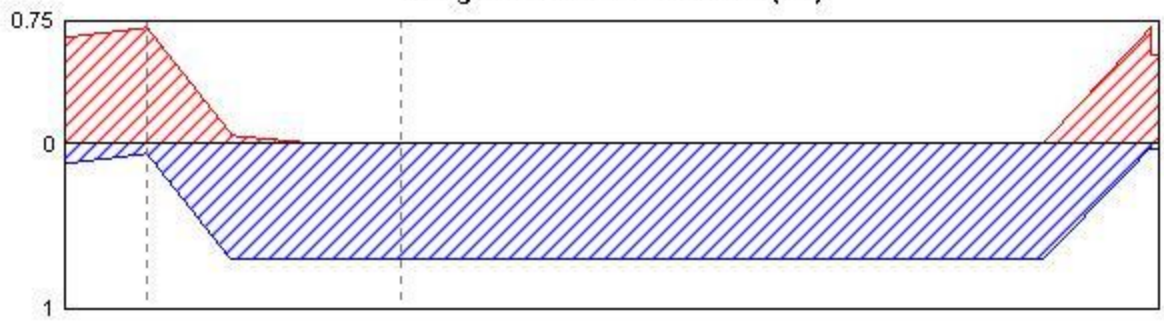
Combination = Overall Envelope
Strip Label = MSB7
Length = 480.72 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2

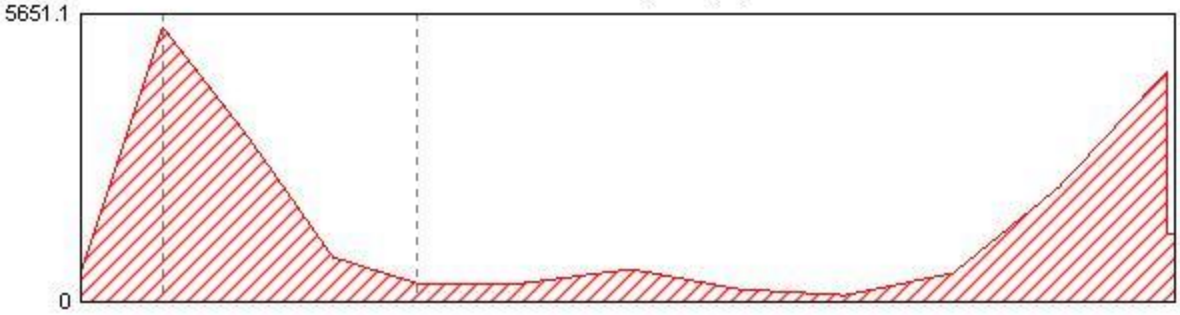


Longitudinal Reinforcement (in2)



As (top)	0.58	0.36	0.36	5.563E-002	0	0	0	0.23
b=1.2	Comb6	LC	Comb					Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	6.30		002	6.307E-002	0.35	0.4	0.41	0.32	0.27
b=1.2	Comb6	LC	Comb					Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	5382	5382	3269.58	336.88	336.88	614.03	2217.81

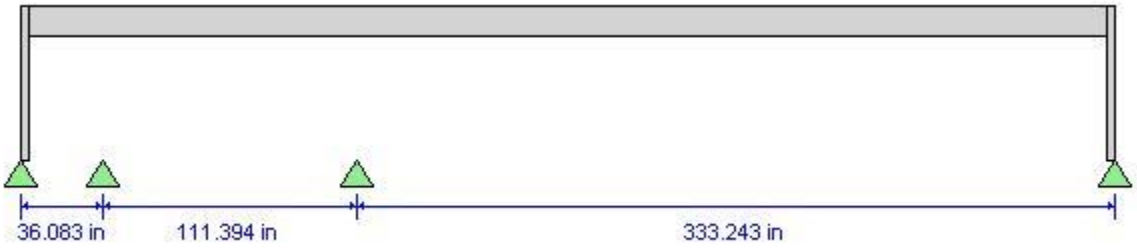
ACI 318-14 Concrete Strip Design

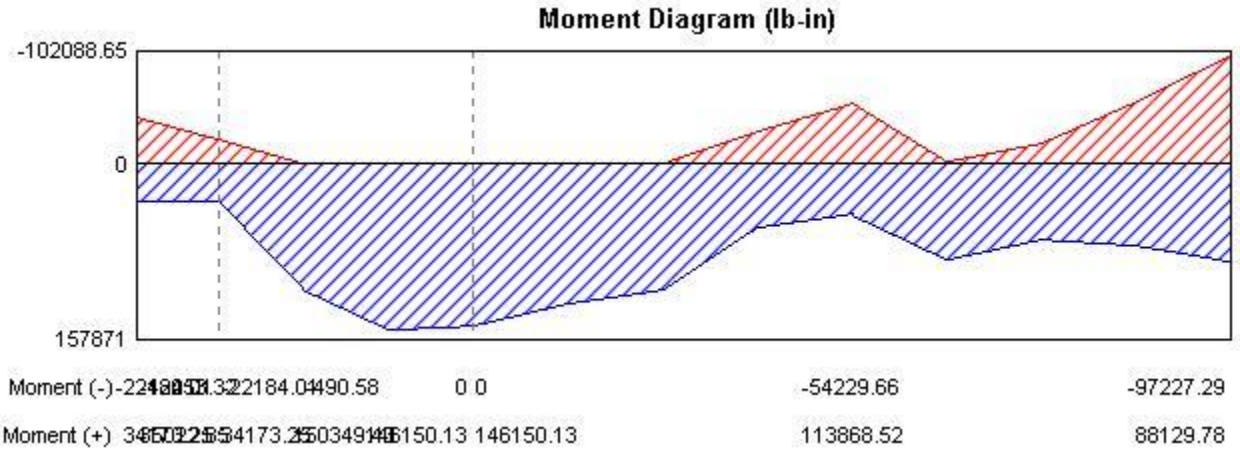
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB8
Length = 480.72 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

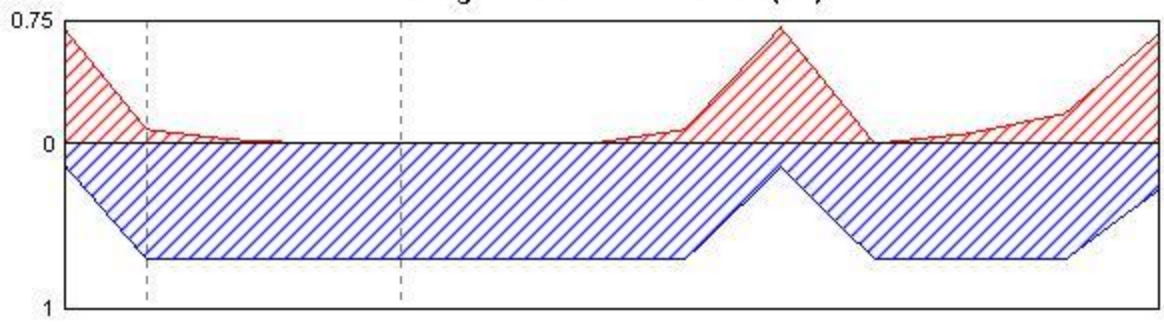
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2



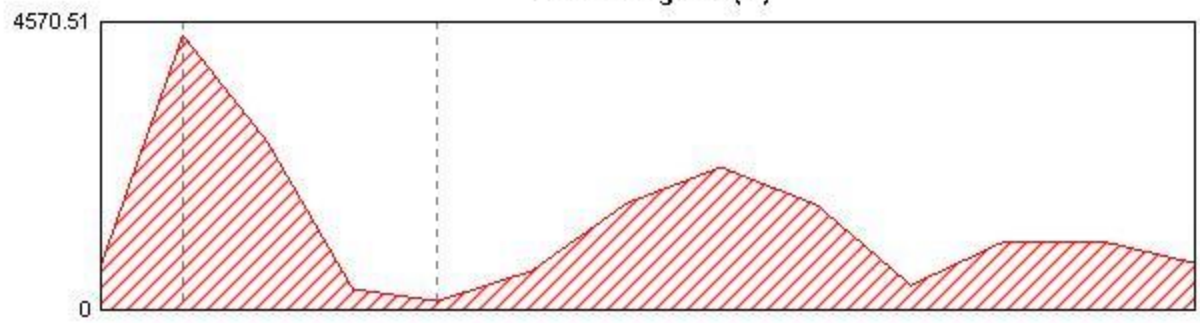


Longitudinal Reinforcement (in2)



As (top)	9.026E-002	9.026E-002	9.026E-002	0	0			0.17		0.31
b=1.2	Comb=1.2 D + 1.6 L +1.6 Lr							Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.13	0.13	0.13	0.48	0.46	0.46		0.35		0.28
b=1.2	Comb=1.2 D + 1.6 L +1.6 Lr						Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	4352.87	4352.87	4352.87	2670.13	122.06	122.06		1676.95		1092.87

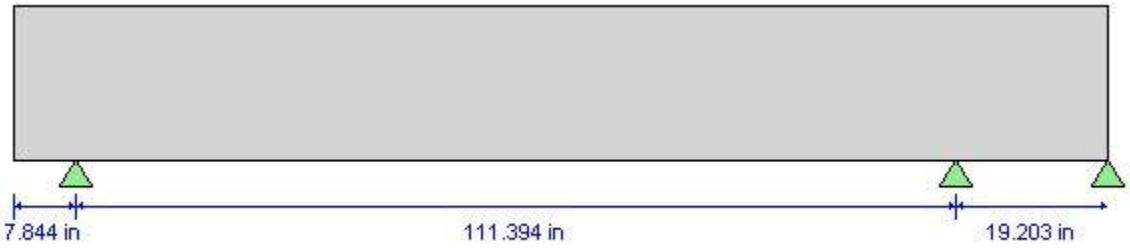
ACI 318-14 Concrete Strip Design

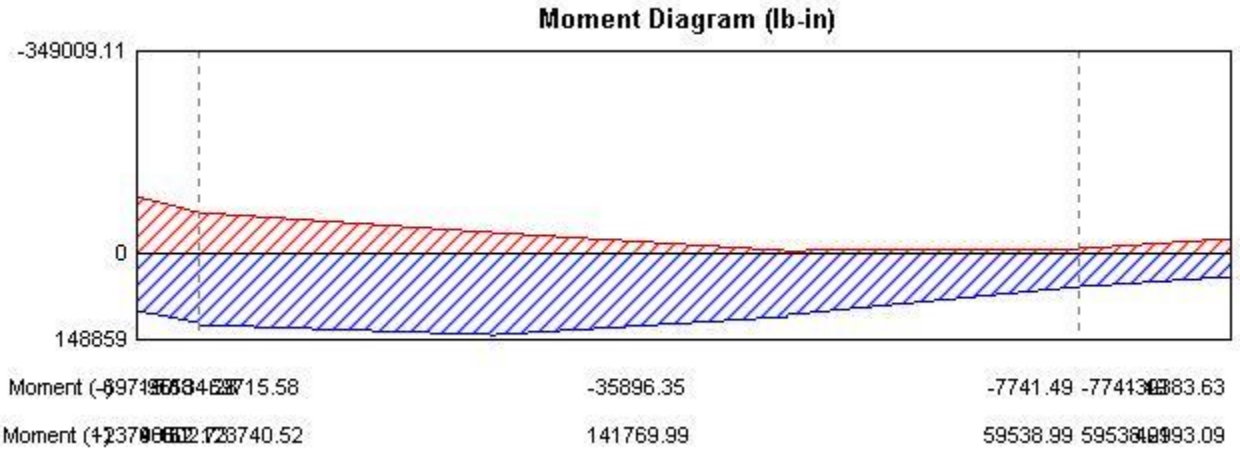
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB10
Length = 388.558 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

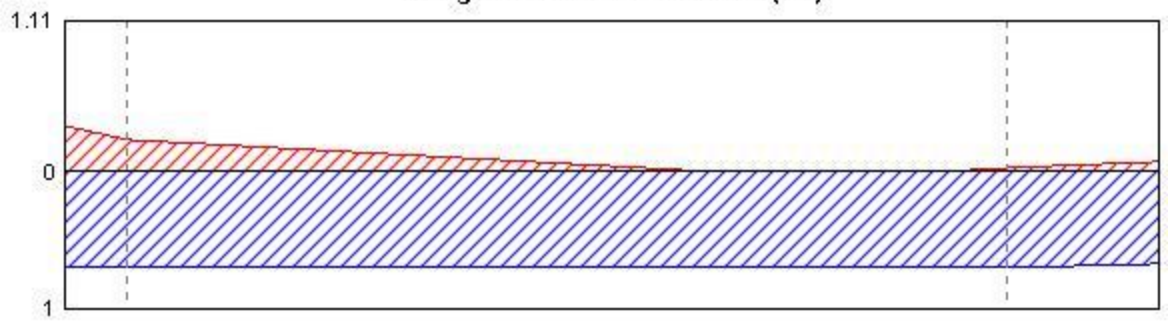
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2



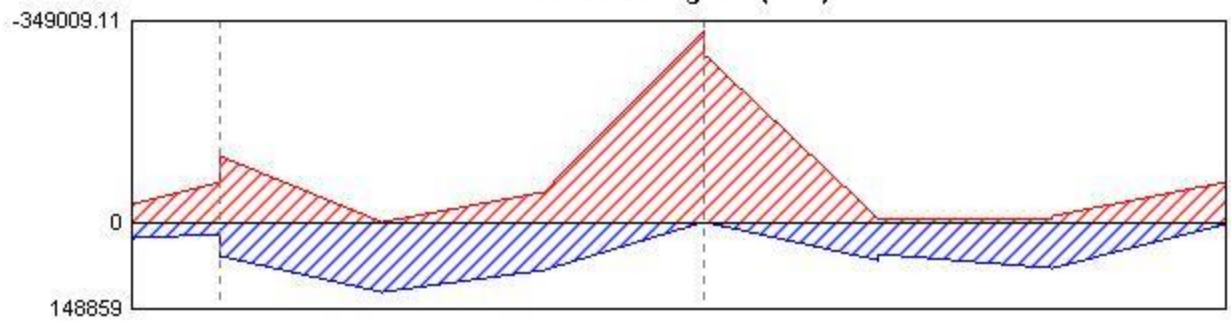


Longitudinal Reinforcement (in2)



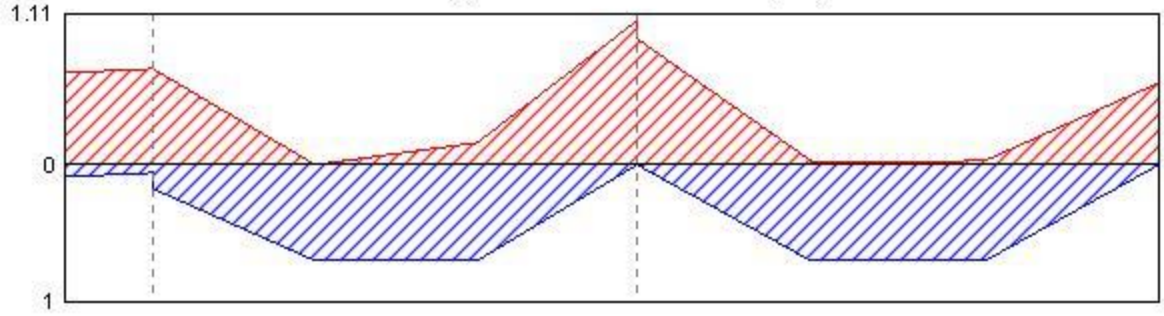
| | | |

Moment Diagram (lb-in)



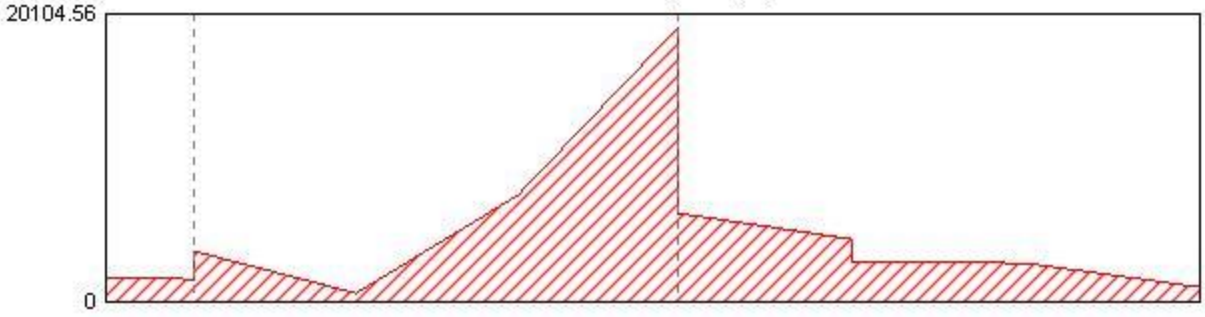
Moment (-)	-11[illegible]	114894.64	-52763.53	-332389.63	-332389.63	-9190.78	-70294.39
Moment (+)	[illegible]	59065.3	120563.97	0	0	80171.14	1981.43

Longitudinal Reinforcement (in2)



As (top)	9.9[illegible]00236	0.16	1.06	1.06	2.819E-002	0.22
	[illegible]					
As (bot)	0.1[illegible]0.18	0.18	0.37	0	0	0.25	0
	[illegible]					

Shear Diagram (lb)



Shear	341[illegible]1470	577.8	19147.2	19147.2	4375.51	965.77

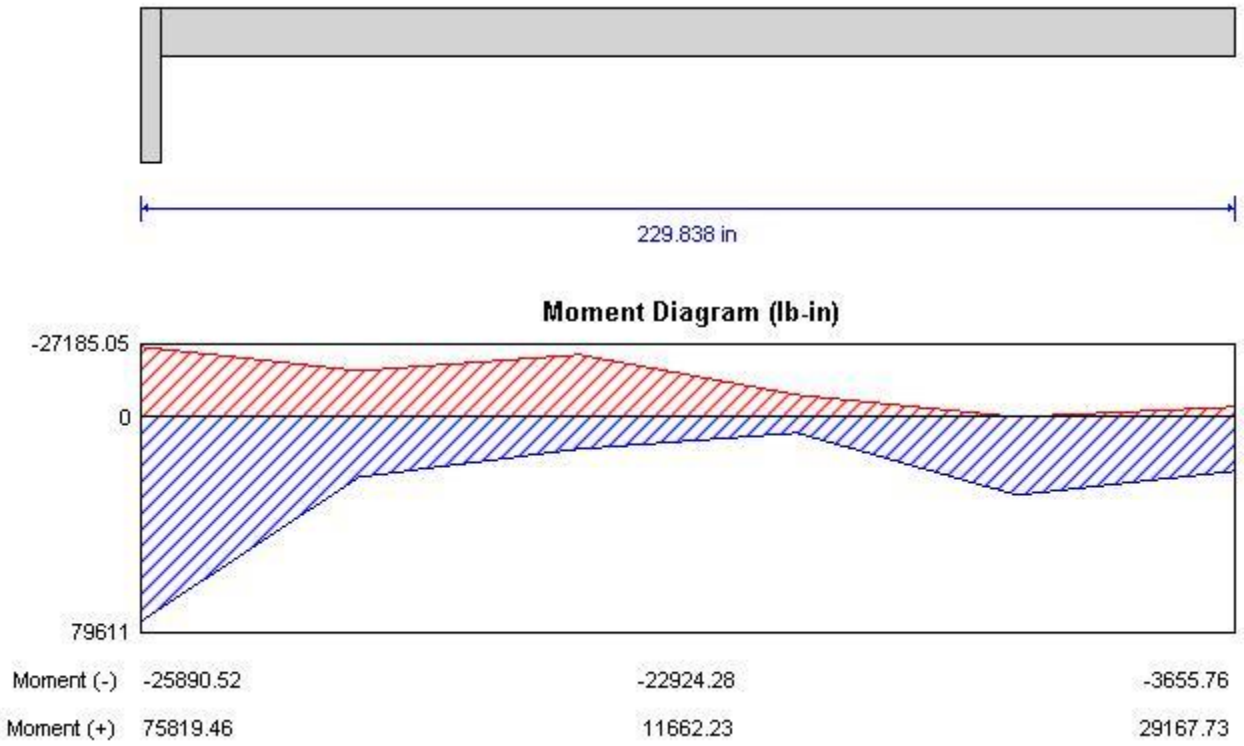
ACI 318-14 Concrete Strip Design

Geometric Properties

Combination = Overall Envelope
Strip Label = MSB11
Length = 229.838 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

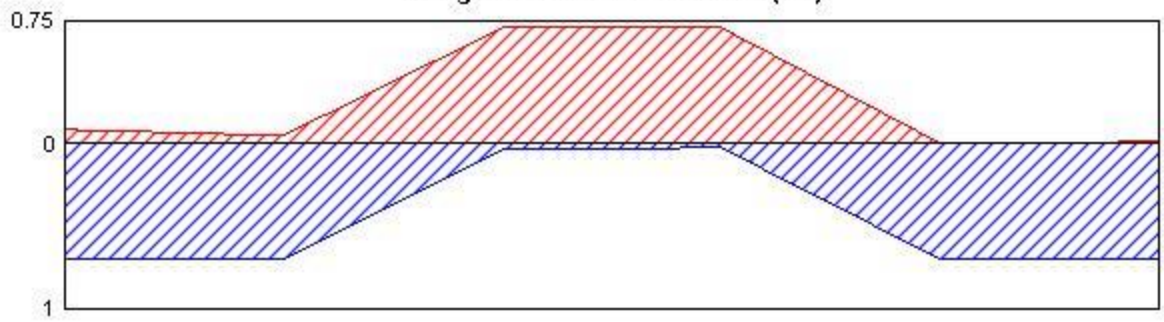
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2



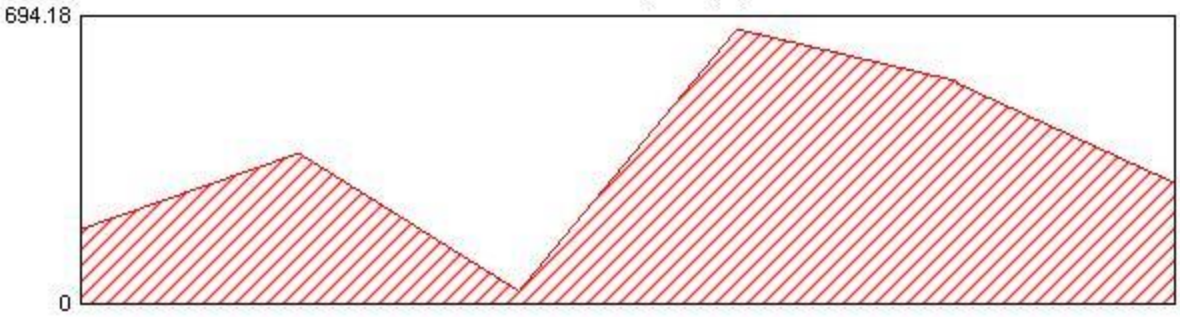
Moment (-)	-25890.52	-22924.28	-3655.76
Moment (+)	75819.46	11662.23	29167.73

Longitudinal Reinforcement (in2)



As (top)	8.853E-002	7.157E-002	1.677E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.24	3.692E-002	0.1
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	29.78	532.87

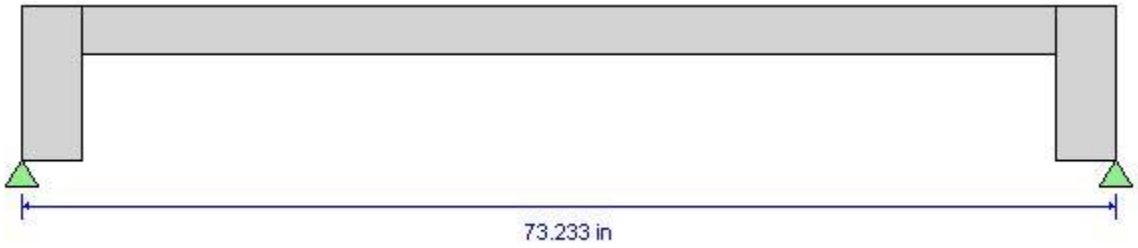
ACI 318-14 Concrete Strip Design

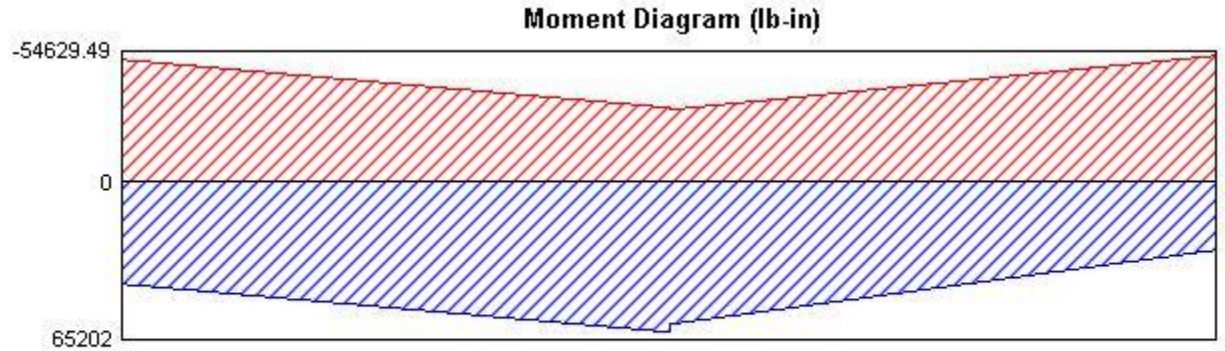
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB12
Length = 73.233 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

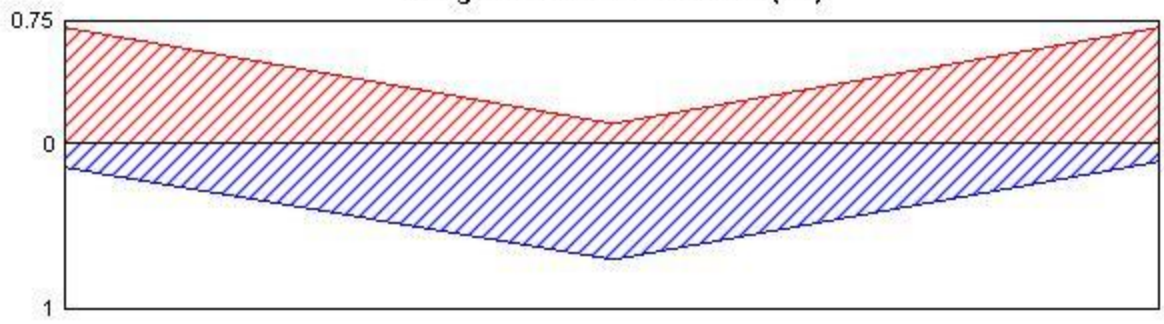
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





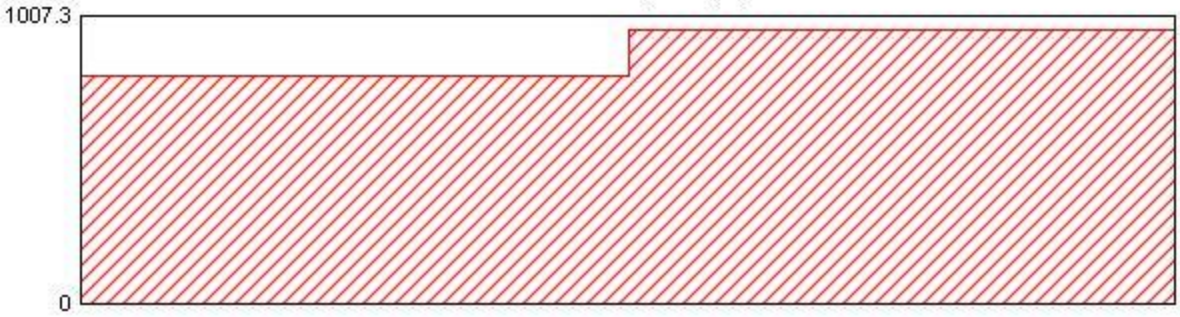
Moment (-)	-50894.44	-29579.36	-52028.09
Moment (+)	42464.62	62096.24	28479.77

Longitudinal Reinforcement (in2)



As (top)	0.18	0.12	0.18
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.15	0.21	0.11
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	798.5	959.33

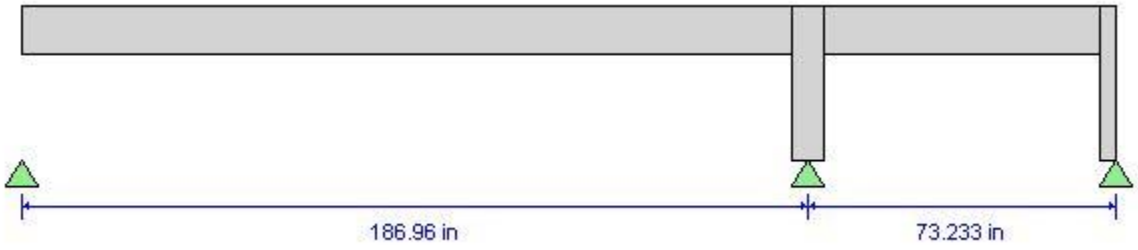
ACI 318-14 Concrete Strip Design

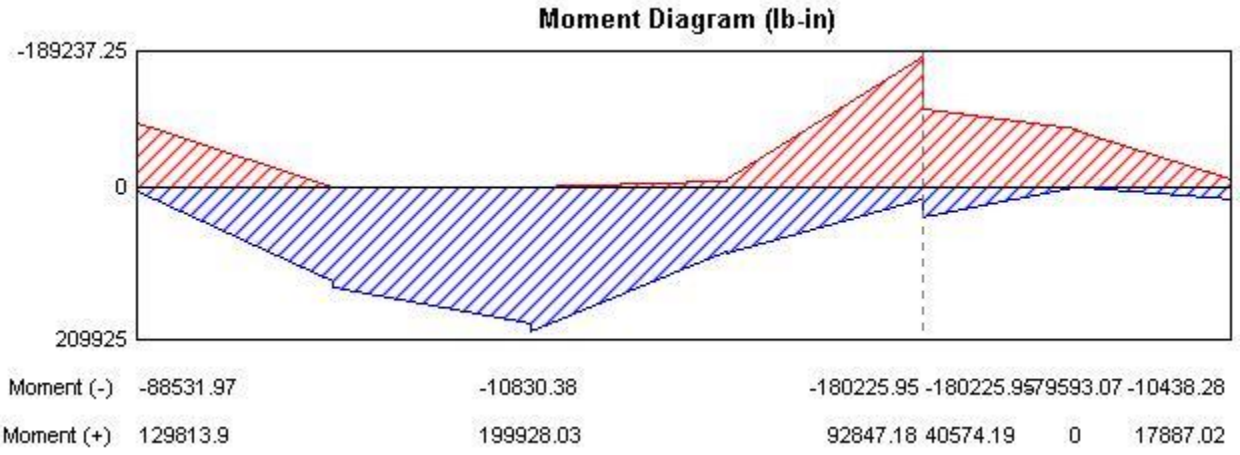
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB13
Length = 260.193 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

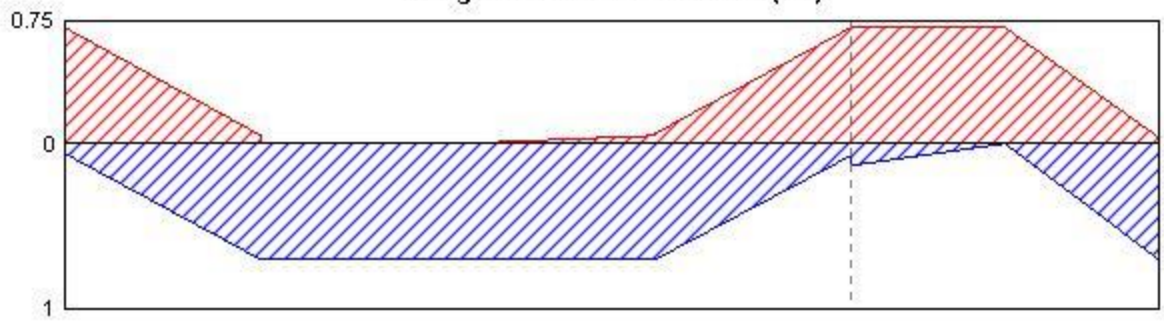
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





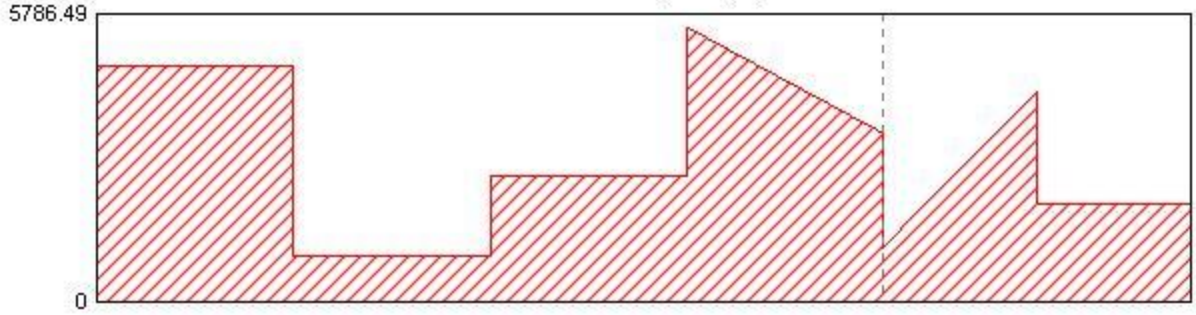
Moment (-)	-88531.97	-10830.38	-180225.95	-180225.95	79593.07	-10438.28	
Moment (+)	129813.9	199928.03	92847.18	40574.19	0	17887.02	

Longitudinal Reinforcement (in2)



As (top)	0.32	5.545E-002	0.57	0.57	0.25	3.202E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr		
As (bot)	0.47	0.65	0.31	0.13	0	5.49E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	0	4752.42	2533.48	3360.31	4215.26	1963.69

ACI 318-14 Concrete Strip Design

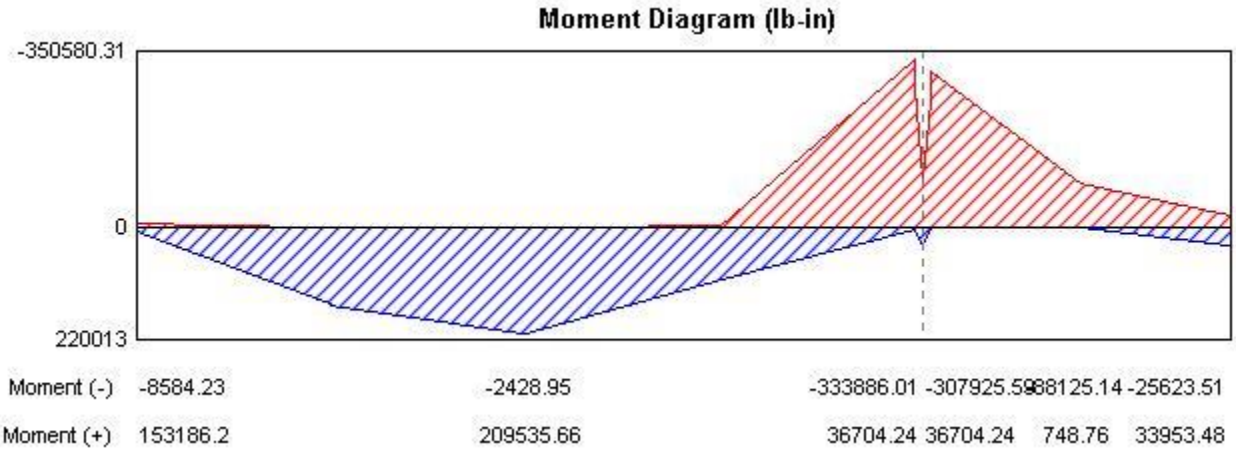
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB14
Length = 260.193 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

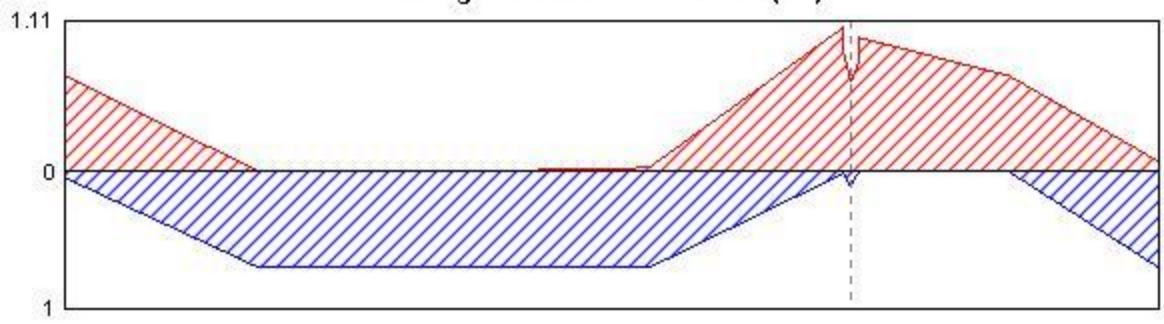
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





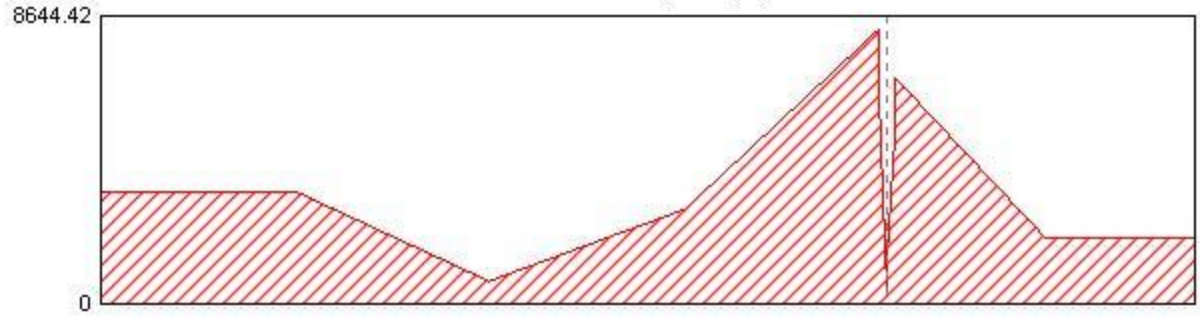
Moment (-)	-8584.23	-2428.95	-333886.01	-307925.59	88125.14	-25623.51	
Moment (+)	153186.2	209535.66	36704.24	36704.24	748.76	33953.48	

Longitudinal Reinforcement (in2)



As (top)	5.742E-002	2.611E-002	1.06	0.98	0.27	7.869E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr			
As (bot)	0.54	0.68	0.12	0.12	0	0.1
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr			

Shear Diagram (lb)



Shear	0	639.89	8232.78	324.73	1983.62	1983.62

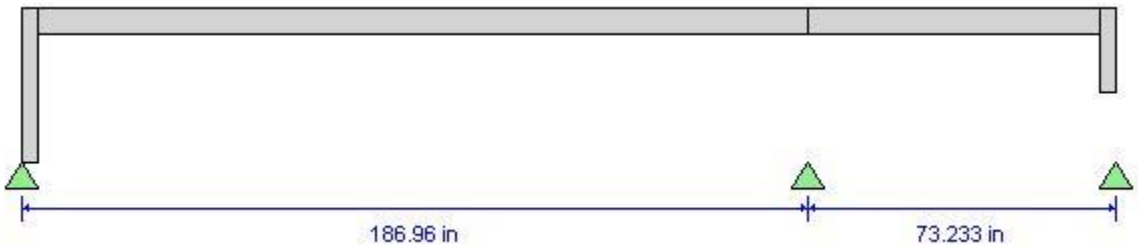
ACI 318-14 Concrete Strip Design

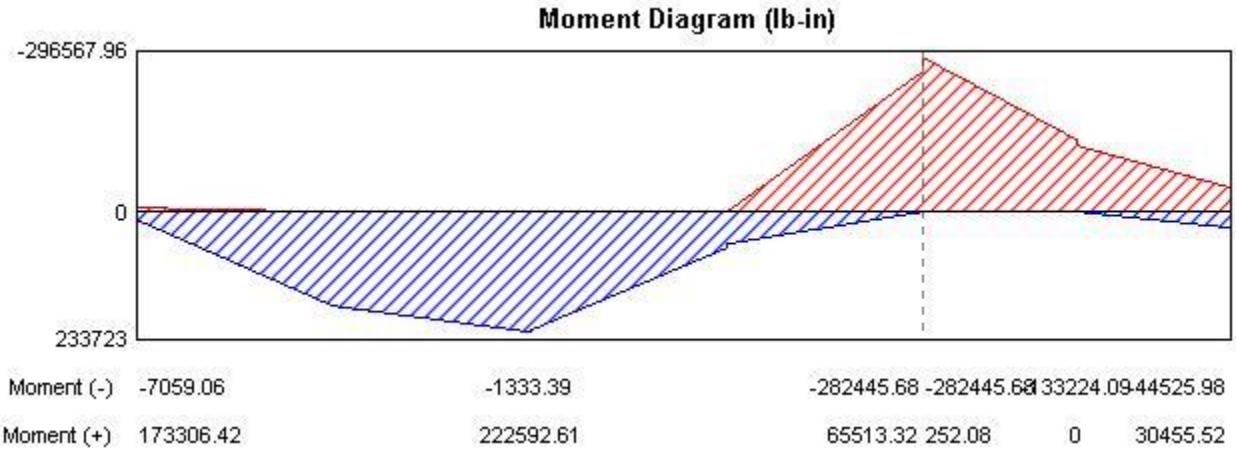
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB15
Length = 260.193 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

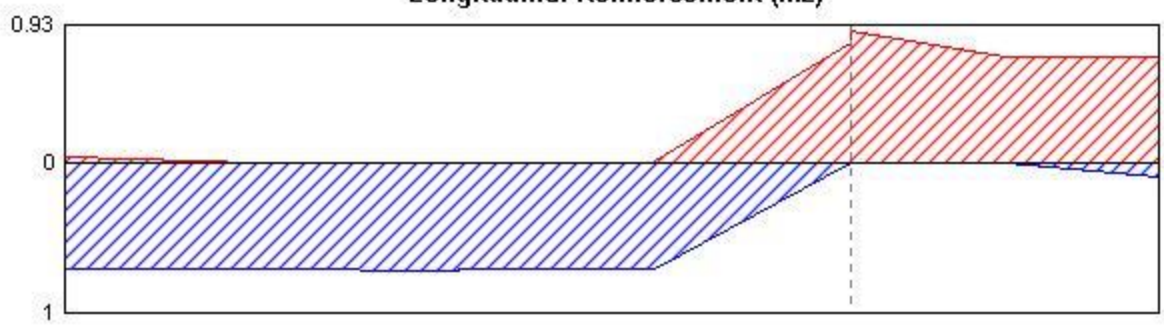
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





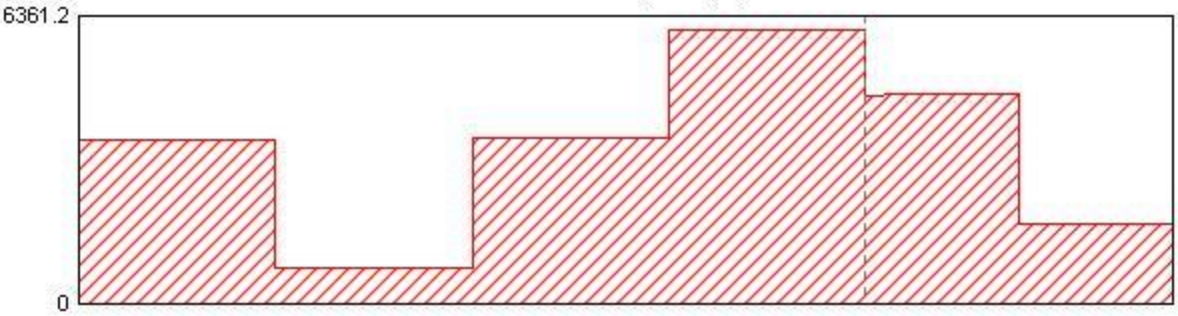
Moment (-)	-7059.06	-1333.39	-282445.68	-282445.68	33224.09	44525.98	
Moment (+)	173306.42	222592.61	65513.32	252.08	0	30455.52	

Longitudinal Reinforcement (in2)



As (top)	4.312E-002	1.599E-002	0.89	0.89	0.41	0.14
Combo	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr			
As (bot)	0.58	0.73	0.22	0	0	9.411E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr			

Shear Diagram (lb)



Shear	0	3632.78	3657.27	6030.09	4619.7	1750.56

ACI 318-14 Concrete Strip Design

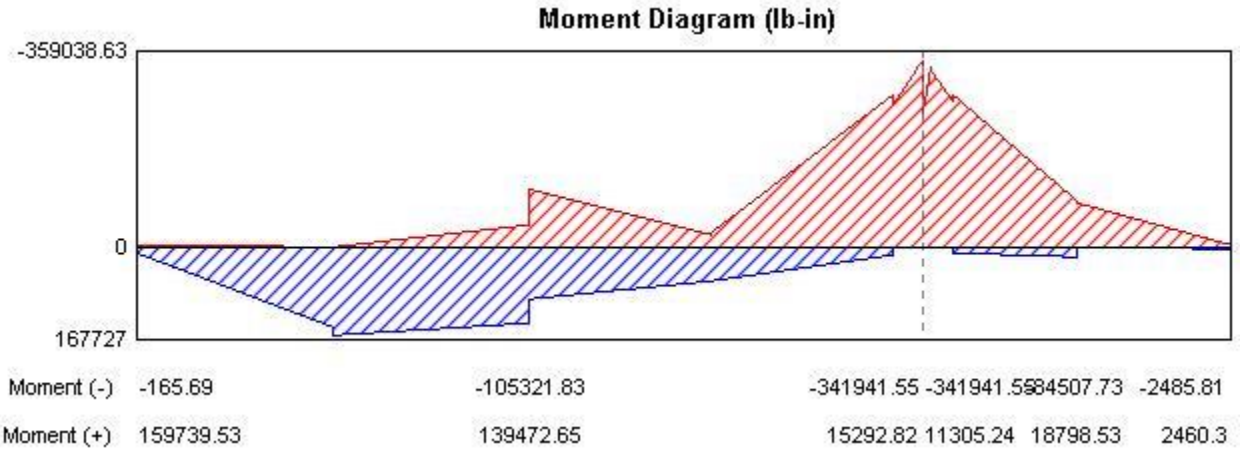
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB16
Length = 260.193 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

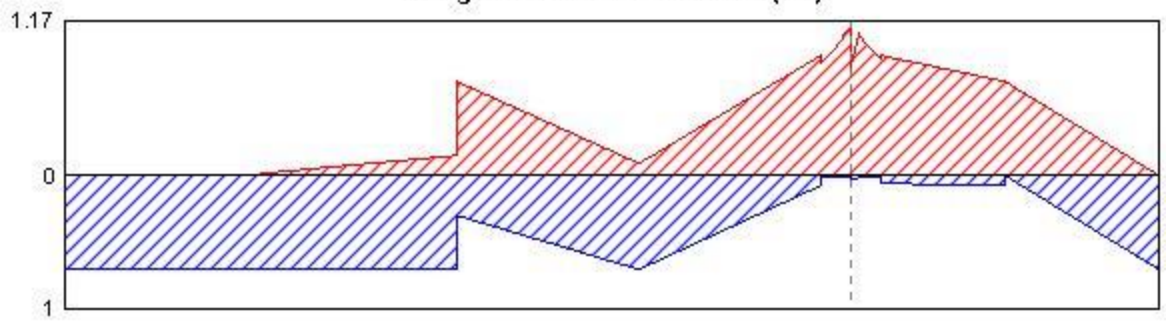
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





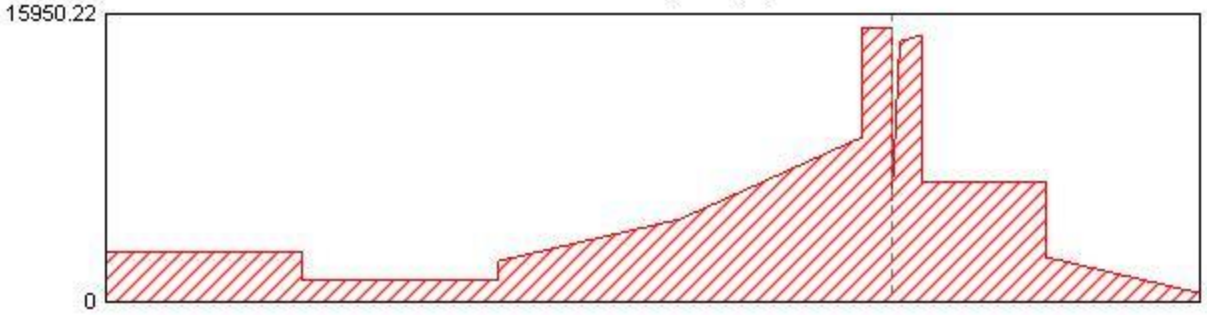
Moment (-)	-165.69	-105321.83	-341941.55	-341941.59	84507.73	-2485.81
Moment (+)	159739.53	139472.65	15292.82	11305.24	18798.53	2460.3

Longitudinal Reinforcement (in2)



As (top)	1.835E-002	0.33	1.11	1.11	0.27	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Co		Lr
As (bot)	0.52	0.46	7.05E-002	5.734E-002	2.051E-002	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Co		U1

Shear Diagram (lb)



Shear	0	1233.36	9084.03	15190.69	6628.36	462.95

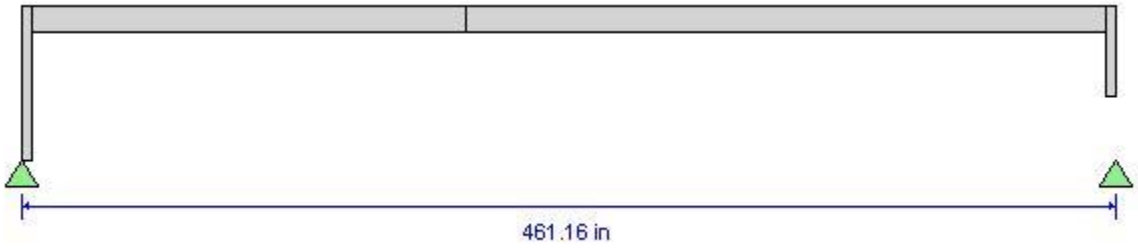
ACI 318-14 Concrete Strip Design

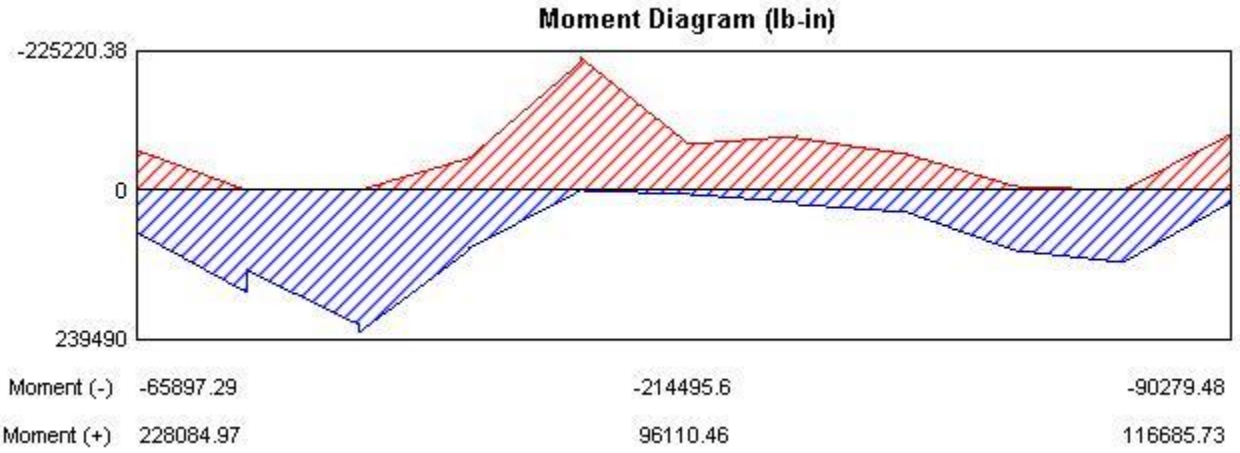
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB17
Length = 461.16 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

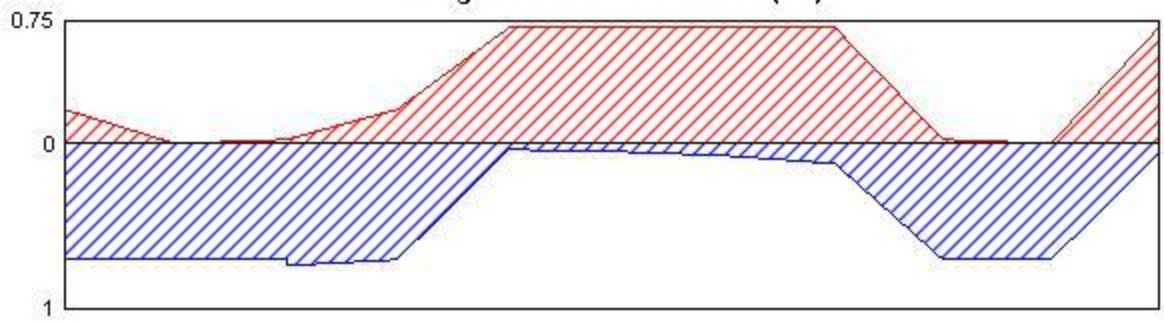
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





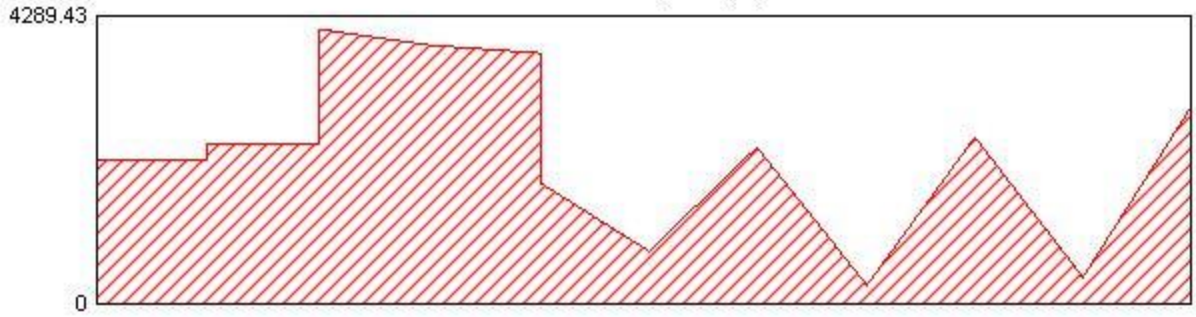
Moment (-)	-65897.29	-214495.6	-90279.48
Moment (+)	228084.97	96110.46	116685.73

Longitudinal Reinforcement (in2)



As (top)	0.21	0.71	0.28
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.74	0.34	0.37
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	3839.59	2486.39

ACI 318-14 Concrete Strip Design

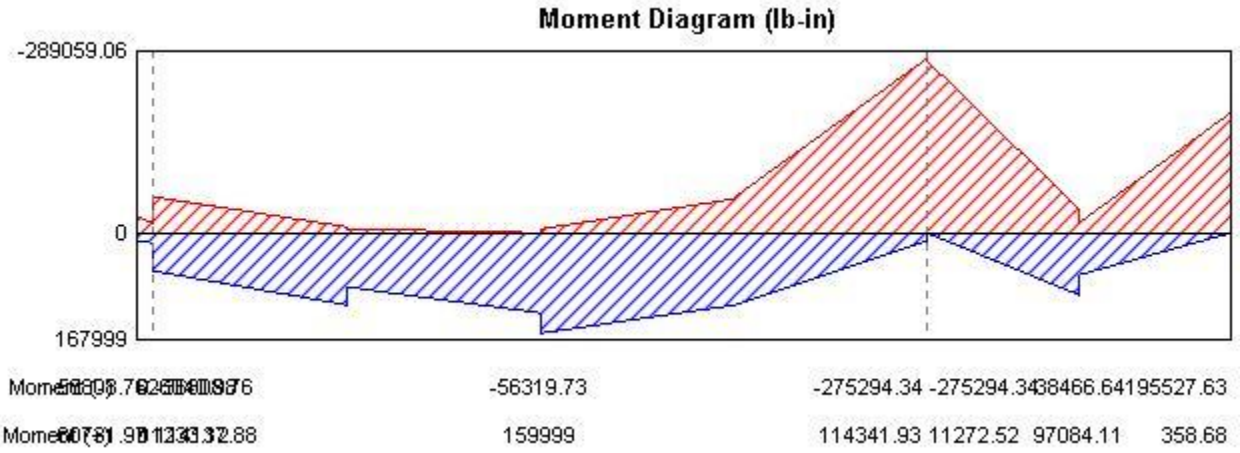
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB18
Length = 465.097 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

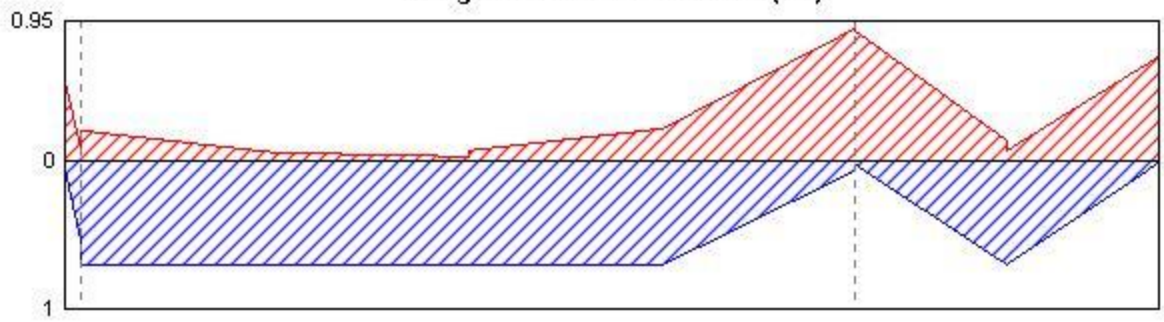
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





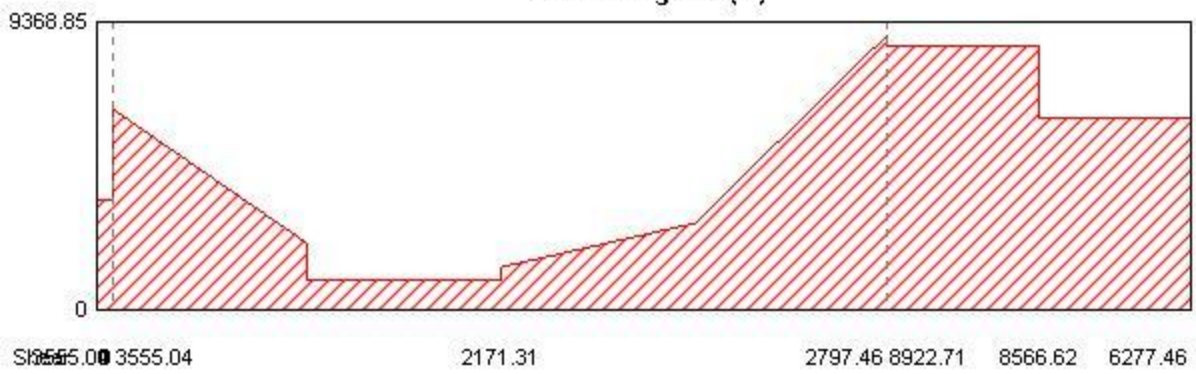
Mome[illegible]76		-56319.73		-275294.34	-275294.34	38466.64 195527.63
Mome[illegible]2.88		159999		114341.93	11272.52	97084.11 358.68

Longitudinal Reinforcement (in2)

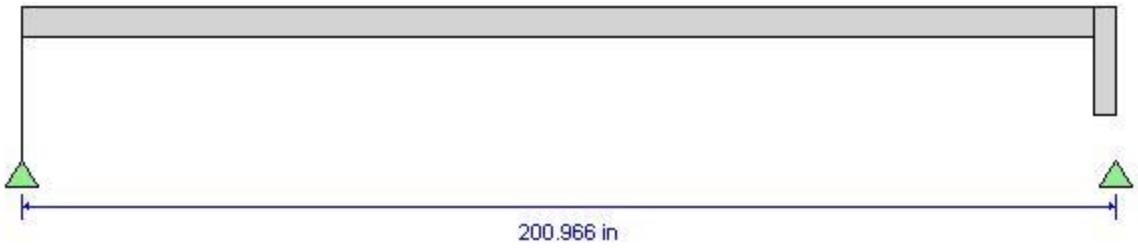


As (top)	0.[illegible]	0.22	0.9	0.9	0.14	0.62
) + 1.6 Lr	Comb=1.2 D + 1.6 L +1.6 L	Comb=1.2 D + 1.6 L +1.6 L	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.2[illegible]E-002	0.54	0.4	7.349E-002	0.32	1.898E-002
) + 1.6 Lr	Comb=1.2 D + 1.6 L +1.6 L	Comb=1.2 D + 1.6 L +1.6 L	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

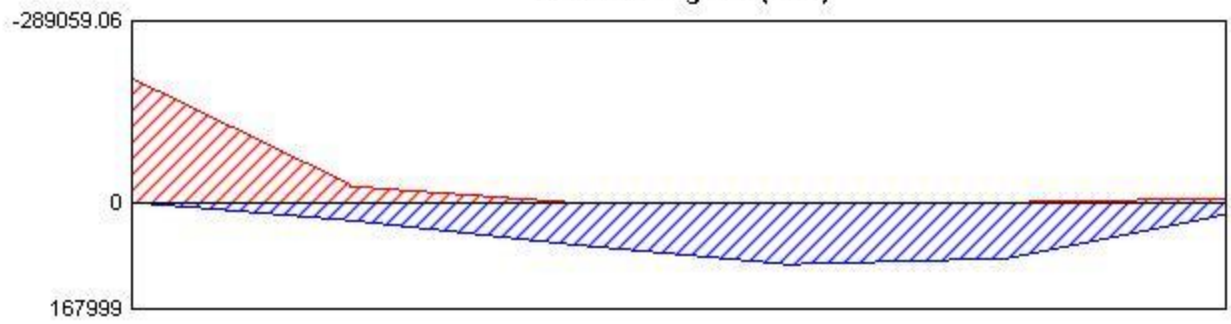
Shear Diagram (lb)



Sh[illegible]	3555.04	2171.31	2797.46	8922.71	8566.62	6277.46

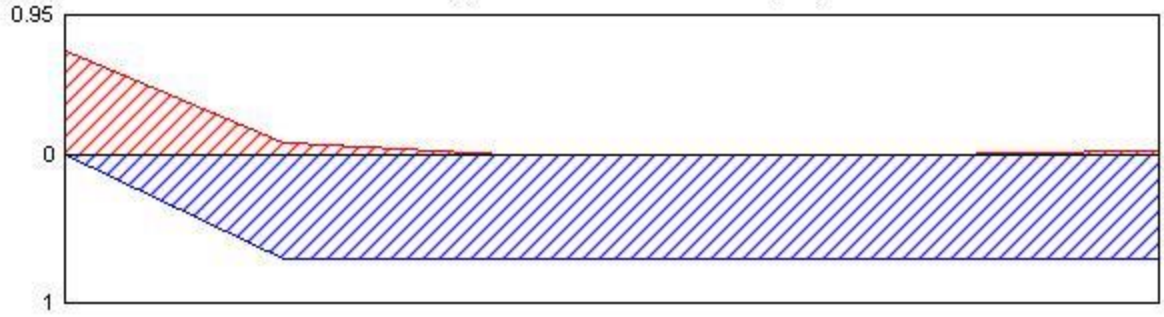


Moment Diagram (lb-in)



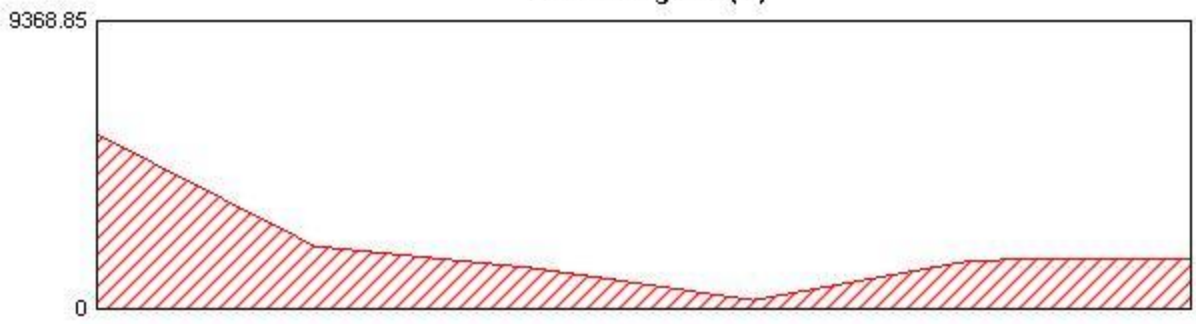
Moment (-)	-195527.63	-482.62	-5821.32
Moment (+)	27446.89	97124.44	88657.76

Longitudinal Reinforcement (in2)



As (top)	0.62	0	2.245E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	8.818E-002	0.3	0.28
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	6277.46	1290.58	1531.65

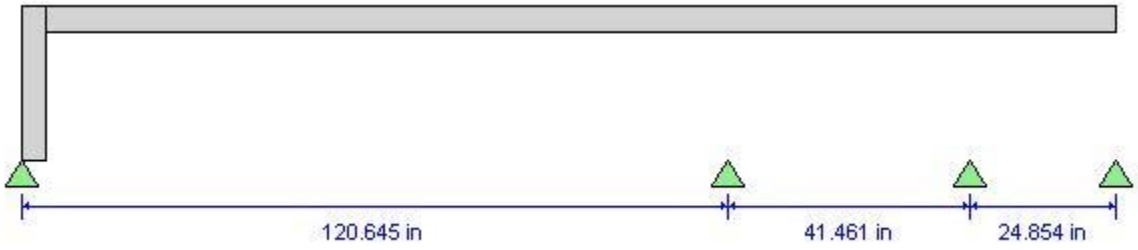
ACI 318-14 Concrete Strip Design

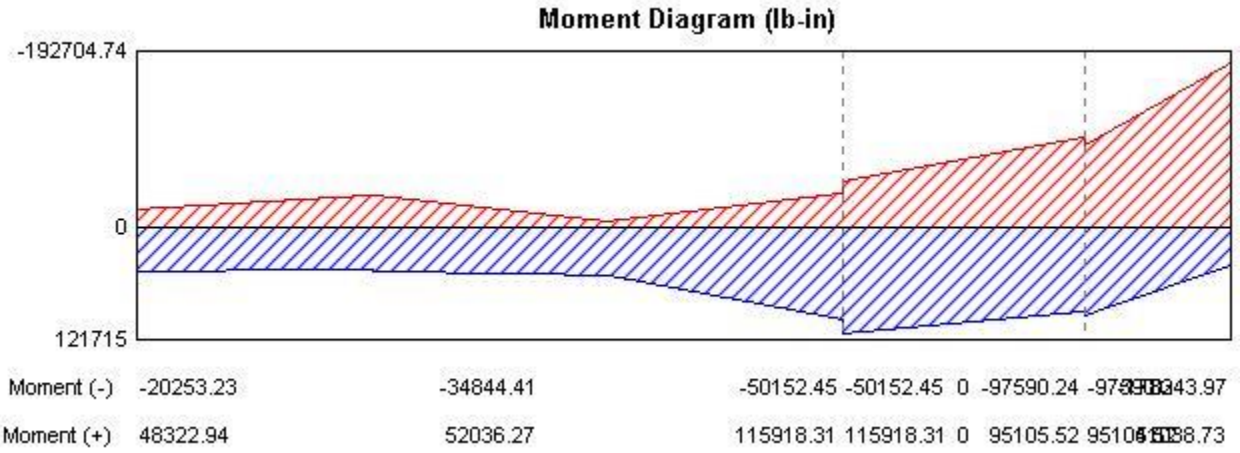
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB19
Length = 461.16 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

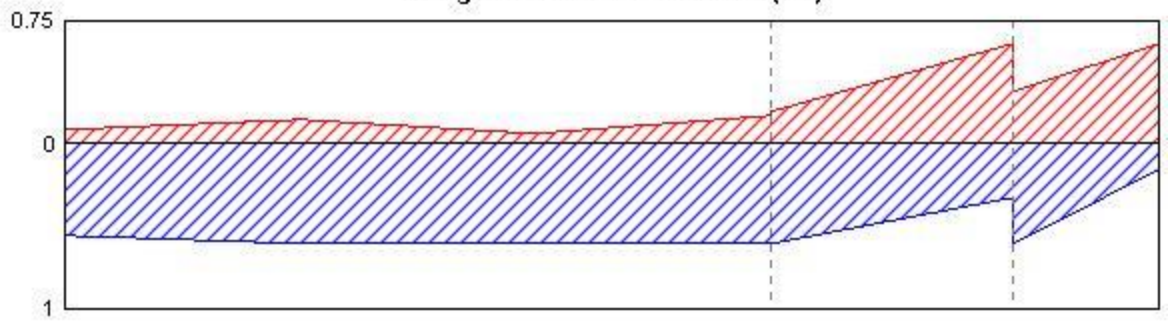
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





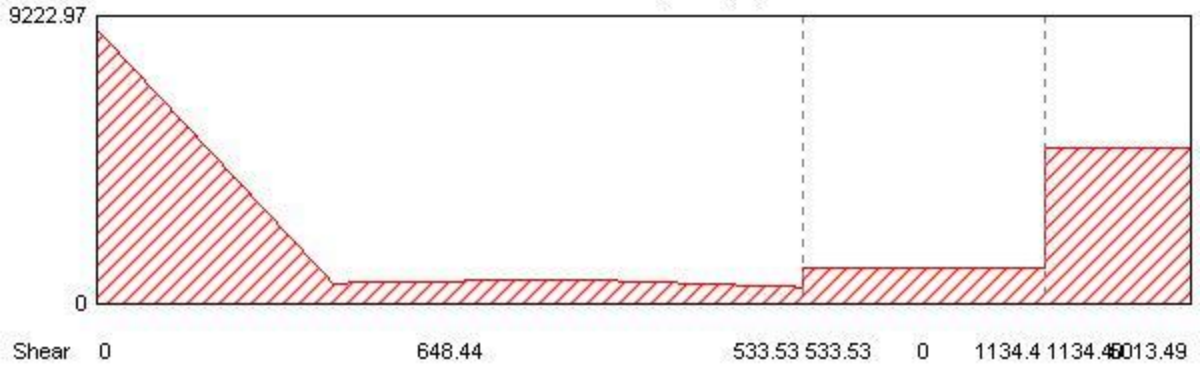
Moment (-)	-20253.23	-34844.41	-50152.45	-50152.45	0	-97590.24	-97[illegible]43.97
Moment (+)	48322.94	52036.27	115918.31	115918.31	0	95105.52	95106[illegible]88.73

Longitudinal Reinforcement (in2)

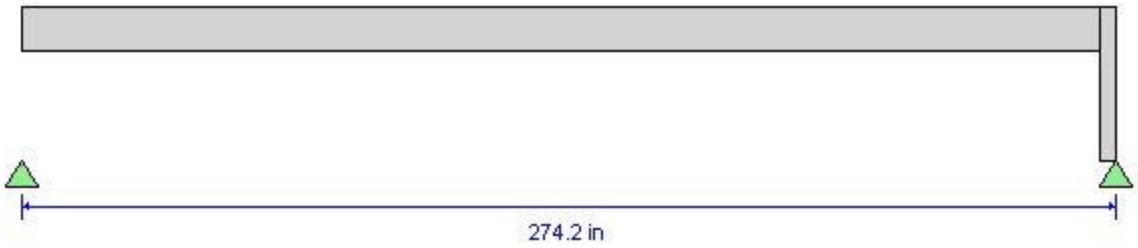


As (top)	7.969E-002	0.14	0.2	0.2	0	0.35	0.35	0	0.59
Combo	Comb=1.2 D + 1.6 L +1	Comb=1.2 D + 1.6 L +1	Comb=1.2 D + 1.6 L +1.6						
As (bot)	0.17	0.21	0.4	0.4	0	0.34	0.34	0	0.16
Combo	Comb=1.2 D + 1.6 L +1	Comb=1.2 D + 1.6 L +1	Comb=1.2 D + 1.6 L +1.6						

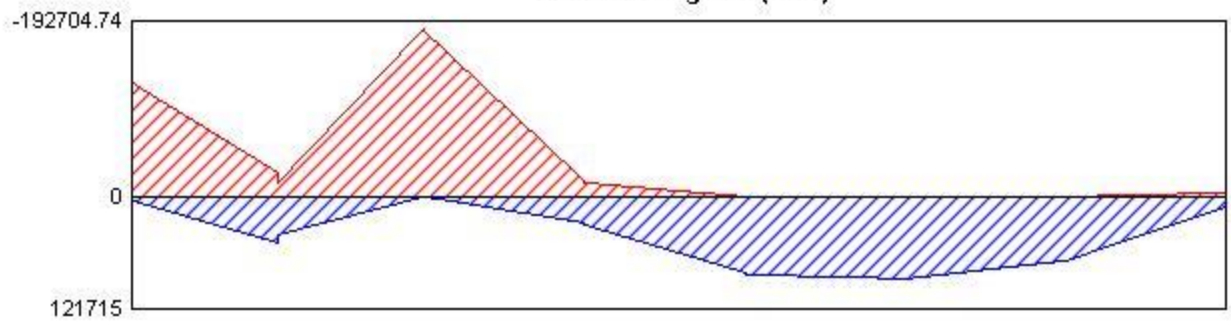
Shear Diagram (lb)



Shear	0	648.44	533.53	533.53	0	1134.4	1134.4	6013.49	

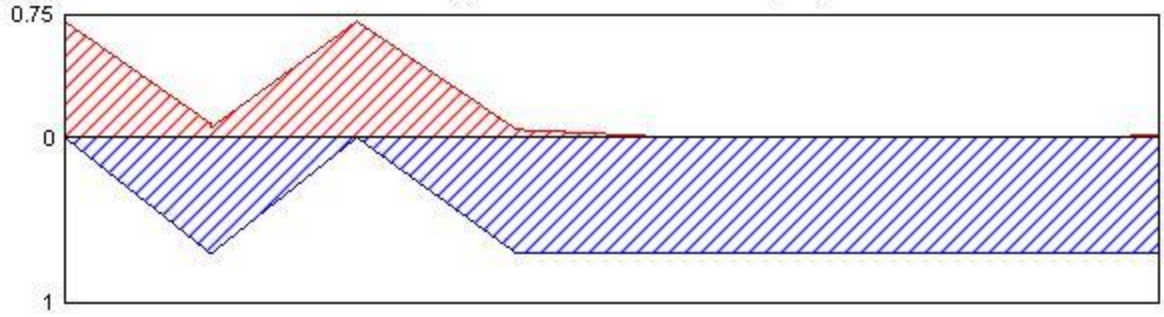


Moment Diagram (lb-in)



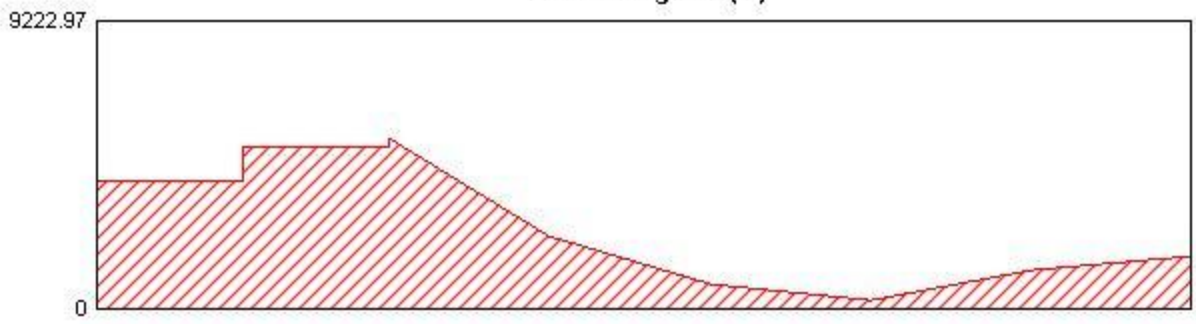
Moment (-)	-178343.97	-183528.32	-4343.28
Moment (+)	41038.73	90346	70608.89

Longitudinal Reinforcement (in2)



As (top)	0.59	0.59	1.615E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.16	0.28	0.22
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	5013.49	5170.3	1235.92

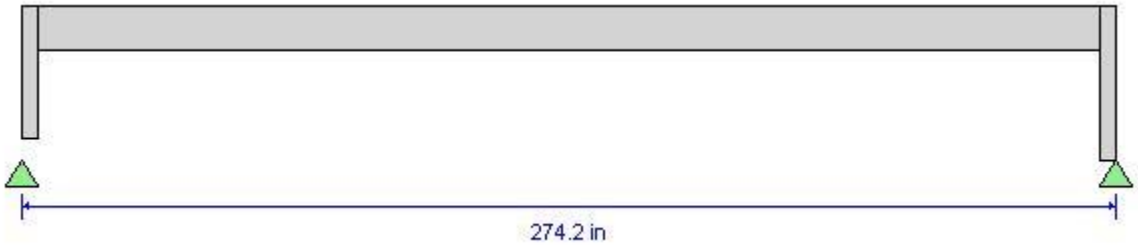
ACI 318-14 Concrete Strip Design

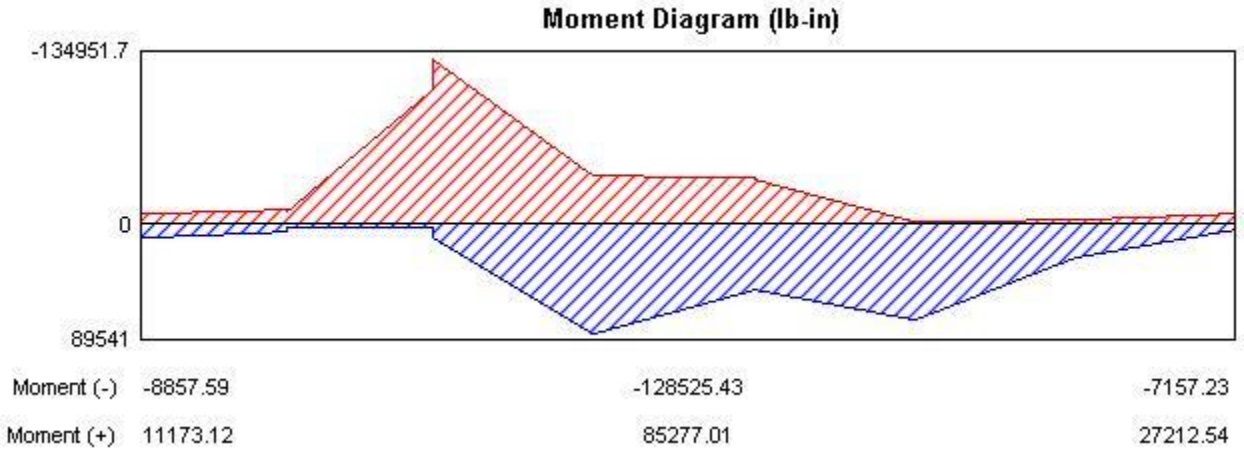
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB20
Length = 274.2 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

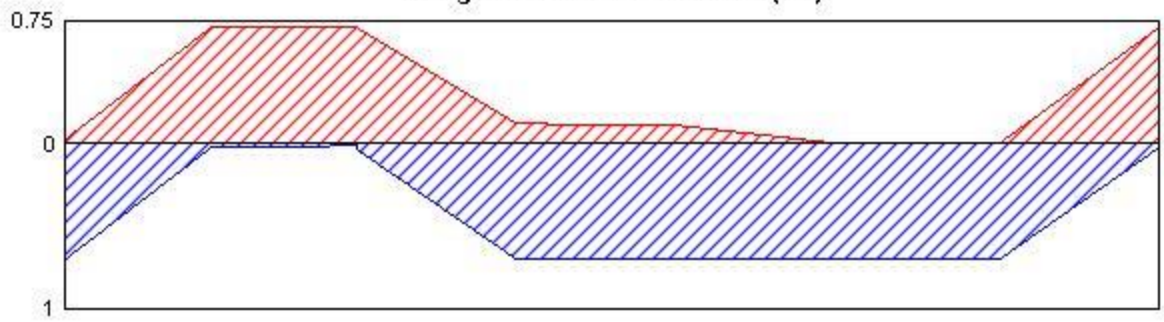
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





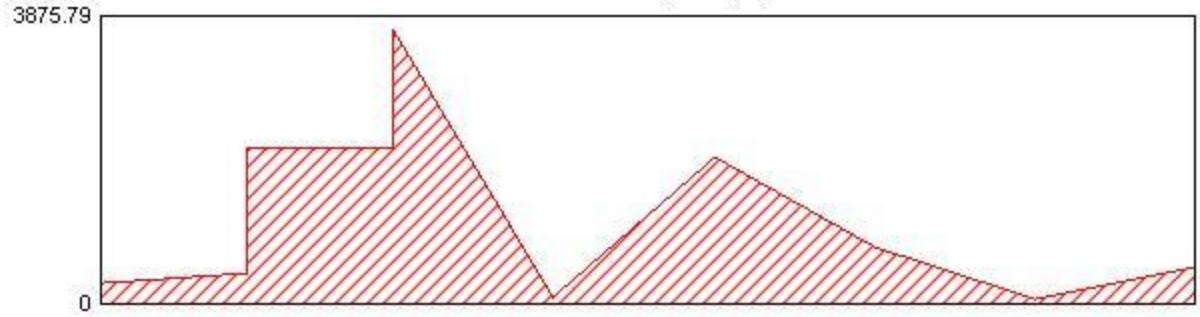
Moment (-)	-8857.59	-128525.43	-7157.23
Moment (+)	11173.12	85277.01	27212.54

Longitudinal Reinforcement (in2)



As (top)	4.301E-002	0.4	2.537E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	4.121E-002	0.27	8.861E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	2094.86	56.61

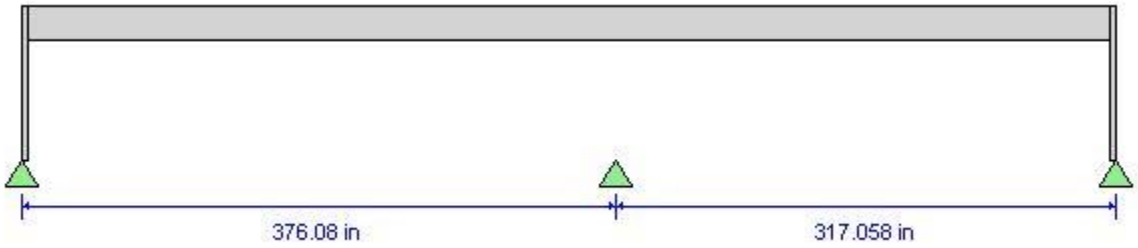
ACI 318-14 Concrete Strip Design

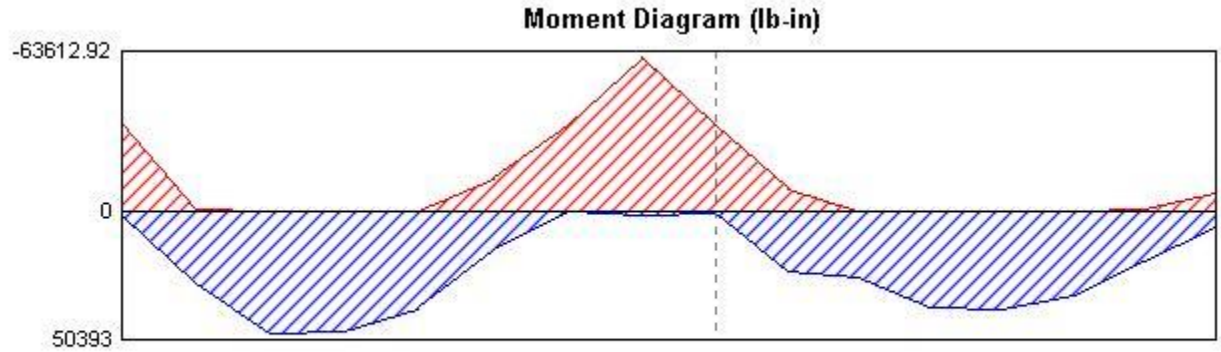
Geometric Properties

Combination = Overall Envelope
Strip Label = MSA13
Length = 693.138 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

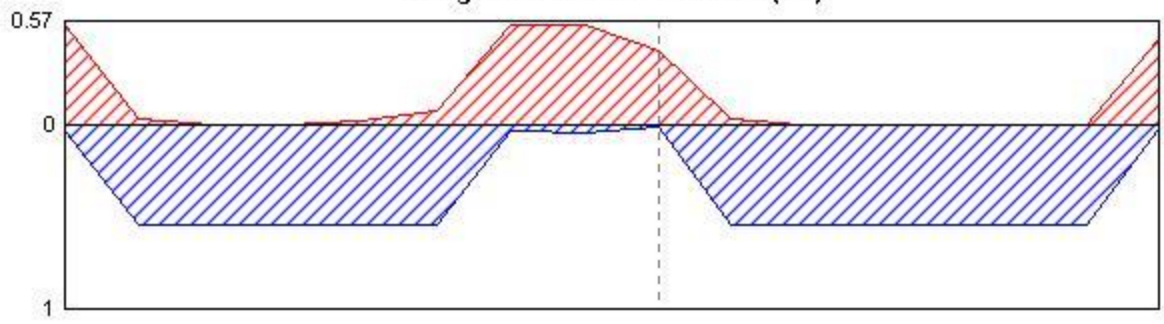
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





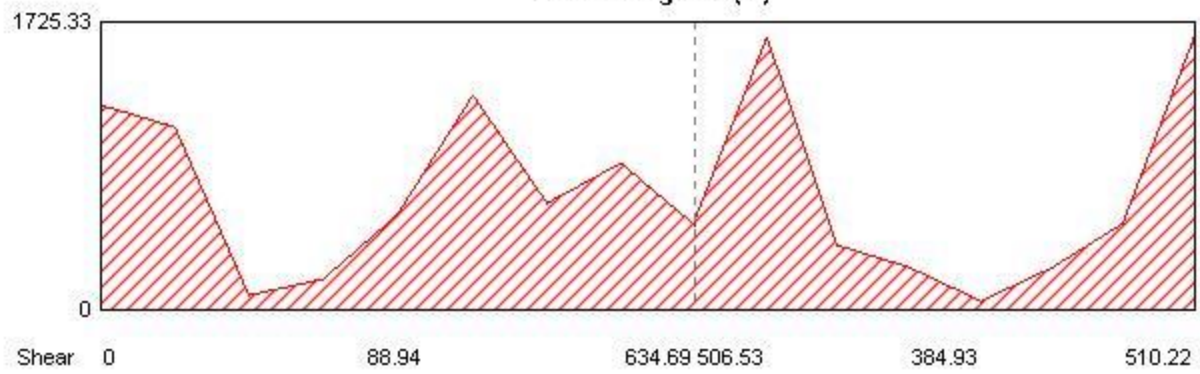
Moment (-)	-35230.77	-34420.62	-60583.73 -34216.59	-173.07	-6832.86
Moment (+)	47993.03	47993.03	1797.58 23650.12	38957.27	19695.36

Longitudinal Reinforcement (in2)



As (top)	0.16	0.18	0.29	0.15	0	2.758E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1
As (bot)	0.22	0.22	4.699E-002	9.57E-002	0.16	7.965E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	88.94	634.69	506.53	384.93	510.22

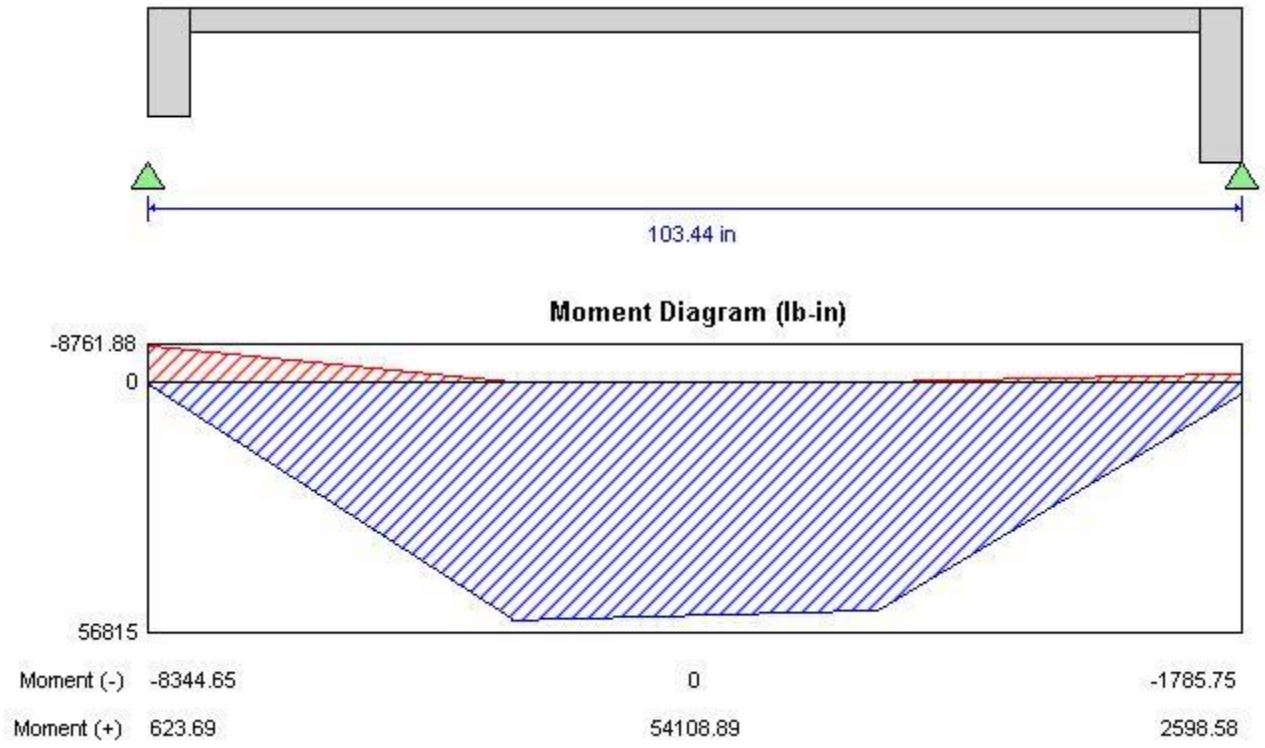
ACI 318-14 Concrete Strip Design

Geometric Properties

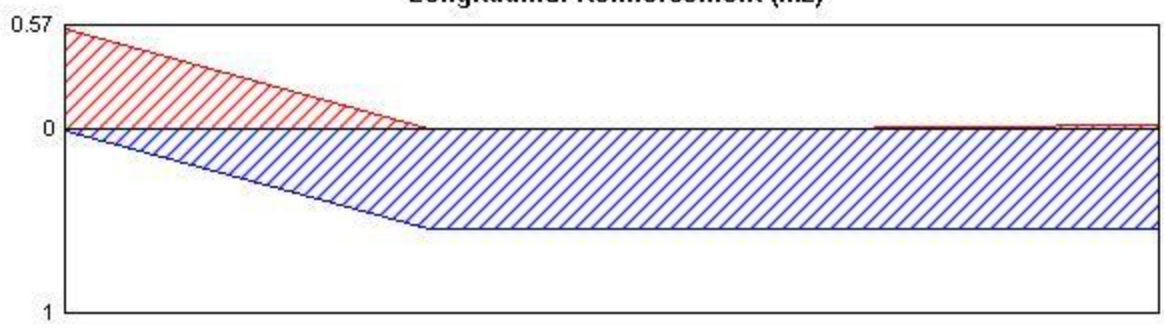
Combination = Overall Envelope
Strip Label = MSB9
Length = 103.44 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2

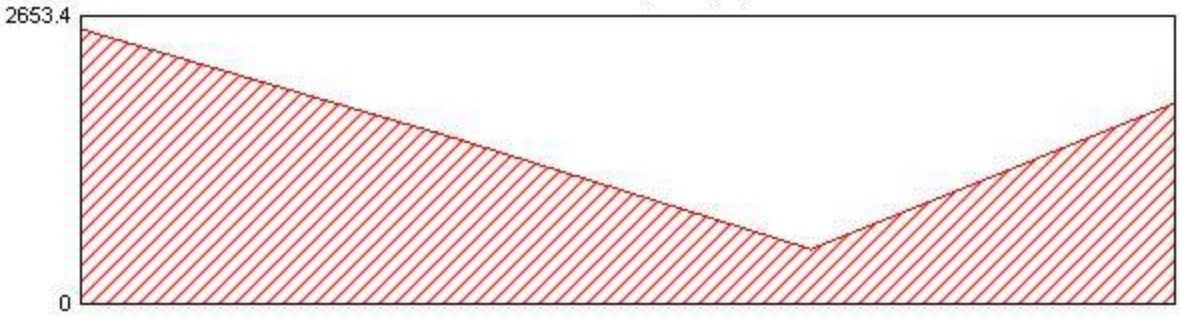


Longitudinal Reinforcement (in2)



As (top)	4.584E-002	0	1.948E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		DCONU1
As (bot)	1.058E-002	0.26	2.28E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	1550.12	1845.4

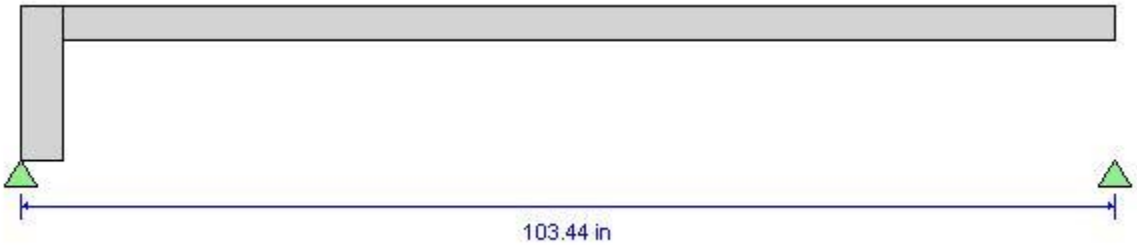
ACI 318-14 Concrete Strip Design

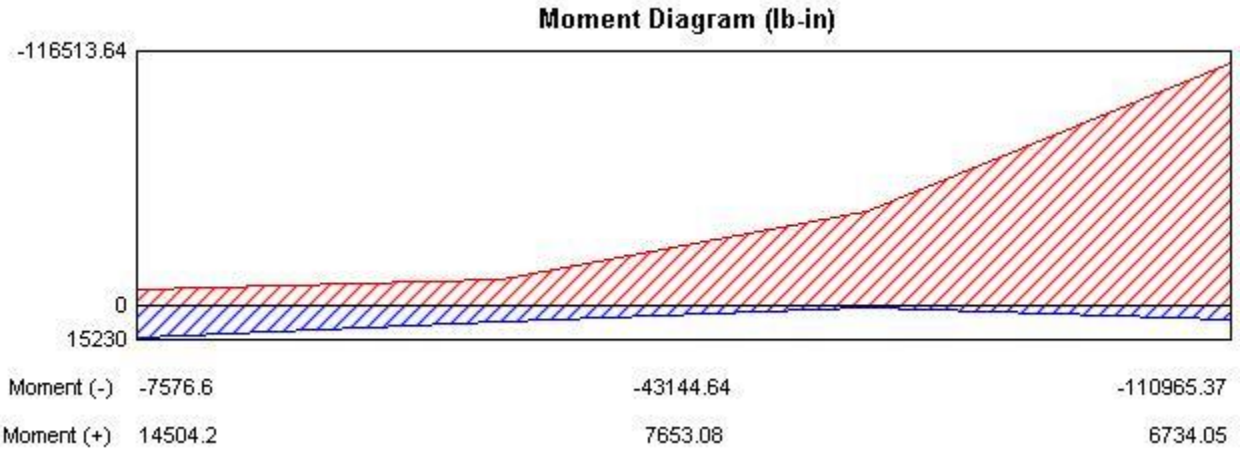
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB21
Length = 103.44 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

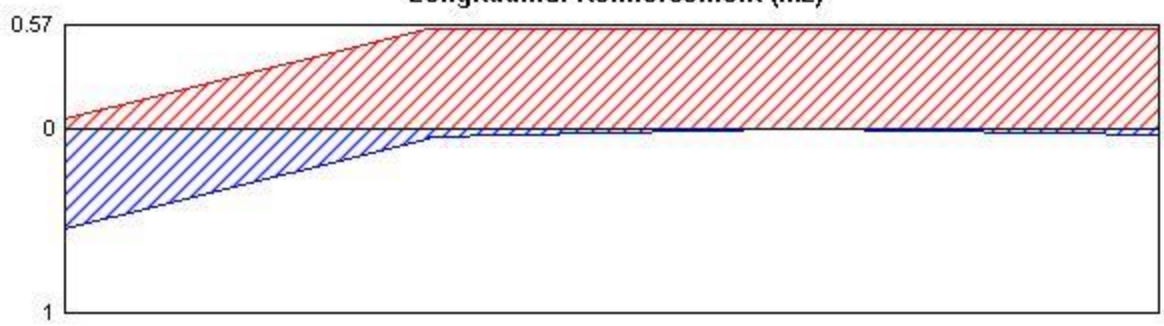
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





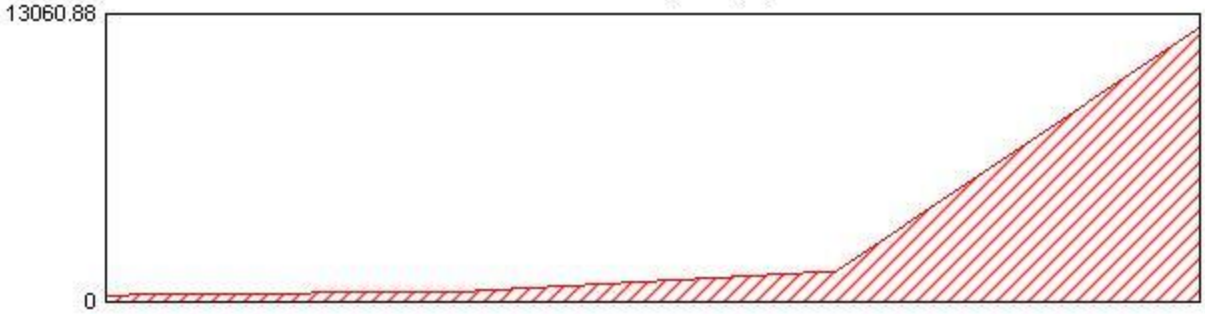
Moment (-)	-7576.6	-43144.64	-110965.37
Moment (+)	14504.2	7653.08	6734.05

Longitudinal Reinforcement (in2)



As (top)	4.912E-002	0.2	0.51
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	8.046E-002	4.91E-002	3.176E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	442	12438.93

ACI 318-14 Concrete Strip Design

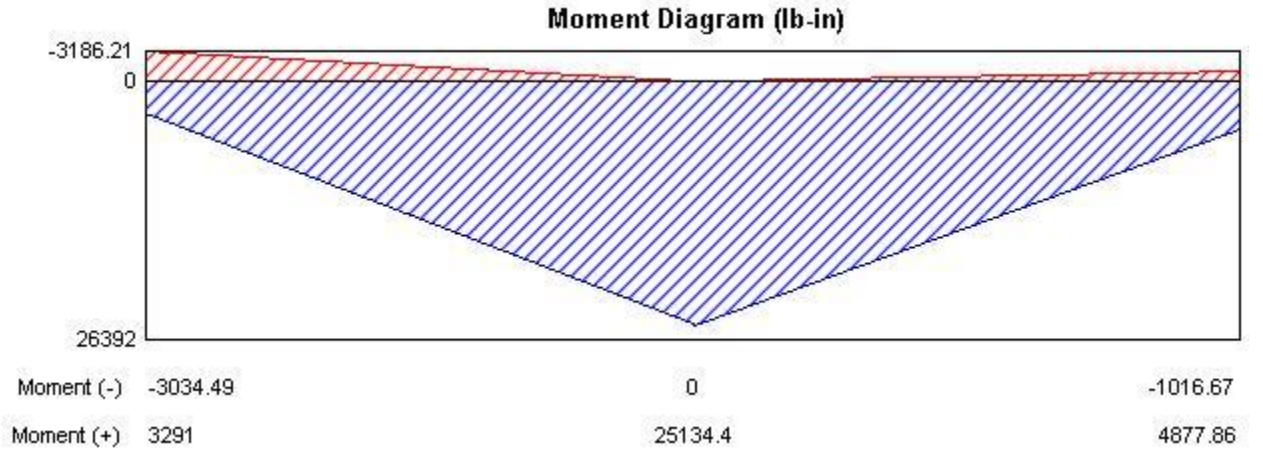
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB22
Length = 58.92 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

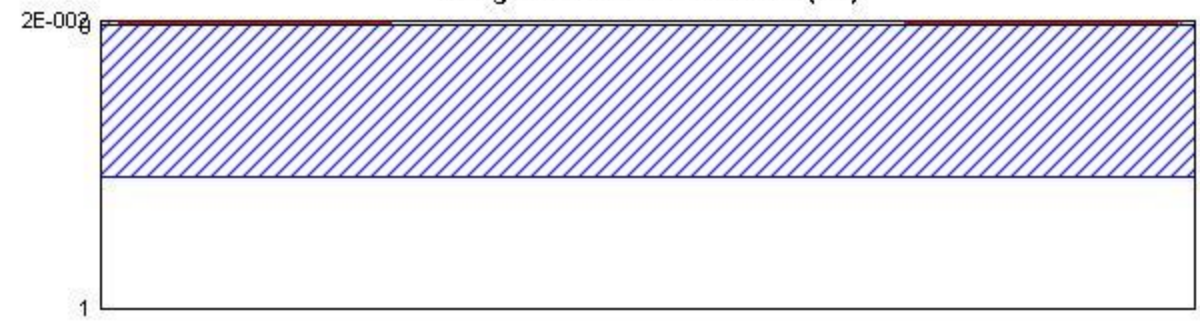
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





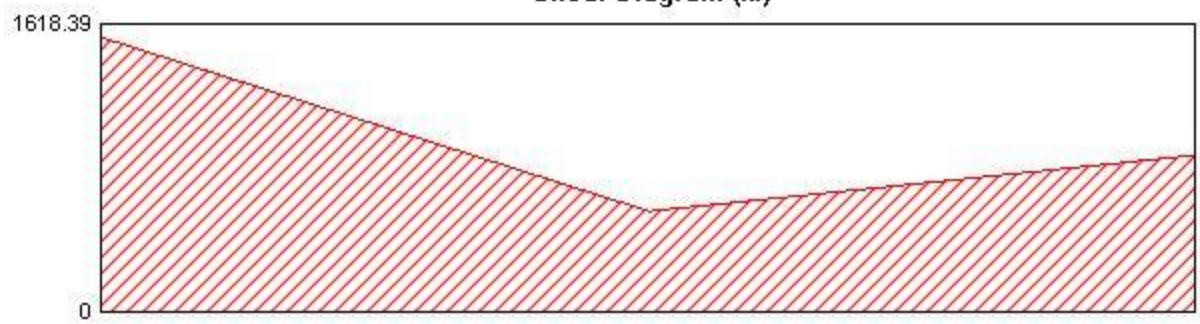
Moment (-)	-3034.49	0	-1016.67
Moment (+)	3291	25134.4	4877.86

Longitudinal Reinforcement (in2)



As (top)	1.534E-002	0	1.086E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	1.65E-002	0.12	2.827E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	558.57	876.56

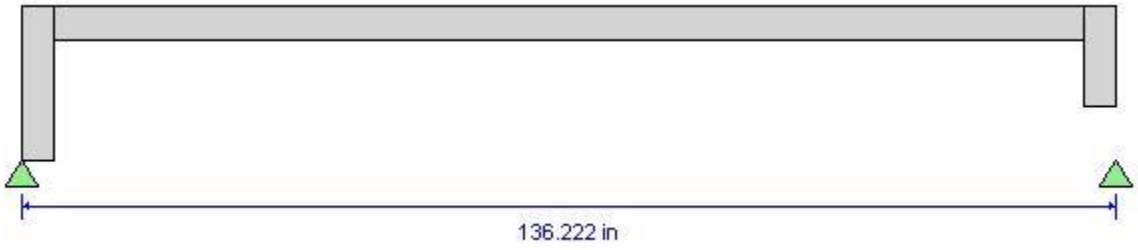
ACI 318-14 Concrete Strip Design

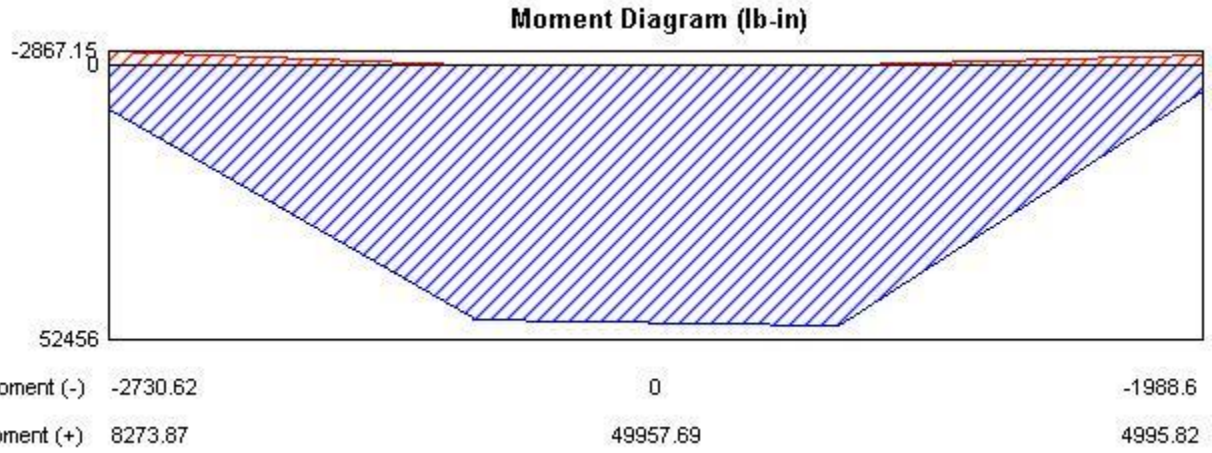
Geometric Properties

Combination = Overall Envelope
Strip Label = MSB23
Length = 136.222 in
Distance to Top Rebar Center = 1.75 in
Distance to Bot Rebar Center = 1.75 in

Material Properties

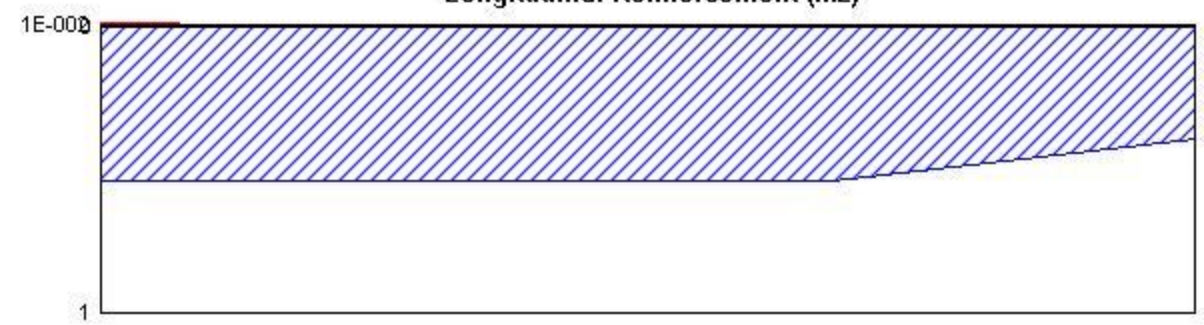
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





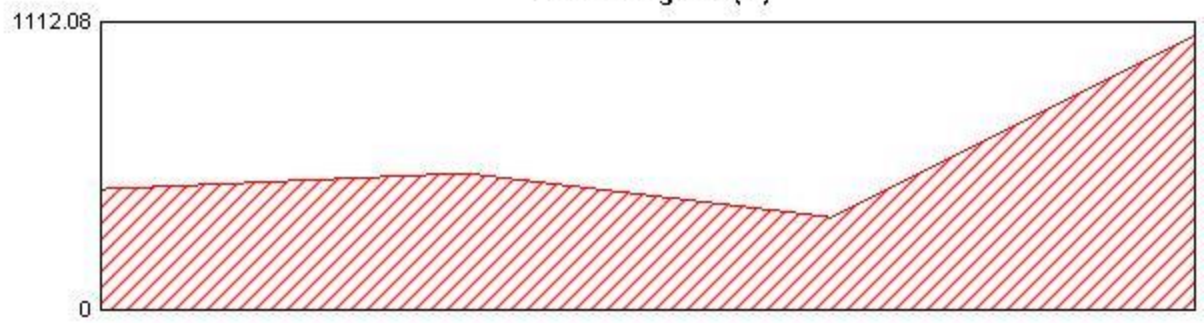
Moment (-)	-2730.62	0	-1988.6
Moment (+)	8273.87	49957.69	4995.82

Longitudinal Reinforcement (in2)



As (top)	1.414E-002	0	1.047E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	3.918E-002	0.23	2.404E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	527.03	1059.12

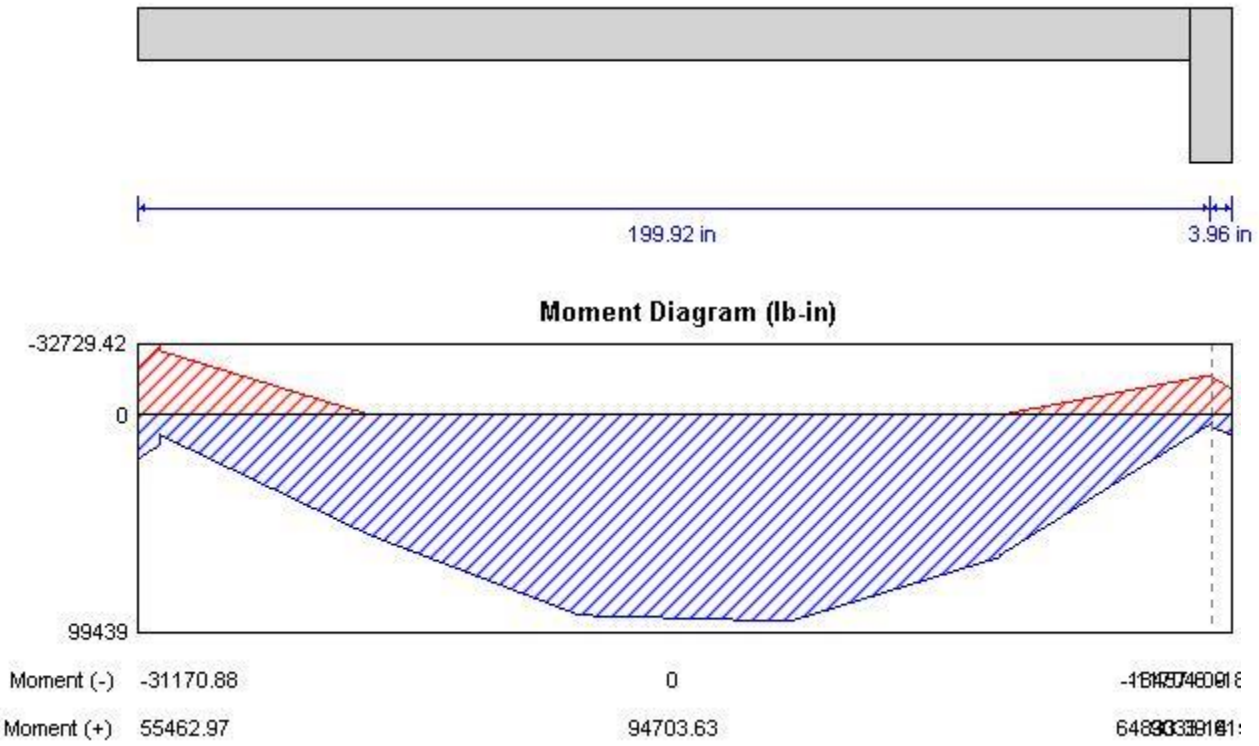
ACI 318-14 Concrete Strip Design

Geometric Properties

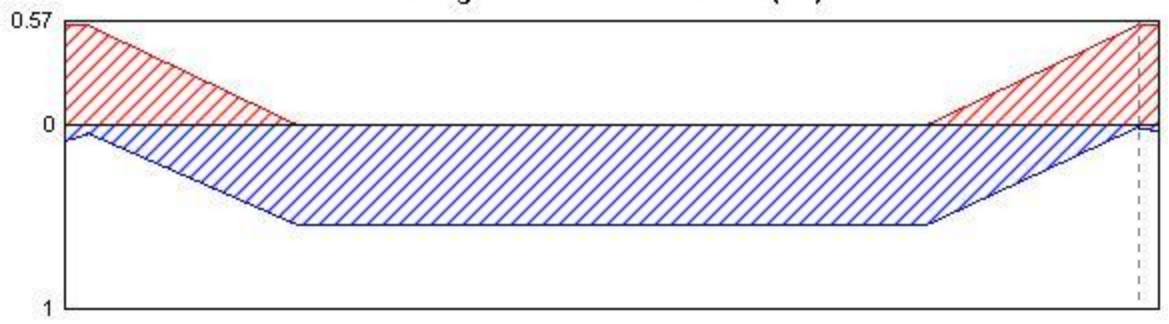
Combination = Overall Envelope
Strip Label = MSA14
Length = 203.879 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2

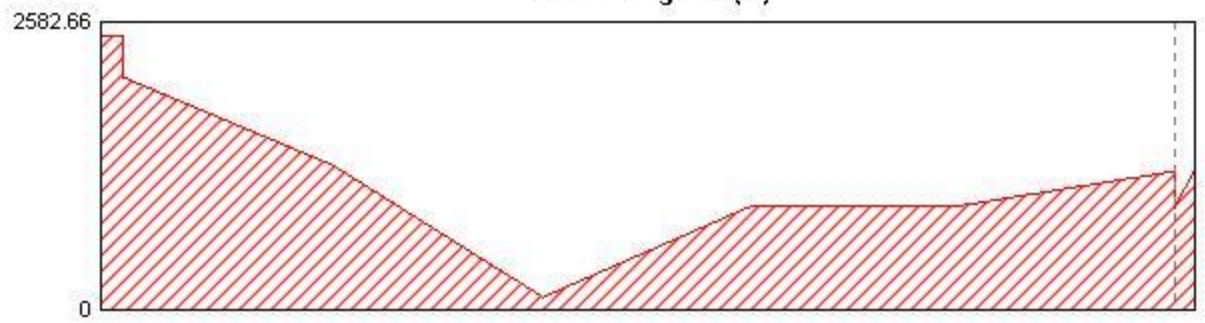


Longitudinal Reinforcement (in2)



As (top)	0.13	0	7.4
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Co
As (bot)	0.23	0.39	3.73
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Co

Shear Diagram (lb)



Shear	0	100.6	9

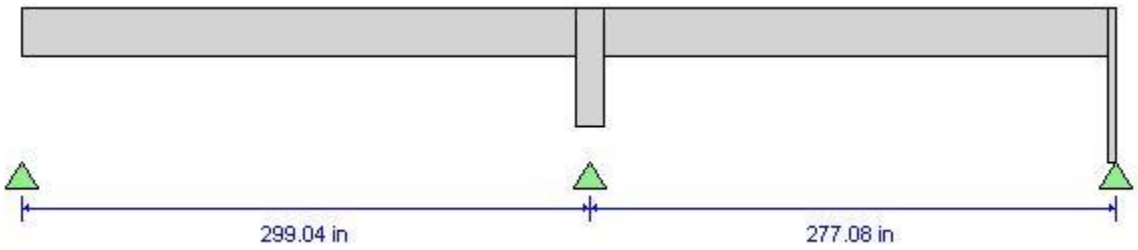
ACI 318-14 Concrete Strip Design

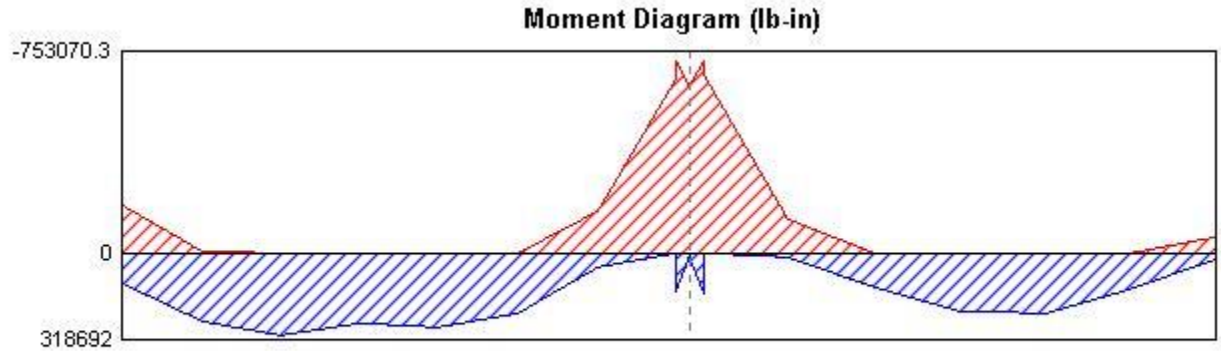
Geometric Properties

Combination = Overall Envelope
Strip Label = CSA1
Length = 576.12 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

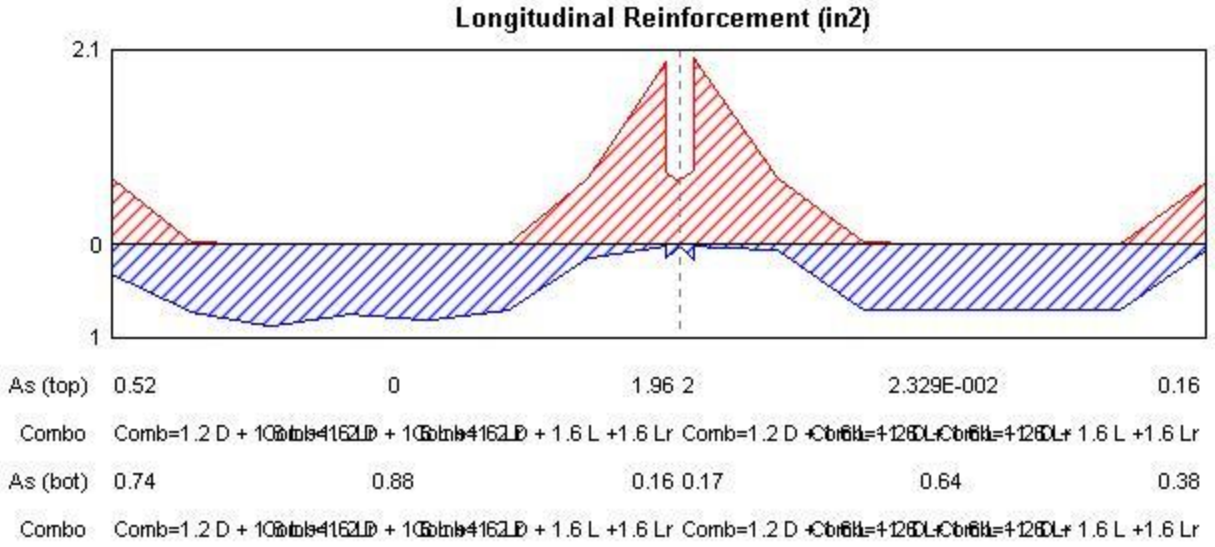
Material Properties

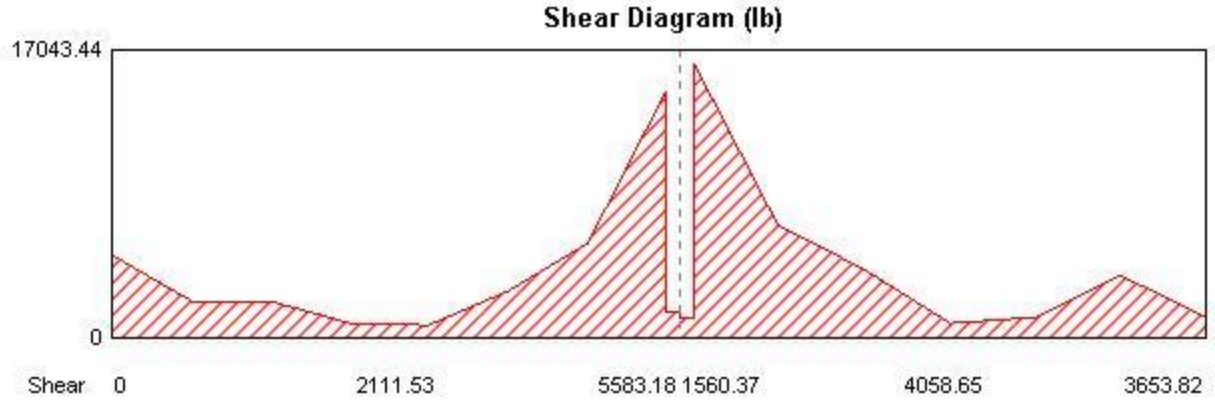
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





Moment (-)	-178680.26	-509.54	-649866.76	-660955.01	-2550.18	-56553.12
Moment (+)	255799.96	303515.32	50242.6	149884.19	219321.06	132750.58


Longitudinal Reinforcement (in2)
2.1
0
1
As (top) 0.52 0 1.96 2 2.329E-002 0.16
Combo Comb=1.2 D + 1 + 1.6 L +1.6 Lr Comb=1.2 D + 1 + 1.6 L +1.6 Lr Comb=1.2 D + + 1.6 L +1.6 Lr
As (bot) 0.74 0.88 0.16 0.17 0.64 0.38
Combo Comb=1.2 D + 1 + 1.6 L +1.6 Lr Comb=1.2 D + 1 + 1.6 L +1.6 Lr Comb=1.2 D + + 1.6 L +1.6 Lr


Shear Diagram (lb)
17043.44
0
Shear 0 2111.53 5583.18 1560.37 4058.65 3653.82

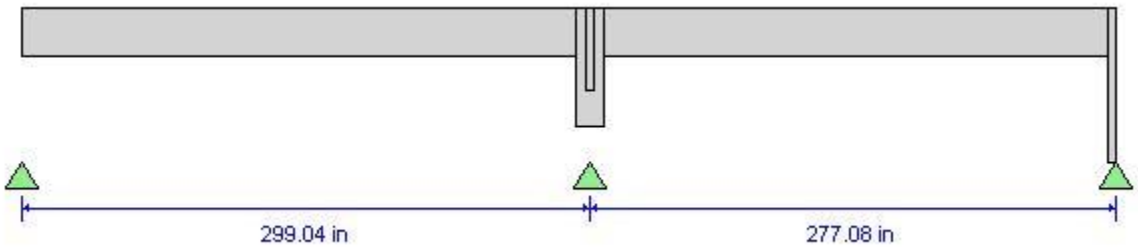
ACI 318-14 Concrete Strip Design

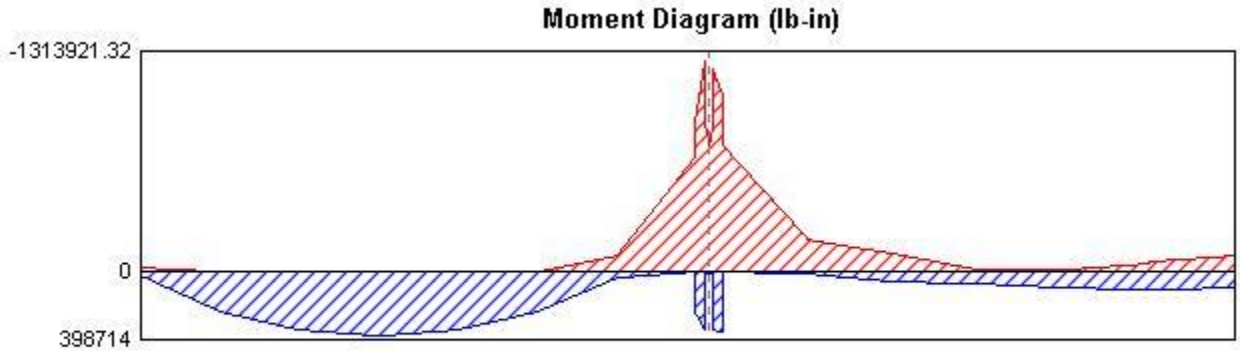
Geometric Properties

Combination = Overall Envelope
Strip Label = CSA2
Length = 576.12 in
Distance to Top Rebar Center = 1.25 in
Distance to Bot Rebar Center = 1.25 in

Material Properties

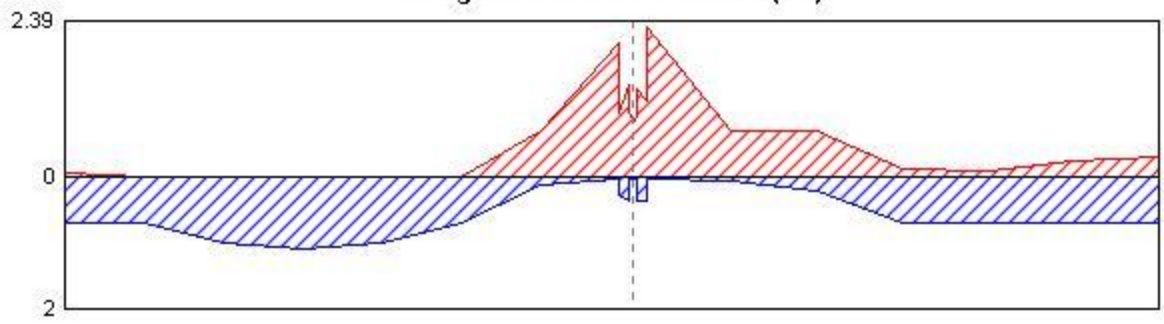
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2





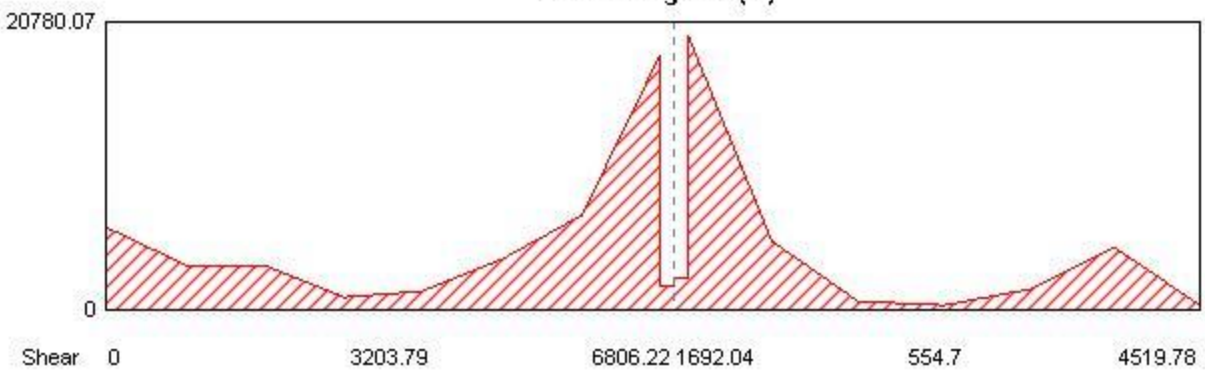
Moment (-)	-18248.21	0	-676077.8	-746469.61	-109077.95	-95336.09
Moment (+)	233529.96	379727.06	341608.65	351834.74	92208.99	104955.72

Longitudinal Reinforcement (in2)



As (top)	5.759E-002	0	2.05	2.28	0.36	0.31
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D +			1.6 L +1.6 Lr
As (bot)	0.68	1.11	0.37	0.38	0.35	0.38
Combo	Comb=1.2 D +		+ 1.6 L +1.6 Lr	Comb=1.2 D +		1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	0	3203.79	6806.22	1692.04	554.7	4519.78

6.4. Beam design

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Line	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
1	Span 1	Start	DCONU1	-3504.17	0.00
1	Span 1	Middle		0.00	0.00
1	Span 1	End		0.00	0.00
3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-25120.22	1.416E-002
3	Span 1	Middle		0.00	0.00
3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-87349.59	6.104E-002
4	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-60134.02	4.499E-002
4	Span 1	Middle		0.00	0.00
4	Span 1	End	DCONU1	-32550.51	2.545E-002
23	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-2829.60	0.00
23	Span 1	Middle		0.00	0.00
23	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-41547.89	5.360E-002
25	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-137429.59	0.12
25	Span 1	Middle		0.00	0.00
25	Span 1	End		0.00	0.00
26	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-377654.93	0.28
26	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-228500.92	0.18
26	Span 1	End		0.00	0.00
27	Span 1	Start		0.00	0.00
27	Span 1	Middle		0.00	0.00
27	Span 1	End		0.00	0.00
42	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-255386.70	0.20
42	Span 1	Middle		0.00	0.00
42	Span 1	End	DCONU1	-197680.54	0.13
43	Span 1	Start	DCONU1	-202798.02	0.14
43	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-177371.87	0.11
43	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-134345.00	8.061E-002
45	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-165858.70	7.884E-002
45	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-42407.88	1.916E-002
45	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1093.21	0.00
46	Span 1	Start	DCONU1	-4230.44	7.058E-003
46	Span 1	Middle		0.00	0.00
46	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1109.55	0.00
47	Span 1	Start		0.00	0.00
47	Span 1	Middle		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Line	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
47	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-35715.59	6.591E-002
58	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-394509.10	0.27
58	Span 1	Middle		0.00	0.00
58	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-61229.81	3.485E-002
61	Span 1	Start		0.00	0.00
61	Span 1	Middle		0.00	0.00
61	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-433301.52	0.32
62	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-440785.82	0.32
62	Span 1	Middle		0.00	0.00
62	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-98128.88	5.819E-002
63	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-88407.38	5.499E-002
63	Span 1	Middle		0.00	0.00
63	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-323257.02	0.28
64	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-1144723.11	0.59
64	Span 1	Middle		0.00	0.00
64	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1241041.93	0.60
65	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-312768.69	0.25
65	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-861569.71	0.66
65	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-606103.73	0.47
69	Span 1	Start		0.00	0.00
69	Span 1	Middle		0.00	0.00
69	Span 1	End		0.00	0.00
71	Span 1	Start	DCONU1	-4729.35	0.00
71	Span 1	Middle		0.00	0.00
71	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-73072.72	7.427E-002
72	Span 1	Start		0.00	0.44
72	Span 1	Middle		0.00	0.00
72	Span 1	End		0.00	0.00
74	Span 1	Start	DCONU1	-31979.66	0.00
74	Span 1	Middle		0.00	0.00
74	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-211119.54	0.18
75	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-267081.94	0.22
75	Span 1	Middle		0.00	0.00
75	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-255548.75	0.20
76	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-267754.95	0.20
76	Span 1	Middle		0.00	0.00
76	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-200586.73	0.15
84	Span 1	Start		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Line	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
84	Span 1	Middle		0.00	0.00
84	Span 1	End		0.00	0.00
85	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-642575.24	0.28
85	Span 1	Middle		0.00	0.00
85	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-599557.49	0.26
86	Span 1	Start	DCONU1	-32717.34	0.00
86	Span 1	Middle		0.00	0.00
86	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-917911.10	0.44
87	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-705844.98	0.75
87	Span 1	Middle		0.00	0.00
87	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-346620.34	0.36
88	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-142950.59	4.426E-002
88	Span 1	Middle		0.00	0.00
88	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-201243.93	7.586E-002
91	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-44616.55	0.00
91	Span 1	Middle		0.00	0.00
91	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-1726398.86	0.76
92	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-2150297.40	1.07
92	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-1483208.73	0.73
92	Span 1	End	DCONU1	-10272.63	0.00
93	Span 1	Start		0.00	0.00
93	Span 1	Middle		0.00	0.00
93	Span 1	End		0.00	0.00
94	Span 1	Start		0.00	3.615E-002
94	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-156551.80	0.16
94	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-242301.19	0.20
95	Span 1	Start		0.00	0.00
95	Span 1	Middle		0.00	0.00
95	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-26812.20	0.00
98	Span 1	Start	DCONU1	-132691.22	4.517E-002
98	Span 1	Middle		0.00	0.00
98	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-706699.76	0.36
99	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-1756252.16	0.65
99	Span 1	Middle		0.00	0.00
99	Span 1	End	DCONU1	-179445.89	6.191E-002
100	Span 1	Start		0.00	0.00
100	Span 1	Middle		0.00	0.00
100	Span 1	End		0.00	0.00
101	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-147200.91	8.879E-002
101	Span 1	Middle		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 1 of 3

Line	SpanID	Location	FTopCombo	FTopMoment	FTopArea
				lb-in	in2
101	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-464773.18	0.34
102	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-365532.83	0.28
102	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-202837.27	0.16
102	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-51497.59	4.252E-002
105	Span 1	Start		0.00	0.00
105	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-189761.26	0.10
105	Span 1	End		0.00	0.00
111	Span 1	Start		0.00	0.00
111	Span 1	Middle		0.00	0.00
111	Span 1	End		0.00	0.00
113	Span 1	Start		0.00	0.00
113	Span 1	Middle		0.00	0.00
113	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-8257.67	0.00
117	Span 1	Start		0.00	0.00
117	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	-256537.81	0.13
117	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-406793.90	0.20
119	Span 1	Start		0.00	0.00
119	Span 1	Middle	DCONU1	-15262.47	1.668E-002
119	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	-142671.99	0.16
120	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	-150365.16	0.17
120	Span 1	Middle		0.00	0.00
120	Span 1	End		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Line	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	126676.60	0.14
1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	281400.67	0.32
1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	174408.51	0.20
3	Span 1	Start		0.00	0.00
3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	33412.14	2.555E-002
3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	261.80	0.00
4	Span 1	Start		0.00	0.00
4	Span 1	Middle		0.00	0.00
4	Span 1	End		0.00	0.00
23	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	41026.26	5.291E-002
23	Span 1	Middle		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Line	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
23	Span 1	End		0.00	0.00
25	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	1.849E-002
25	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	128031.84	9.779E-002
25	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	31843.10	2.184E-002
26	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	1.688E-002
26	Span 1	Middle		0.00	0.00
26	Span 1	End		0.00	0.00
27	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	82843.41	4.355E-002
27	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	158173.64	7.890E-002
27	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	121096.31	5.683E-002
42	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	103598.84	7.110E-002
42	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	158951.96	0.10
42	Span 1	End	DCONU1	25953.19	0.00
43	Span 1	Start		0.00	0.00
43	Span 1	Middle		0.00	0.00
43	Span 1	End		0.00	0.00
45	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6794.80	0.00
45	Span 1	Middle	DCONU1	34914.26	1.558E-002
45	Span 1	End	DCONU1	743.03	0.00
46	Span 1	Start	DCONU1	51.37	0.00
46	Span 1	Middle		0.00	0.00
46	Span 1	End		0.00	0.00
47	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	13740.56	2.576E-002
47	Span 1	Middle		0.00	0.00
47	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1099.47	0.00
58	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	17685.16	0.00
58	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	542158.70	0.38
58	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	372727.63	0.26
61	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	106846.78	7.129E-002
61	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	150001.52	9.787E-002
61	Span 1	End		0.00	0.00
62	Span 1	Start		0.00	0.00
62	Span 1	Middle		0.00	0.00
62	Span 1	End		0.00	0.00
63	Span 1	Start		0.00	0.00
63	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	100770.05	8.466E-002
63	Span 1	End		0.00	0.00
64	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	566931.24	0.30

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Line	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
64	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1811841.88	0.91
64	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	311432.61	0.15
65	Span 1	Start		0.00	0.00
65	Span 1	Middle		0.00	0.00
65	Span 1	End		0.00	0.00
69	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3109.22	1.218E-002
69	Span 1	Middle		0.00	0.00
69	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5324.02	1.466E-002
71	Span 1	Start		0.00	0.00
71	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	34025.72	3.406E-002
71	Span 1	End		0.00	0.00
72	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	597306.55	0.86
72	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1013876.88	0.73
72	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	703774.00	0.50
74	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	122862.93	8.119E-002
74	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	204099.16	0.16
74	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	0.00	2.164E-002
75	Span 1	Start		0.00	0.00
75	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	158548.43	0.13
75	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	93736.99	7.375E-002
76	Span 1	Start		0.00	0.00
76	Span 1	Middle		0.00	0.00
76	Span 1	End		0.00	0.00
84	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	49598.91	2.428E-002
84	Span 1	Middle		0.00	0.00
84	Span 1	End	DCONU1	64767.24	4.194E-002
85	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	694281.30	0.31
85	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1515276.18	0.73
85	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	708805.88	0.32
86	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	550726.38	0.26
86	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	900143.40	0.43
86	Span 1	End		0.00	0.00
87	Span 1	Start		0.00	0.00
87	Span 1	Middle		0.00	0.00
87	Span 1	End		0.00	0.00
88	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1044787.08	0.50

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Line	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
88	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1860974.10	0.91
88	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	737013.83	0.35
91	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1834875.75	0.81
91	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2697130.00	1.20
91	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	774482.99	0.33
92	Span 1	Start		0.00	0.00
92	Span 1	Middle		0.00	0.00
92	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	519.28	0.00
93	Span 1	Start		0.00	0.00
93	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	338132.07	0.16
93	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	223732.59	0.10
94	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	76339.66	0.10
94	Span 1	Middle		0.00	5.022E-002
94	Span 1	End		0.00	3.840E-002
95	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	199670.07	0.19
95	Span 1	Middle		0.00	0.00
95	Span 1	End		0.00	0.00
98	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	792919.46	0.39
98	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1119208.26	0.56
98	Span 1	End	DCONU1	236577.30	0.11
99	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1254856.76	0.47
99	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2532170.16	1.02
99	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2140672.18	0.86
100	Span 1	Start	DCONU1	30794.59	2.121E-002
100	Span 1	Middle	DCONU1	118888.86	8.332E-002
100	Span 1	End	DCONU1	158019.34	0.11
101	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	184826.36	0.12
101	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	412489.30	0.29
101	Span 1	End		0.00	0.00
102	Span 1	Start		0.00	0.00
102	Span 1	Middle		0.00	0.00
102	Span 1	End		0.00	0.00
105	Span 1	Start		0.00	0.00
105	Span 1	Middle		0.00	0.00
105	Span 1	End		0.00	0.00
111	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	10534.70	0.00
111	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	19064.72	1.362E-002

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 2 of 3

Line	SpanID	Location	FBotCombo	FBotMoment	FBotArea
				lb-in	in2
111	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2130.17	0.00
113	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2468.79	0.00
113	Span 1	Middle	DCONU1	9578.38	0.00
113	Span 1	End		0.00	0.00
117	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	115503.80	5.073E-002
117	Span 1	Middle		0.00	0.00
117	Span 1	End		0.00	0.00
119	Span 1	Start	DCONU1	19708.19	2.027E-002
119	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	29951.63	3.280E-002
119	Span 1	End		0.00	0.00
120	Span 1	Start		0.00	0.00
120	Span 1	Middle		0.00	0.00
120	Span 1	End		0.00	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Line	SpanID	Location	VCombo	VForce	VArea
				lb	in2/ft
1	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4526.63	0.00
1	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2807.75	0.00
1	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	7214.13	7.874E-002
3	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2078.87	0.00
3	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1434.26	0.00
3	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	3698.27	0.00
4	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1446.36	0.00
4	Span 1	Middle		0.00	0.00
4	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	775.64	0.00
23	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2208.60	0.00
23	Span 1	Middle		0.00	0.00
23	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2647.85	0.00
25	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5309.98	0.00
25	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4361.66	0.00
25	Span 1	End	DCONU1	2693.65	0.00
26	Span 1	Start	DCONU1	4221.95	0.00
26	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4505.17	0.00
26	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	3563.71	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Line	SpanID	Location	VCombo	VForce	VArea
				lb	in2/ft
27	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3506.52	0.00
27	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2949.38	0.00
27	Span 1	End	DCONU1	1708.96	0.00
42	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6284.24	0.00
42	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1461.42	0.00
42	Span 1	End	DCONU1	3325.49	0.00
43	Span 1	Start	DCONU1	1818.20	0.00
43	Span 1	Middle	DCONU1	1761.63	0.00
43	Span 1	End	DCONU1	932.21	0.00
45	Span 1	Start	DCONU1	3606.44	0.00
45	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1951.43	0.00
45	Span 1	End	DCONU1	1641.61	0.00
46	Span 1	Start	DCONU1	346.12	0.00
46	Span 1	Middle		0.00	0.00
46	Span 1	End	DCONU1	209.92	0.00
47	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	766.96	0.00
47	Span 1	Middle		0.00	0.00
47	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1243.56	0.00
58	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	10070.13	8.004E-002
58	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	6530.15	0.00
58	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	7964.99	0.00
61	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	3146.76	0.00
61	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	7109.67	0.00
61	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	9763.22	8.004E-002
62	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	18943.04	8.004E-002
62	Span 1	Middle		0.00	0.00
62	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	18546.78	8.004E-002
63	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2682.79	0.00
63	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3378.62	0.00
63	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4174.84	0.00
64	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	39250.60	7.874E-002
64	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	25057.19	7.874E-002
64	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	40996.09	8.495E-002
65	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1200.14	0.00
65	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	2578.36	0.00

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Line	SpanID	Location	VCombo	VForce	VArea
				lb	in2/ft
65	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2004.23	0.00
69	Span 1	Start	DCONU1	467.38	0.00
69	Span 1	Middle		0.00	0.00
69	Span 1	End	DCONU1	313.61	0.00
71	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	1374.58	0.00
71	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	1970.95	0.00
71	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2709.81	0.00
72	Span 1	Start	DCONU1	296.60	0.12
72	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4475.84	0.00
72	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	10975.77	8.004E-002
74	Span 1	Start	DCONU1	3632.58	0.00
74	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	5177.60	0.00
74	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	6148.44	0.00
75	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	5266.23	0.00
75	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4594.23	0.00
75	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5900.16	0.00
76	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6108.58	0.00
76	Span 1	Middle		0.00	0.00
76	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5865.38	0.00
84	Span 1	Start	DCONU1	1236.83	0.00
84	Span 1	Middle		0.00	0.00
84	Span 1	End	DCONU1	837.22	0.00
85	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	19337.08	7.874E-002
85	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	12203.20	0.00
85	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	18944.42	7.874E-002
86	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	13653.33	0.00
86	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	14269.51	0.00
86	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	18945.39	7.874E-002
87	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	15725.37	7.874E-002
87	Span 1	Middle		0.00	0.00
87	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	15352.11	7.874E-002
88	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	25560.54	7.874E-002
88	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	15315.40	7.874E-002
88	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	20313.92	7.874E-002

Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Line	SpanID	Location	VCombo	VForce	VArea
				lb	in2/ft
91	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	15232.24	0.00
91	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	11755.10	0.00
91	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	30667.70	7.874E-002
92	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	23974.39	7.874E-002
92	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	23294.47	7.874E-002
92	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	20840.91	7.874E-002
93	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2022.12	0.00
93	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4230.90	0.00
93	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	5040.54	0.00
94	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	2364.70	0.00
94	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	4556.56	0.00
94	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4570.78	0.00
95	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6659.77	0.00
95	Span 1	Middle		0.00	0.00
95	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	7187.91	7.874E-002
98	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	14275.79	7.874E-002
98	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	11025.88	0.00
98	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	18734.30	7.874E-002
99	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	14708.96	0.00
99	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	7819.10	0.00
99	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	30712.68	8.004E-002
100	Span 1	Start	DCONU1	3798.63	0.00
100	Span 1	Middle	DCONU1	3104.49	0.00
100	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	1814.49	0.00
101	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	8273.73	0.00
101	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	6659.71	0.00
101	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	10781.36	8.004E-002
102	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4736.71	0.00
102	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3853.70	0.00
102	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	2309.34	0.00
105	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4581.70	0.00

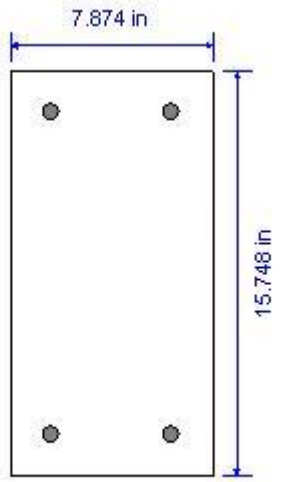
Table 46: Concrete Beam Design Summary 01 - Flexural And Shear Data, Part 3 of 3

Line	SpanID	Location	VCombo	VForce	VArea
				lb	in2/ft
105	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	5783.03	0.00
105	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4298.75	0.00
111	Span 1	Start	DCONU1	639.97	0.00
111	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	453.21	0.00
111	Span 1	End	DCONU1	957.43	0.00
113	Span 1	Start	DCONU1	600.24	0.00
113	Span 1	Middle	DCONU1	162.66	0.00
113	Span 1	End	DCONU1	883.61	0.00
117	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	6972.10	0.00
117	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	7736.97	0.00
117	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	6765.48	0.00
119	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	479.99	0.00
119	Span 1	Middle	Comb=1.2 D + 1.6 L +1.6 Lr	3686.35	0.00
119	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4169.61	0.00
120	Span 1	Start	Comb=1.2 D + 1.6 L +1.6 Lr	4673.49	0.00
120	Span 1	Middle		0.00	0.00
120	Span 1	End	Comb=1.2 D + 1.6 L +1.6 Lr	4171.52	0.00

ACI 318-14 Concrete Beam Design

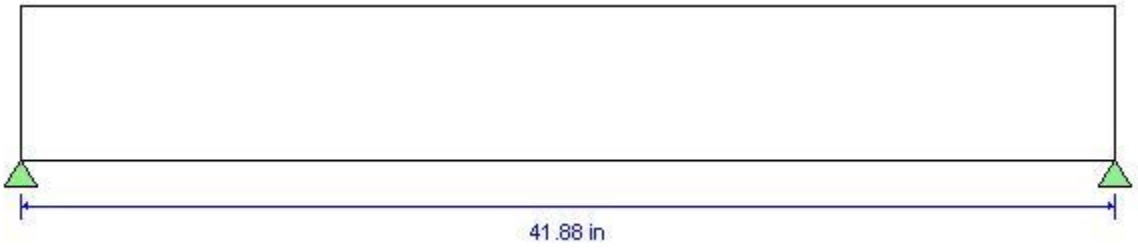
Geometric Properties

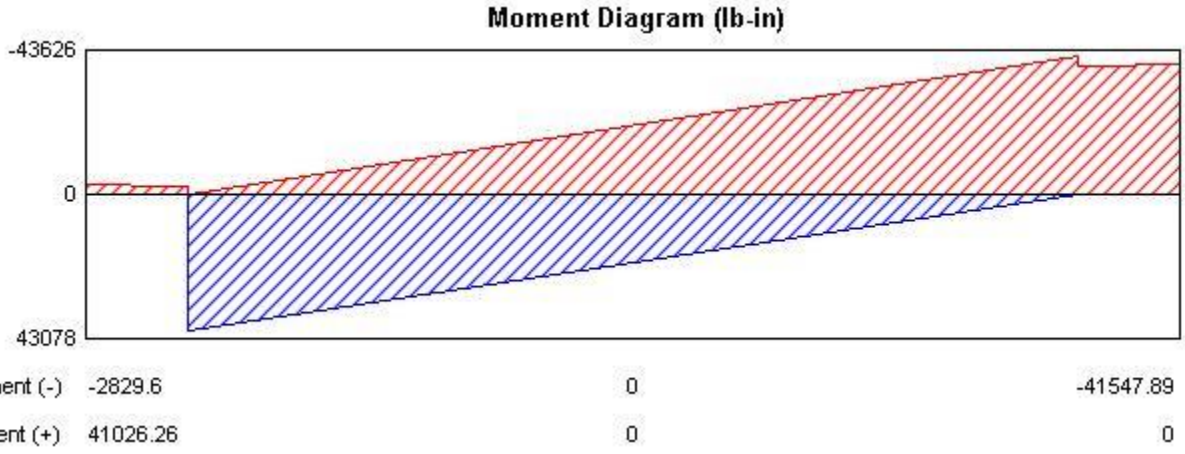
Combination = Overall Envelope
Beam Label = 23
Section Property = B4-8x16- 4 ksi
Length = 41.88 in
Section Width = 7.874 in
Section Depth = 15.748 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



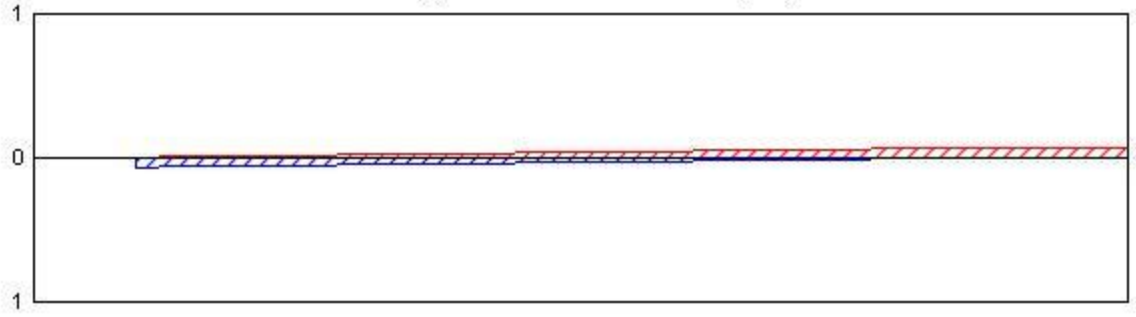
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



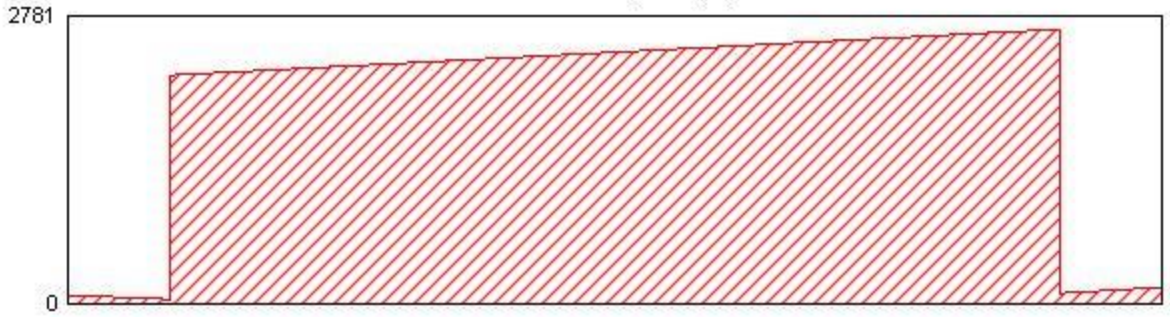


Longitudinal Reinforcement (in2)



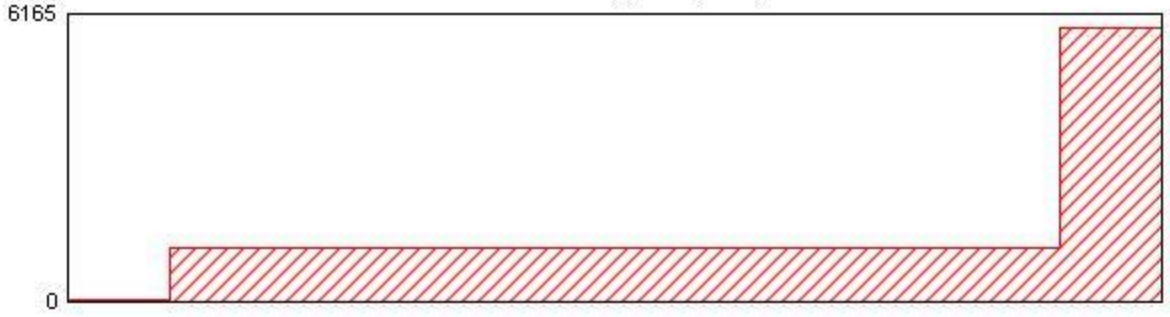
As (top)	0	0	5.36E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	5.291E-002	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	2208.6	0	2647.85

Torsion Diagram (lb-in)

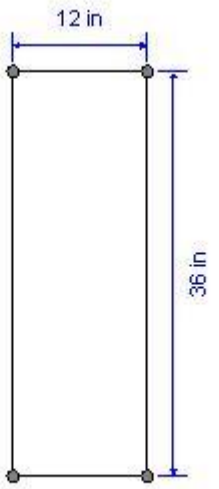


Torsion	1167.49	0	5871.24

ACI 318-14 Concrete Beam Design

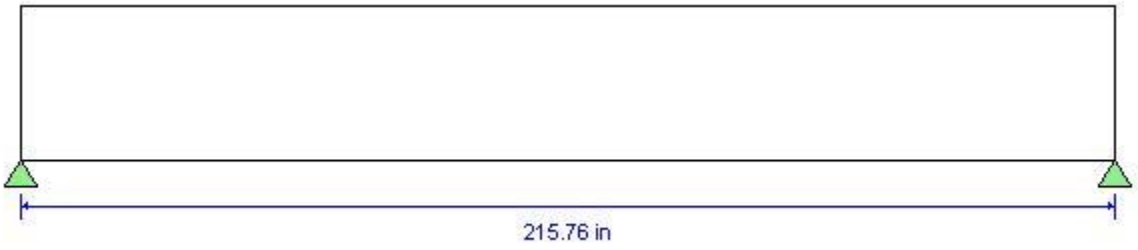
Geometric Properties

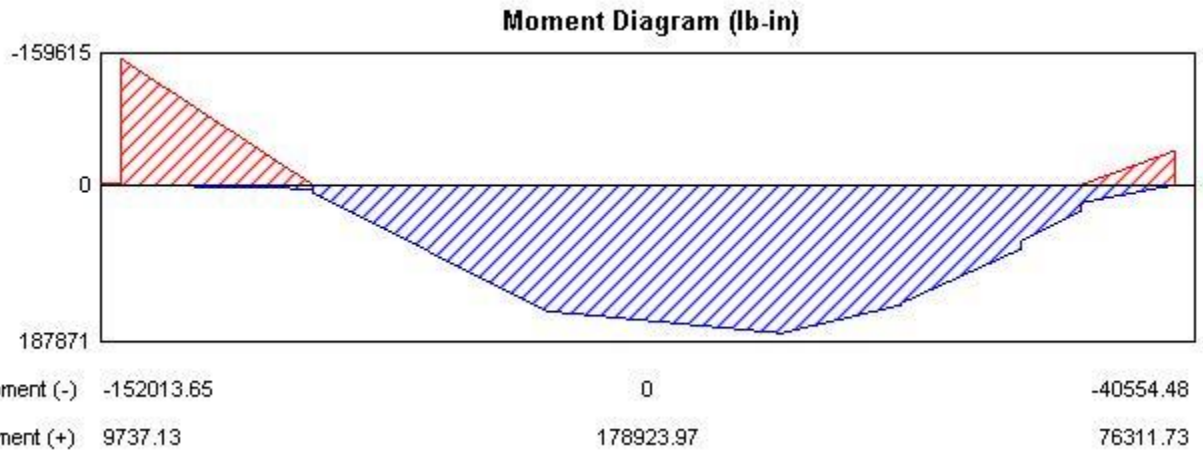
Combination = Overall Envelope
Beam Label = 24
Section Property =
Length = 215.76 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



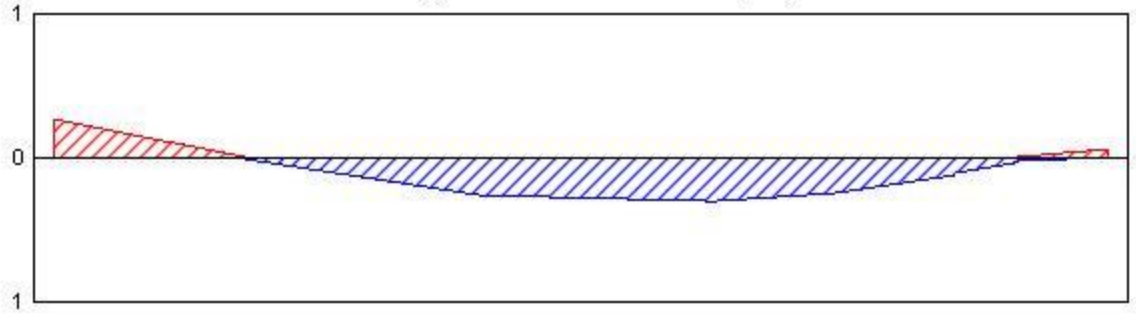
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2



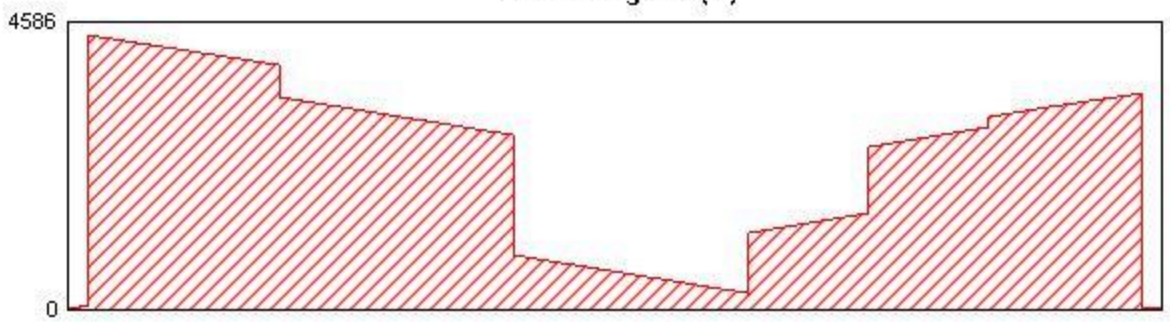


Longitudinal Reinforcement (in2)



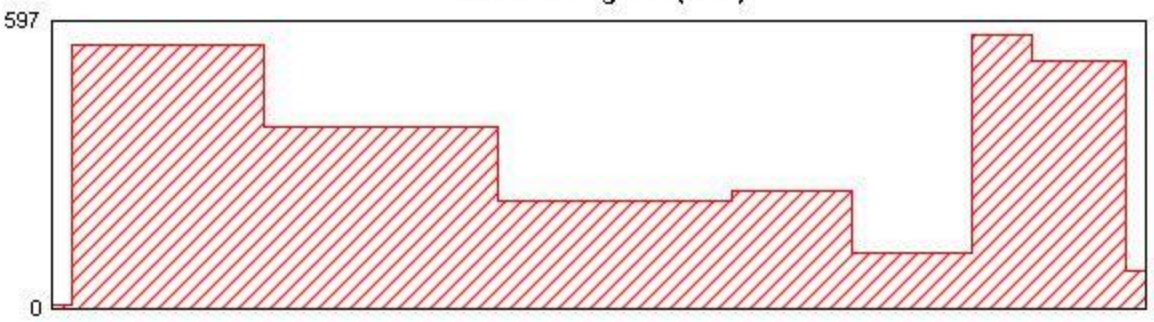
As (top)	0.2	0	4.519E-002
Combo	DCONU2		DCONU2
As (bot)	9.848E-003	0.24	9.682E-002
Combo	DCONU2	DCONU2	DCONU2

Shear Diagram (lb)



Shear	4366.91	2780.56	3439.09

Torsion Diagram (lb-in)

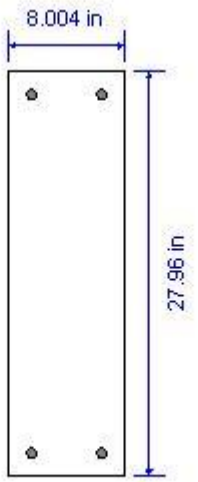


Torsion	548.27	375.79	568.44

ACI 318-14 Concrete Beam Design

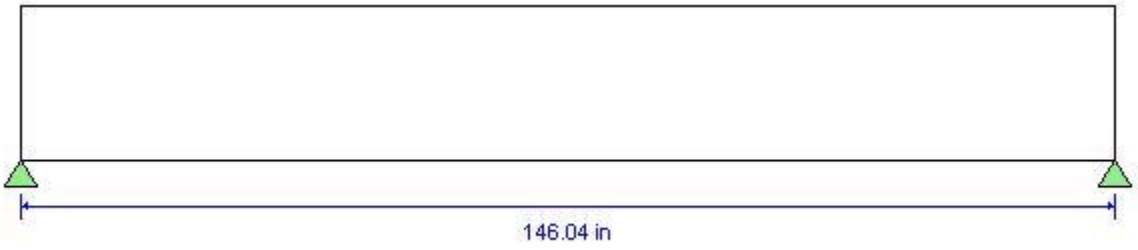
Geometric Properties

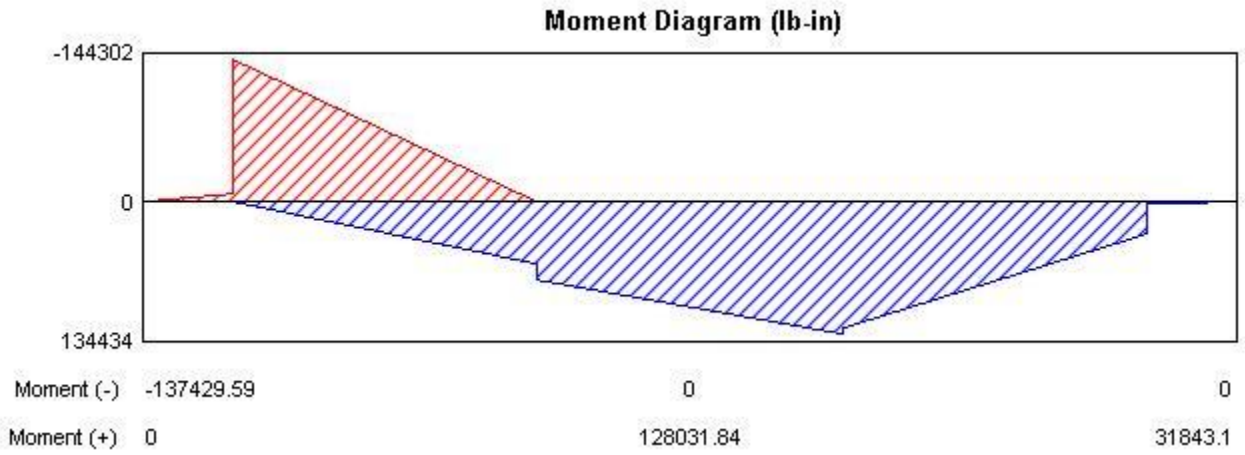
Combination = Overall Envelope
Beam Label = 25
Section Property = B8-8x28-4 ksi
Length = 146.04 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



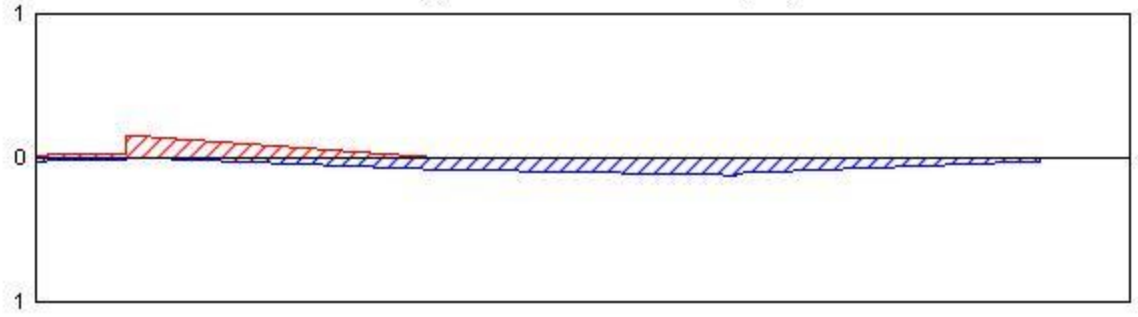
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



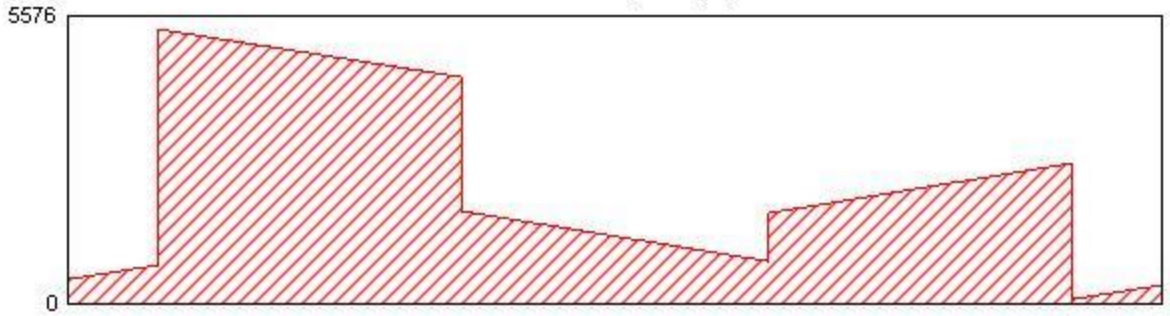


Longitudinal Reinforcement (in2)



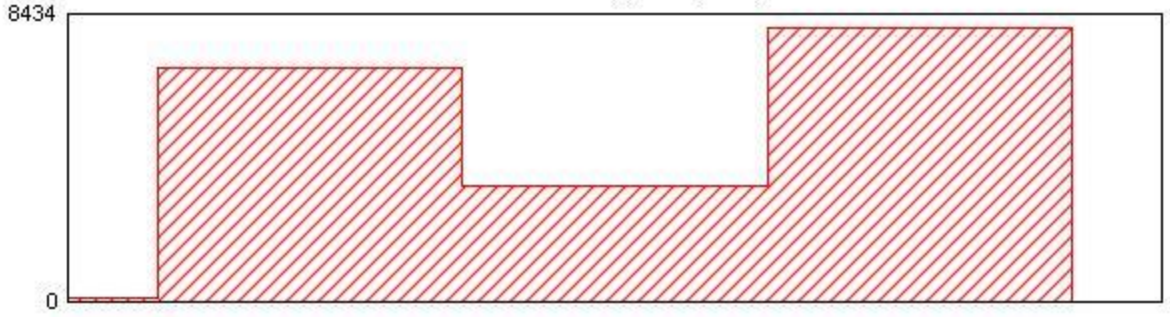
As (top)	0.12	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		
As (bot)	1.849E-002	9.779E-002	2.184E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	5309.98	4361.66	2693.65

Torsion Diagram (lb-in)

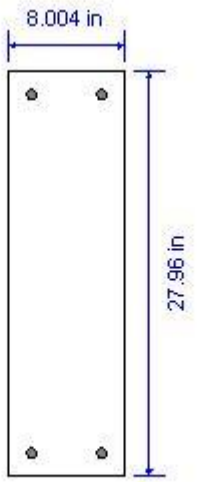


Torsion	6871.68	8031.62	8031.62

ACI 318-14 Concrete Beam Design

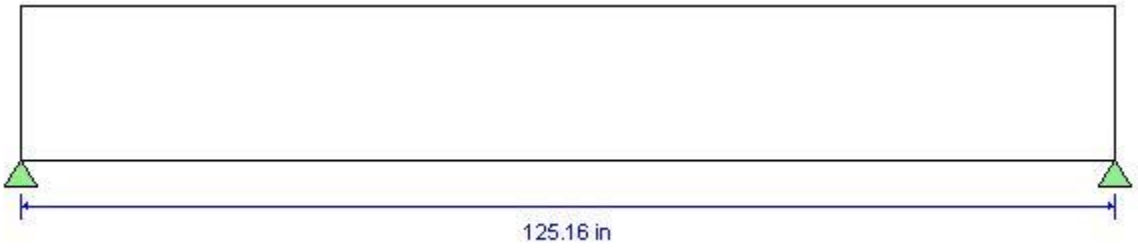
Geometric Properties

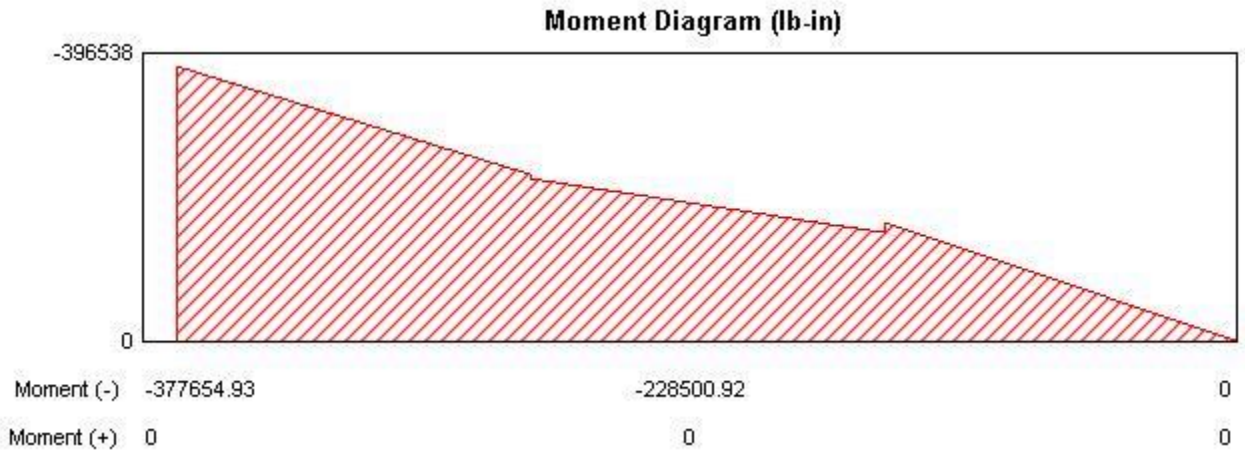
Combination = Overall Envelope
Beam Label = 26
Section Property = B8-8x28-4 ksi
Length = 125.16 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



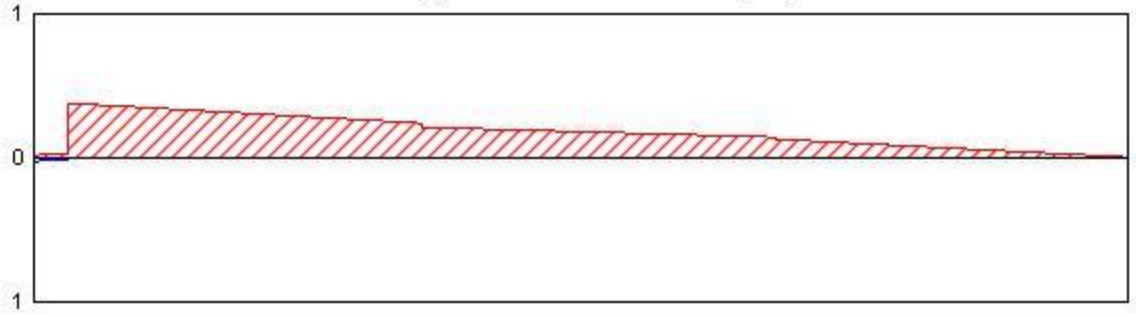
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



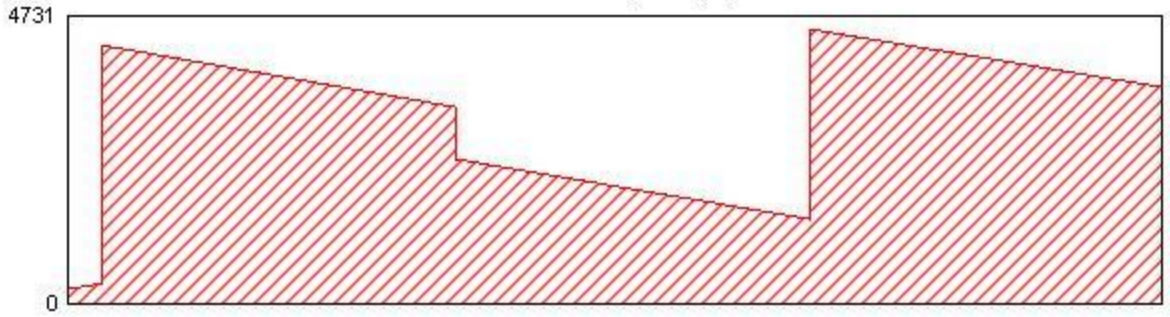


Longitudinal Reinforcement (in2)



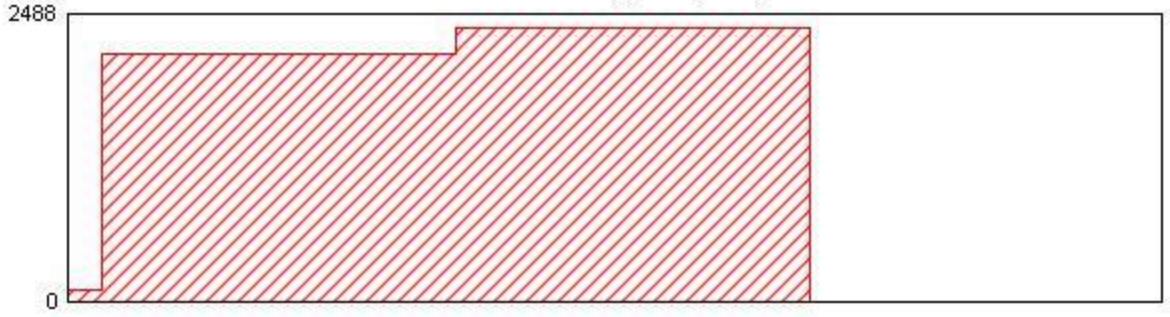
As (top)	0.28	0.18	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	
As (bot)	1.688E-002	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	4221.95	4505.17	3563.71

Torsion Diagram (lb-in)

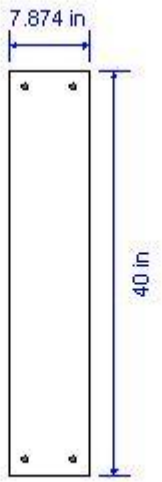


Torsion	2144.24	2369.16	0

ACI 318-14 Concrete Beam Design

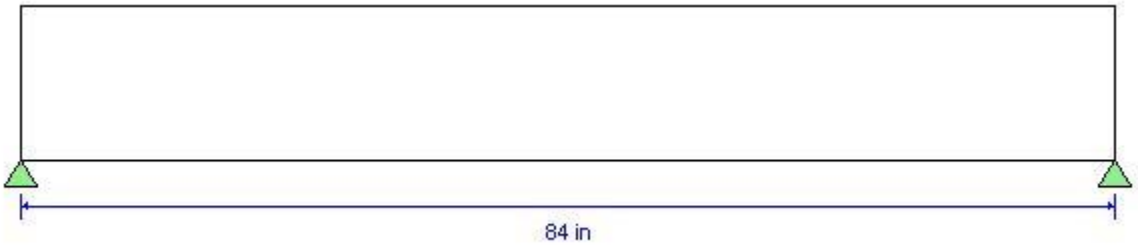
Geometric Properties

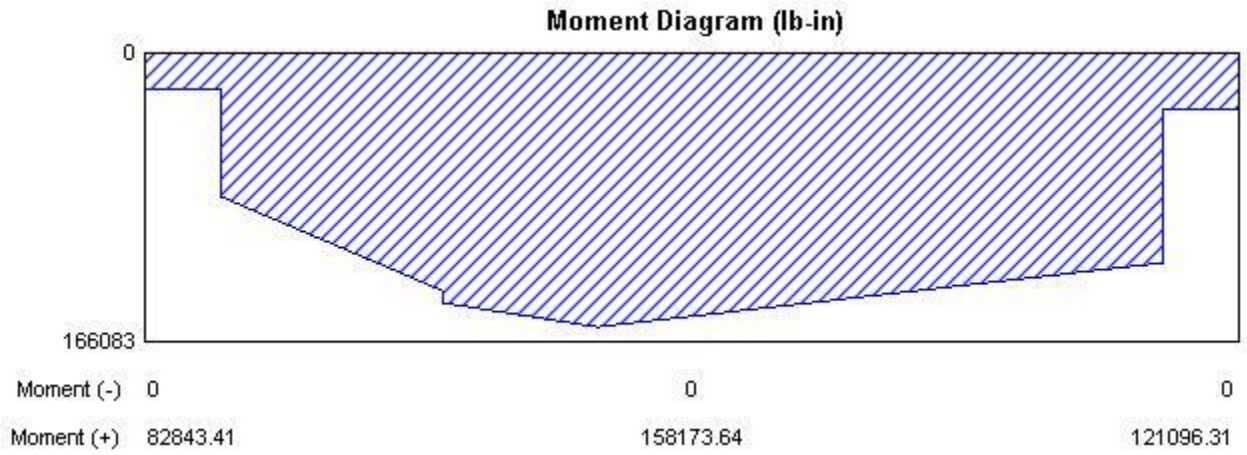
Combination = Overall Envelope
Beam Label = 27
Section Property = B10-8x40-4 ksi
Length = 84 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



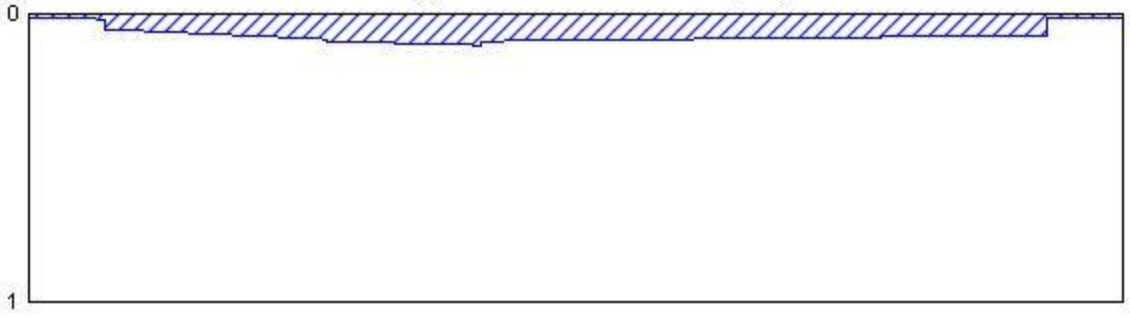
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



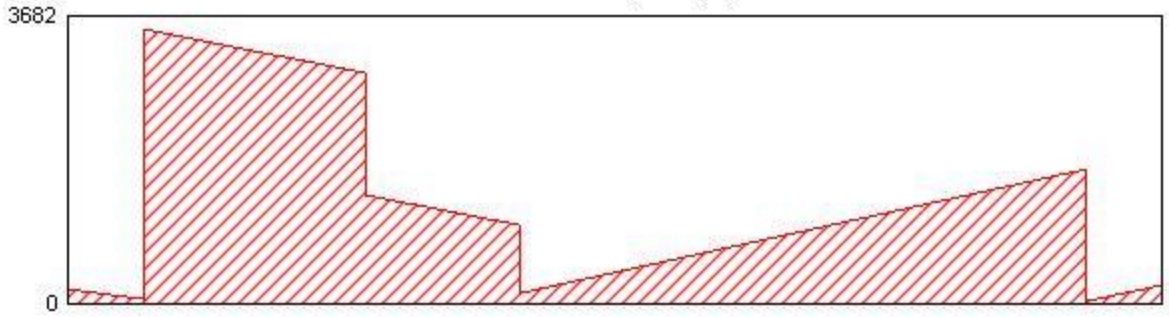


Longitudinal Reinforcement (in2)



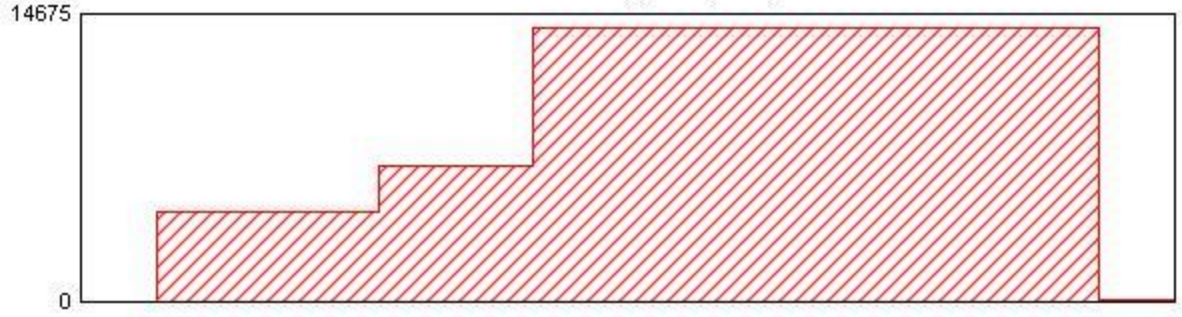
As (top)	0	0	0
Combo			
As (bot)	4.355E-002	7.89E-002	5.683E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	3506.52	2949.38	1708.96

Torsion Diagram (lb-in)

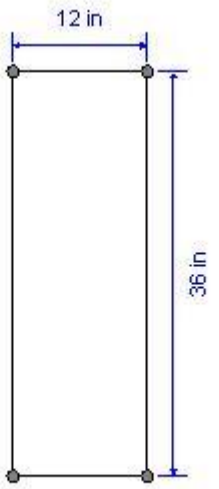


Torsion	4556.69	13975.37	13975.37

ACI 318-14 Concrete Beam Design

Geometric Properties

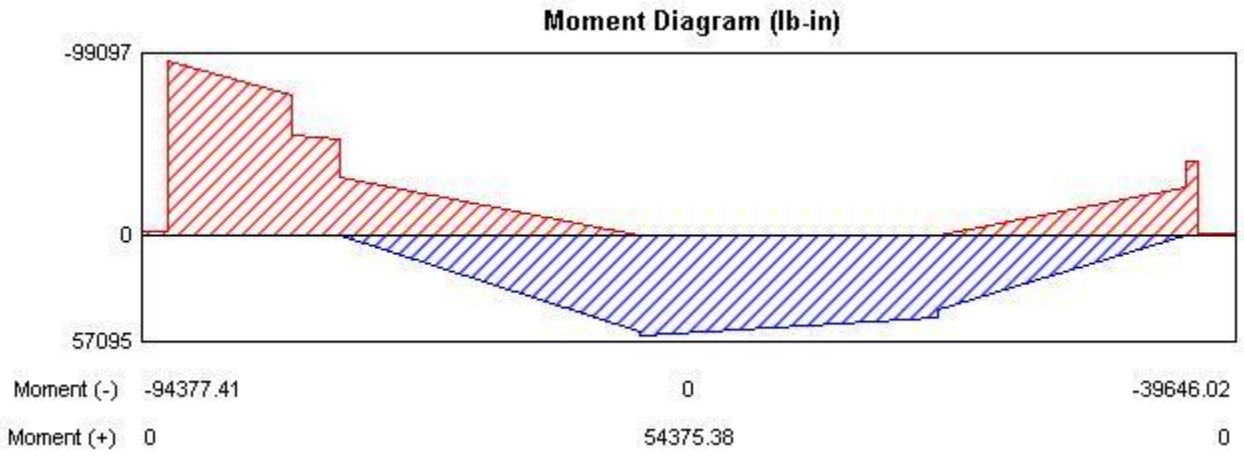
Combination = Overall Envelope
Beam Label = 28
Section Property =
Length = 171.12 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



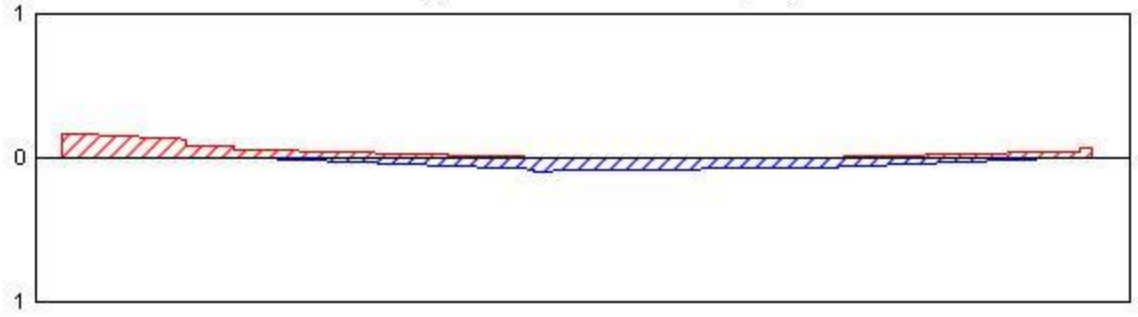
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2



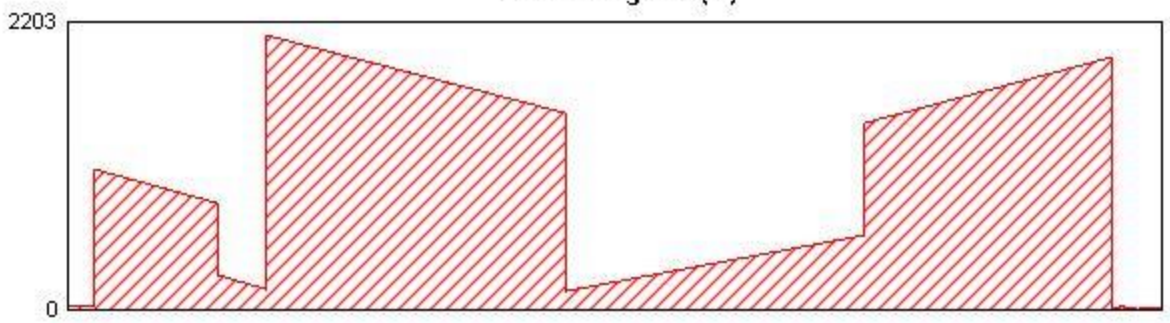


Longitudinal Reinforcement (in2)



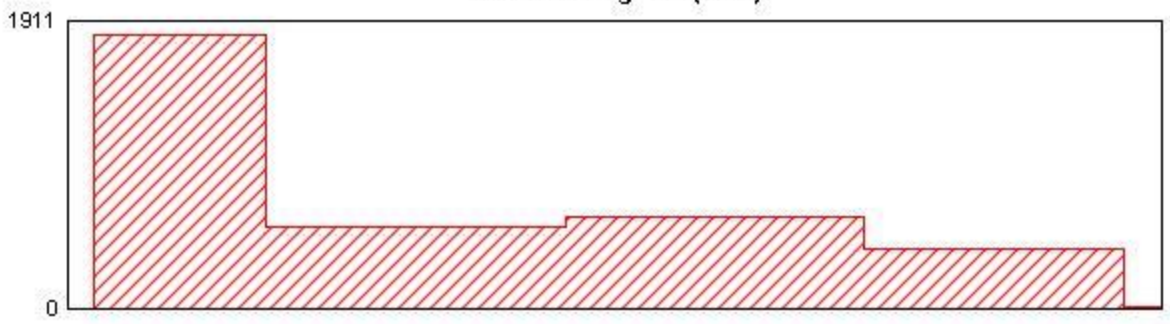
As (top)	0.12	0	4.741E-002
Combo	DCONU2		DCONU1
As (bot)	0	6.798E-002	0
Combo		DCONU2	

Shear Diagram (lb)



Shear	2097.37	1494.42	1928.3

Torsion Diagram (lb-in)

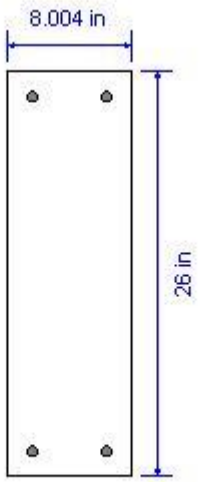


Torsion	1819.34	614.26	399.53

ACI 318-14 Concrete Beam Design

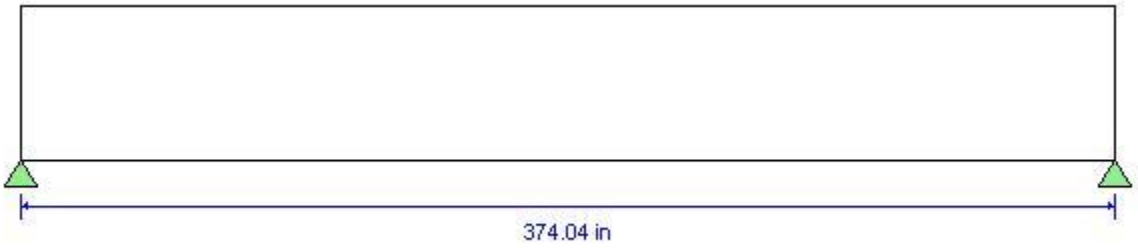
Geometric Properties

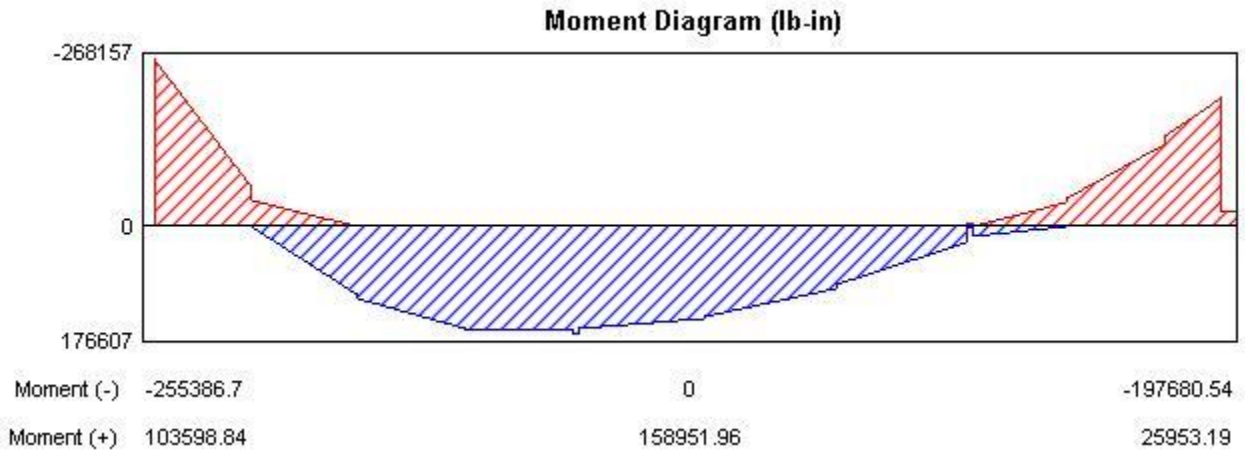
Combination = Overall Envelope
Beam Label = 42
Section Property = B9-8x26-4 ksi
Length = 374.04 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



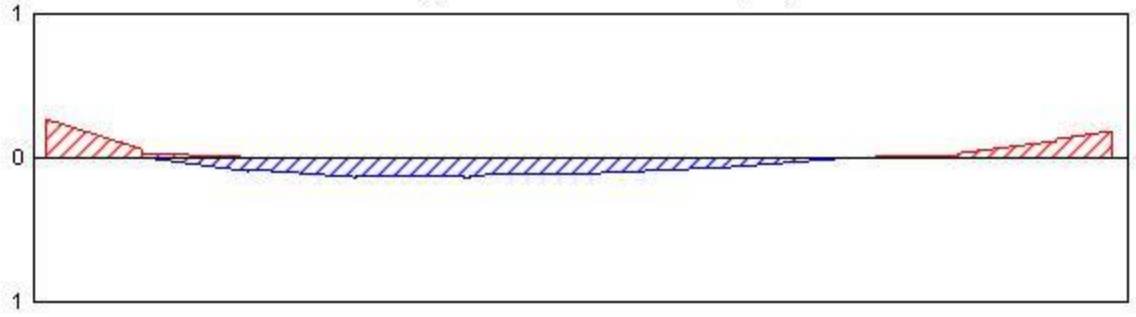
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



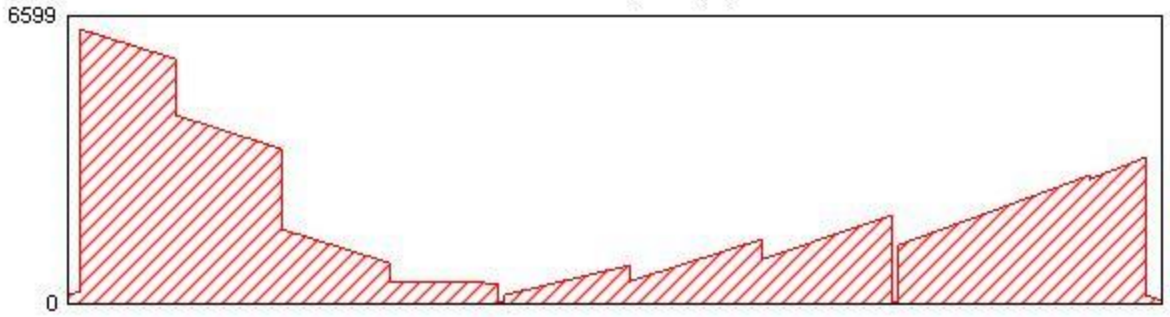


Longitudinal Reinforcement (in2)



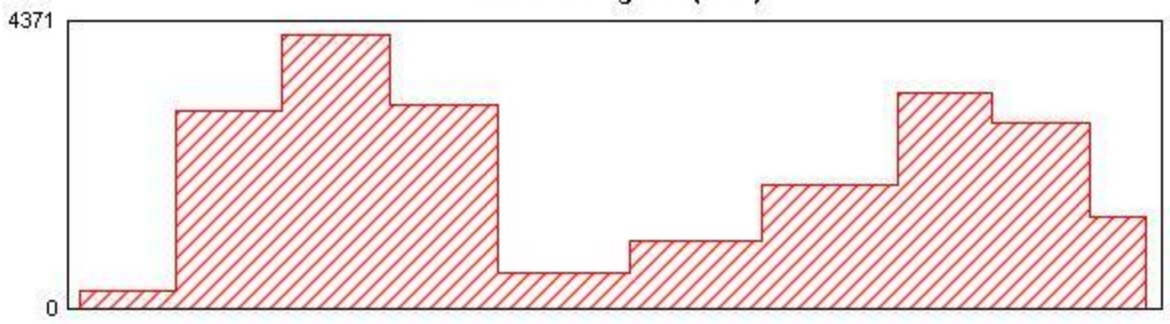
As (top)	0.2	0	0.13
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		DCONU1
As (bot)	7.11E-002	0.1	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1

Shear Diagram (lb)



Shear	6284.24	1461.42	3325.49

Torsion Diagram (lb-in)

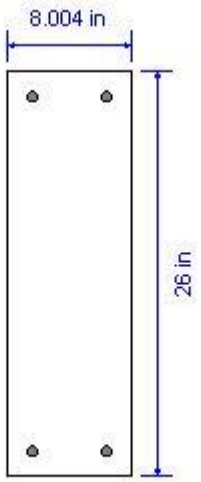


Torsion	4162.62	4162.62	3287.38

ACI 318-14 Concrete Beam Design

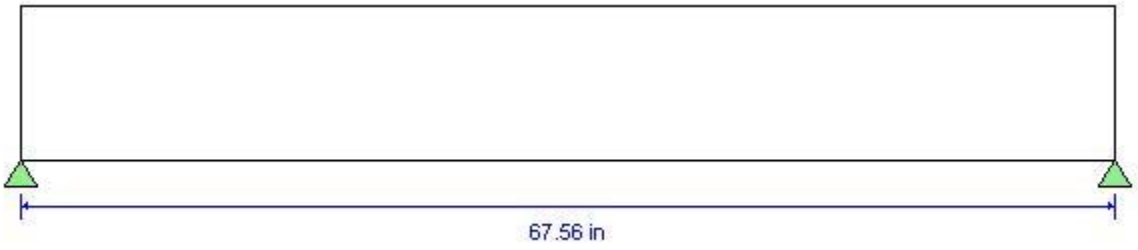
Geometric Properties

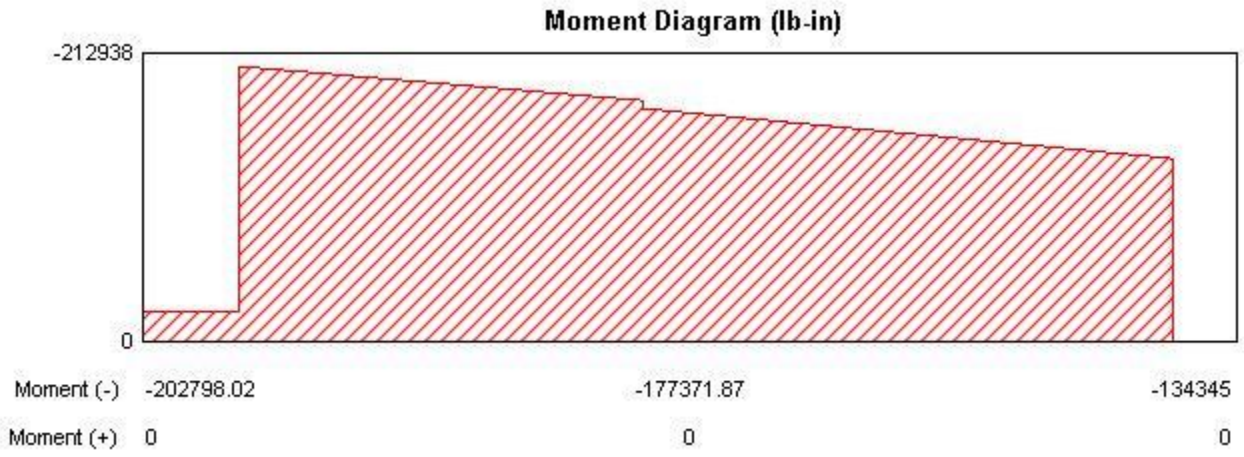
Combination = Overall Envelope
Beam Label = 43
Section Property = B9-8x26-4 ksi
Length = 67.56 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



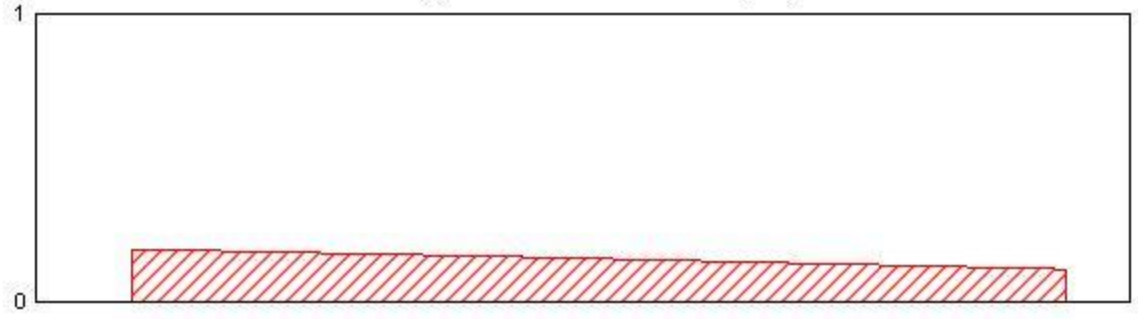
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



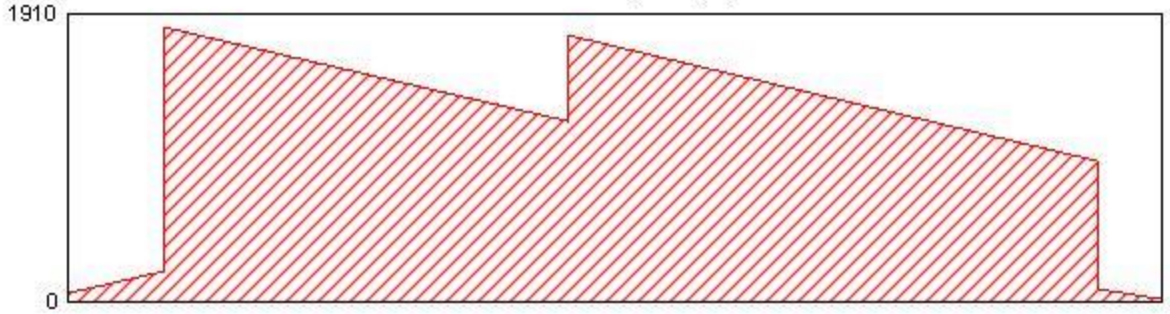


Longitudinal Reinforcement (in2)



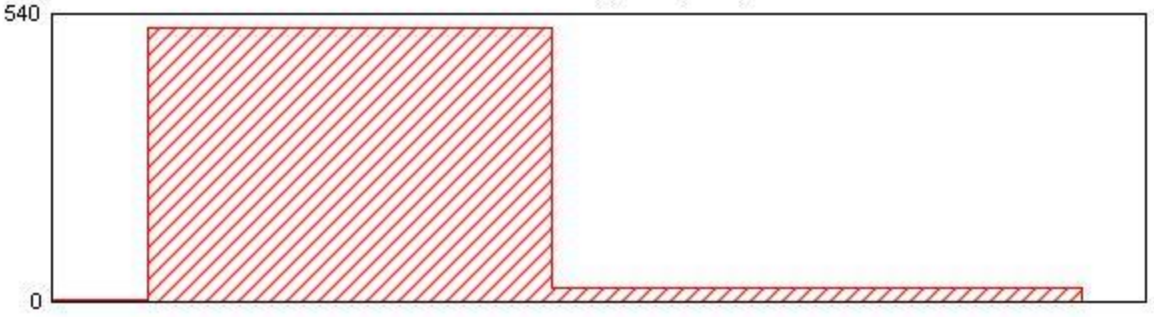
As (top)	0.14	0.11	8.061E-002
Combo	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	1818.2	1761.63	932.21

Torsion Diagram (lb-in)

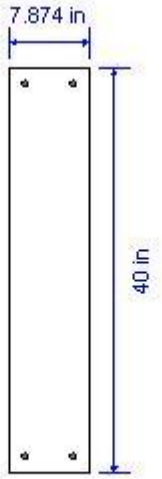


Torsion	513.61	513.61	26.34

ACI 318-14 Concrete Beam Design

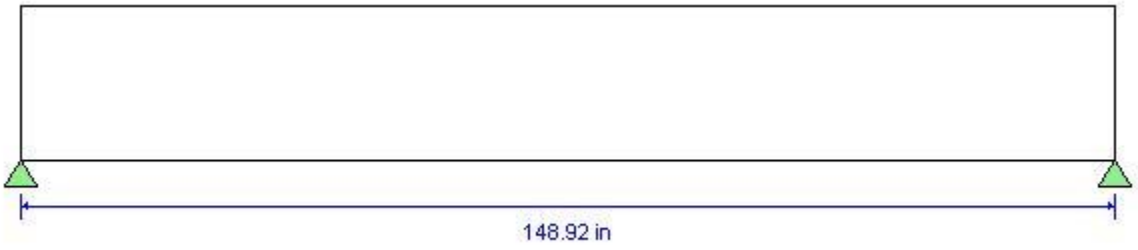
Geometric Properties

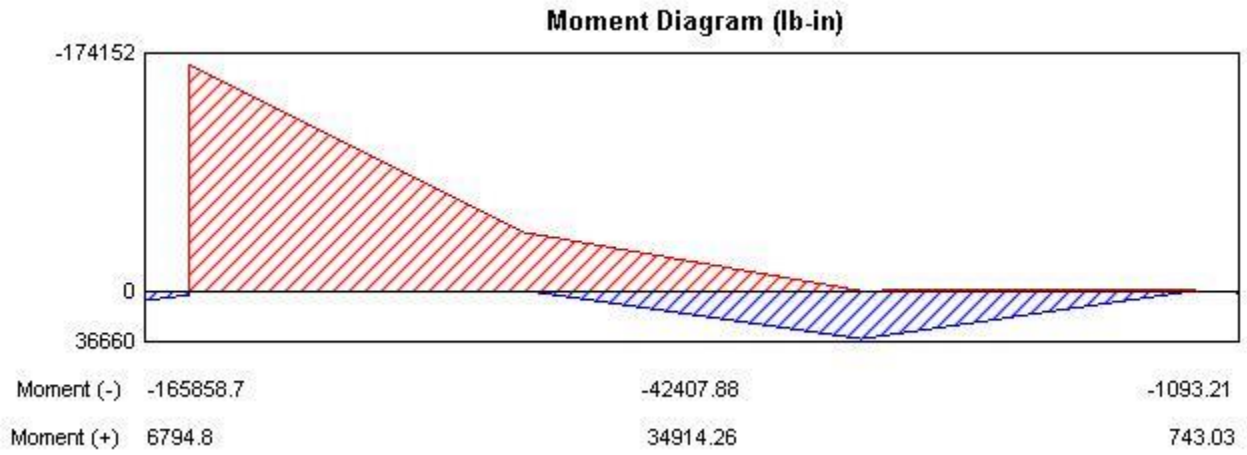
Combination = Overall Envelope
Beam Label = 45
Section Property = B10-8x40-4 ksi
Length = 148.92 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



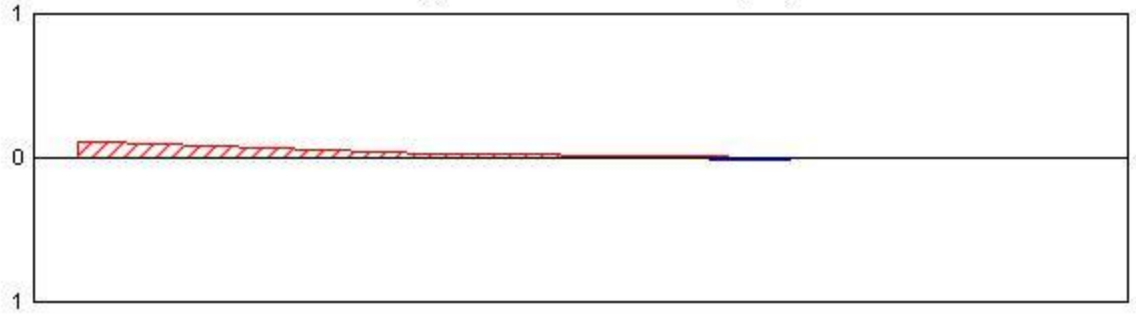
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



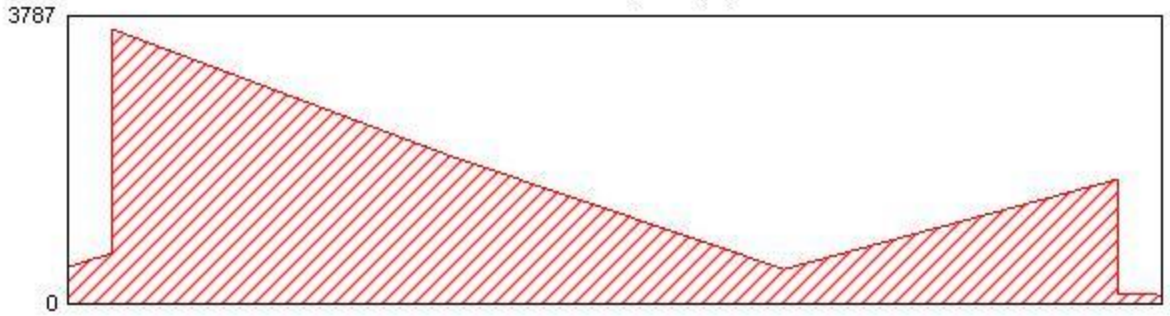


Longitudinal Reinforcement (in2)



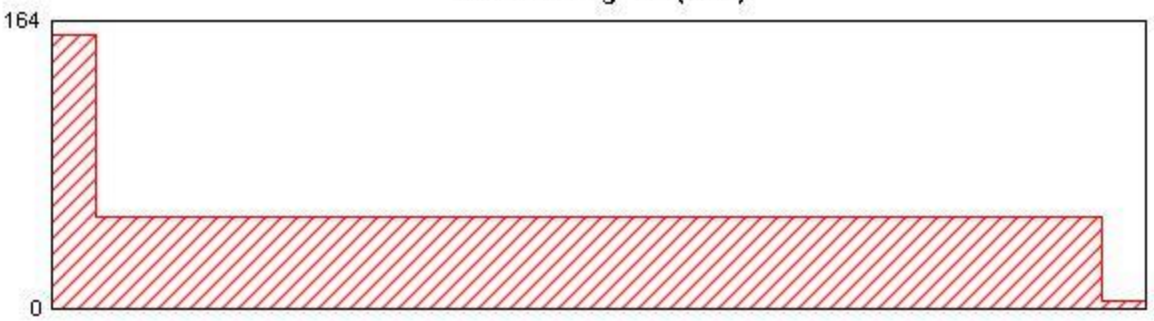
As (top)	7.884E-002	1.916E-002	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	1.558E-002	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	DCONU1

Shear Diagram (lb)



Shear	3606.44	1951.43	1641.61

Torsion Diagram (lb-in)

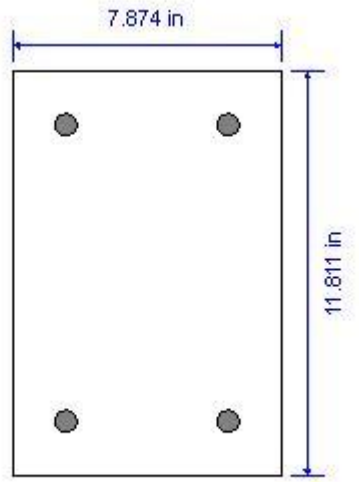


Torsion	155.64	51.84	51.84

ACI 318-14 Concrete Beam Design

Geometric Properties

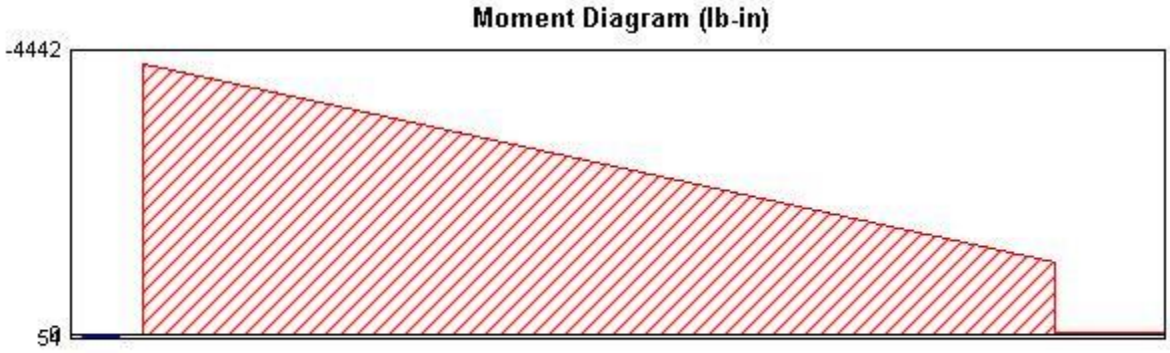
Combination = Overall Envelope
Beam Label = 46
Section Property = B2-8x12- 4ksi
Length = 59.04 in
Section Width = 7.874 in
Section Depth = 11.811 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

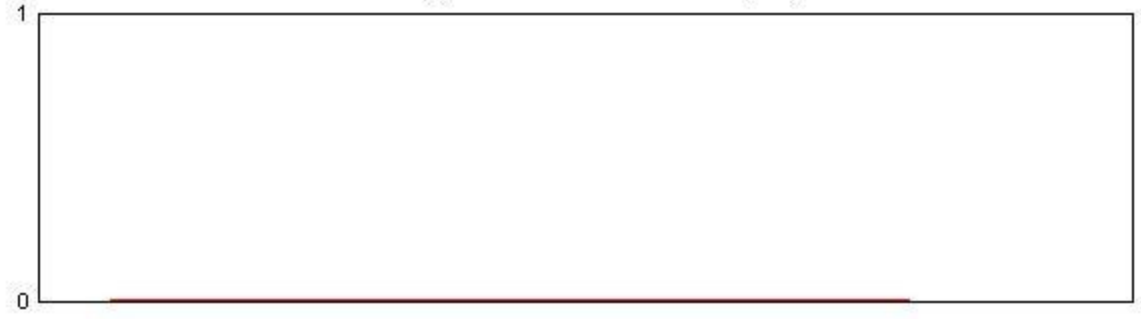
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2





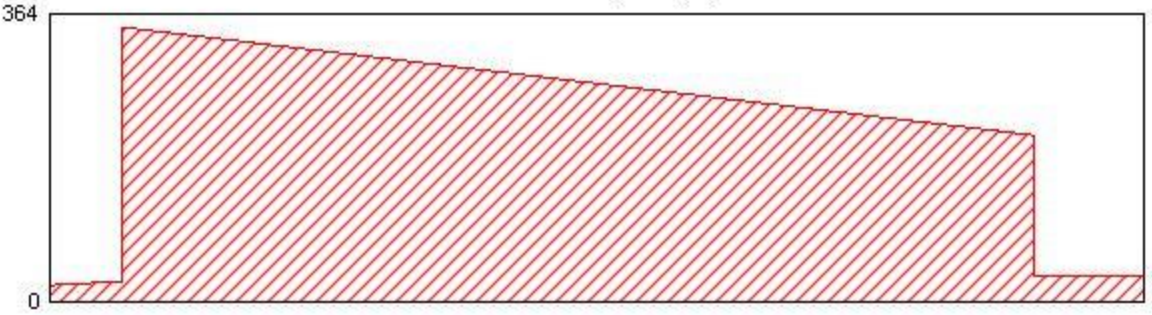
Moment (-)	-4230.44	0	-1109.55
Moment (+)	51.37	0	0

Longitudinal Reinforcement (in2)



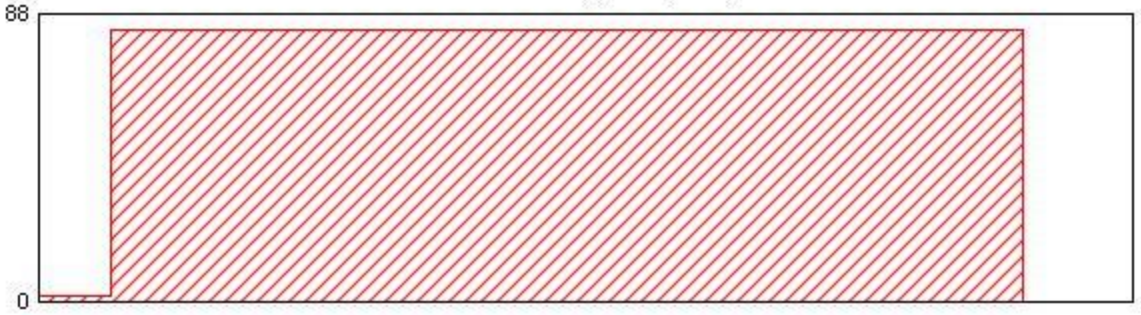
As (top)	7.058E-003	0	0
Combo	DCONU1		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo	DCONU1		

Shear Diagram (lb)



Shear	346.12	0	209.92

Torsion Diagram (lb-in)

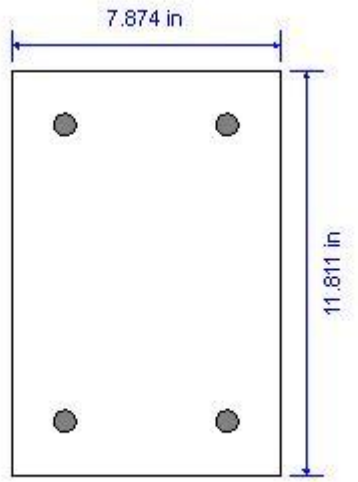


Torsion	83.08	0	83.08

ACI 318-14 Concrete Beam Design

Geometric Properties

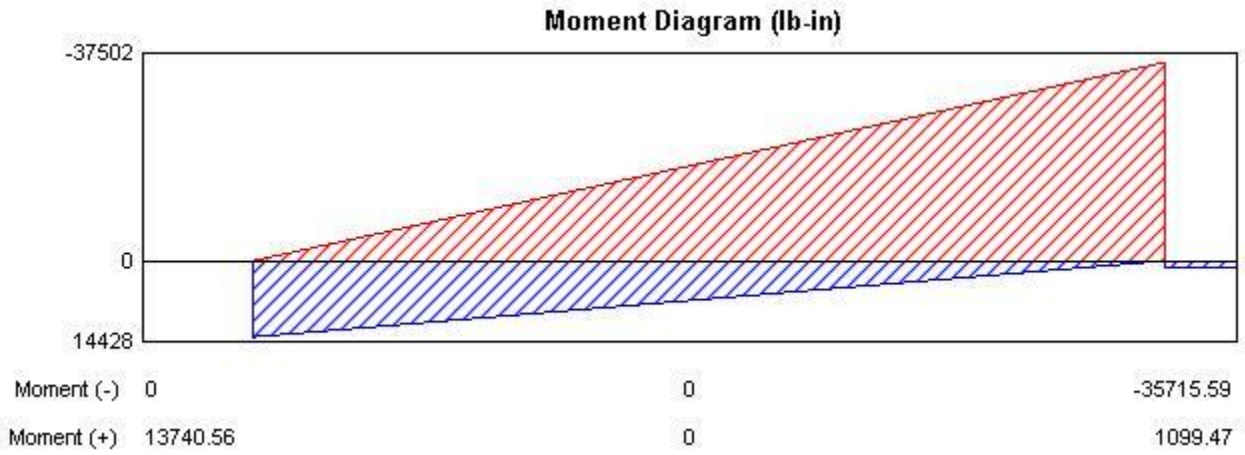
Combination = Overall Envelope
Beam Label = 47
Section Property = B2-8x12- 4ksi
Length = 59.04 in
Section Width = 7.874 in
Section Depth = 11.811 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



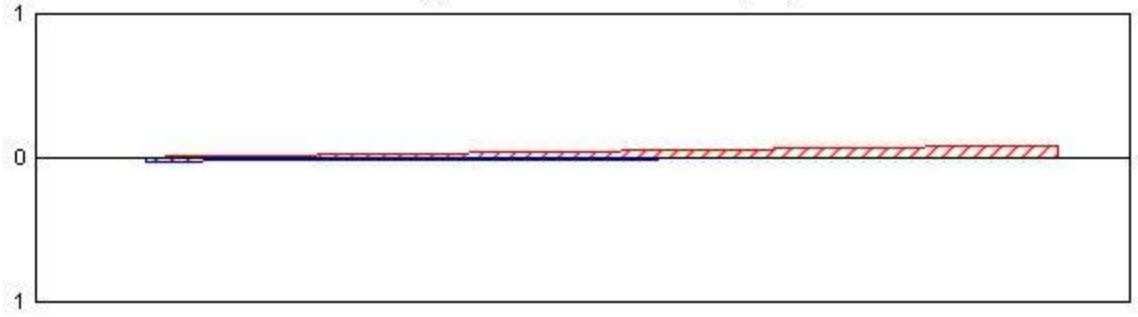
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



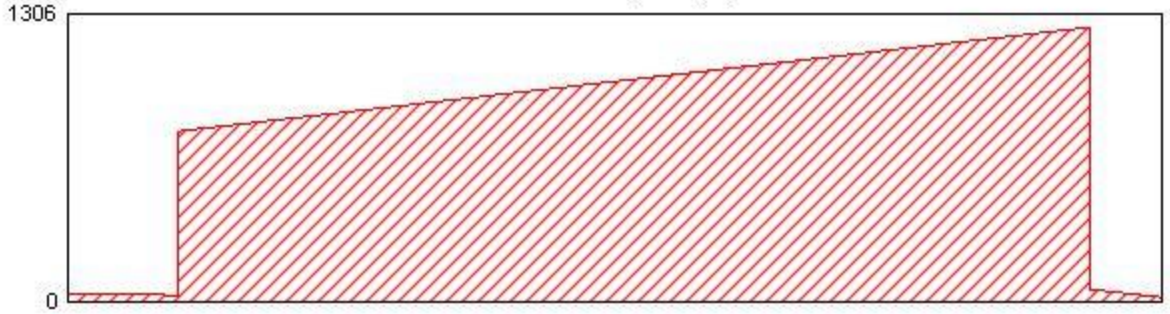


Longitudinal Reinforcement (in2)



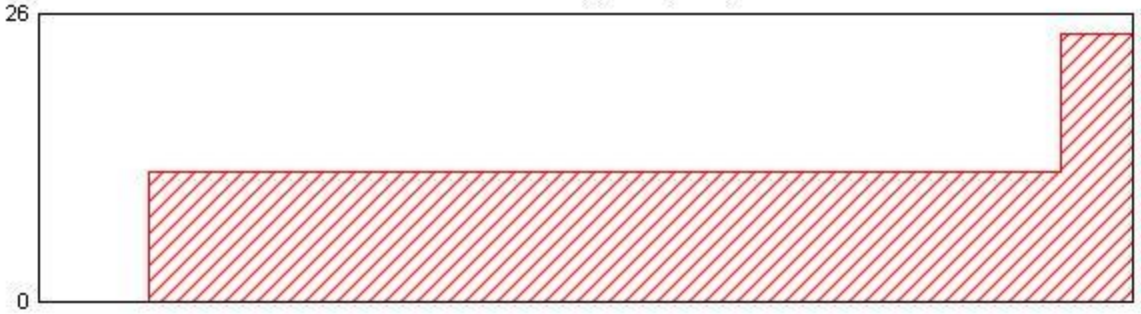
As (top)	0	0	6.591E-002
Combo			Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	2.576E-002	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	766.96	0	1243.56

Torsion Diagram (lb-in)

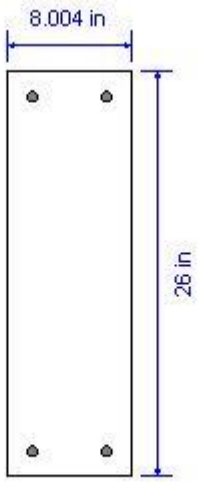


Torsion	11.71	0	24.16

ACI 318-14 Concrete Beam Design

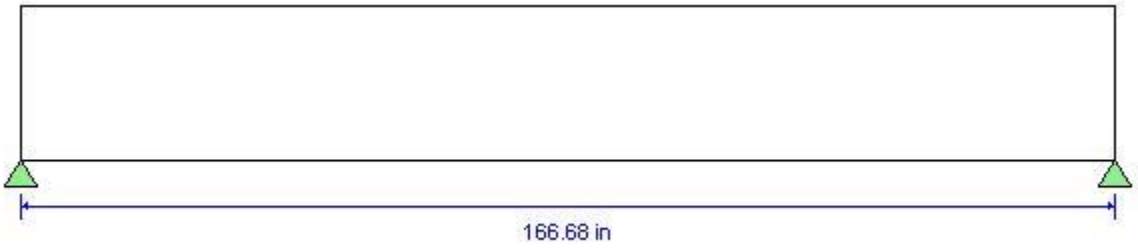
Geometric Properties

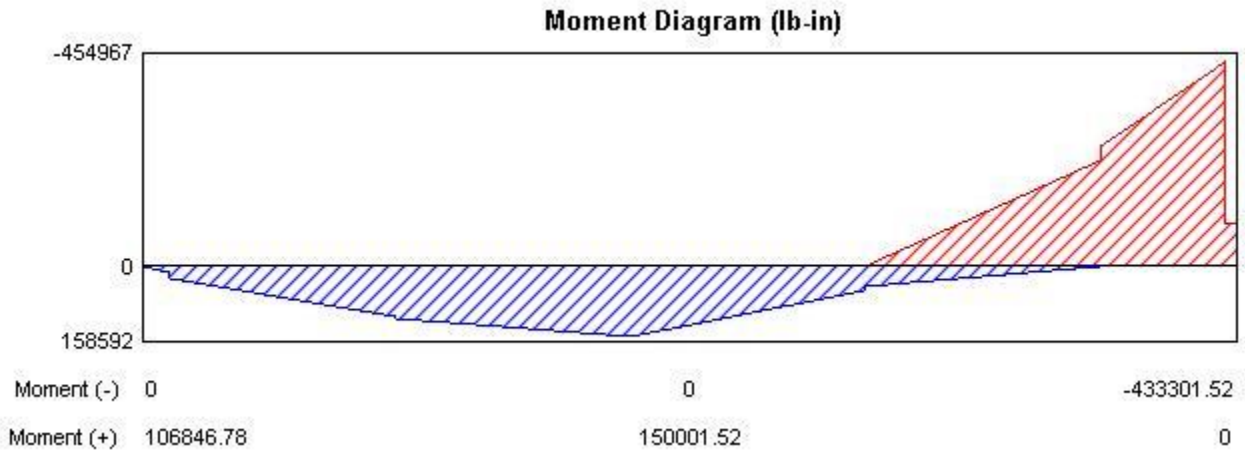
Combination = Overall Envelope
Beam Label = 61
Section Property = B9-8x26-4 ksi
Length = 166.68 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



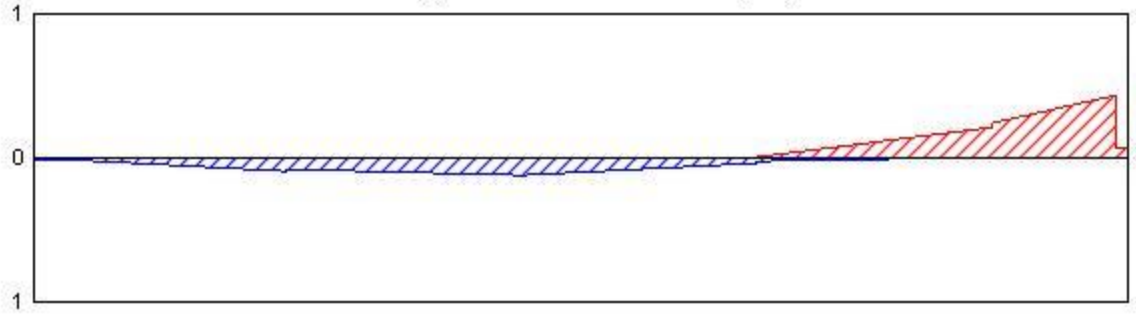
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



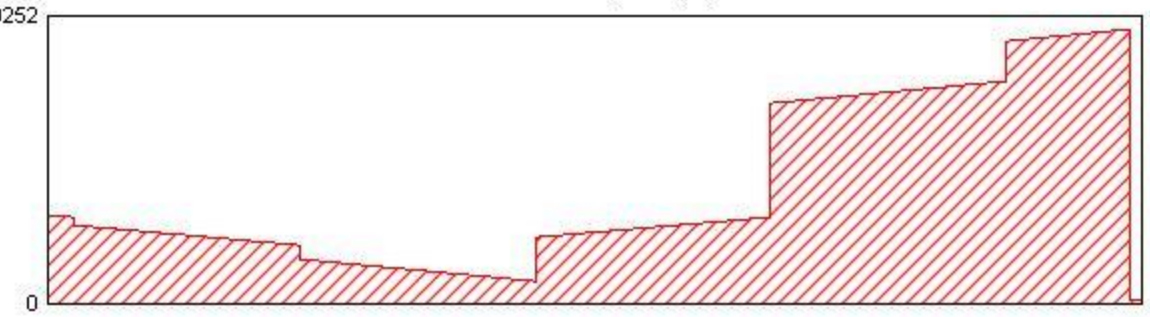


Longitudinal Reinforcement (in2)



As (top)	0	0	0.32
Combo			Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	7.129E-002	9.787E-002	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)

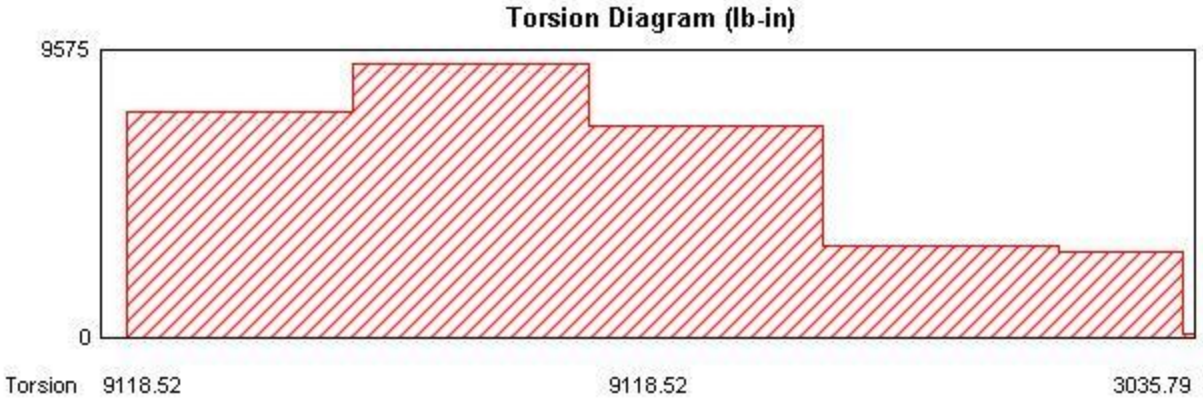


Shear	3146.76	7109.67	9763.22

Transverse Reinforcement (in2/ft)



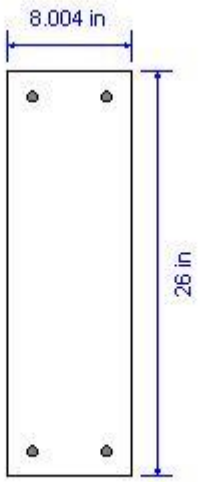
Asv/s reqd	0	0	8.004E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
9575
0
Torsion
9118.52
9118.52
3035.79

ACI 318-14 Concrete Beam Design

Geometric Properties

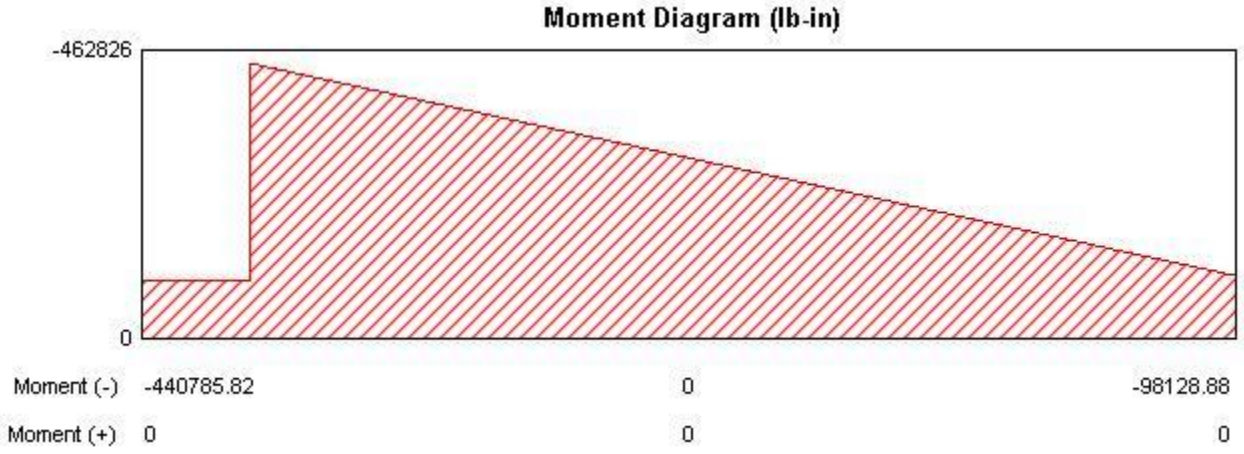
Combination = Overall Envelope
Beam Label = 62
Section Property = B9-8x26-4 ksi
Length = 20.28 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



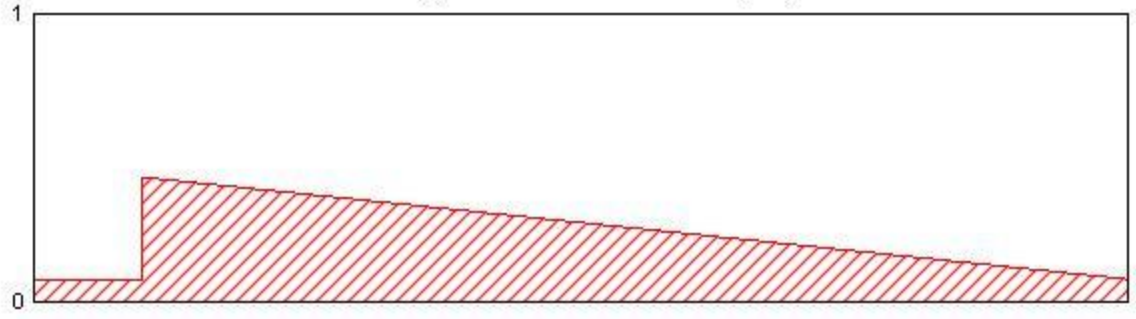
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



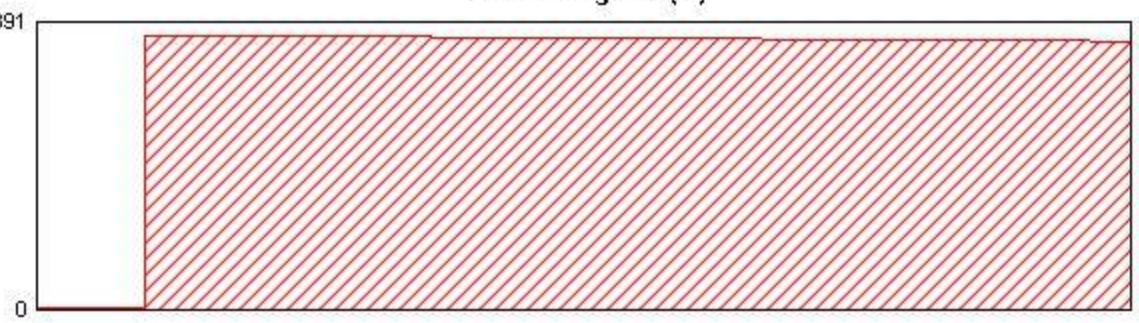


Longitudinal Reinforcement (in2)



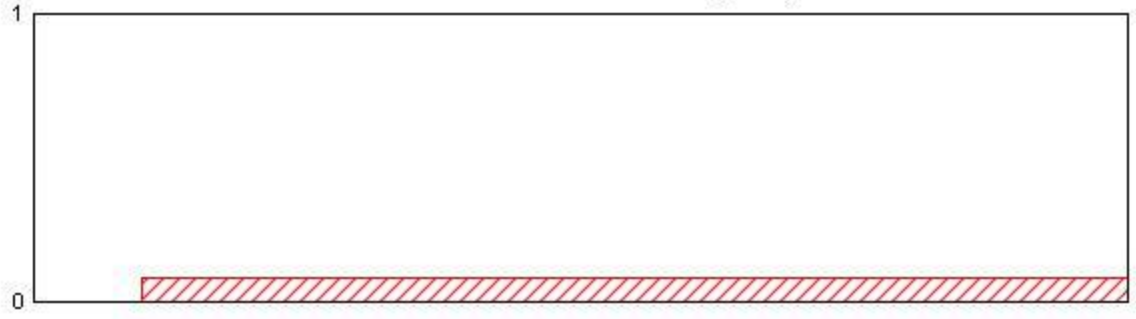
As (top)	0.32	0	5.819E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)

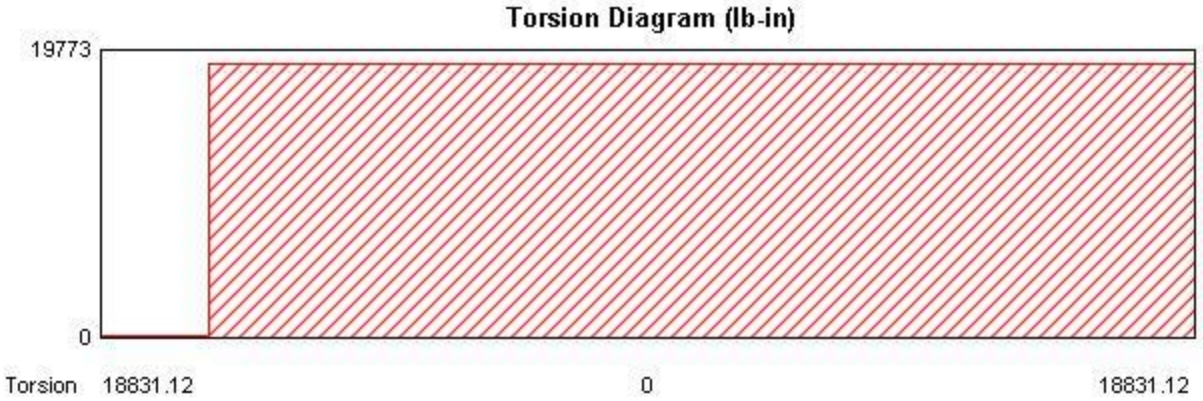


Shear	18943.04	0	18546.78

Transverse Reinforcement (in2/ft)



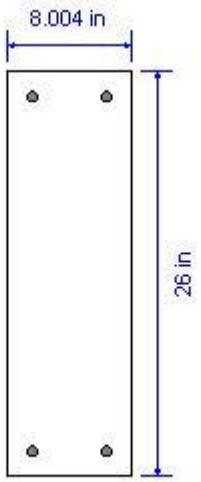
Asv/s reqd	8.004E-002	0	8.004E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
19773
0
Torsion
18831.12
0
18831.12

ACI 318-14 Concrete Beam Design

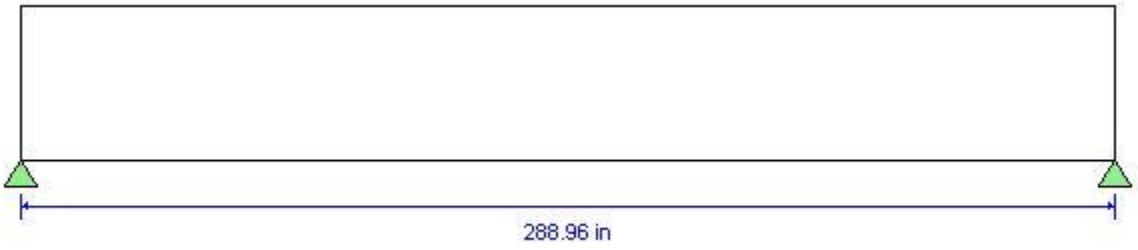
Geometric Properties

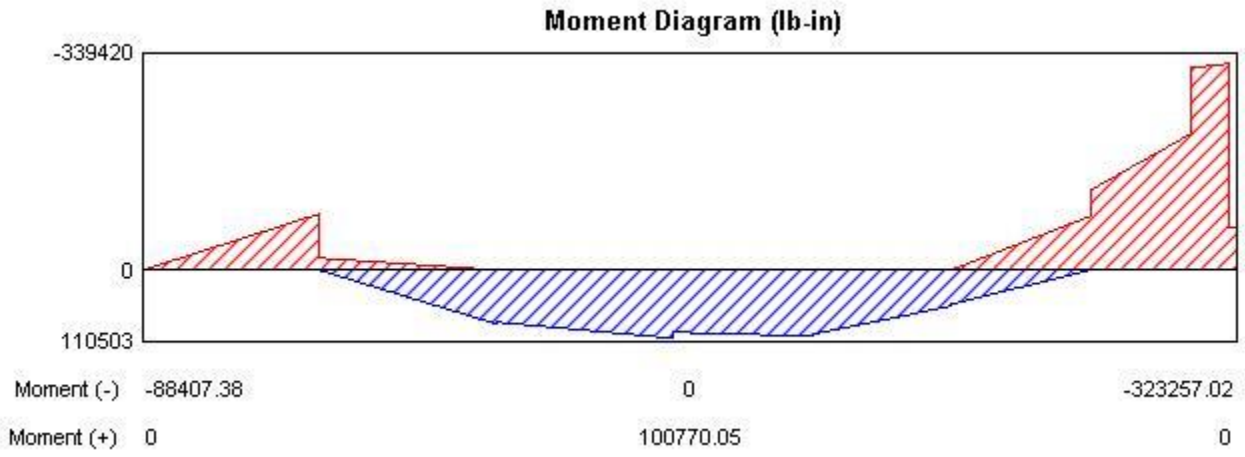
Combination = Overall Envelope
Beam Label = 63
Section Property = B9-8x26-4 ksi
Length = 288.96 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



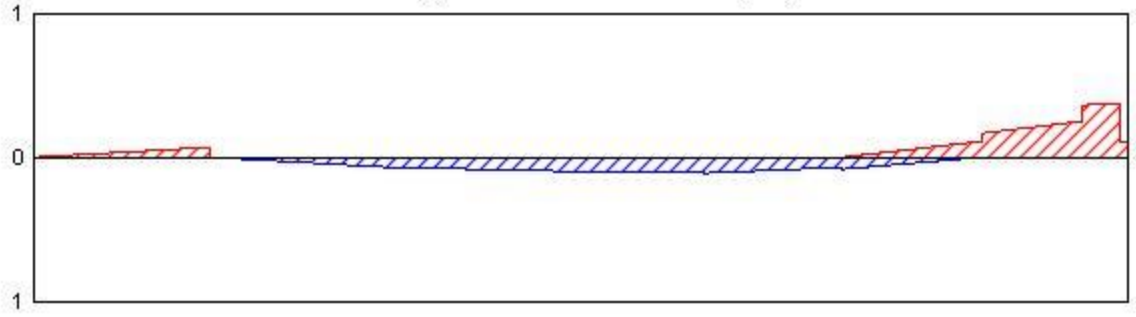
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



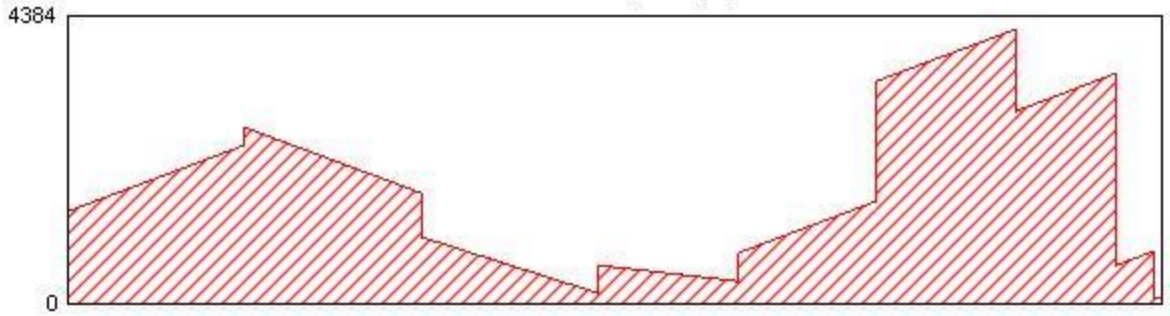


Longitudinal Reinforcement (in2)



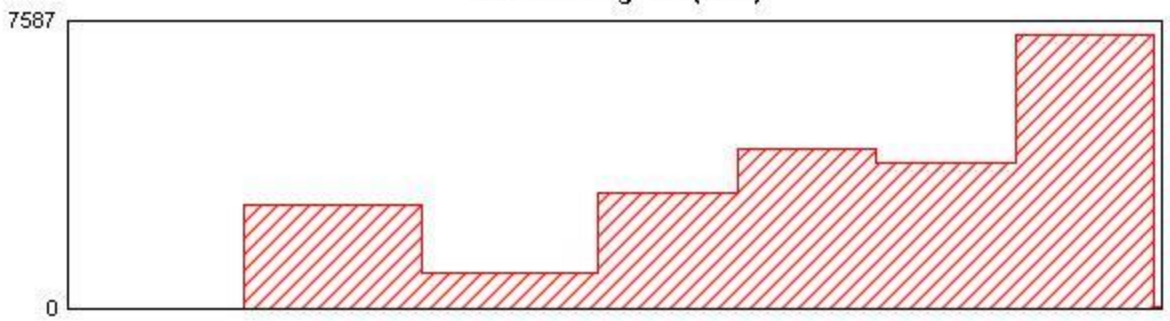
As (top)	5.499E-002	0	0.28
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	8.466E-002	0
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)



Shear	2682.79	3378.62	4174.84

Torsion Diagram (lb-in)

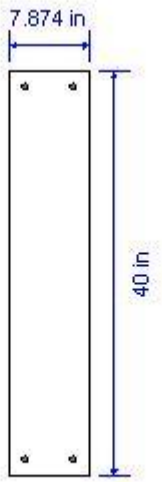


Torsion	2761.87	4200.55	7225.1

ACI 318-14 Concrete Beam Design

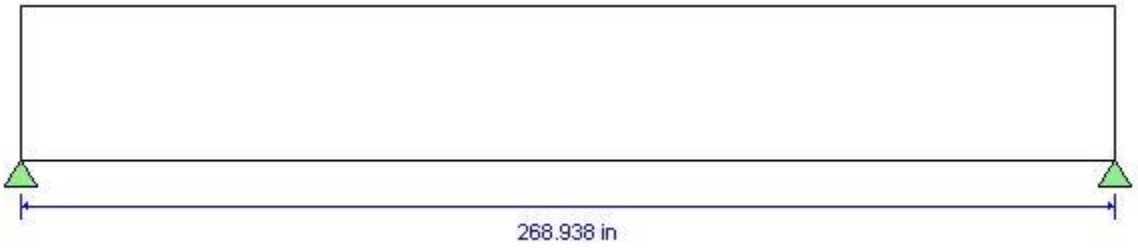
Geometric Properties

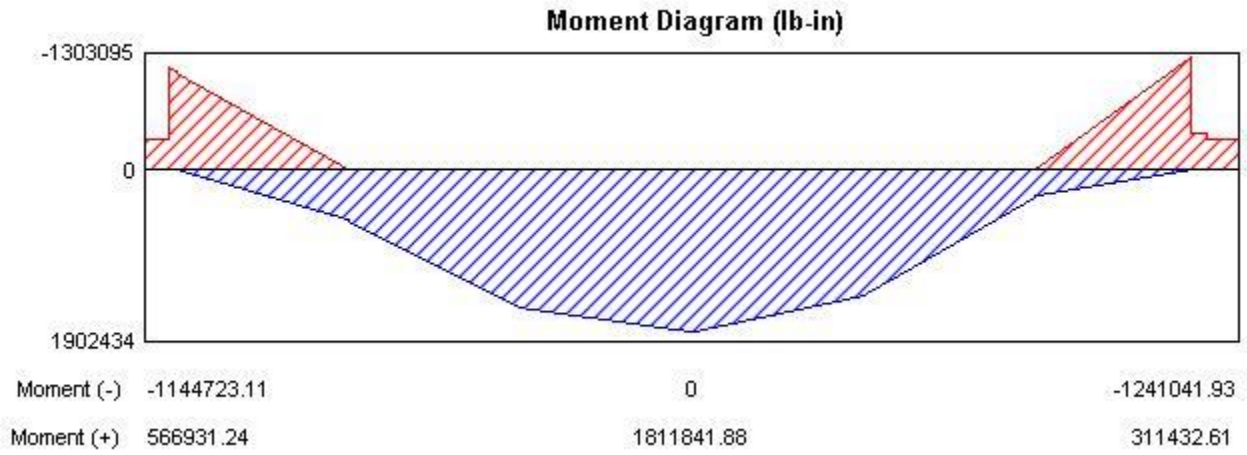
Combination = Overall Envelope
Beam Label = 64
Section Property = B12-12x40- 4 ksi
Length = 268.938 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



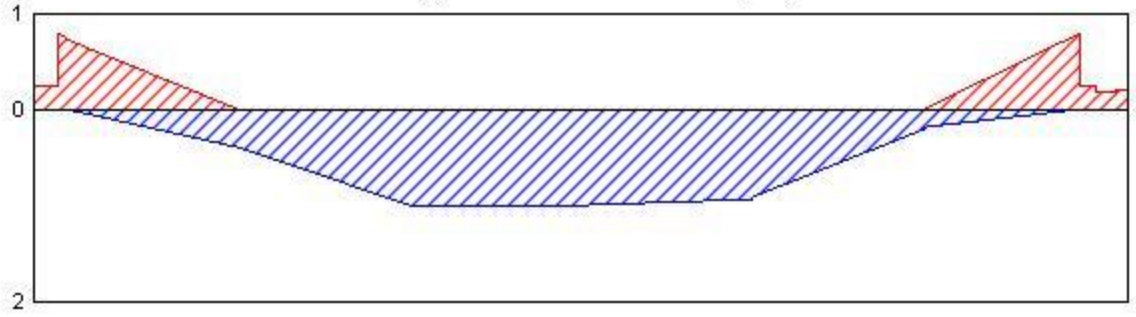
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



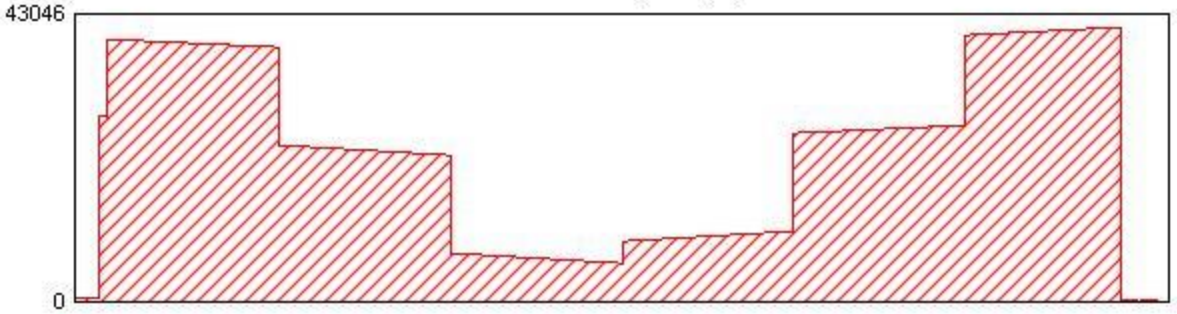


Longitudinal Reinforcement (in2)



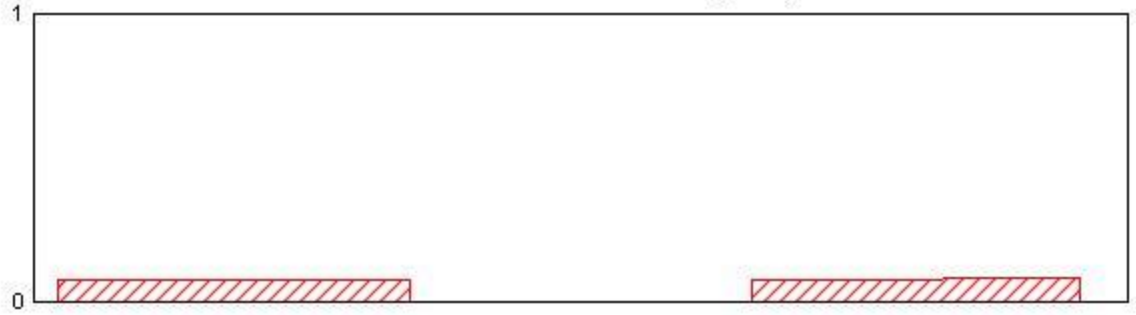
As (top)	0.59	0	0.6
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.3	0.91	0.15
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

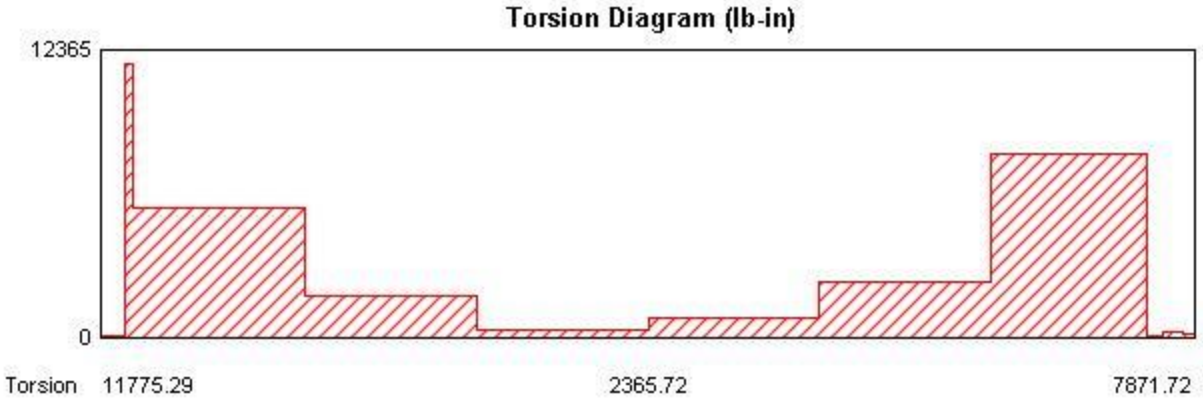


Shear	39250.6	25057.19	40996.09

Transverse Reinforcement (in2/ft)



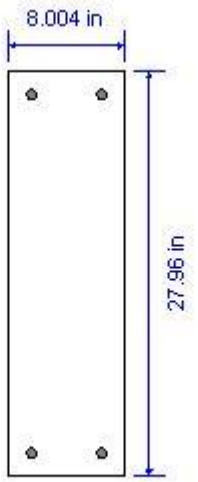
Asv/s reqd	7.874E-002	7.874E-002	8.495E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
12365
0
Torsion
11775.29
2365.72
7871.72

ACI 318-14 Concrete Beam Design

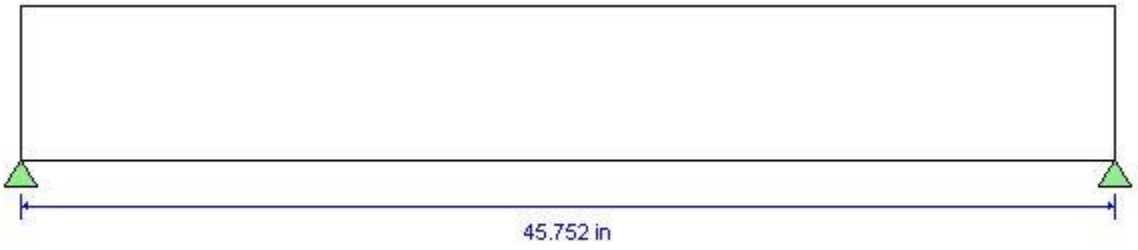
Geometric Properties

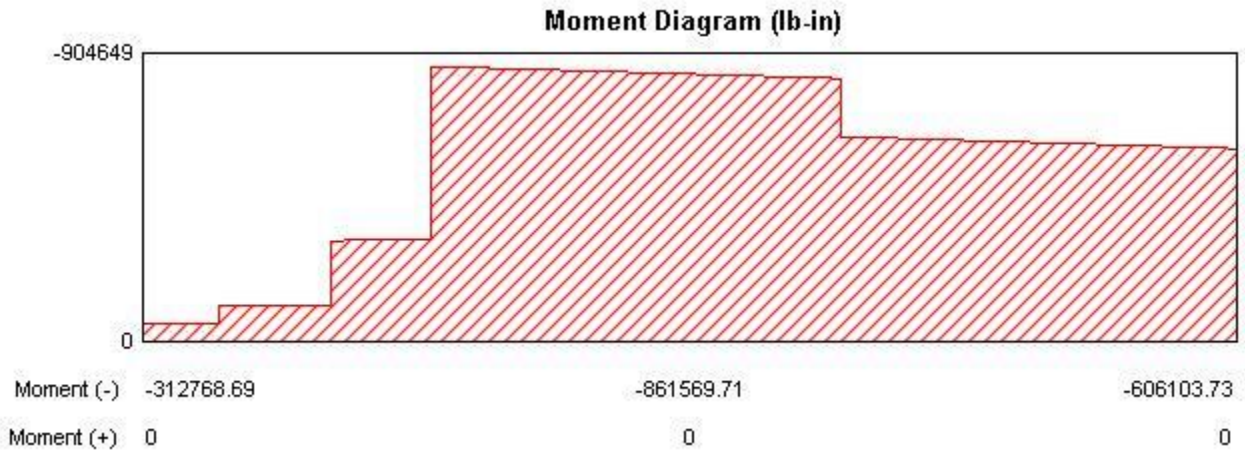
Combination = Overall Envelope
Beam Label = 65
Section Property = B8-8x28-4 ksi
Length = 45.752 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



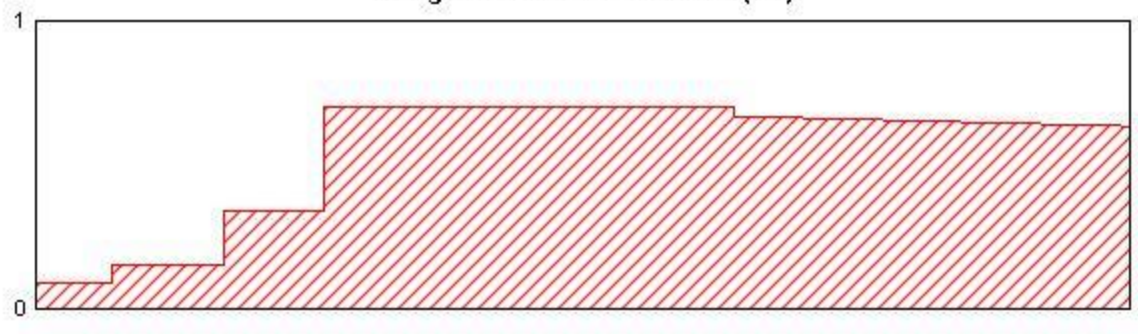
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



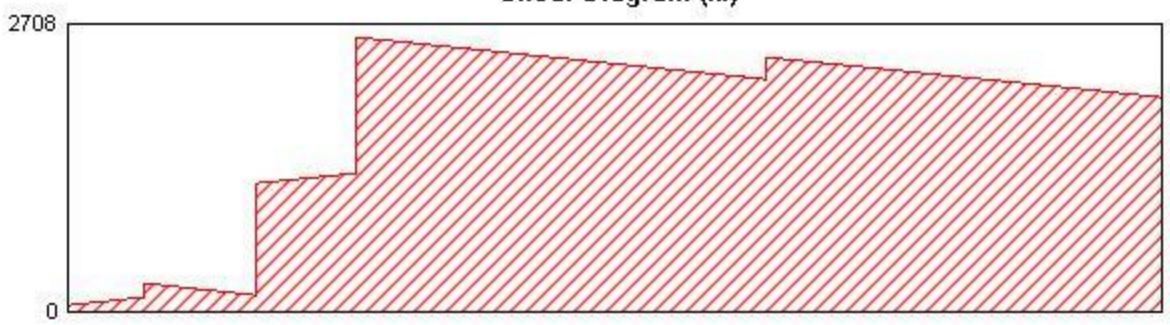


Longitudinal Reinforcement (in2)



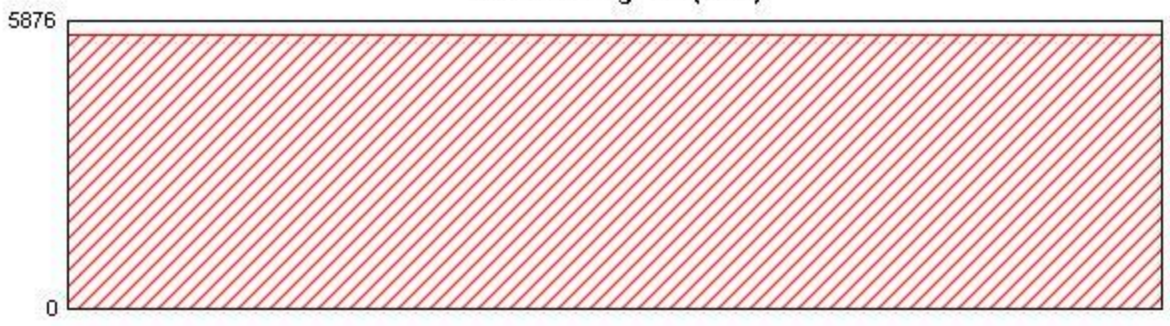
As (top)	0.25	0.66	0.47
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	1200.14	2578.36	2004.23

Torsion Diagram (lb-in)

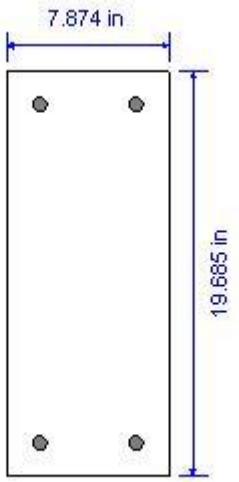


Torsion	5595.75	5595.75	5594.97

ACI 318-14 Concrete Beam Design

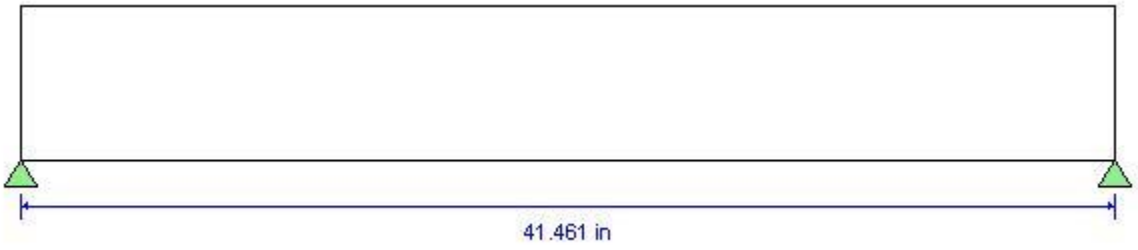
Geometric Properties

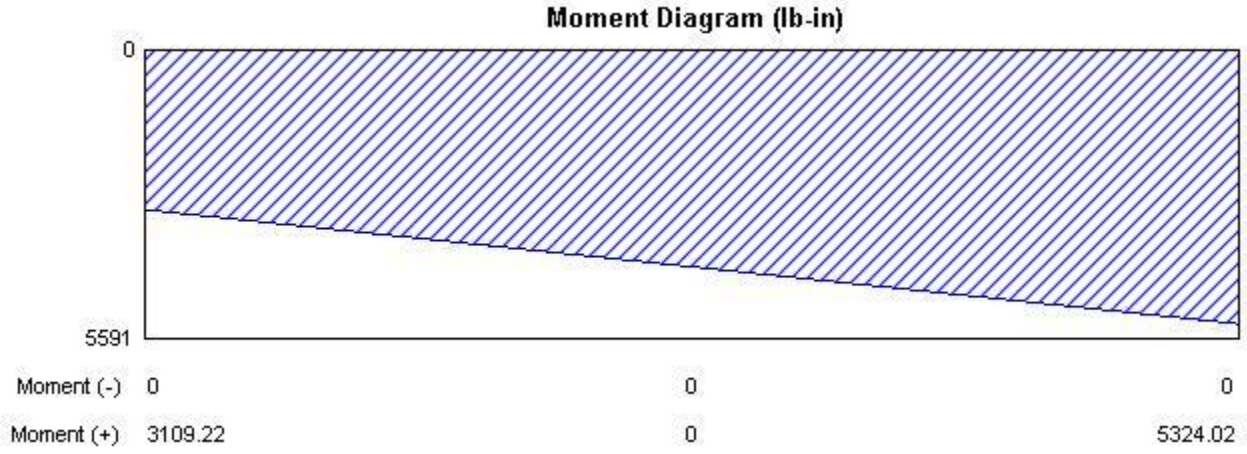
Combination = Overall Envelope
Beam Label = 69
Section Property = B3-8x20- 4 ksi
Length = 41.461 in
Section Width = 7.874 in
Section Depth = 19.685 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



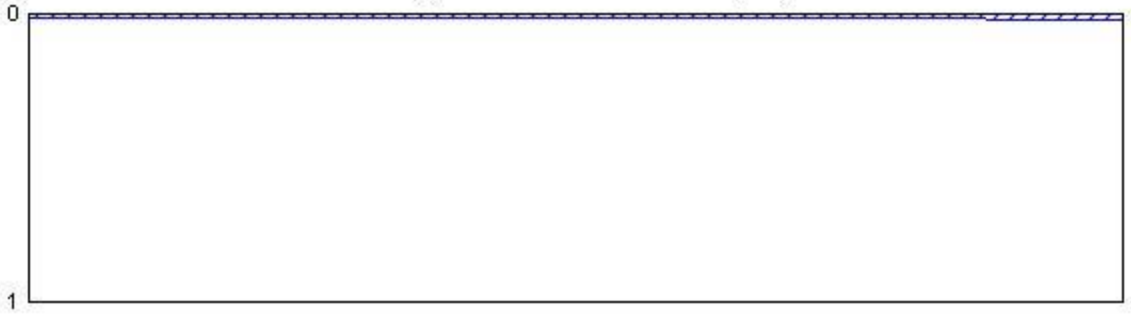
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



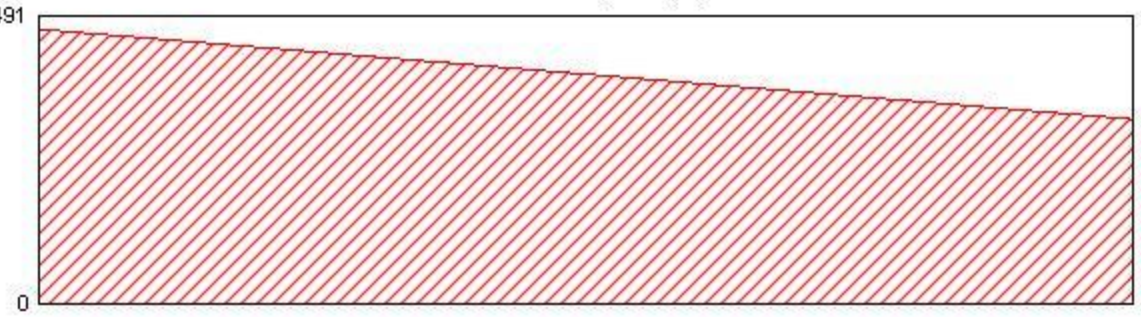


Longitudinal Reinforcement (in2)



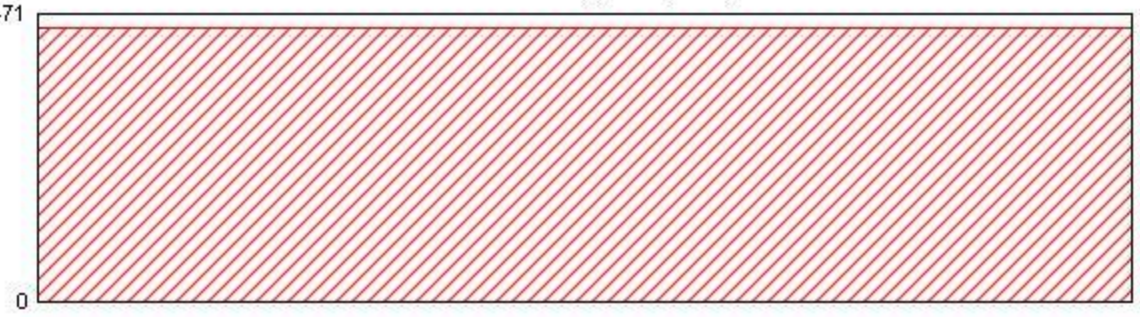
As (top)	0	0	0
Combo			
As (bot)	1.218E-002	0	1.466E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	467.38	0	313.61

Torsion Diagram (lb-in)

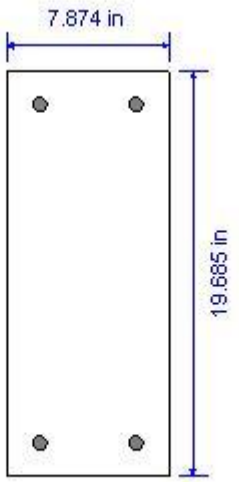


Torsion	6162.55	0	6162.55

ACI 318-14 Concrete Beam Design

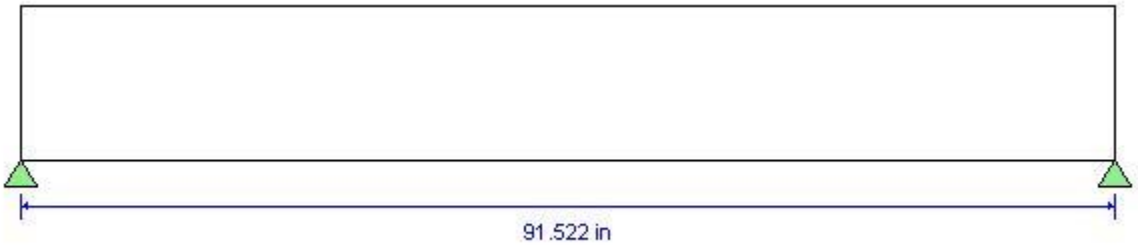
Geometric Properties

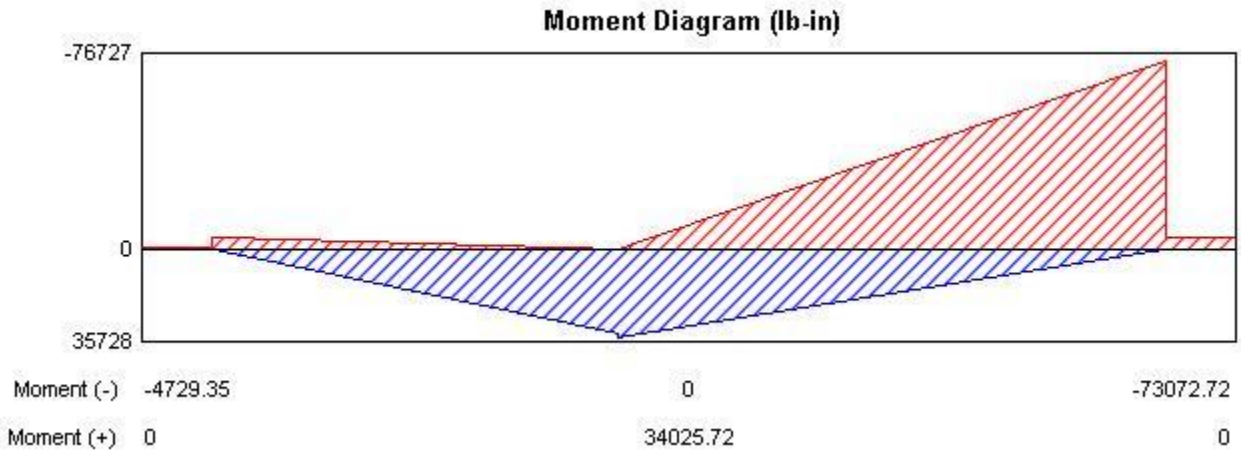
Combination = Overall Envelope
Beam Label = 71
Section Property = B3-8x20- 4 ksi
Length = 91.522 in
Section Width = 7.874 in
Section Depth = 19.685 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



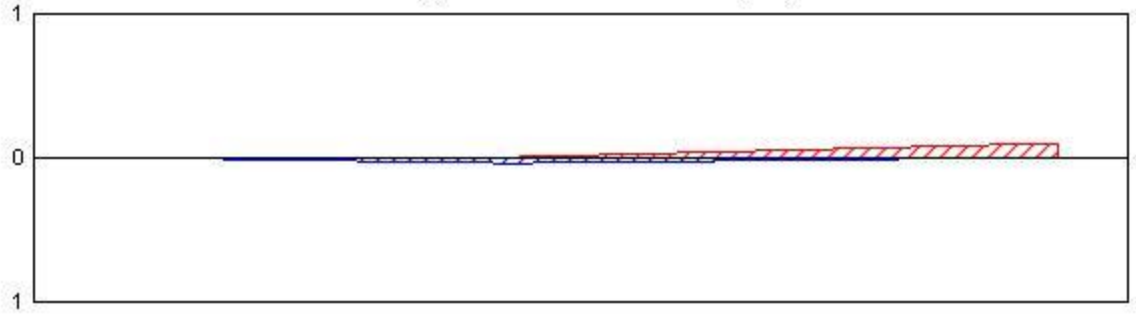
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



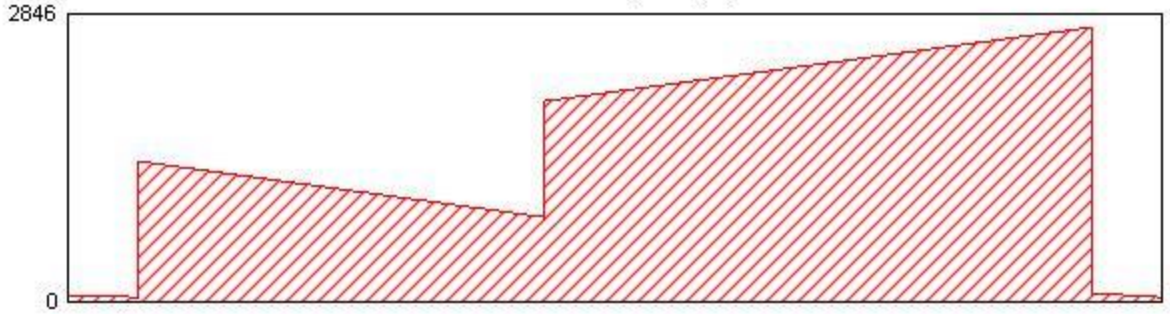


Longitudinal Reinforcement (in2)



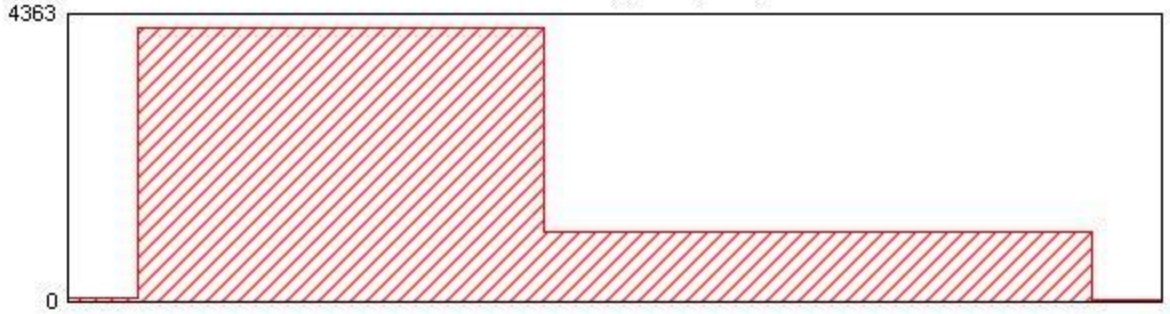
As (top)	0	0	7.427E-002
Combo	DCONU1		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	3.406E-002	0
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)



Shear	1374.58	1970.95	2709.81

Torsion Diagram (lb-in)

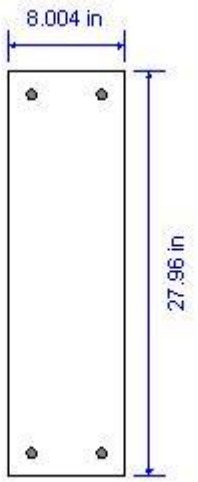


Torsion	4154.73	4154.73	1071.82

ACI 318-14 Concrete Beam Design

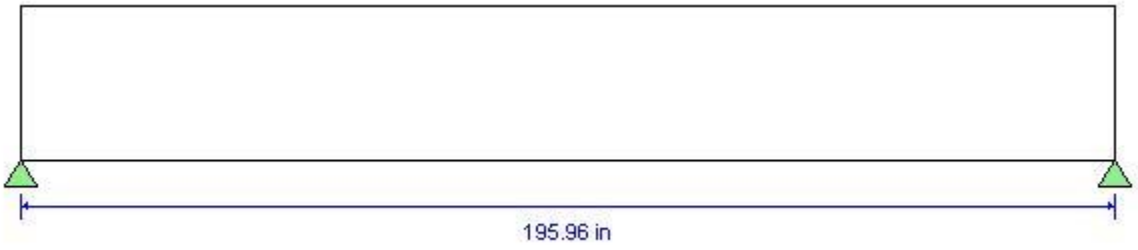
Geometric Properties

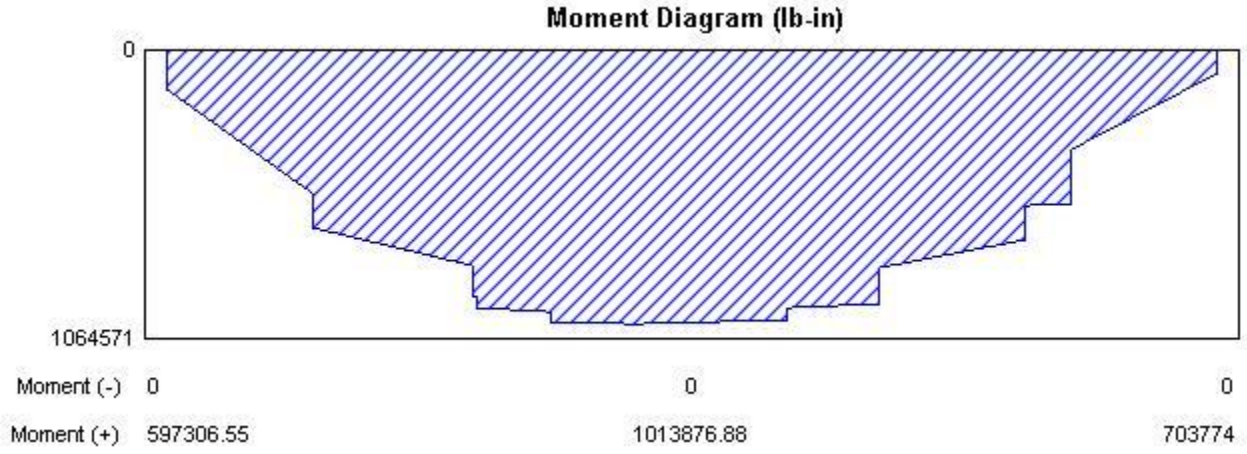
Combination = Overall Envelope
Beam Label = 72
Section Property = B8-8x28-4 ksi
Length = 195.96 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



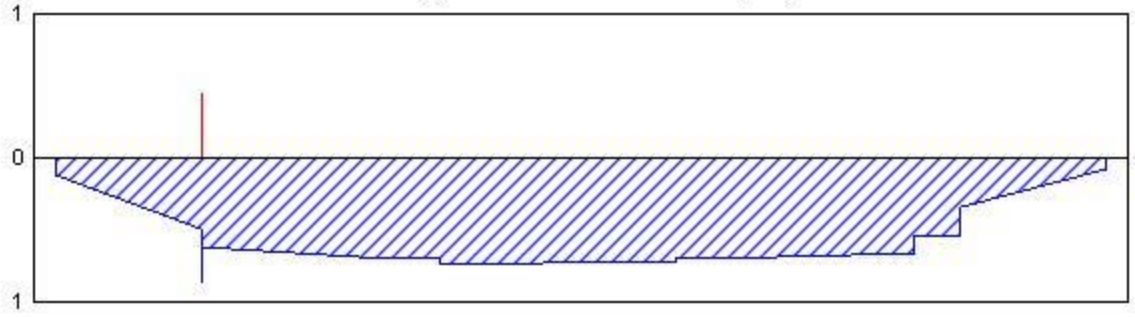
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



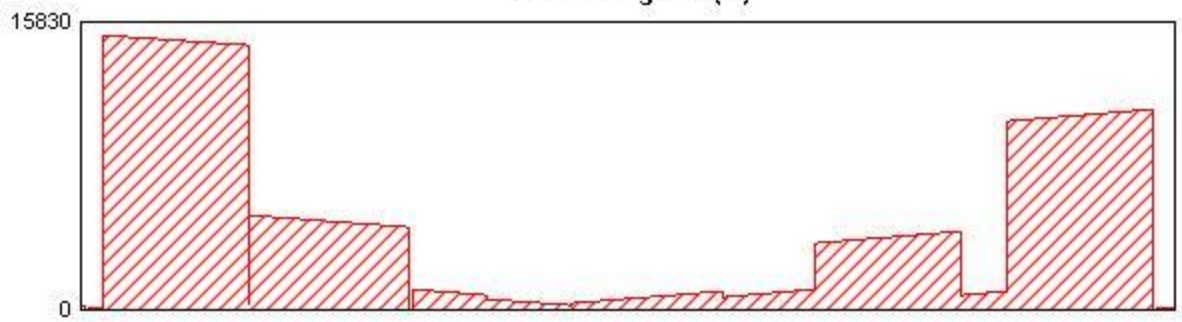


Longitudinal Reinforcement (in2)



As (top)	0.44	0	0
Combo			
As (bot)	0.86	0.73	0.5
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

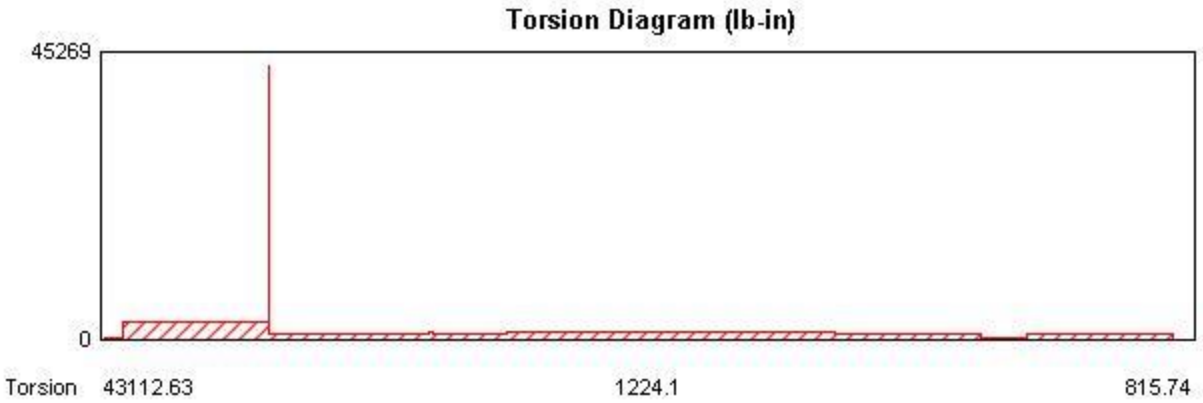


Shear	296.6	4475.84	10975.77

Transverse Reinforcement (in2/ft)



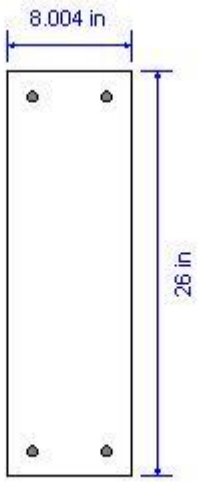
Asv/s reqd	0.12	0	8.004E-002
Combo	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
45269
0
Torsion
43112.63
1224.1
815.74

ACI 318-14 Concrete Beam Design

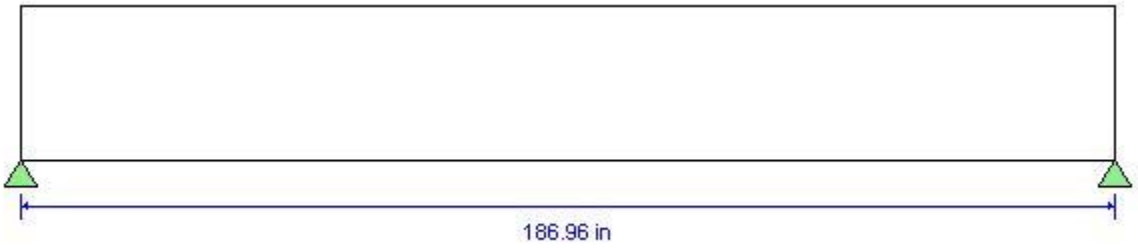
Geometric Properties

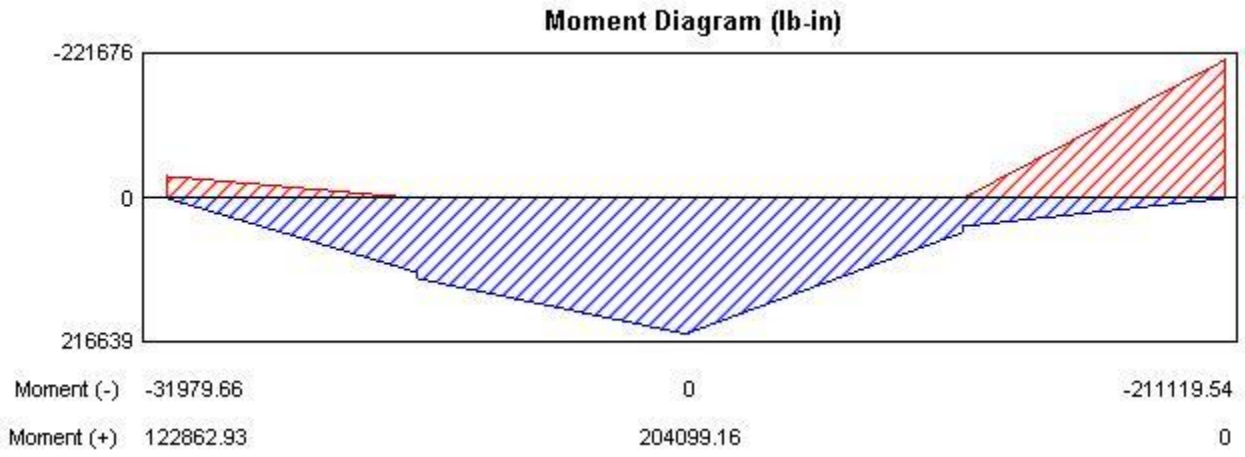
Combination = Overall Envelope
Beam Label = 74
Section Property = B9-8x26-4 ksi
Length = 186.96 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



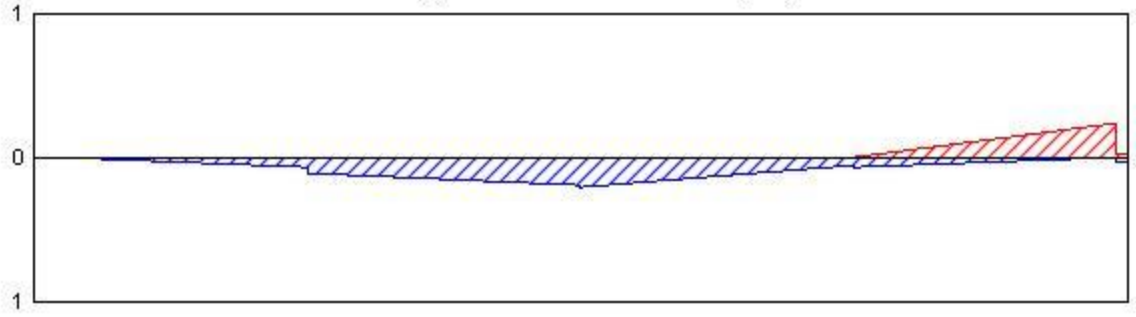
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



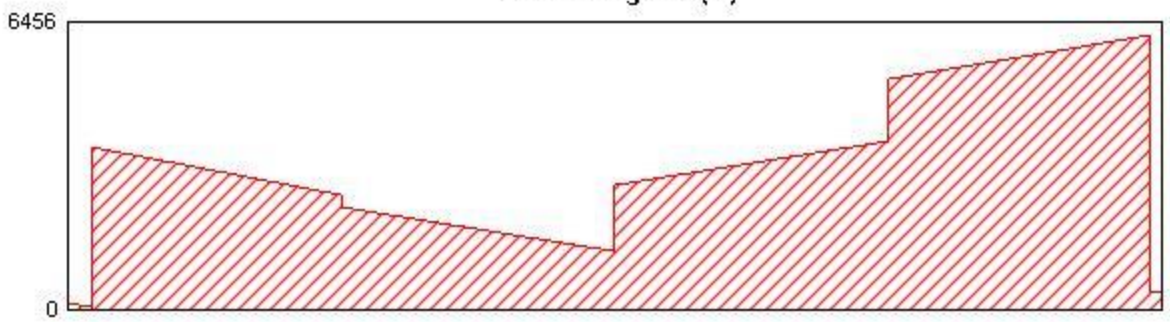


Longitudinal Reinforcement (in2)



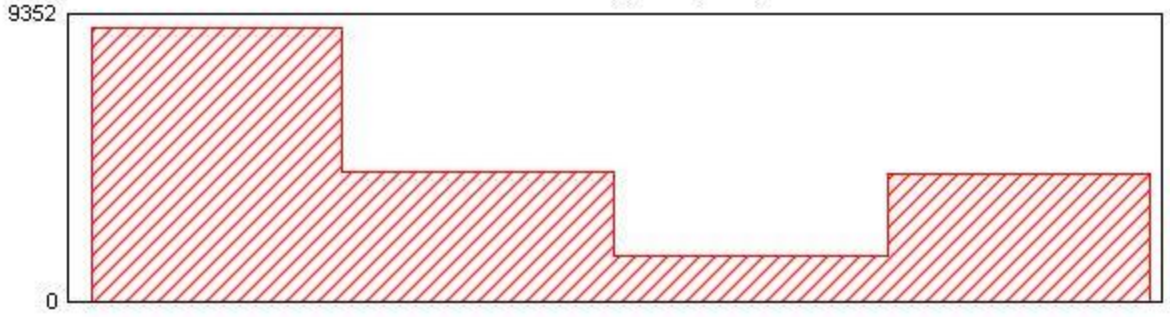
As (top)	0	0	0.18
Combo	DCONU1		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	8.119E-002	0.16	2.164E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	3632.58	5177.6	6148.44

Torsion Diagram (lb-in)

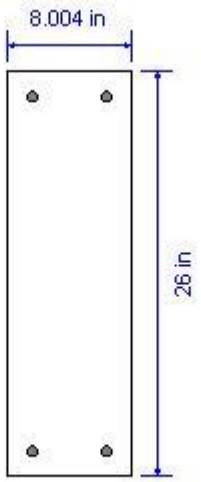


Torsion	8905.87	4207.09	4174.82

ACI 318-14 Concrete Beam Design

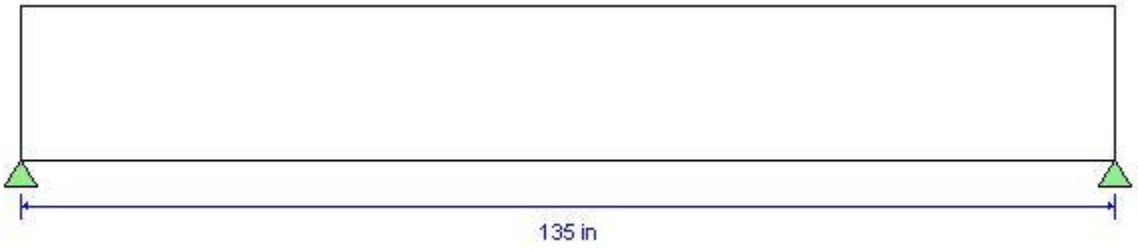
Geometric Properties

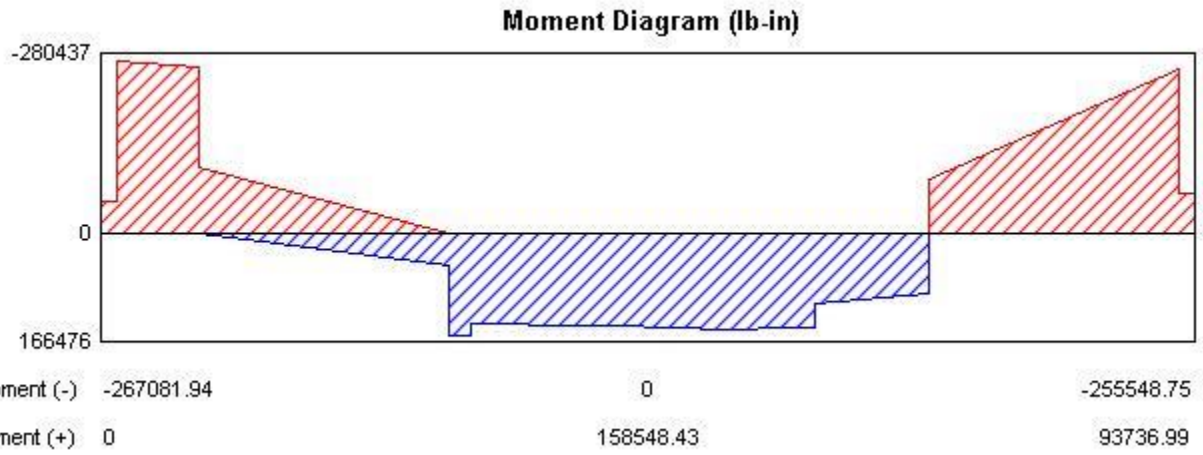
Combination = Overall Envelope
Beam Label = 75
Section Property = B9-8x26-4 ksi
Length = 135 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



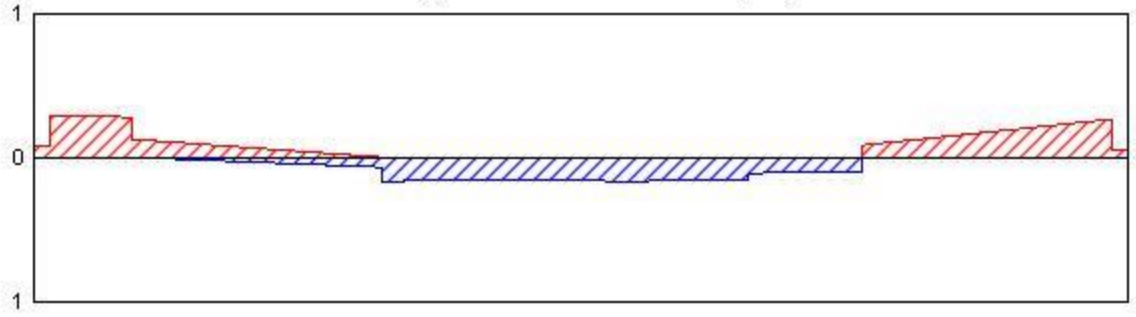
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



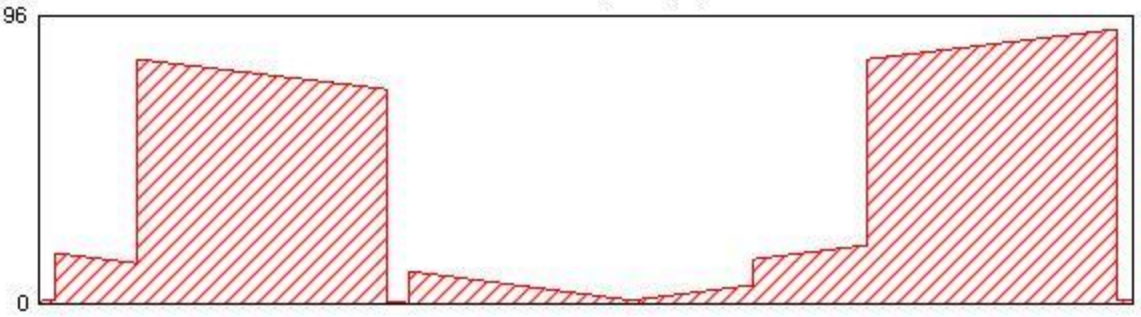


Longitudinal Reinforcement (in2)



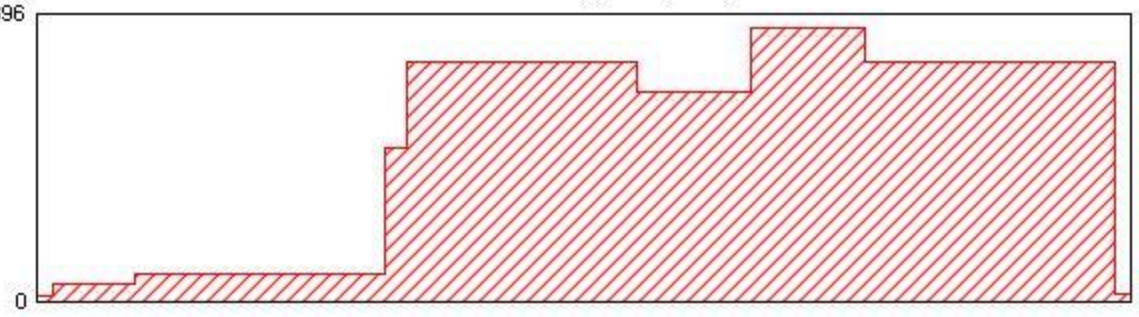
As (top)	0.22	0	0.2
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0.13	7.375E-002
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	5266.23	4594.23	5900.16

Torsion Diagram (lb-in)

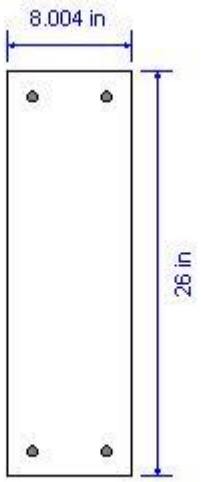


Torsion	285.38	2757.4	2757.4

ACI 318-14 Concrete Beam Design

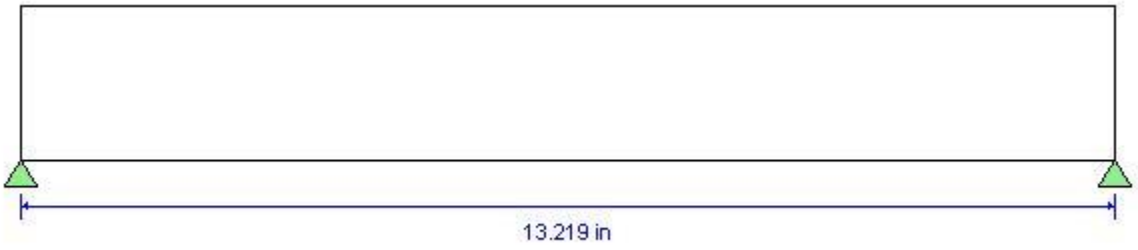
Geometric Properties

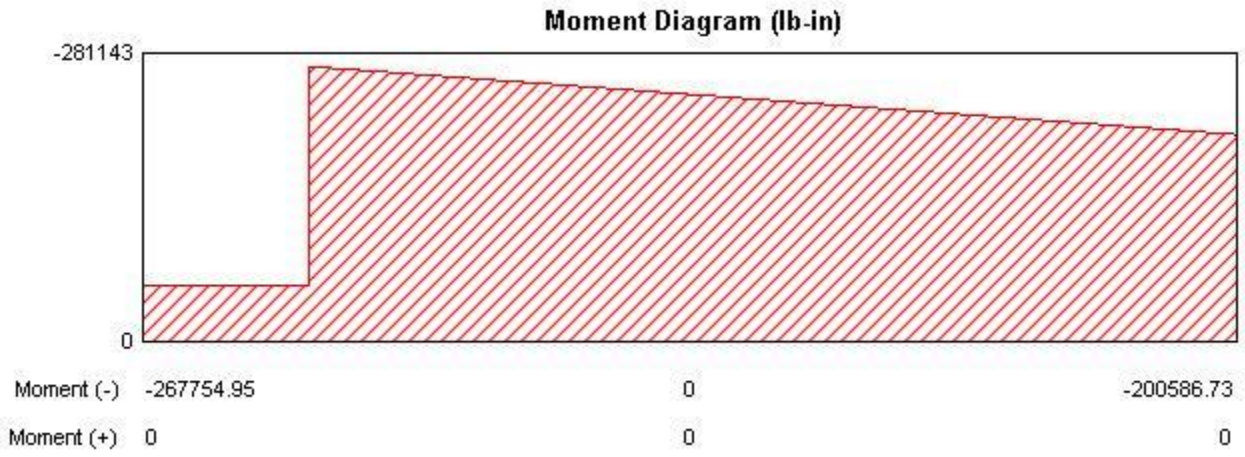
Combination = Overall Envelope
Beam Label = 76
Section Property = B9-8x26-4 ksi
Length = 13.219 in
Section Width = 8.004 in
Section Depth = 26 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



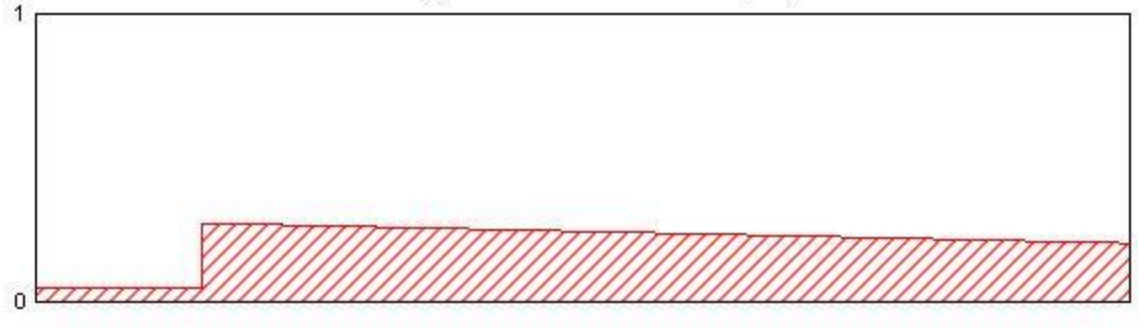
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



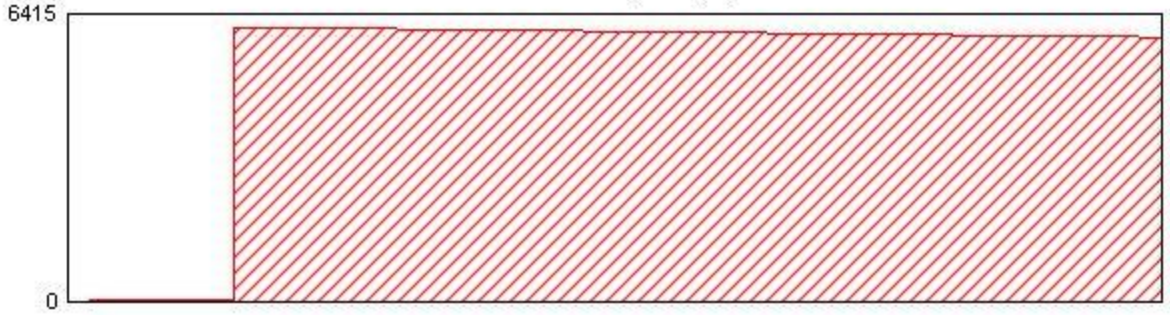


Longitudinal Reinforcement (in2)



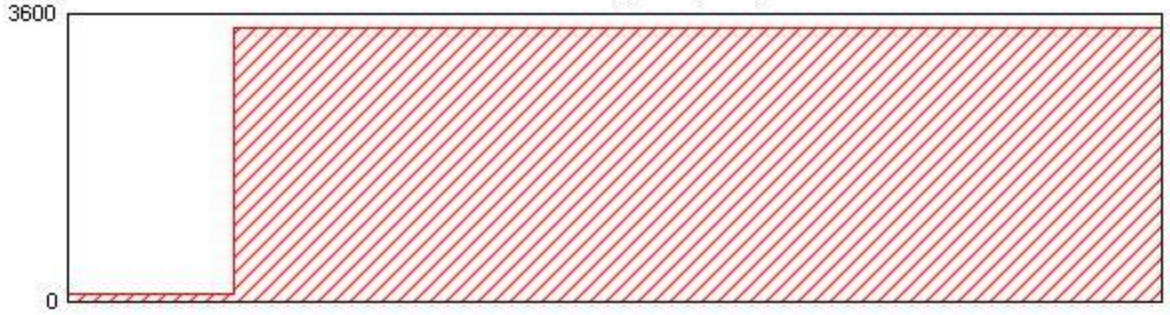
As (top)	0.2	0	0.15
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	6108.58	0	5865.38

Torsion Diagram (lb-in)

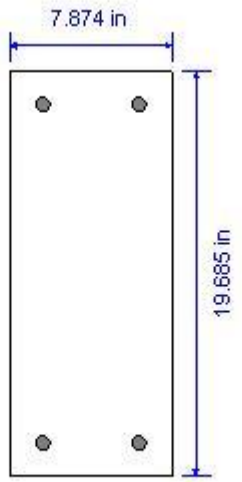


Torsion	3428.43	0	3428.43

ACI 318-14 Concrete Beam Design

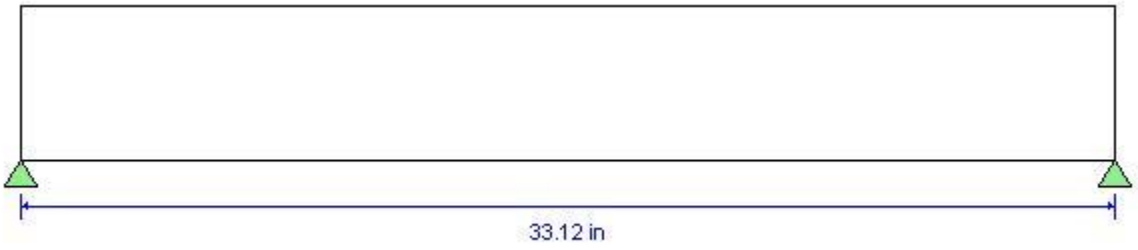
Geometric Properties

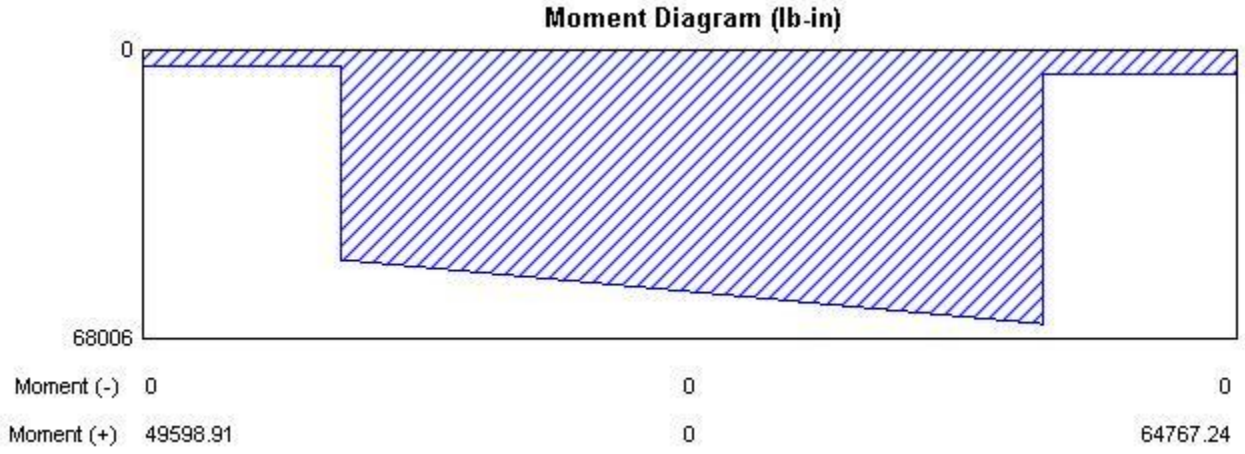
Combination = Overall Envelope
Beam Label = 84
Section Property = B3-8x20- 4 ksi
Length = 33.12 in
Section Width = 7.874 in
Section Depth = 19.685 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



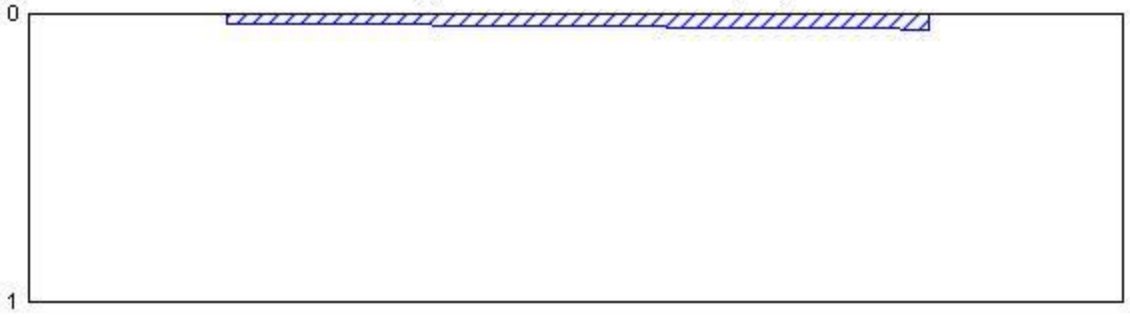
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



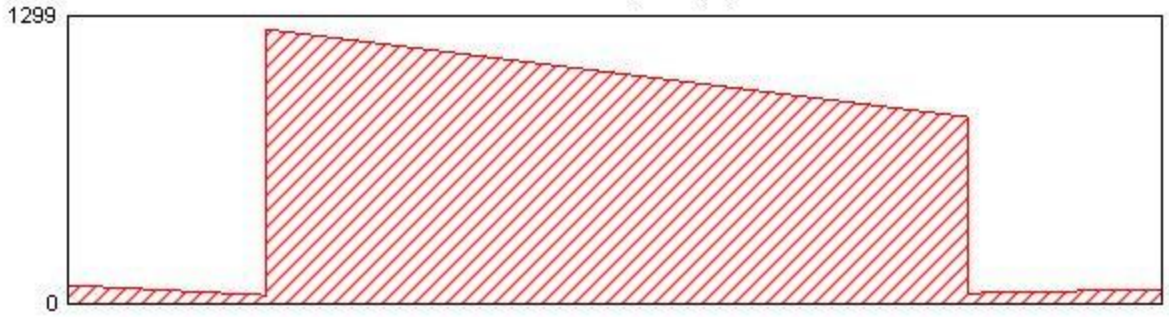


Longitudinal Reinforcement (in2)



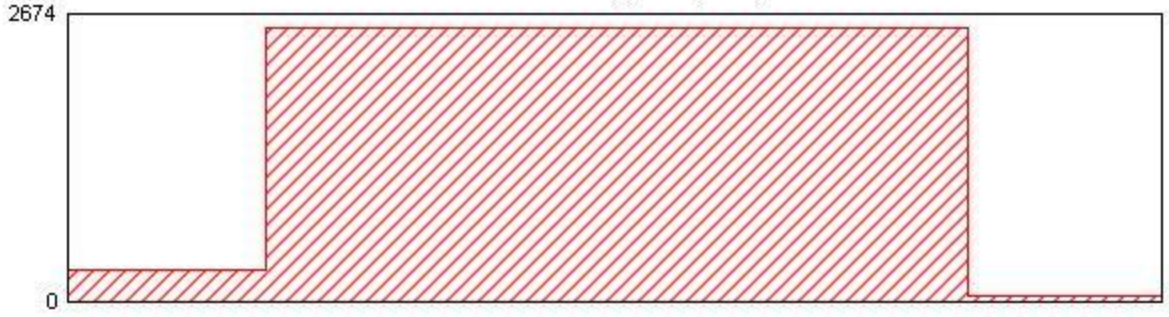
As (top)	0	0	0
Combo			
As (bot)	2.428E-002	0	4.194E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		DCONU1

Shear Diagram (lb)



Shear	1236.83	0	837.22

Torsion Diagram (lb-in)

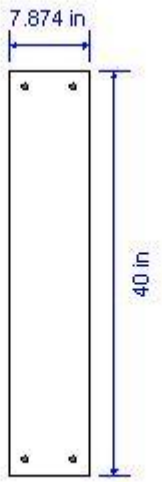


Torsion	2545.99	0	2545.99

ACI 318-14 Concrete Beam Design

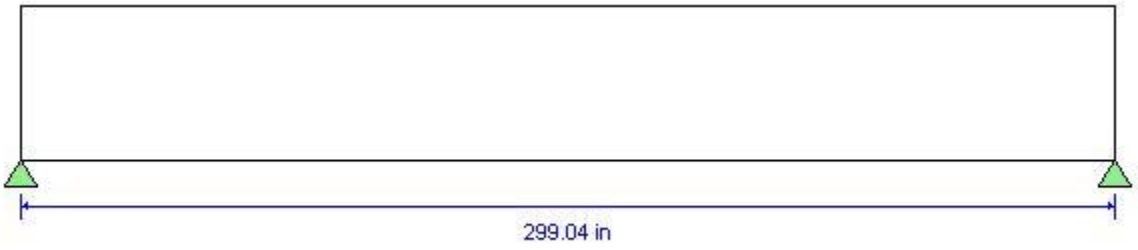
Geometric Properties

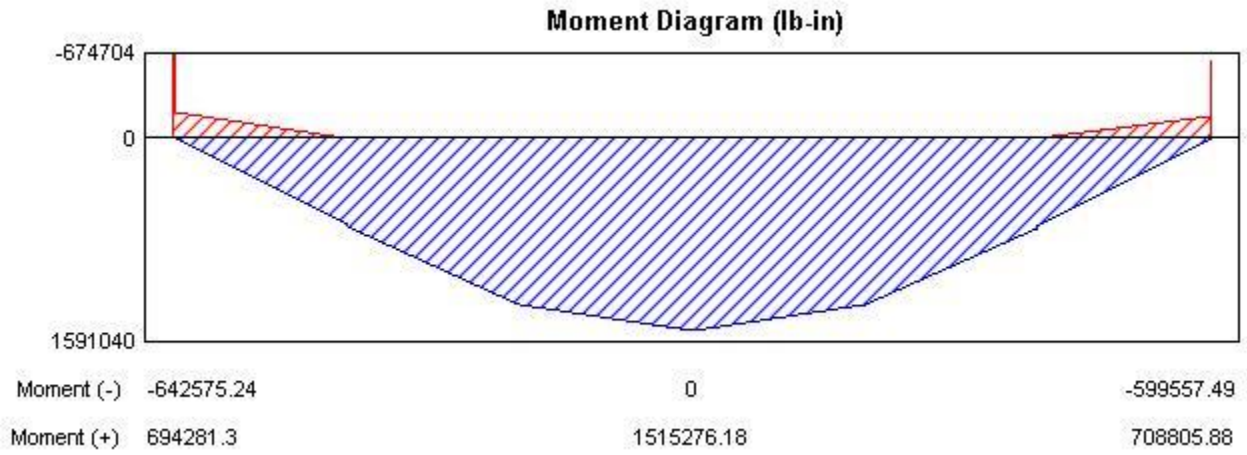
Combination = Overall Envelope
Beam Label = 85
Section Property = B12-12x40- 4 ksi
Length = 299.04 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



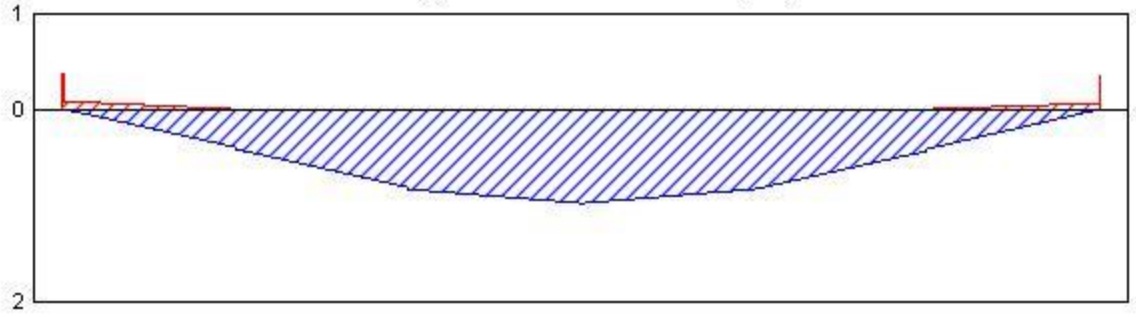
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



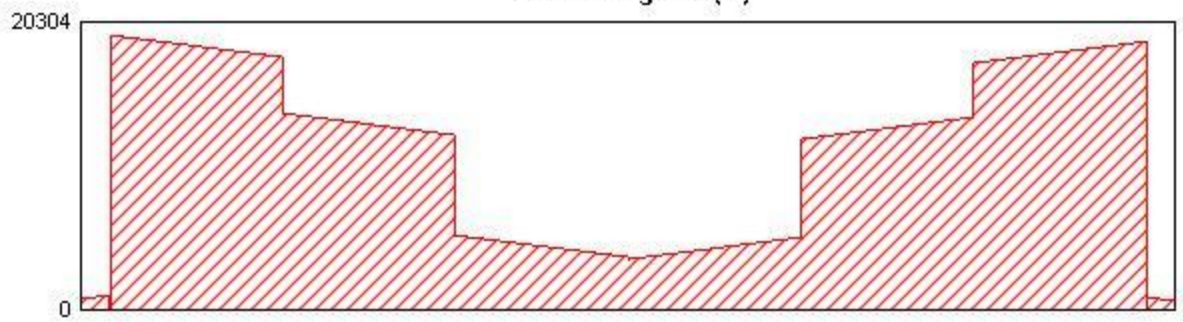


Longitudinal Reinforcement (in2)



As (top)	0.28	0	0.26
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.31	0.73	0.32
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

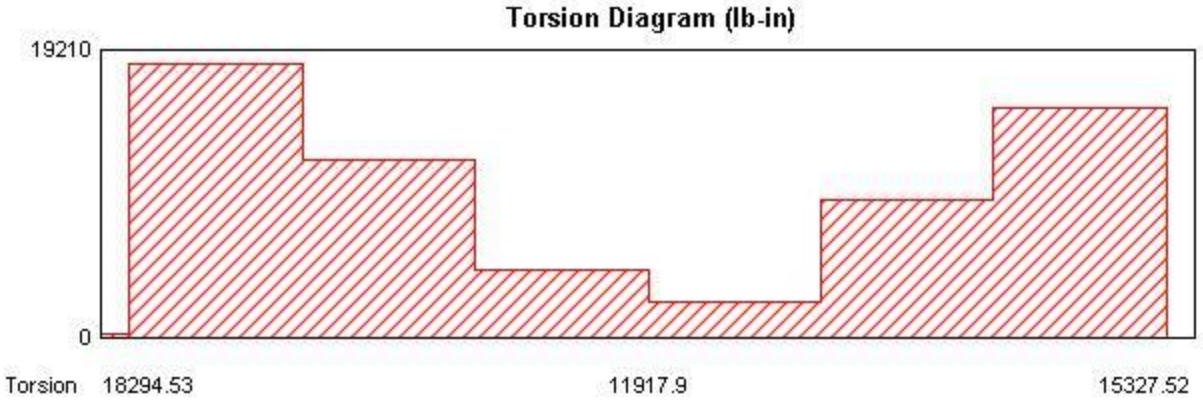


Shear	19337.08	12203.2	18944.42

Transverse Reinforcement (in2/ft)



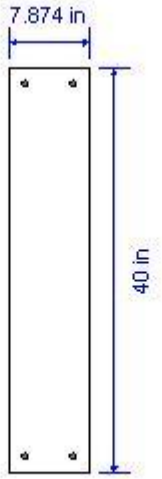
Asv/s reqd	7.874E-002	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
19210
0
Torsion
18294.53
11917.9
15327.52

ACI 318-14 Concrete Beam Design

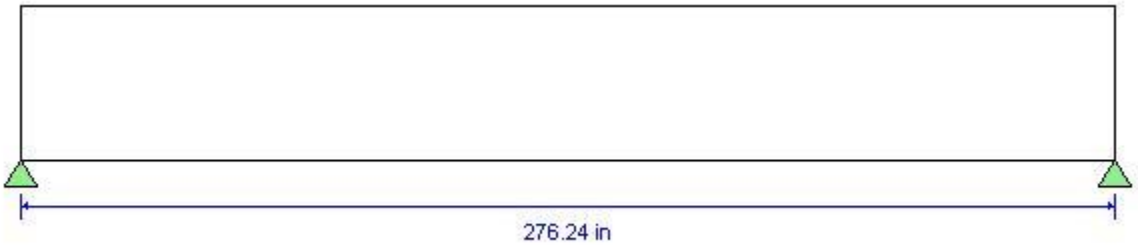
Geometric Properties

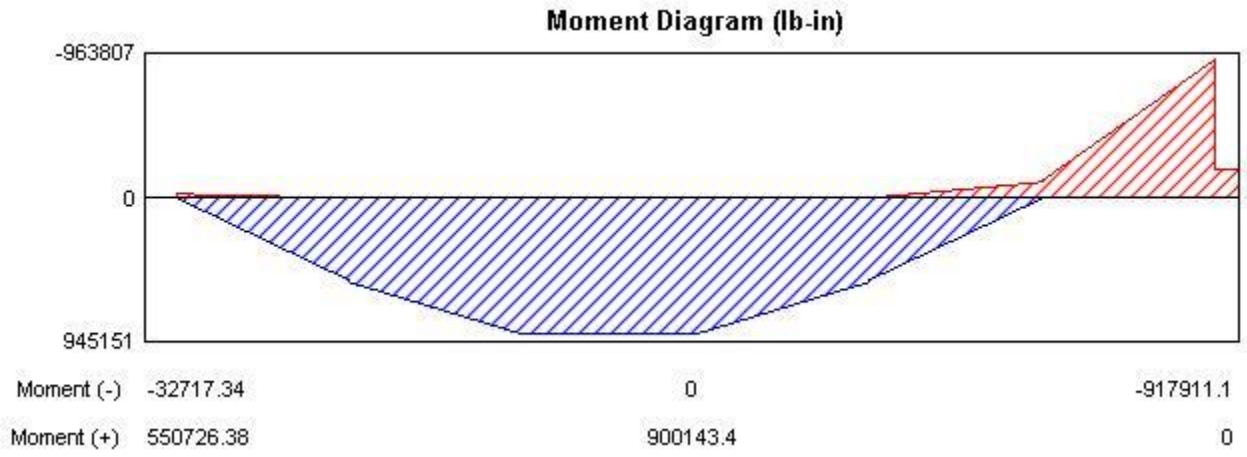
Combination = Overall Envelope
Beam Label = 86
Section Property = B12-12x40- 4 ksi
Length = 276.24 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



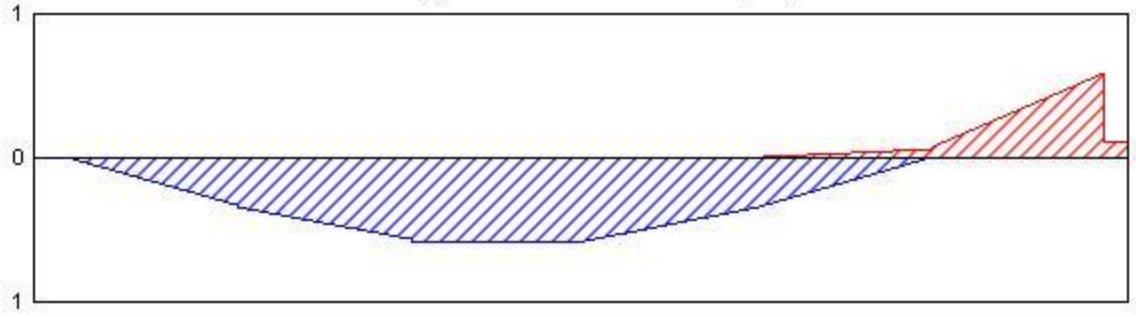
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



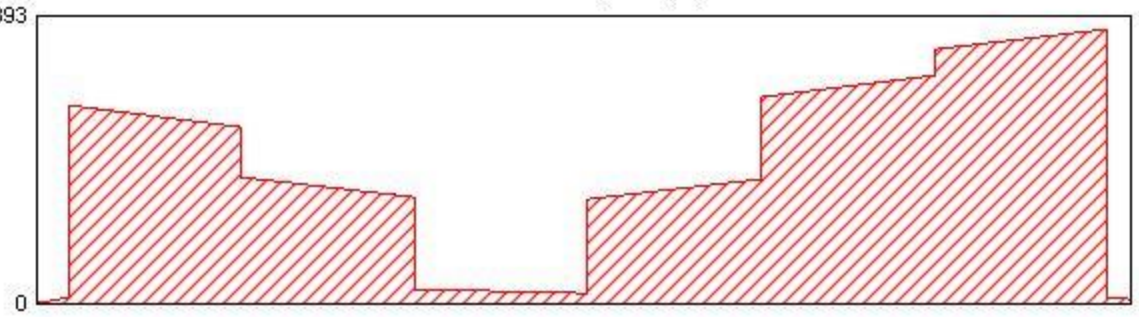


Longitudinal Reinforcement (in2)



As (top)	0	0	0.44
Combo	DCONU1		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.26	0.43	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)

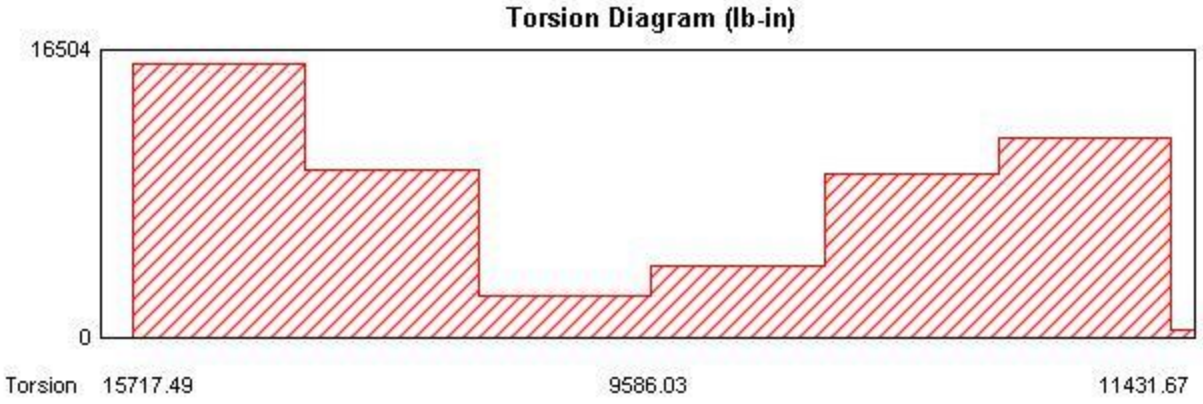


Shear	13653.33	14269.51	18945.39

Transverse Reinforcement (in2/ft)



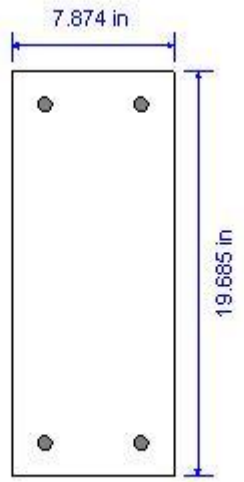
Asv/s reqd	0	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
16504
0
Torsion
15717.49
9586.03
11431.67

ACI 318-14 Concrete Beam Design

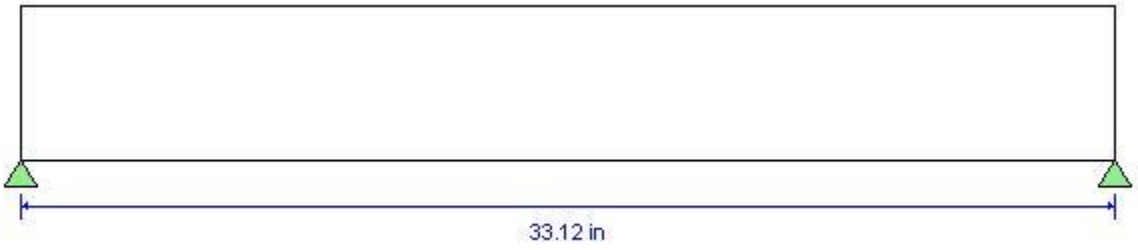
Geometric Properties

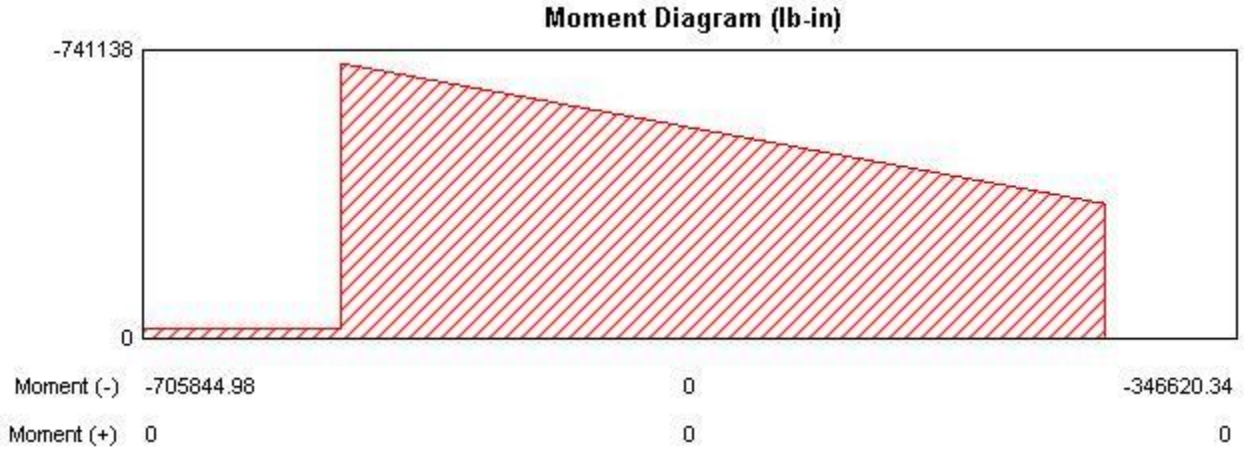
Combination = Overall Envelope
Beam Label = 87
Section Property = B3-8x20- 4 ksi
Length = 33.12 in
Section Width = 7.874 in
Section Depth = 19.685 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



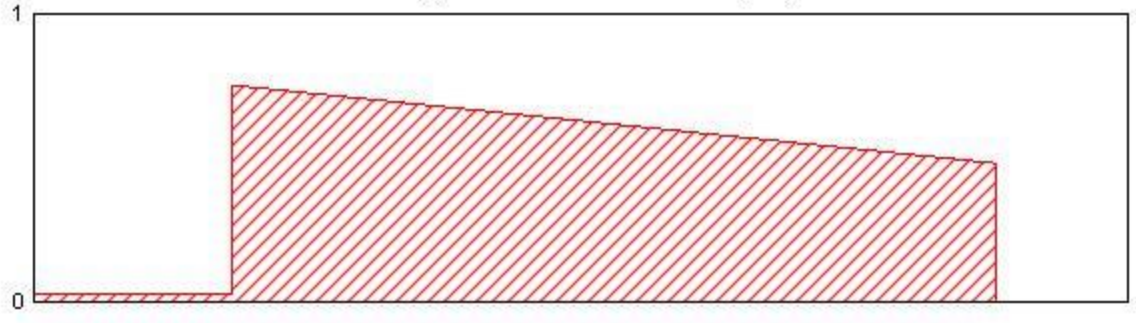
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



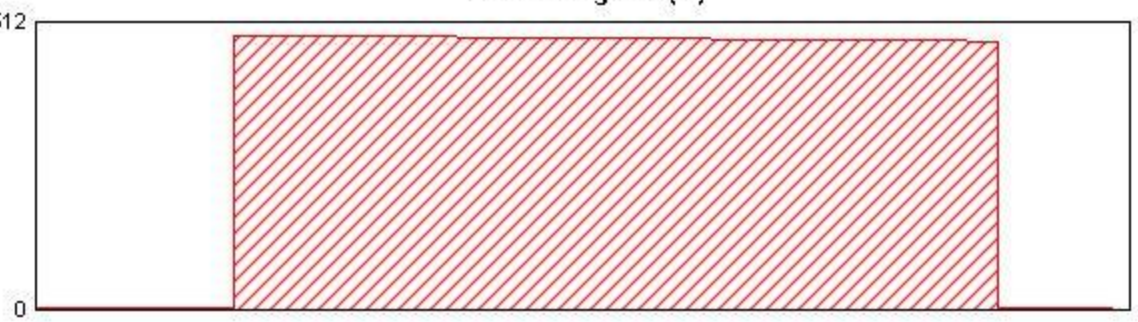


Longitudinal Reinforcement (in2)



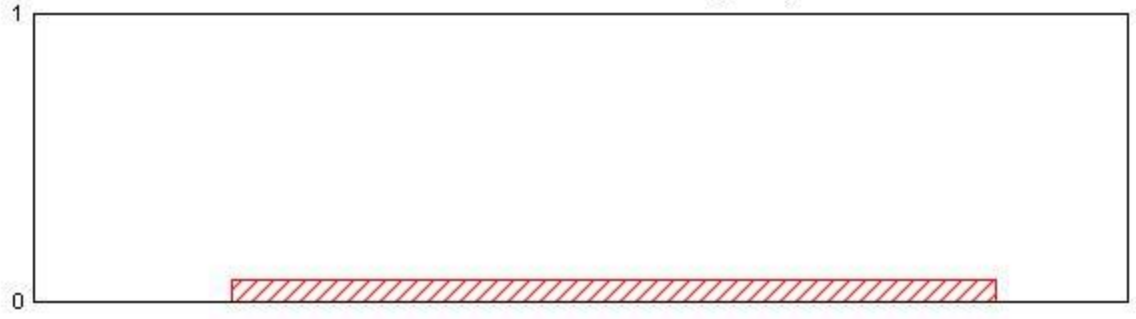
As (top)	0.75	0	0.36
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)

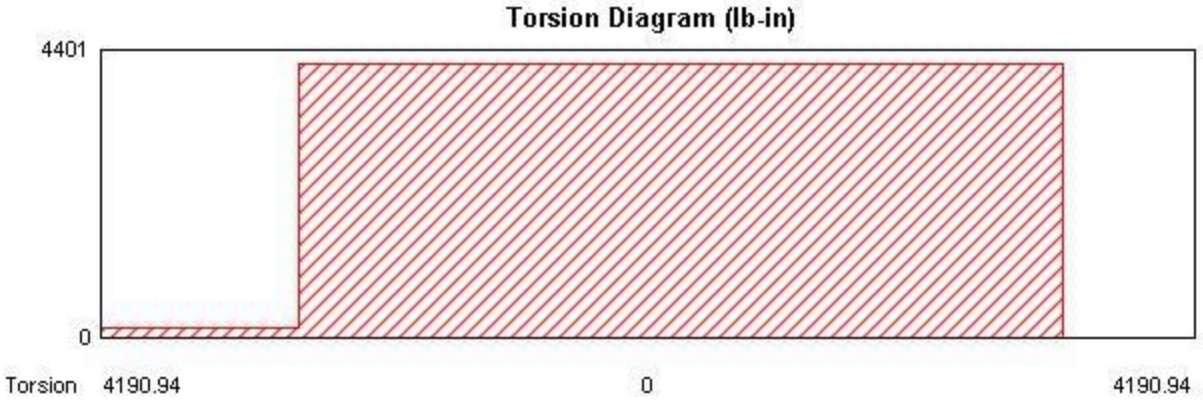


Shear	15725.37	0	15352.11

Transverse Reinforcement (in2/ft)



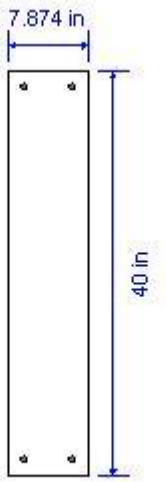
Asv/s reqd	7.874E-002	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
4401
0
Torsion
4190.94
0
4190.94

ACI 318-14 Concrete Beam Design

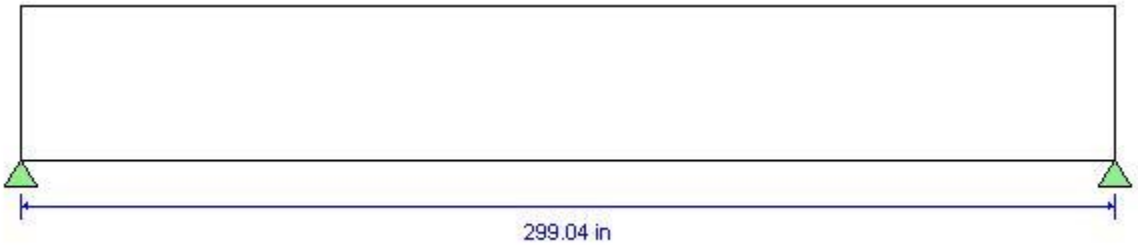
Geometric Properties

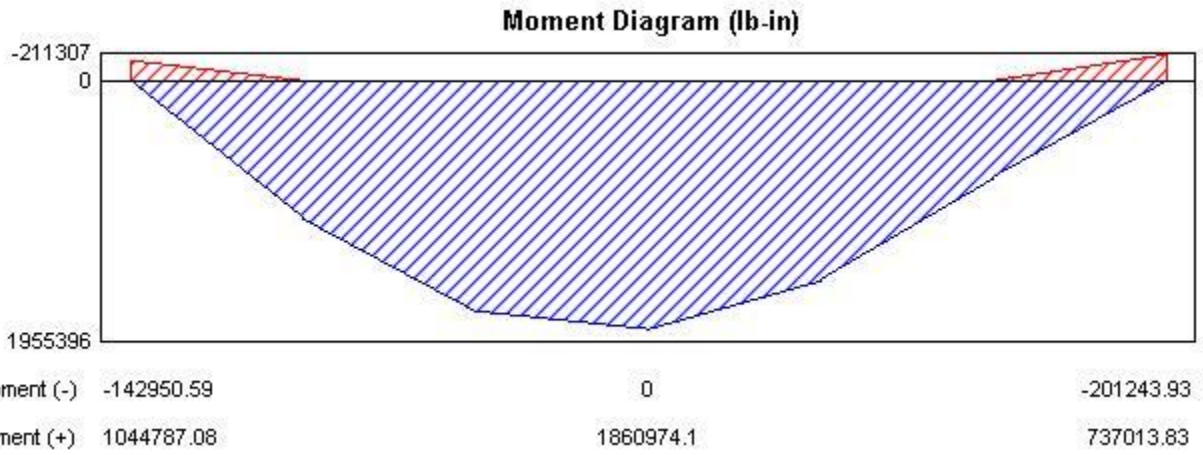
Combination = Overall Envelope
Beam Label = 88
Section Property = B10-8x40-4 ksi
Length = 299.04 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



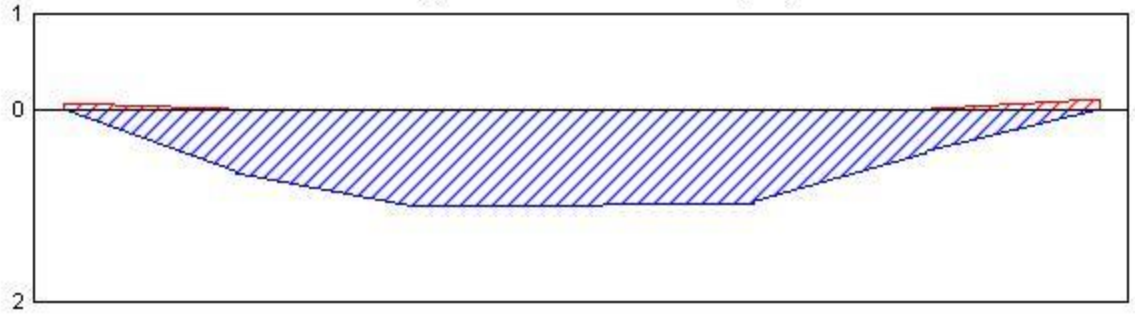
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



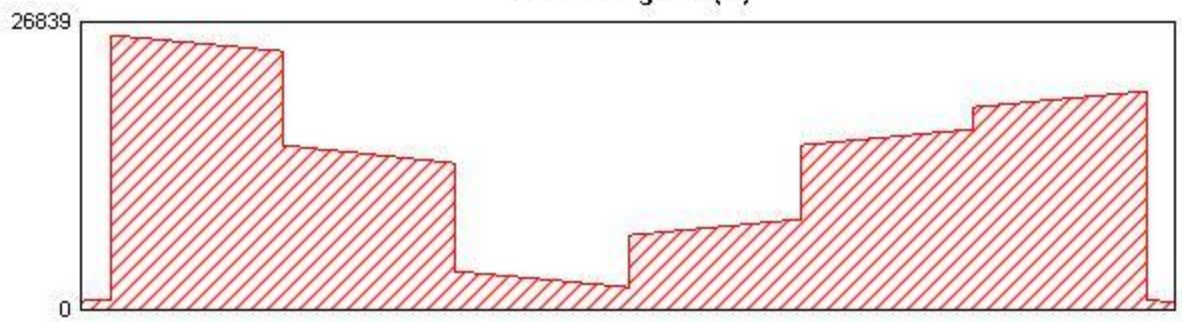


Longitudinal Reinforcement (in2)



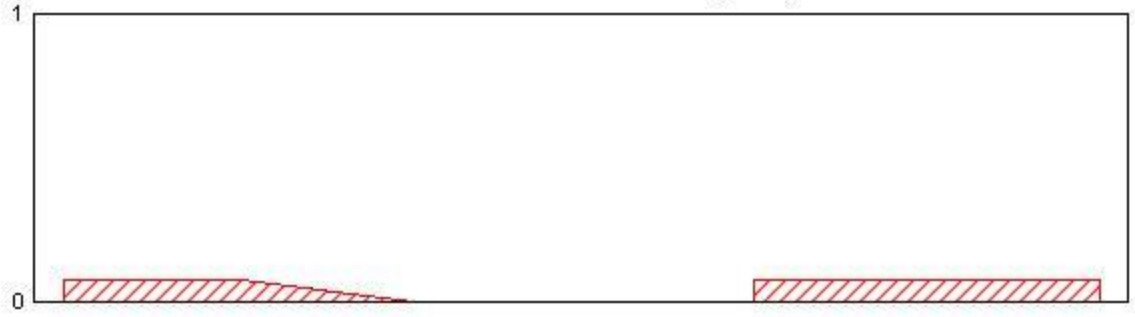
As (top)	4.426E-002	0	7.586E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.5	0.91	0.35
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

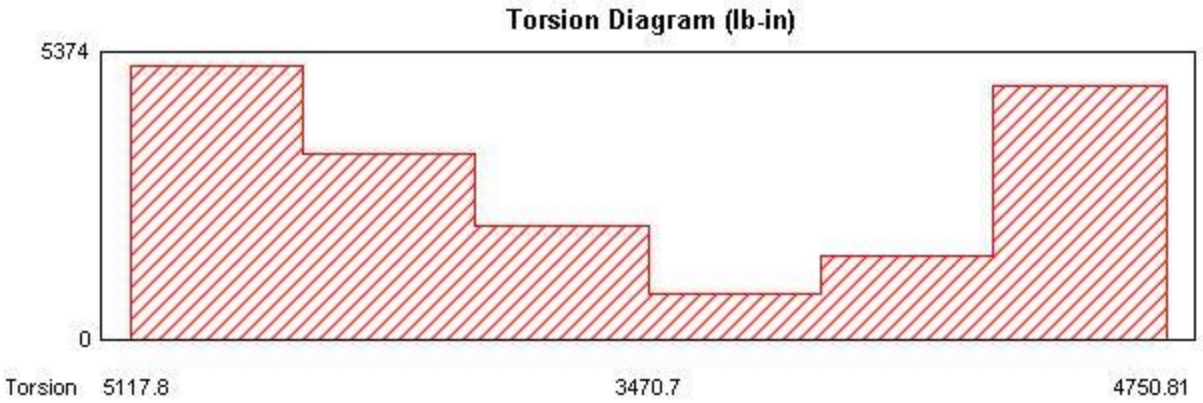


Shear	25560.54	15315.4	20313.92

Transverse Reinforcement (in2/ft)



Asv/s reqd	7.874E-002	7.874E-002	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
5374
0
Torsion
5117.8
3470.7
4750.81

ACI 318-14 Concrete Beam Design

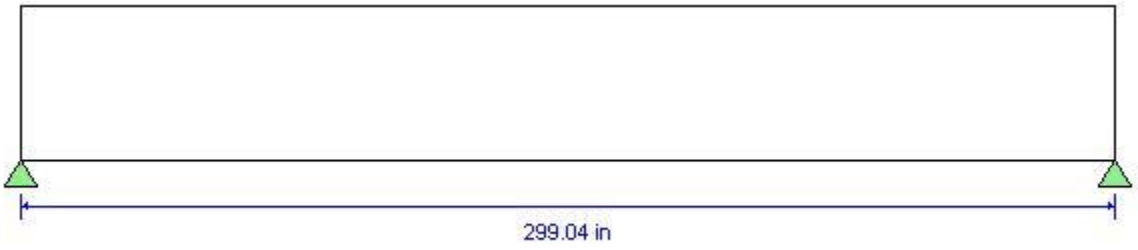
Geometric Properties

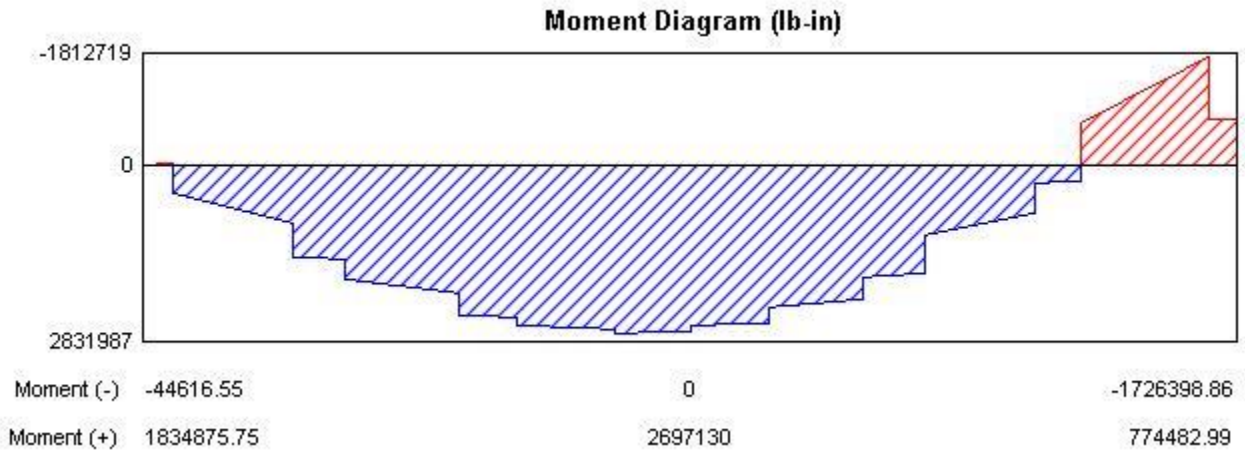
Combination = Overall Envelope
Beam Label = 91
Section Property = B11-8x44-4 ksi
Length = 299.04 in
Section Width = 7.874 in
Section Depth = 44 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



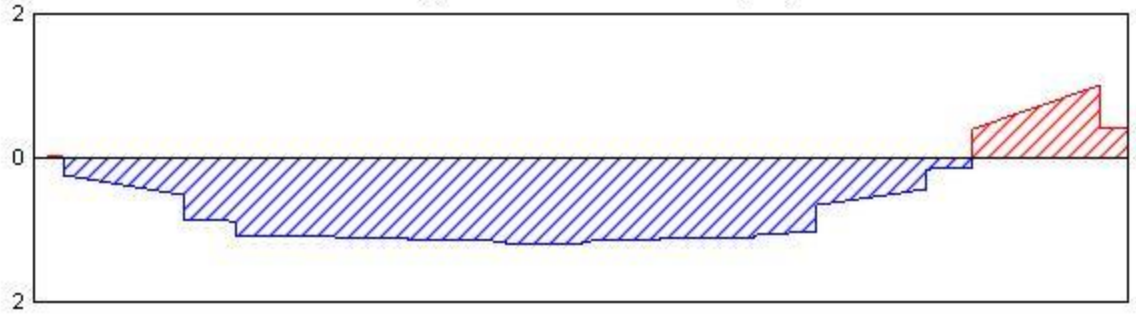
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



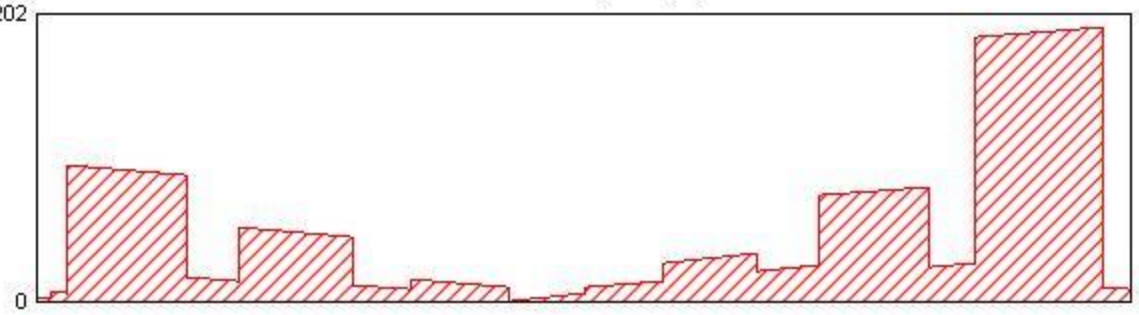


Longitudinal Reinforcement (in2)



As (top)	0	0	0.76
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.81	1.2	0.33
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

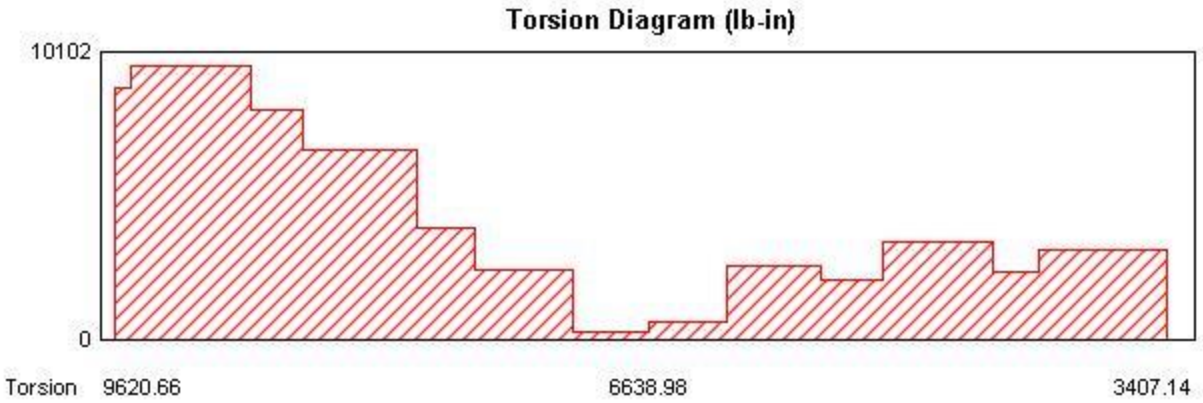


Shear	15232.24	11755.1	30667.7

Transverse Reinforcement (in2/ft)



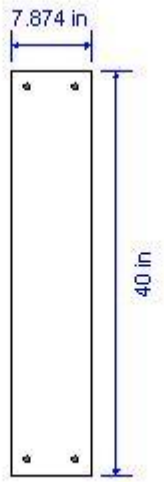
Asv/s reqd	0	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
10102
0
Torsion
9620.66
6638.98
3407.14

ACI 318-14 Concrete Beam Design

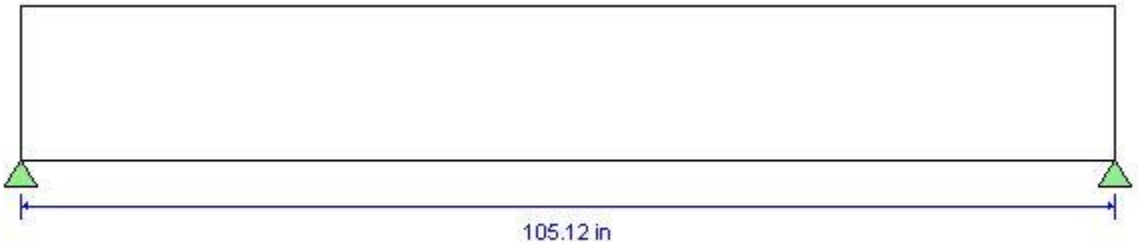
Geometric Properties

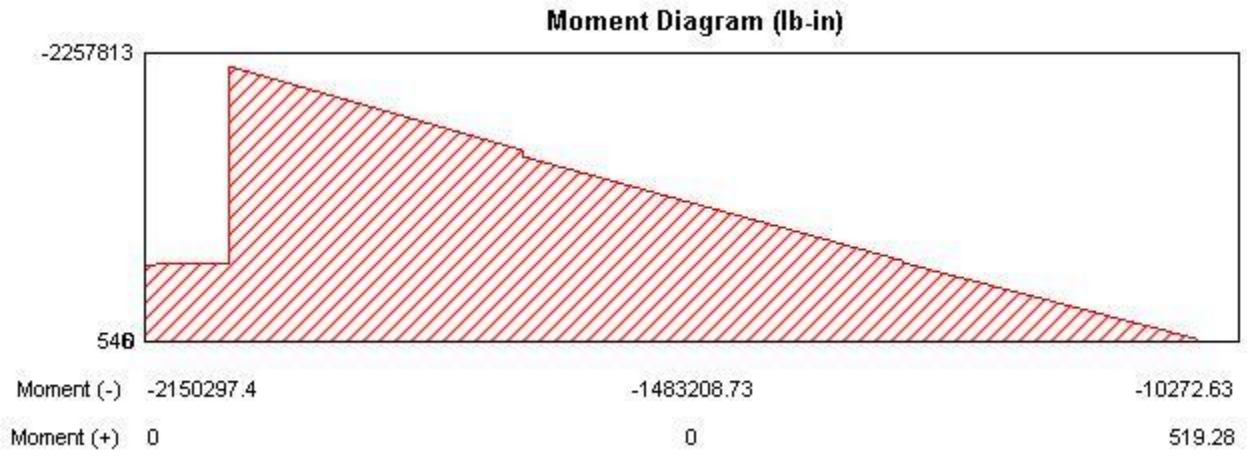
Combination = Overall Envelope
Beam Label = 92
Section Property = B10-8x40-4 ksi
Length = 105.12 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



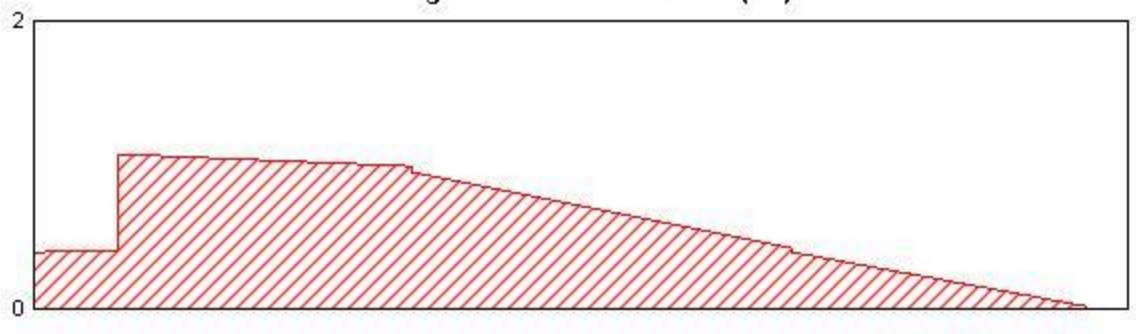
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



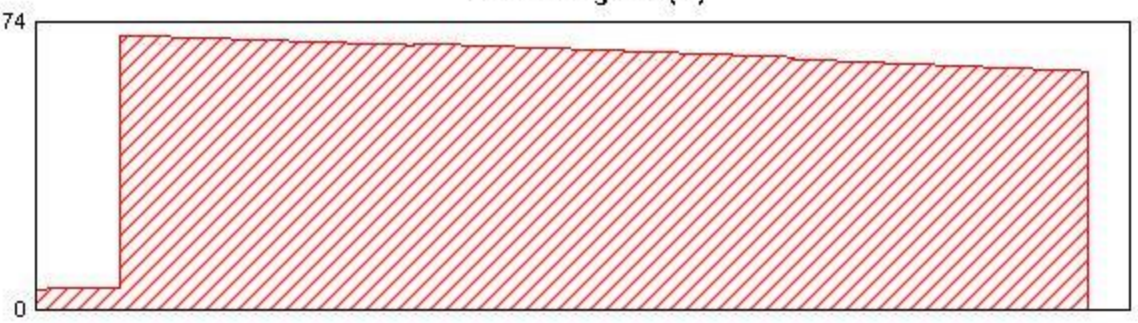


Longitudinal Reinforcement (in2)



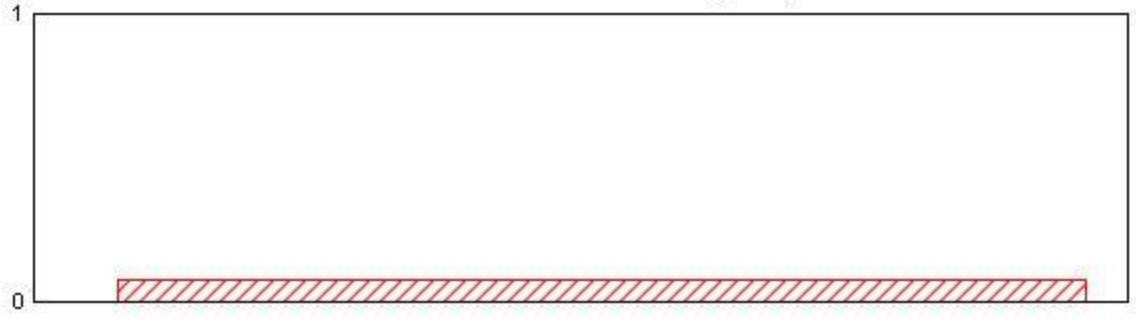
As (top)	1.07	0.73	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1
As (bot)	0	0	0
Combo			Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

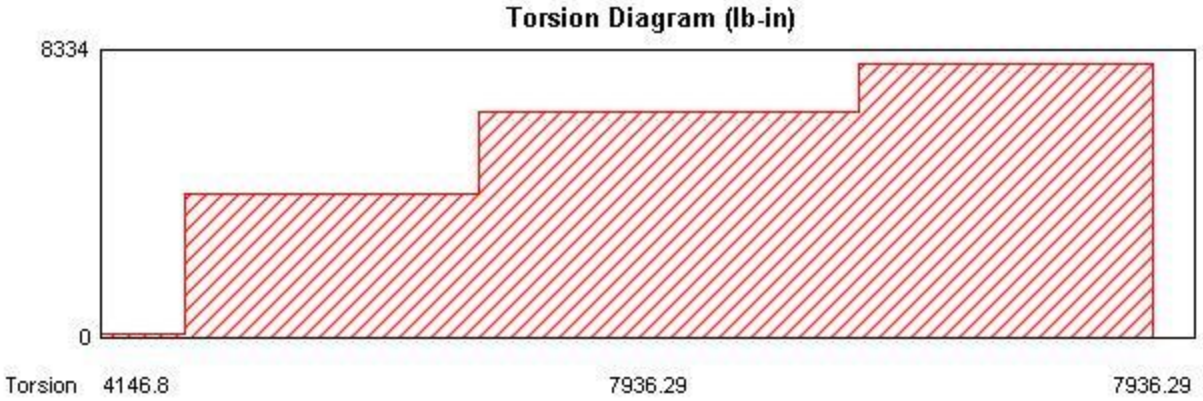


Shear	23974.39	23294.47	20840.91

Transverse Reinforcement (in2/ft)



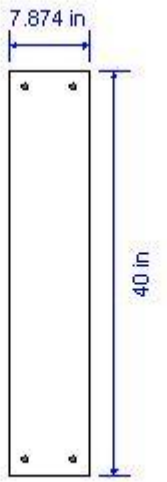
Asv/s reqd	7.874E-002	7.874E-002	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
8334
0
Torsion
4146.8
7936.29
7936.29

ACI 318-14 Concrete Beam Design

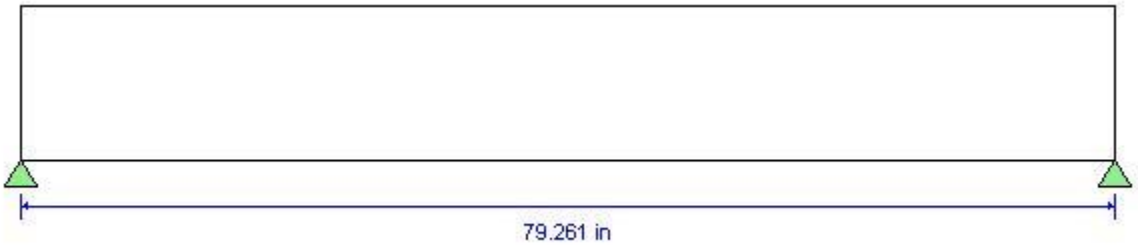
Geometric Properties

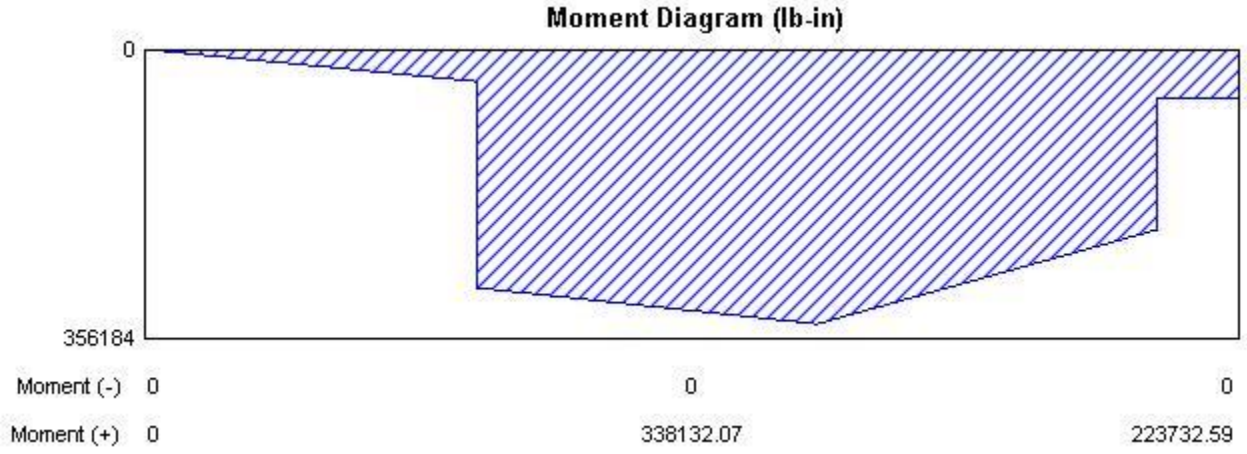
Combination = Overall Envelope
Beam Label = 93
Section Property = B10-8x40-4 ksi
Length = 79.261 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



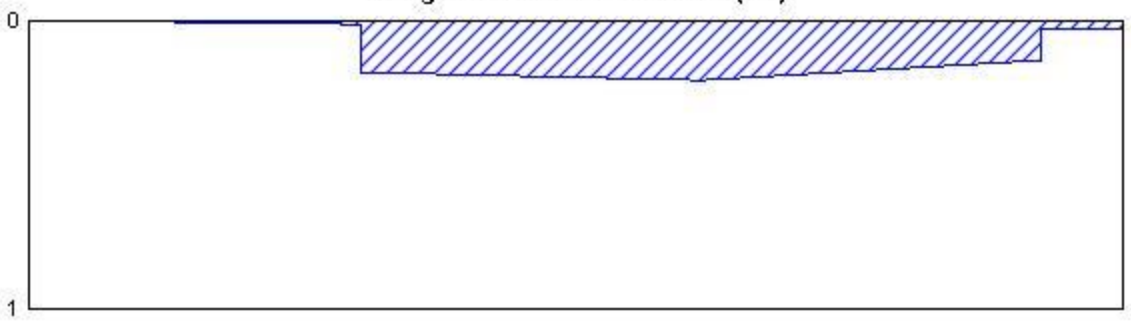
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



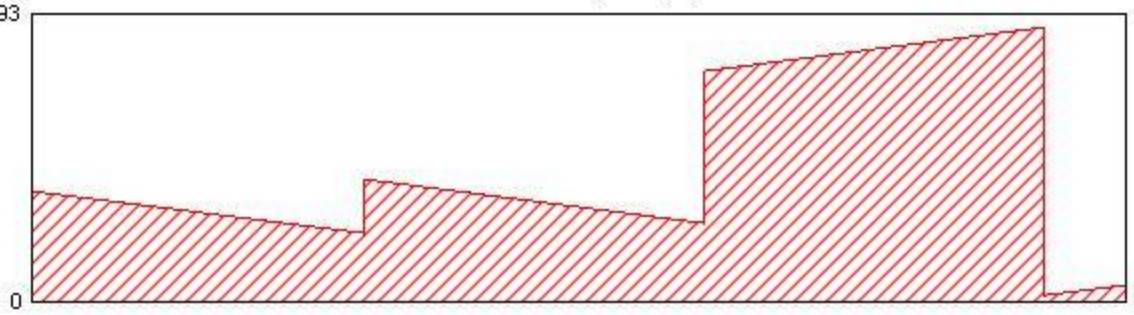


Longitudinal Reinforcement (in2)



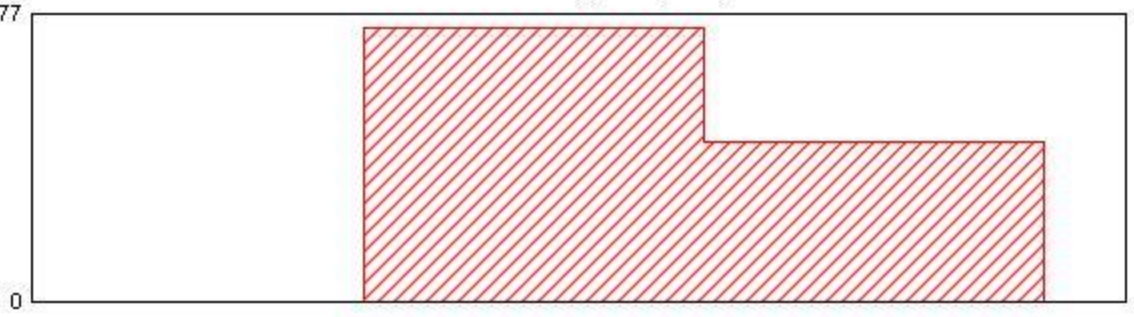
As (top)	0	0	0
Combo			
As (bot)	0	0.16	0.1
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	2022.12	4230.9	5040.54

Torsion Diagram (lb-in)

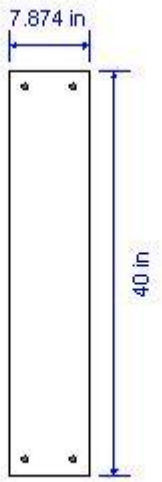


Torsion	0	10454.14	6062.28

ACI 318-14 Concrete Beam Design

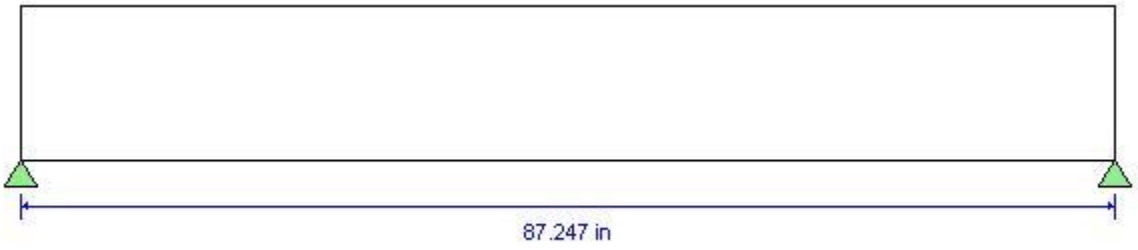
Geometric Properties

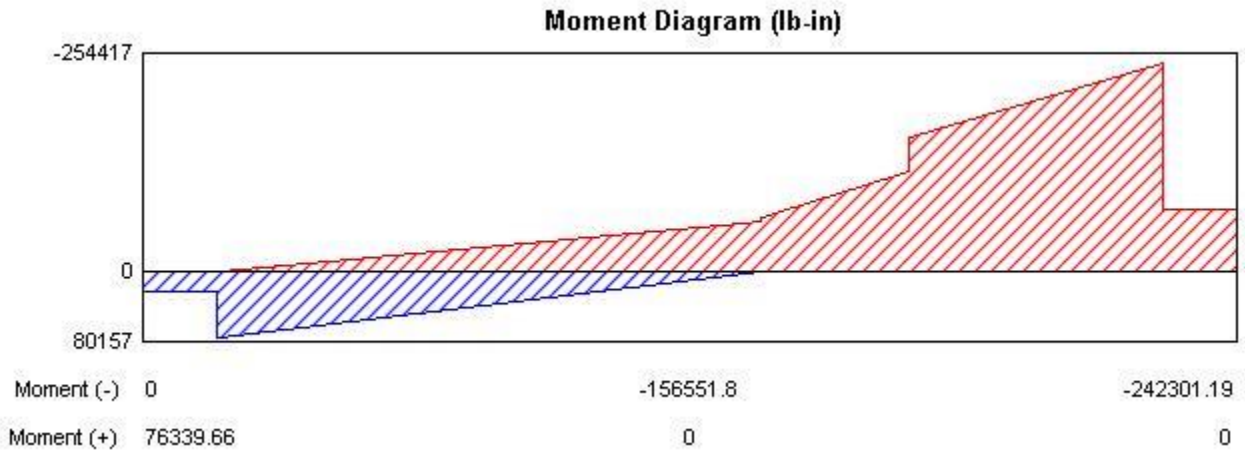
Combination = Overall Envelope
Beam Label = 94
Section Property = B12-12x40- 4 ksi
Length = 87.247 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



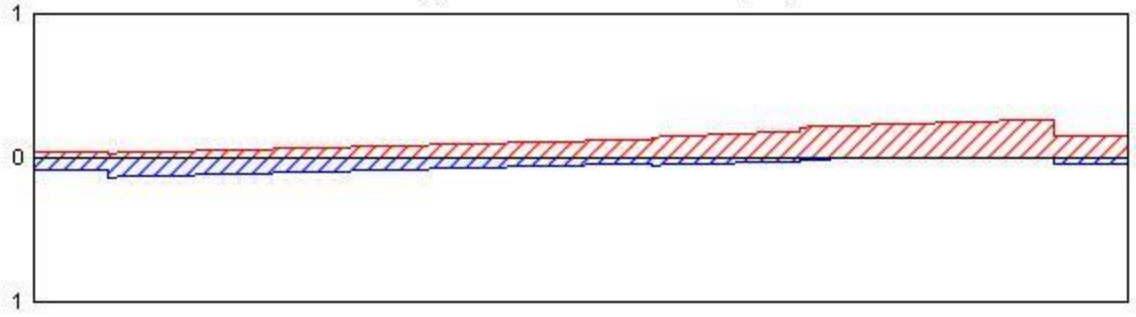
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



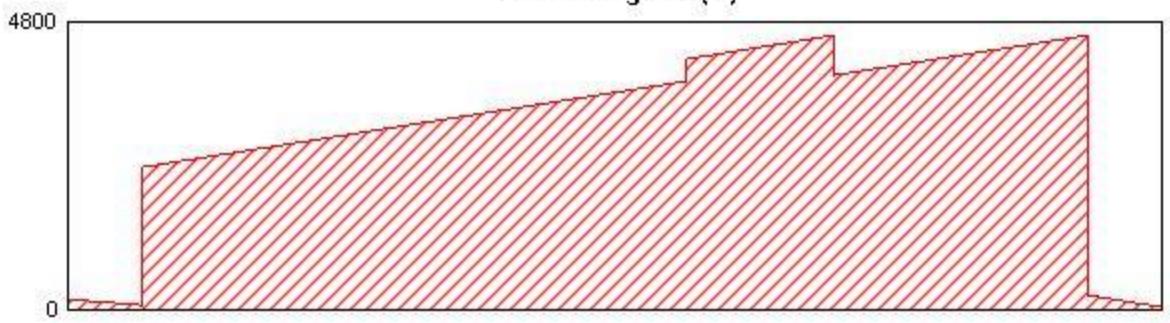


Longitudinal Reinforcement (in2)



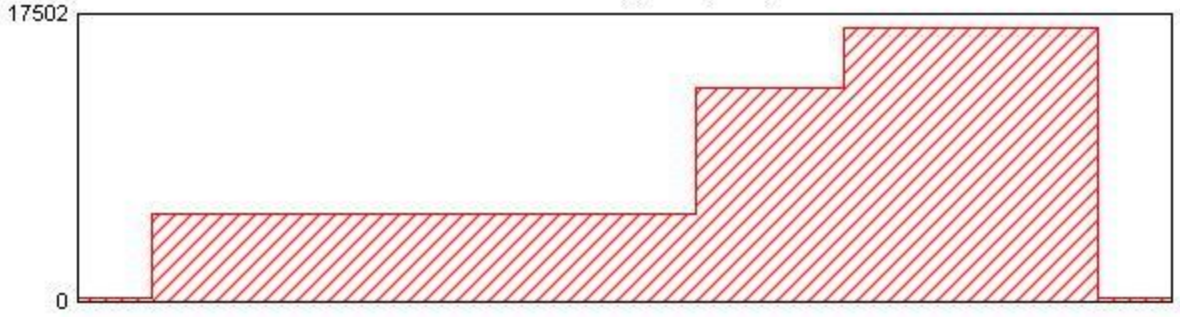
As (top)	3.615E-002	0.16	0.2
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.1	5.022E-002	3.84E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	2364.7	4556.56	4570.78

Torsion Diagram (lb-in)

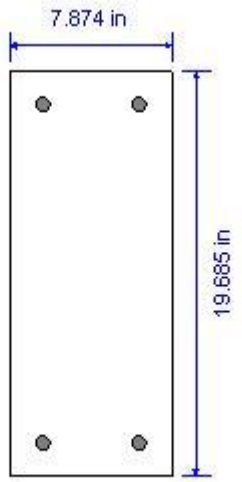


Torsion	5360.48	16667.82	16667.82

ACI 318-14 Concrete Beam Design

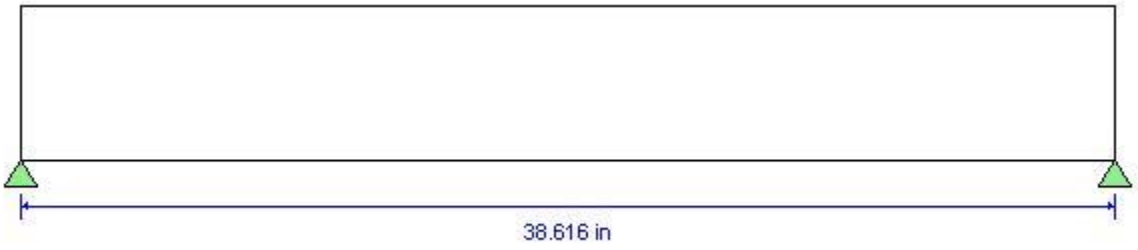
Geometric Properties

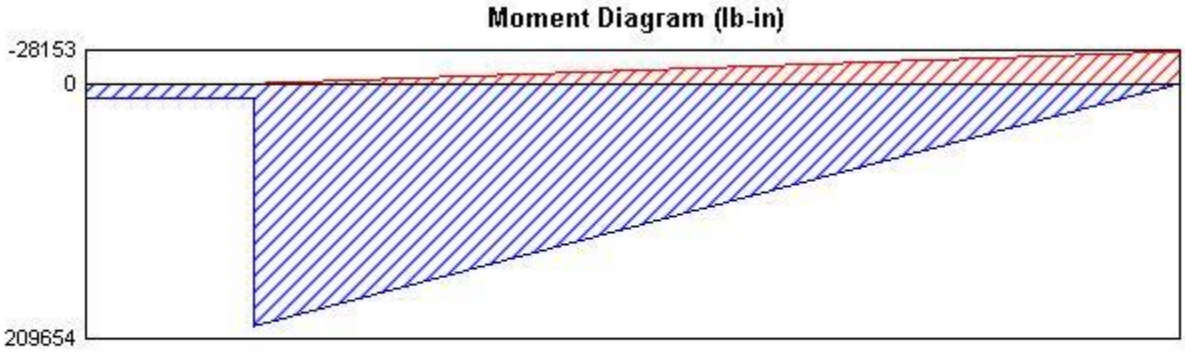
Combination = Overall Envelope
Beam Label = 95
Section Property = B3-8x20- 4 ksi
Length = 38.616 in
Section Width = 7.874 in
Section Depth = 19.685 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

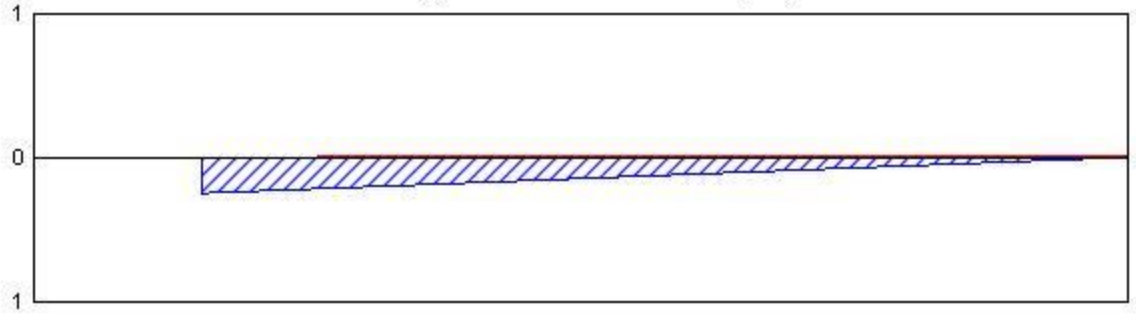
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2





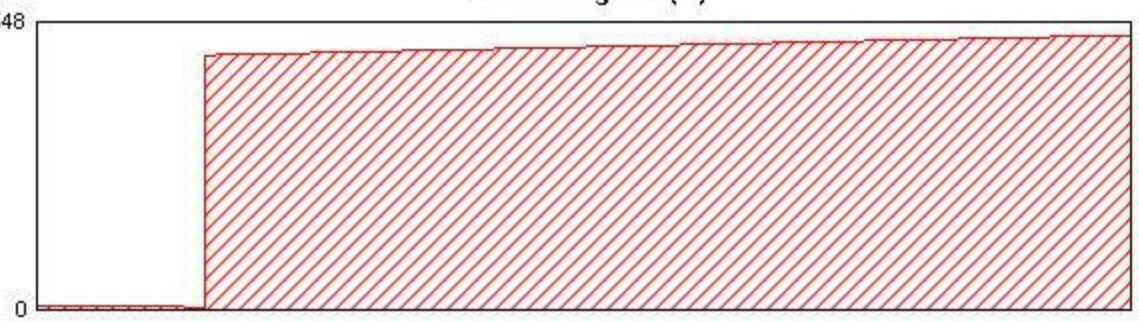
Moment (-)	0	0	-26812.2
Moment (+)	199670.07	0	0

Longitudinal Reinforcement (in2)



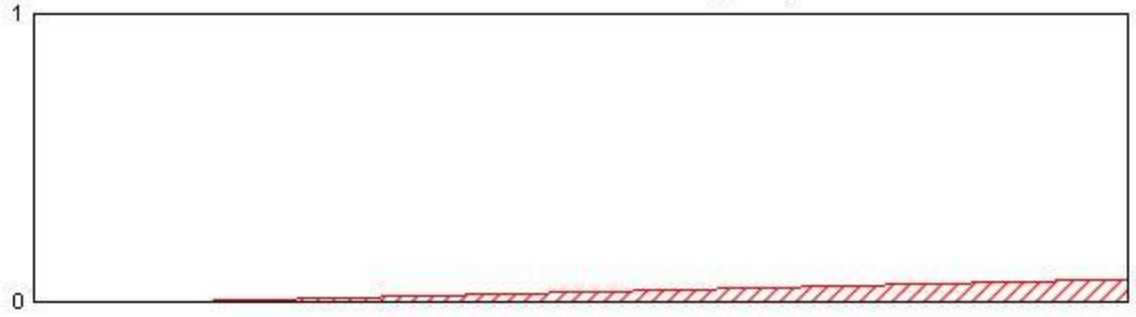
As (top)	0	0	0
Combo			Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.19	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	6659.77	0	7187.91

Transverse Reinforcement (in2/ft)

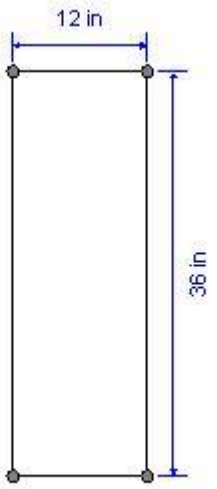


Asv/s reqd	0	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr

ACI 318-14 Concrete Beam Design

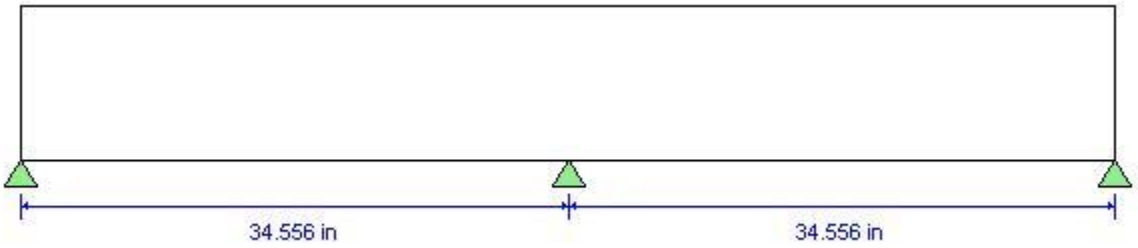
Geometric Properties

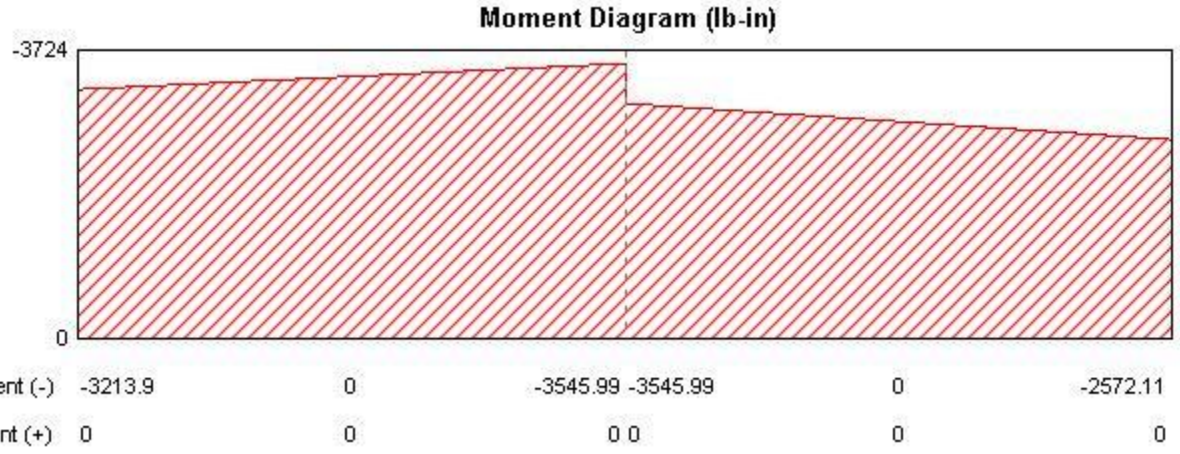
Combination = Overall Envelope
Beam Label = 96
Section Property =
Length = 69.112 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

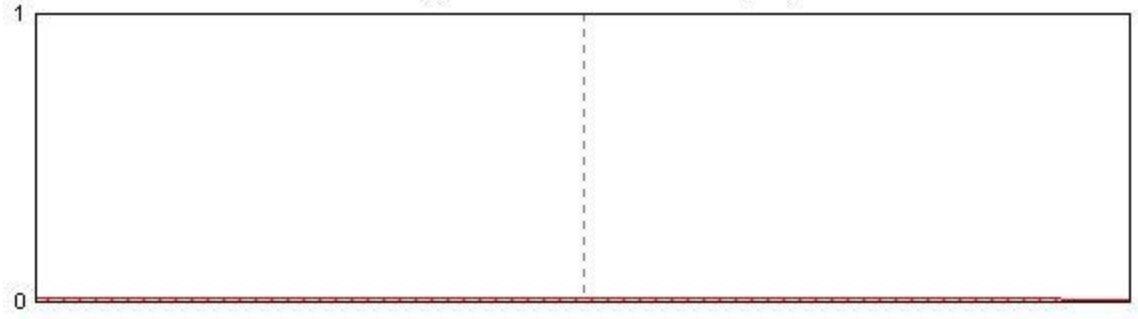
Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





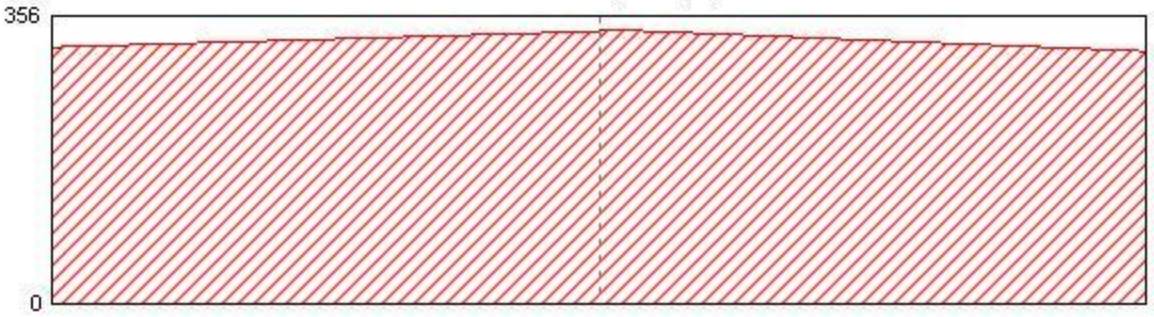
Moment (-)	-3213.9	0	-3545.99 -3545.99	0	-2572.11
Moment (+)	0	0	0 0	0	0

Longitudinal Reinforcement (in2)



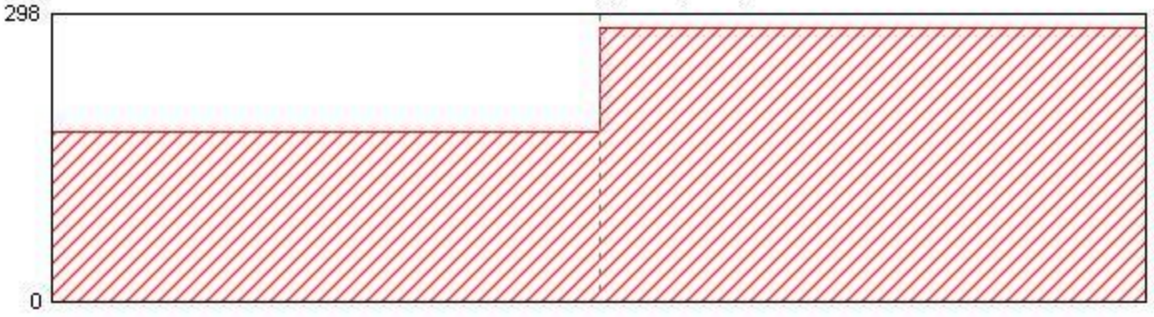
As (top)	9.133E-003	0	9.505E-003	9.505E-003	0	7.776E-003
Combo	DCONU1		DCONU1	DCONU1		DCONU1
As (bot)	0	0	0	0	0	0
Combo						

Shear Diagram (lb)



Shear	315.85	0	338.29	338.29	0	312.64

Torsion Diagram (lb-in)

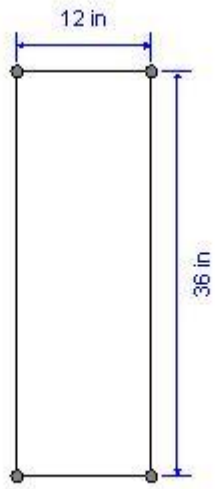


Torsion	176.7	0	283.02	283.02	0	283.02

ACI 318-14 Concrete Beam Design

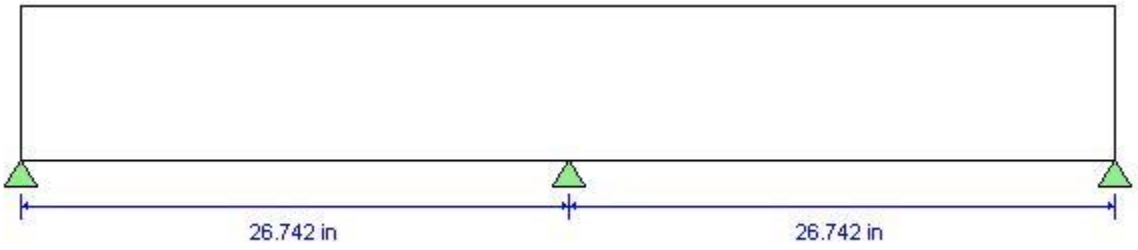
Geometric Properties

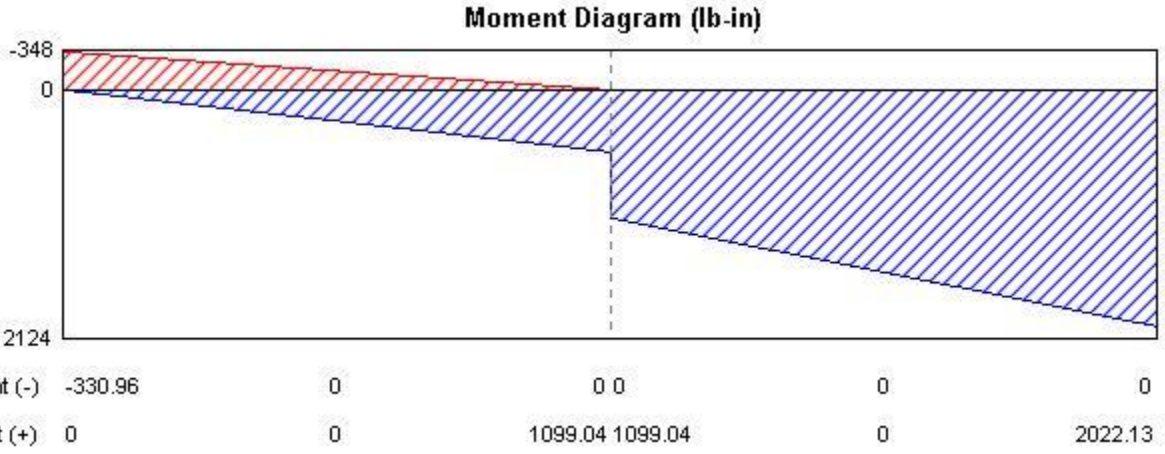
Combination = Overall Envelope
Beam Label = 97
Section Property =
Length = 53.484 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



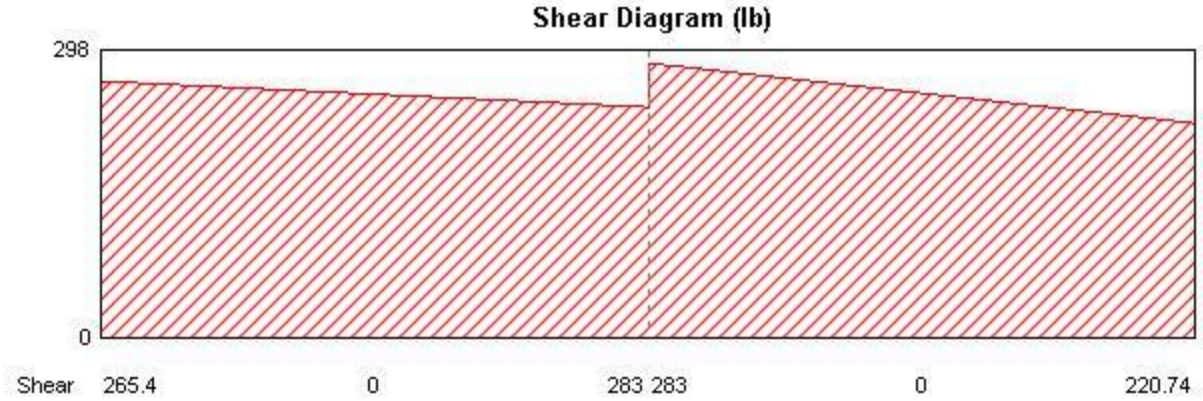
Material Properties

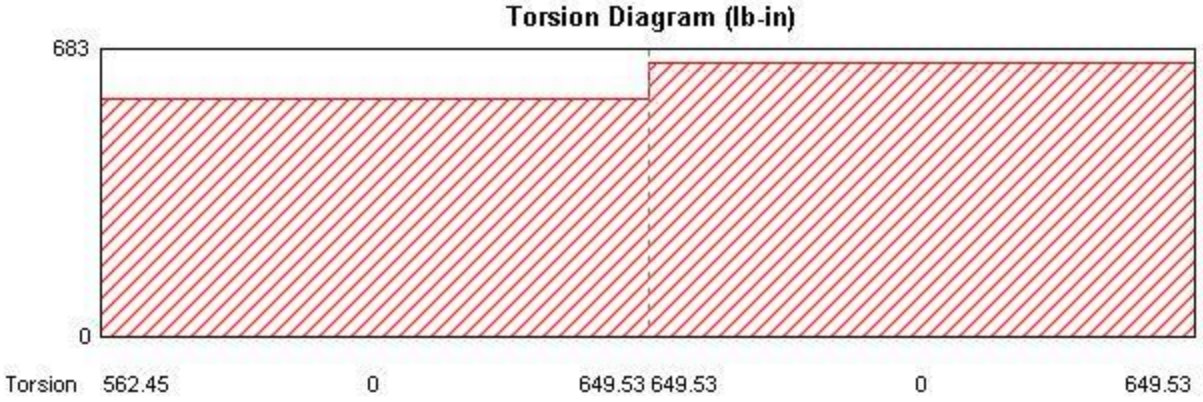
Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





Moment (-)	-330.96	0	0 0	0	0
Moment (+)	0	0	1099.04 1099.04	0	2022.13

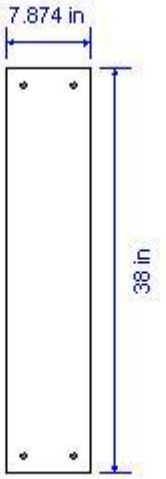

Shear Diagram (lb)
298
0
Shear 265.4 0 283 283 0 220.74


Torsion Diagram (lb-in)
683
0
Torsion 562.45 0 649.53 649.53 0 649.53

ACI 318-14 Concrete Beam Design

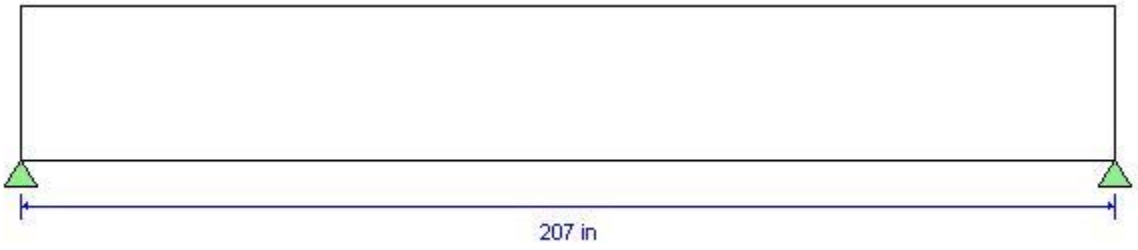
Geometric Properties

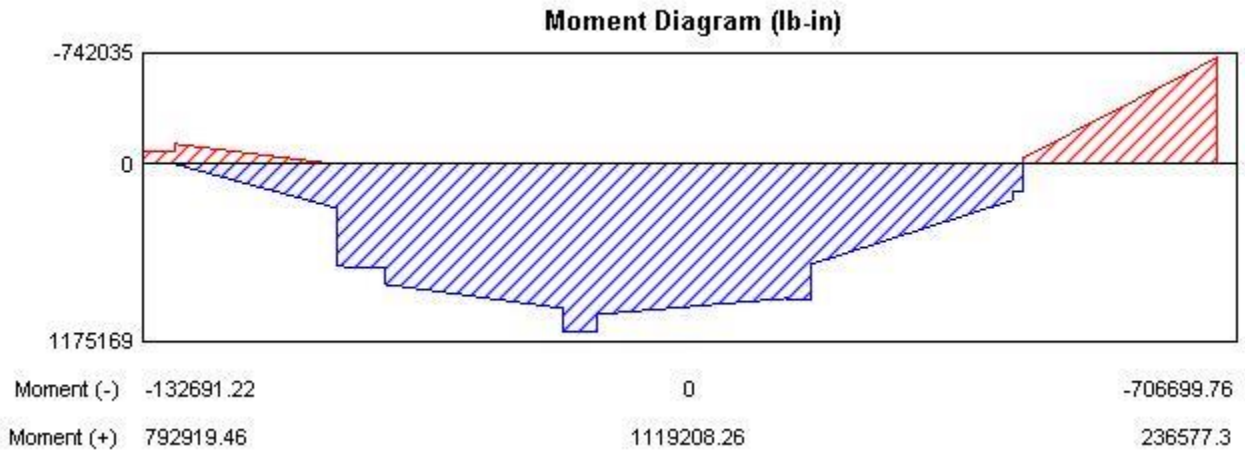
Combination = Overall Envelope
Beam Label = 98
Section Property = B13-8x38-4 ksi
Length = 207 in
Section Width = 7.874 in
Section Depth = 38 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



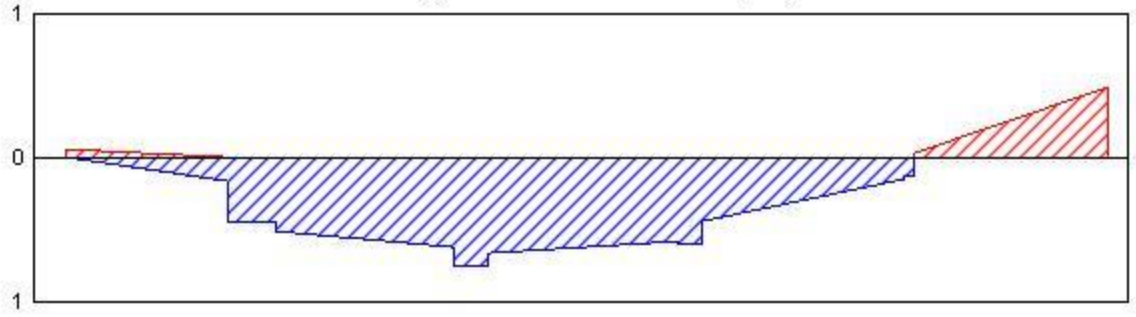
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



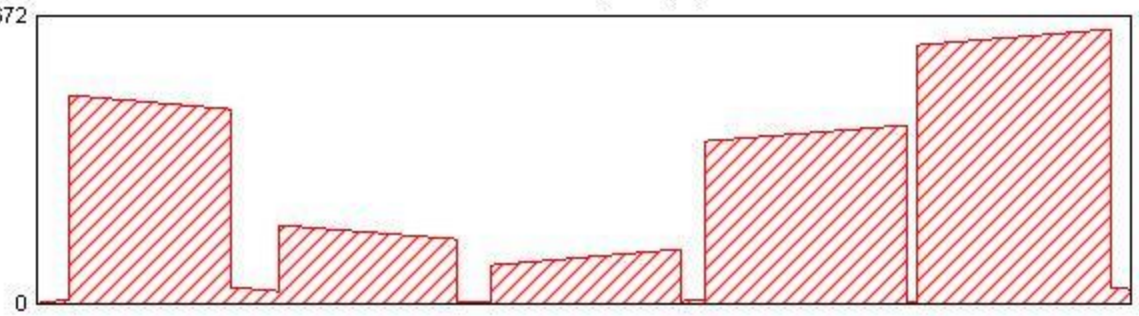


Longitudinal Reinforcement (in2)



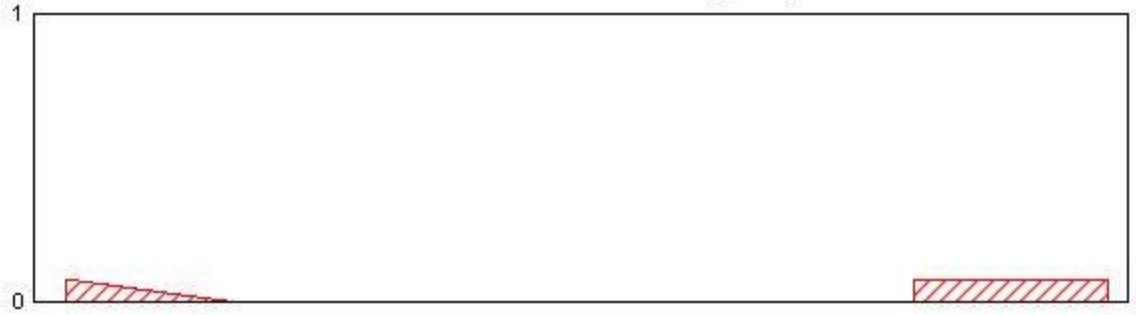
As (top)	4.517E-002	0	0.36
Combo	DCONU1		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.39	0.56	0.11
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1

Shear Diagram (lb)

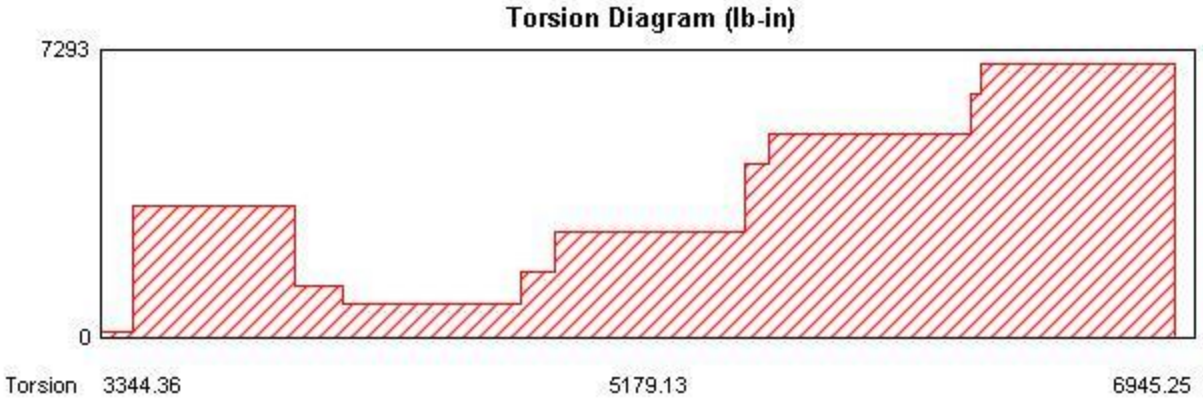


Shear	14275.79	11025.88	18734.3

Transverse Reinforcement (in2/ft)



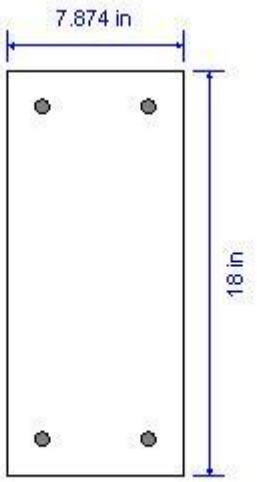
Asv/s reqd	7.874E-002	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
7293
0
Torsion
3344.36
5179.13
6945.25

ACI 318-14 Concrete Beam Design

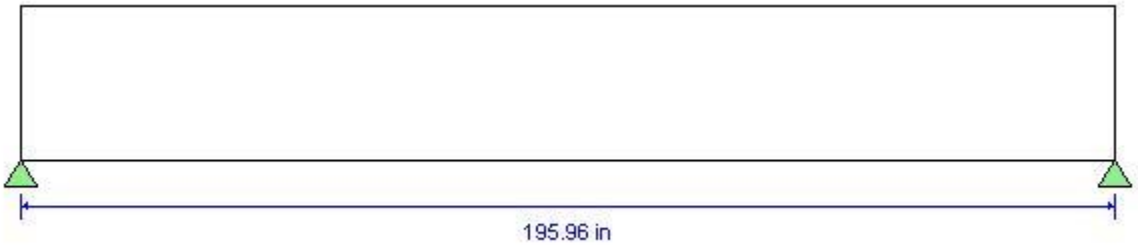
Geometric Properties

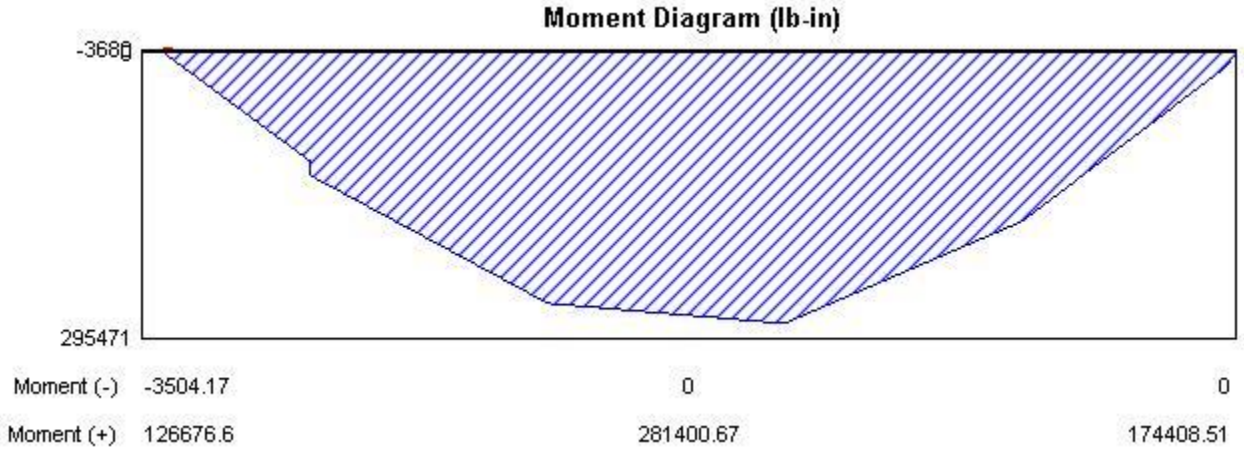
Combination = Overall Envelope
Beam Label = 1
Section Property = B7-8x18-4 ksi
Length = 195.96 in
Section Width = 7.874 in
Section Depth = 18 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



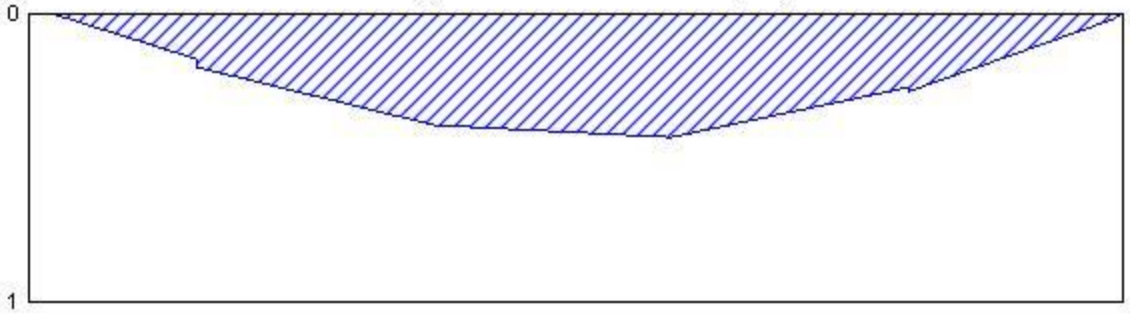
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



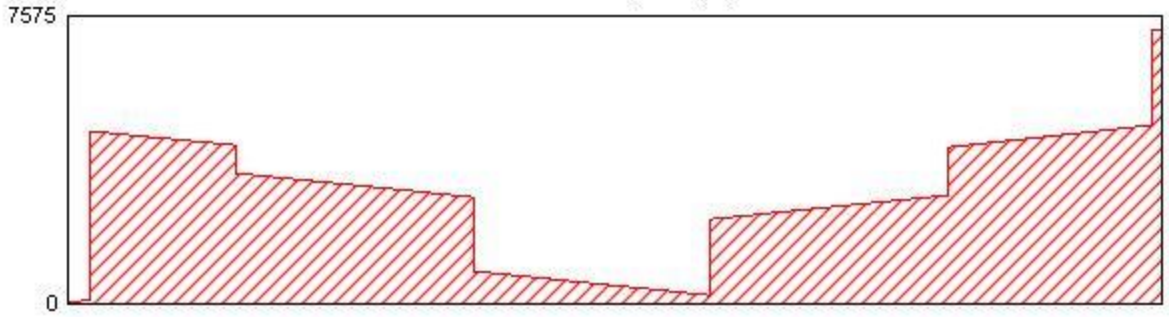


Longitudinal Reinforcement (in2)



As (top)	0	0	0
Combo	DCONU1		
As (bot)	0.14	0.32	0.2
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

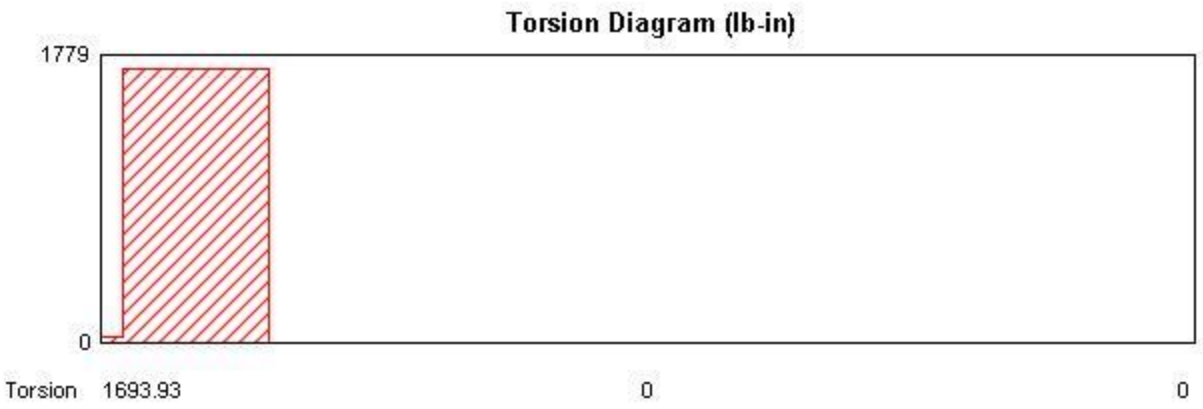


Shear	4526.63	2807.75	7214.13

Transverse Reinforcement (in2/ft)



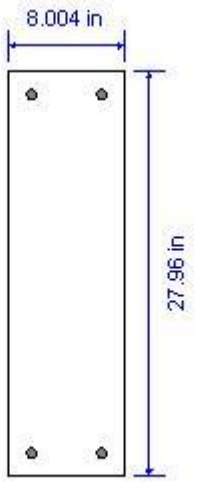
Asv/s reqd	0	0	7.874E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
1779
0
Torsion
1693.93
0
0

ACI 318-14 Concrete Beam Design

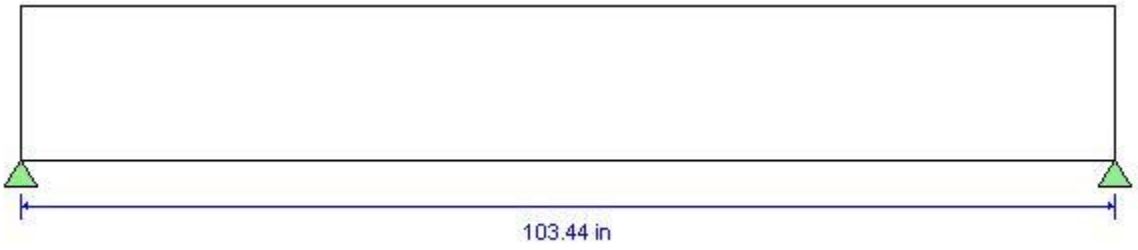
Geometric Properties

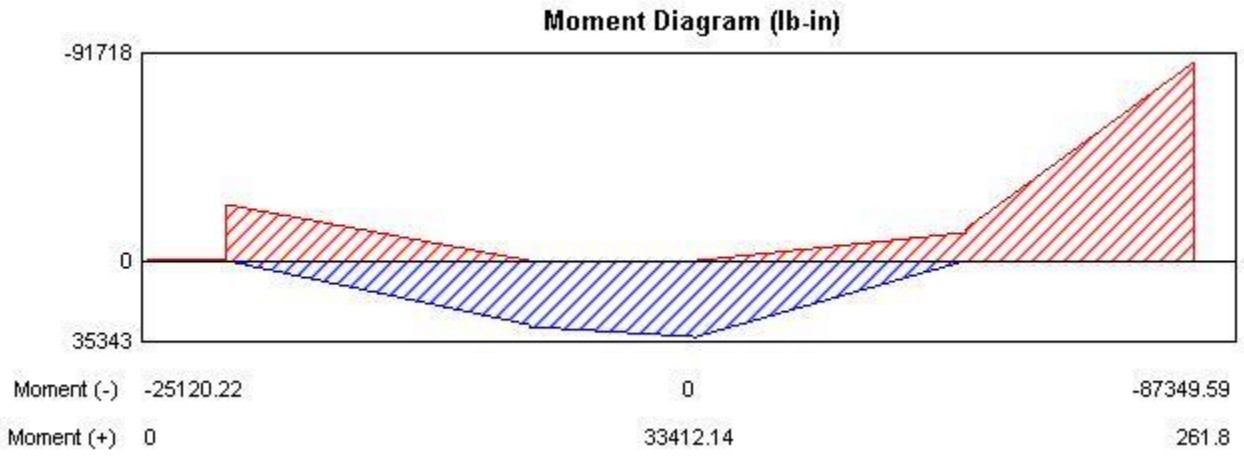
Combination = Overall Envelope
Beam Label = 3
Section Property = B8-8x28-4 ksi
Length = 103.44 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



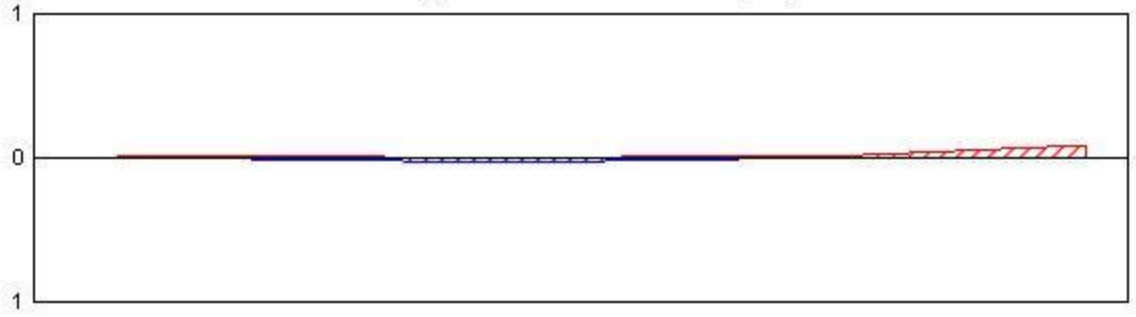
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



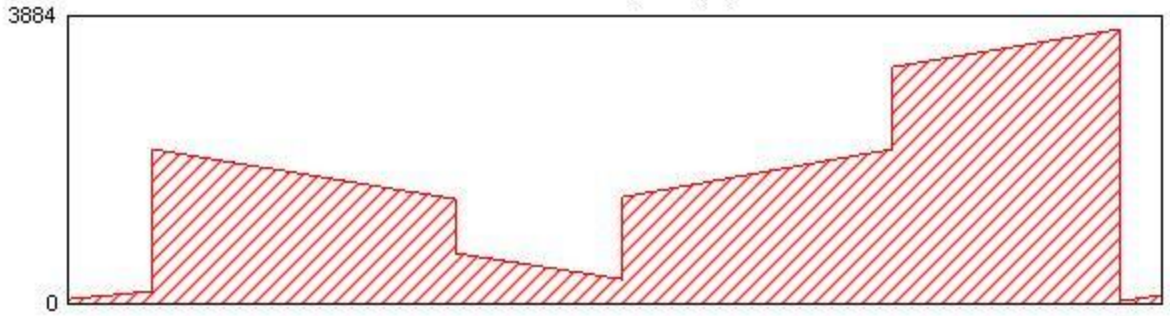


Longitudinal Reinforcement (in2)



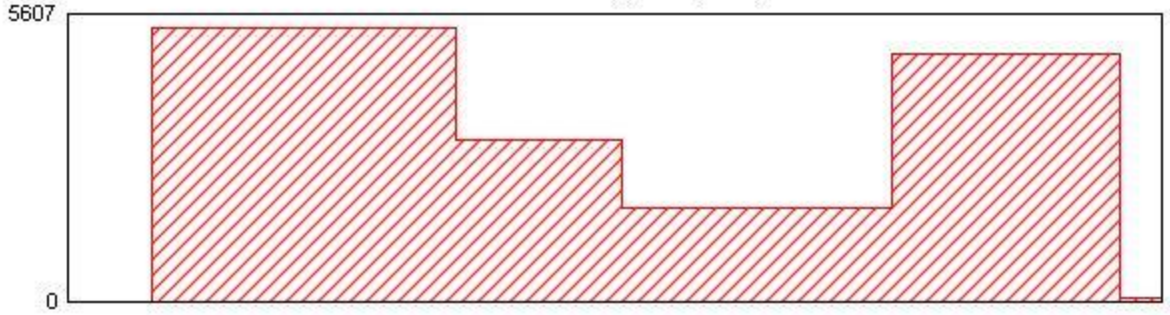
As (top)	1.416E-002	0	6.104E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	2.555E-002	0
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	2078.87	1434.26	3698.27

Torsion Diagram (lb-in)

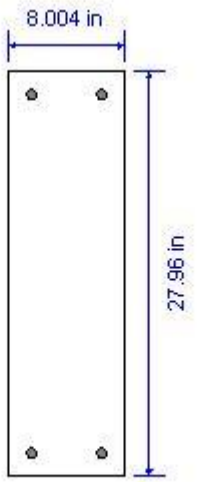


Torsion	5339.53	5339.53	4825.01

ACI 318-14 Concrete Beam Design

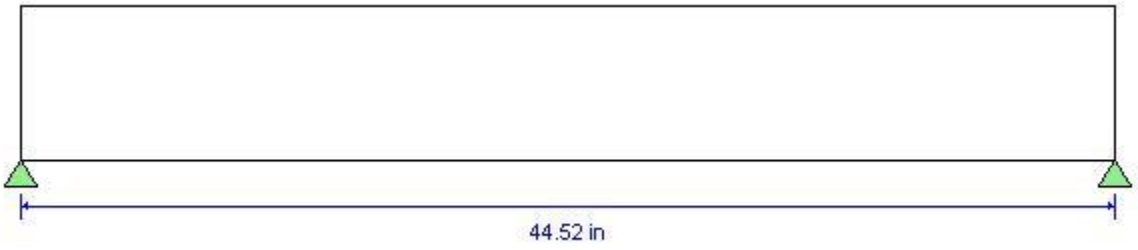
Geometric Properties

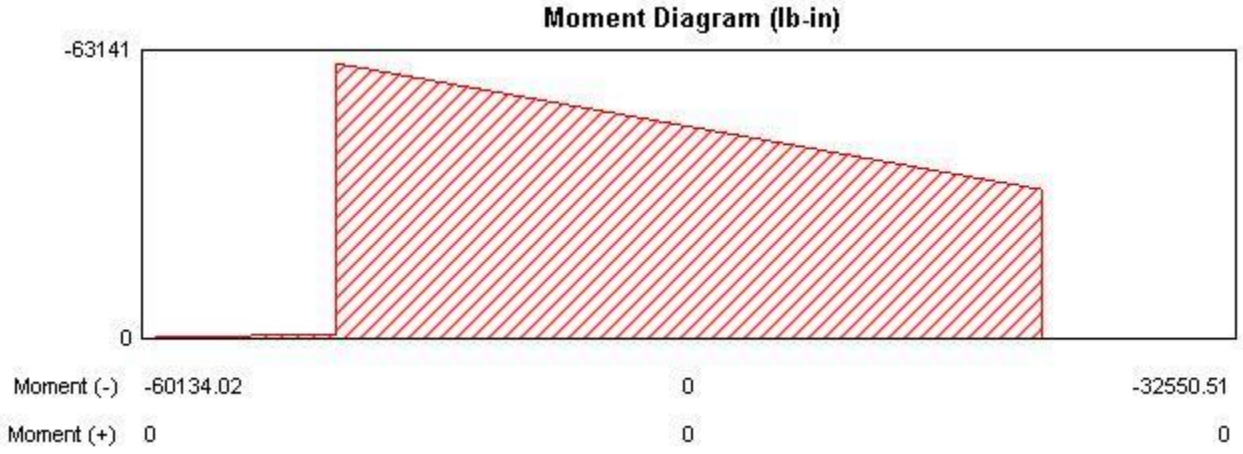
Combination = Overall Envelope
Beam Label = 4
Section Property = B8-8x28-4 ksi
Length = 44.52 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



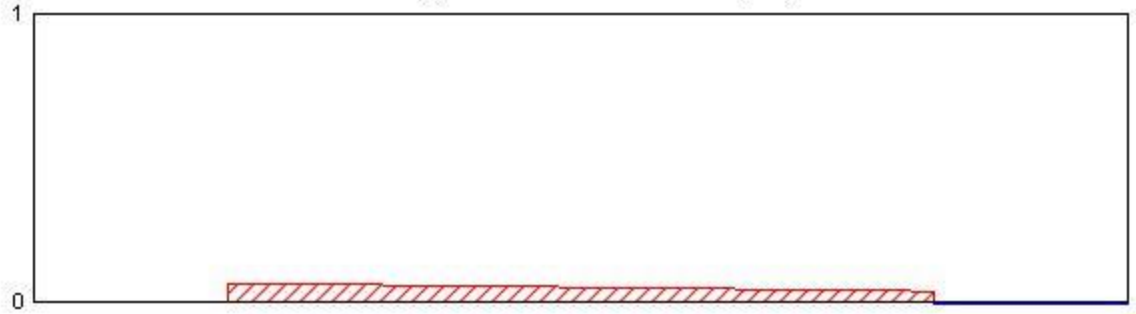
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



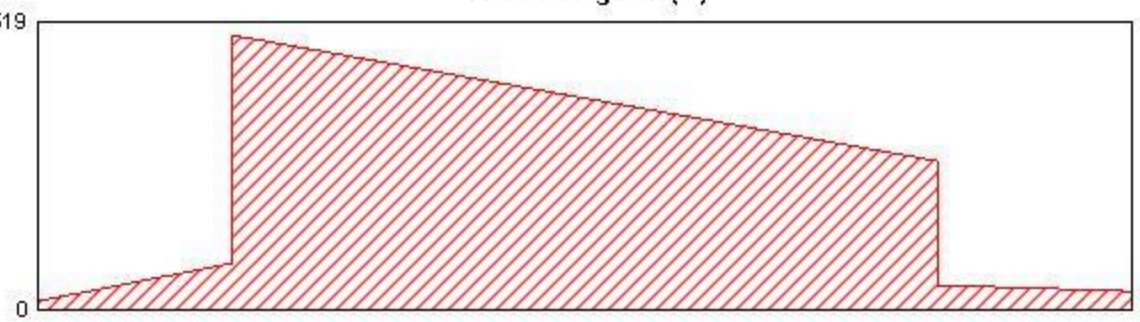


Longitudinal Reinforcement (in2)



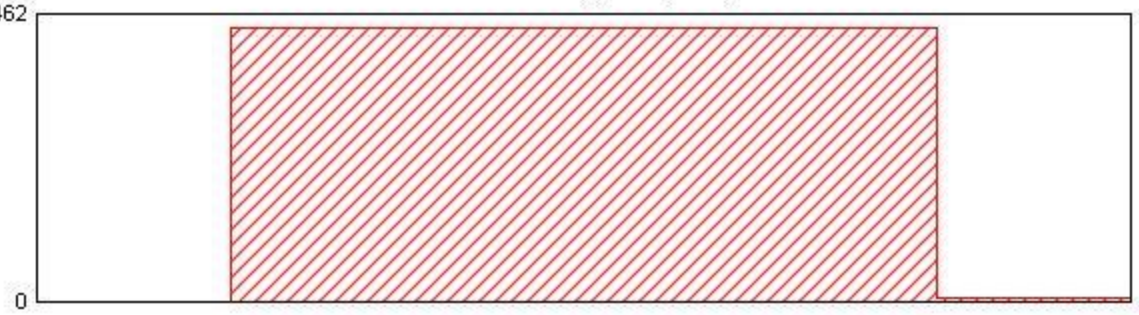
As (top)	4.499E-002	0	2.545E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		DCONU1
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	1446.36	0	775.64

Torsion Diagram (lb-in)

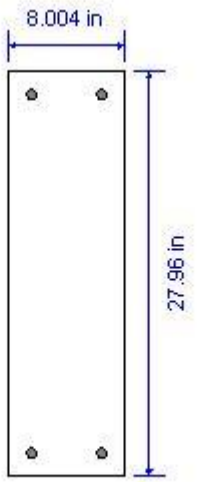


Torsion	10916.19	0	10916.19

ACI 318-14 Concrete Beam Design

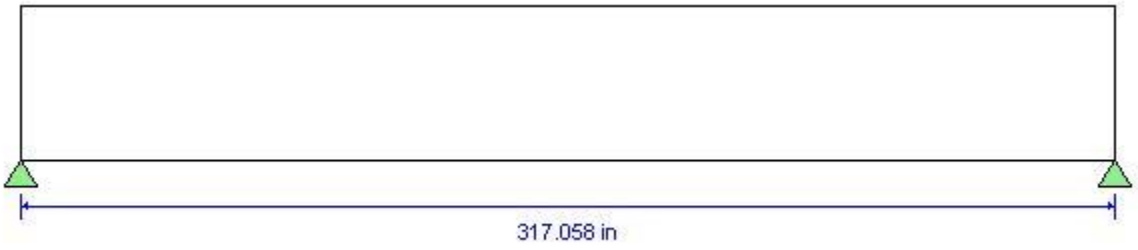
Geometric Properties

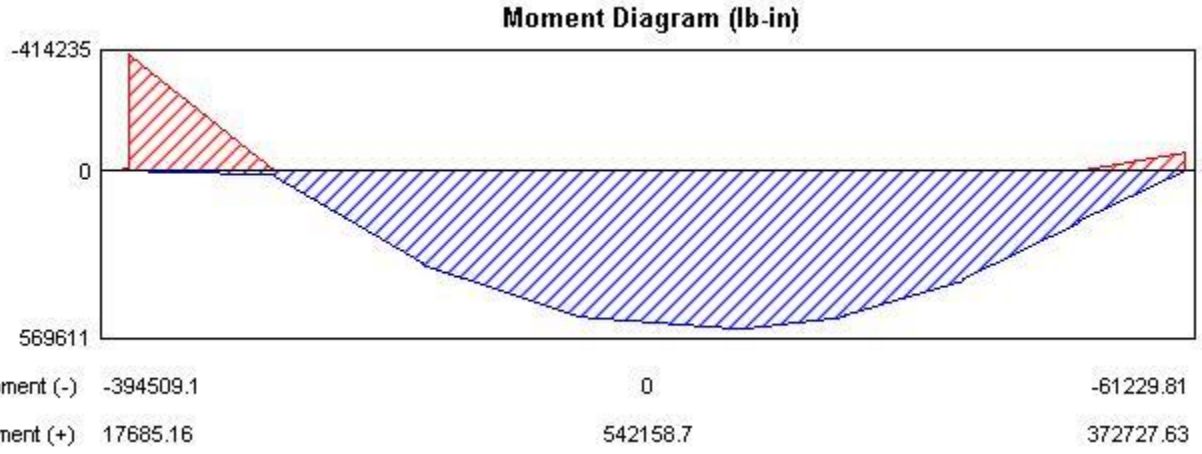
Combination = Overall Envelope
Beam Label = 58
Section Property = B8-8x28-4 ksi
Length = 317.058 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



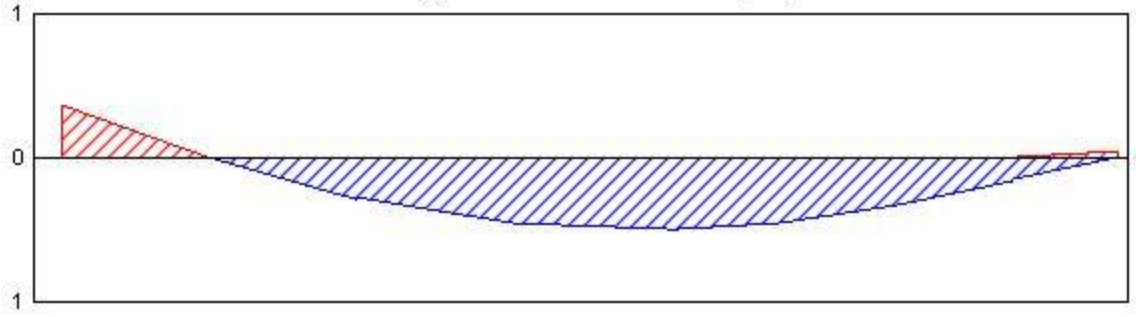
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



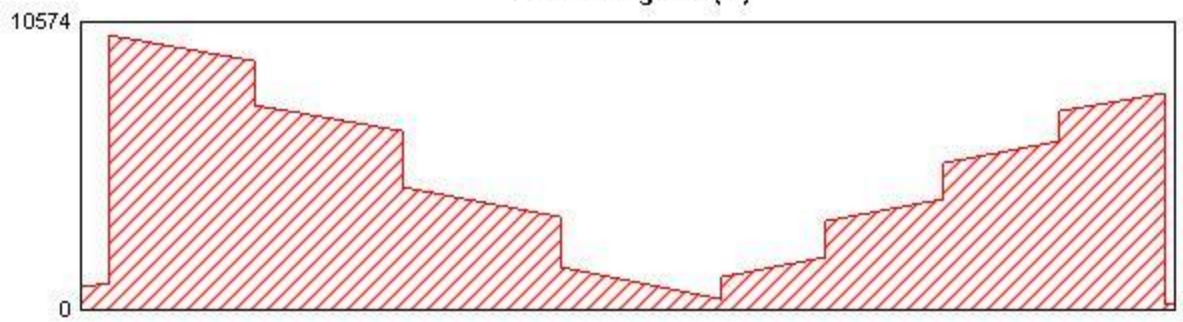


Longitudinal Reinforcement (in2)



As (top)	0.27	0	3.485E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0.38	0.26
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

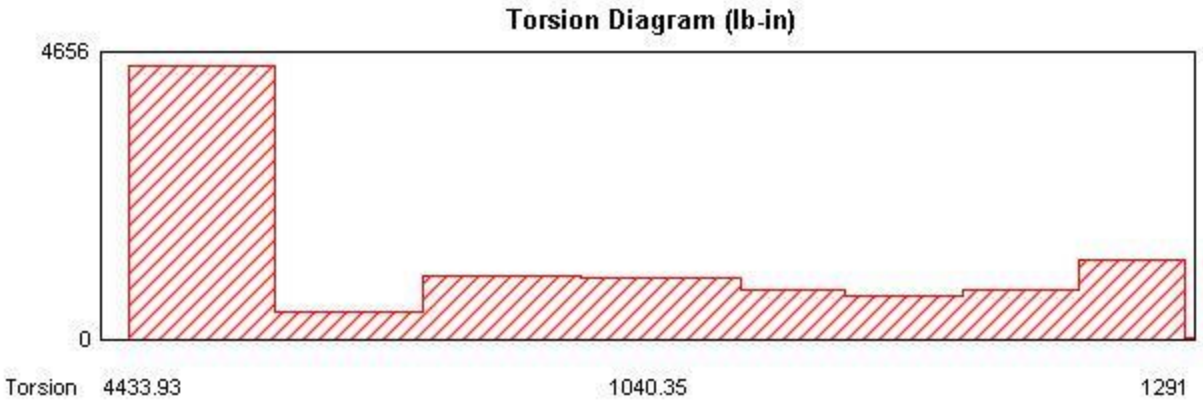


Shear	10070.13	6530.15	7964.99

Transverse Reinforcement (in2/ft)



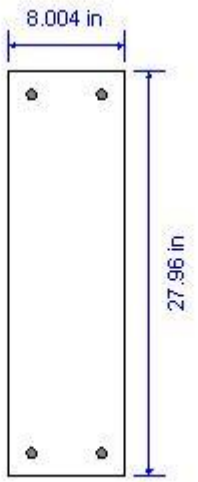
Asv/s reqd	8.004E-002	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
4656
0
Torsion
4433.93
1040.35
1291

ACI 318-14 Concrete Beam Design

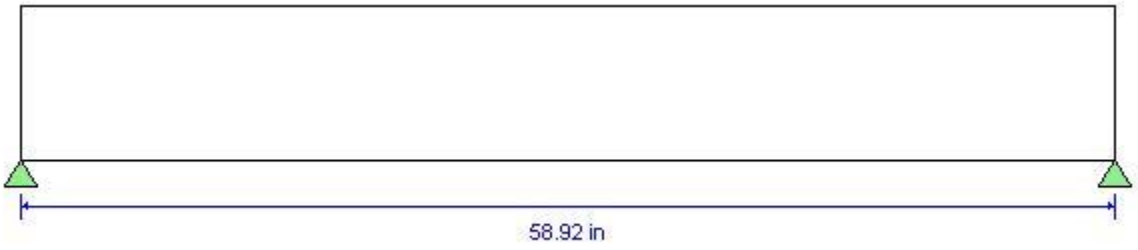
Geometric Properties

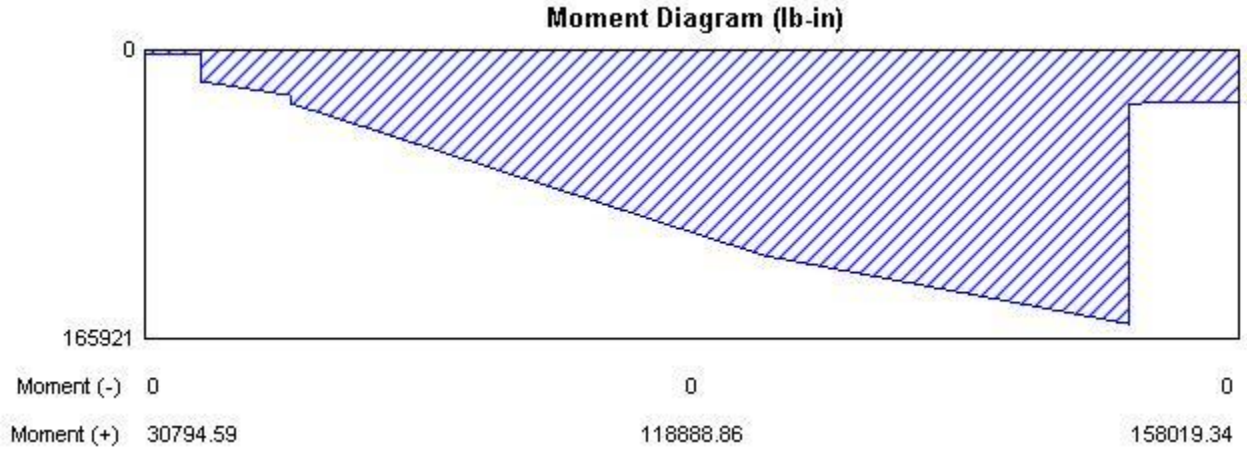
Combination = Overall Envelope
Beam Label = 100
Section Property = B8-8x28-4 ksi
Length = 58.92 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



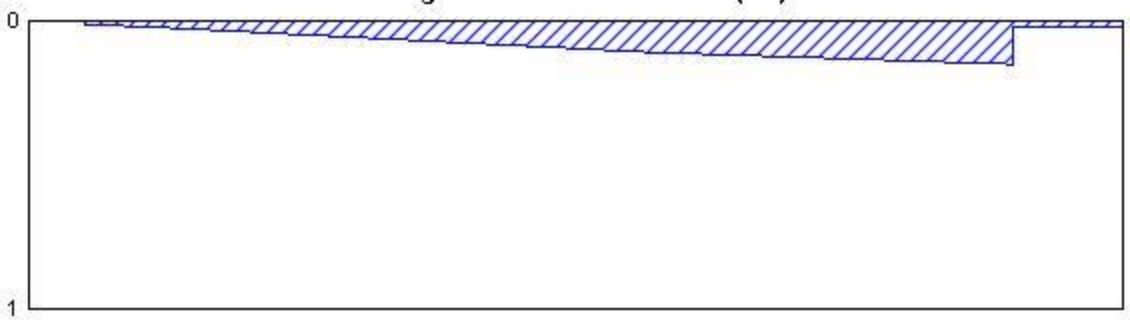
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



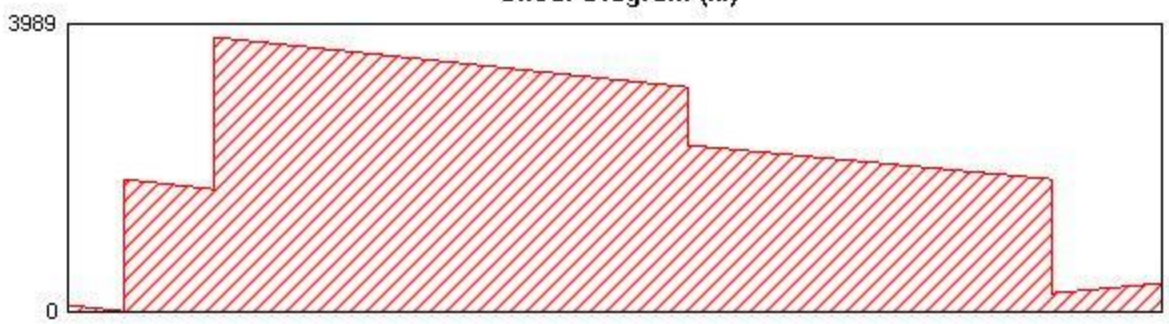


Longitudinal Reinforcement (in2)



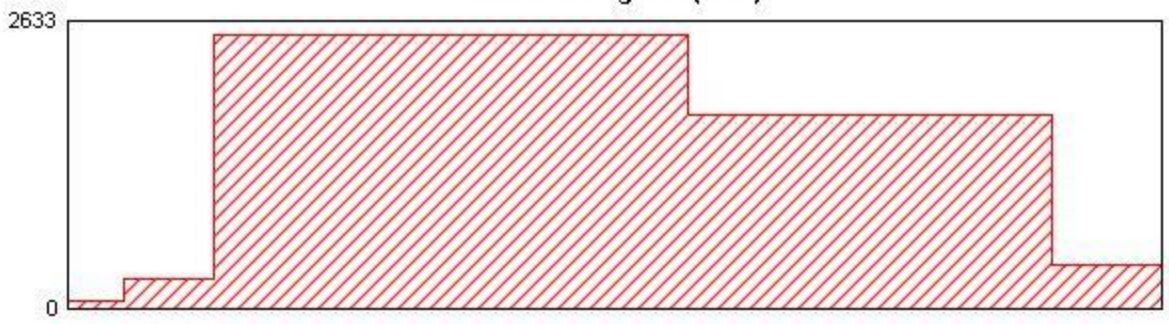
As (top)	0	0	0
Combo			
As (bot)	2.121E-002	8.332E-002	0.11
Combo	DCONU1	DCONU1	DCONU1

Shear Diagram (lb)



Shear	3798.63	3104.49	1814.49

Torsion Diagram (lb-in)

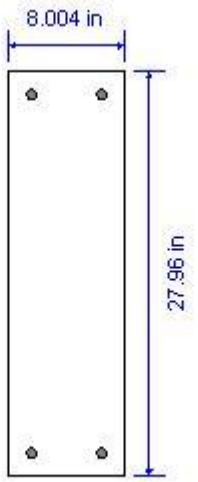


Torsion	2507.06	2507.06	1770.85

ACI 318-14 Concrete Beam Design

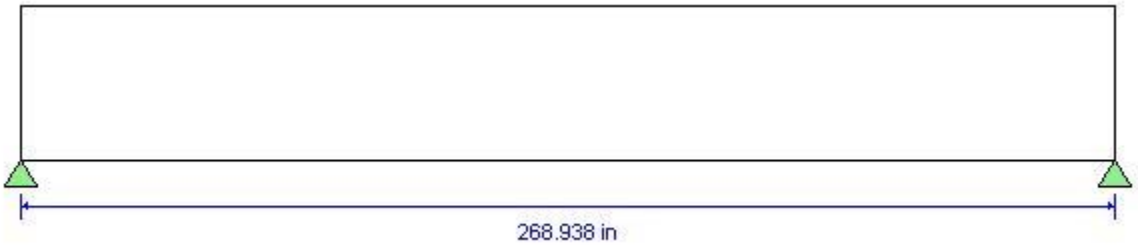
Geometric Properties

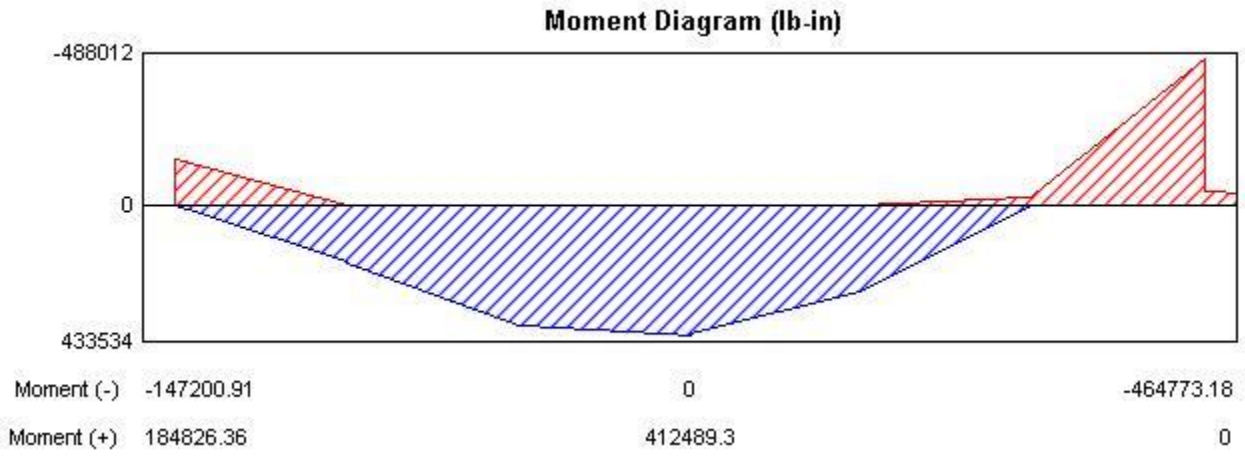
Combination = Overall Envelope
Beam Label = 101
Section Property = B8-8x28-4 ksi
Length = 268.938 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



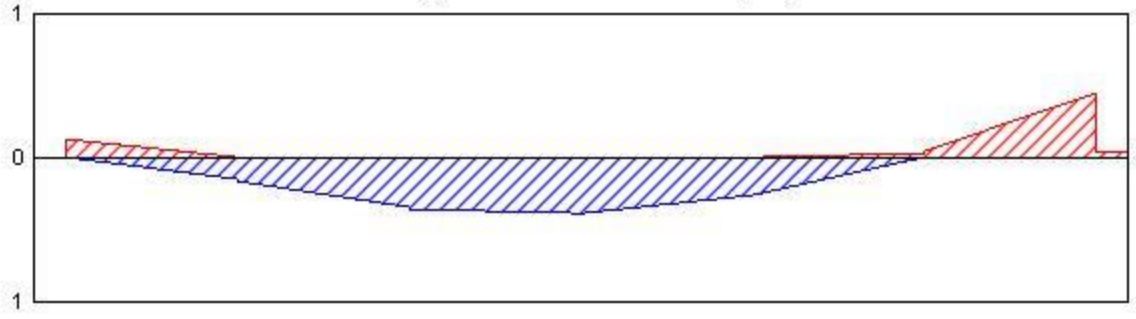
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



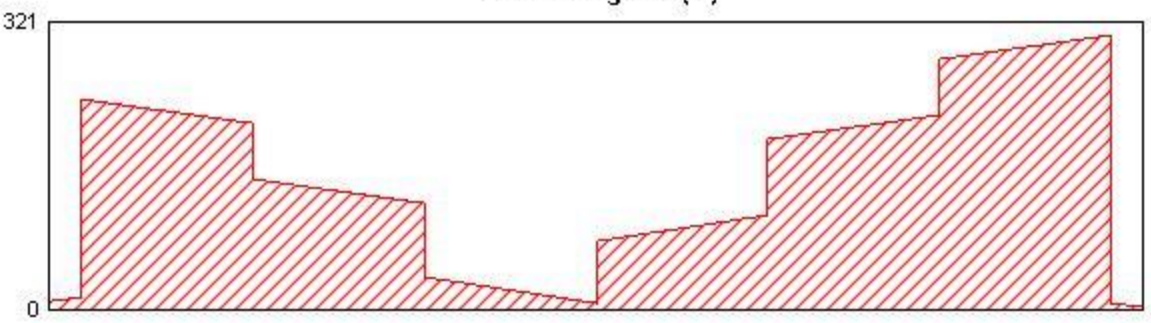


Longitudinal Reinforcement (in2)



As (top)	8.879E-002	0	0.34
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0.12	0.29	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)

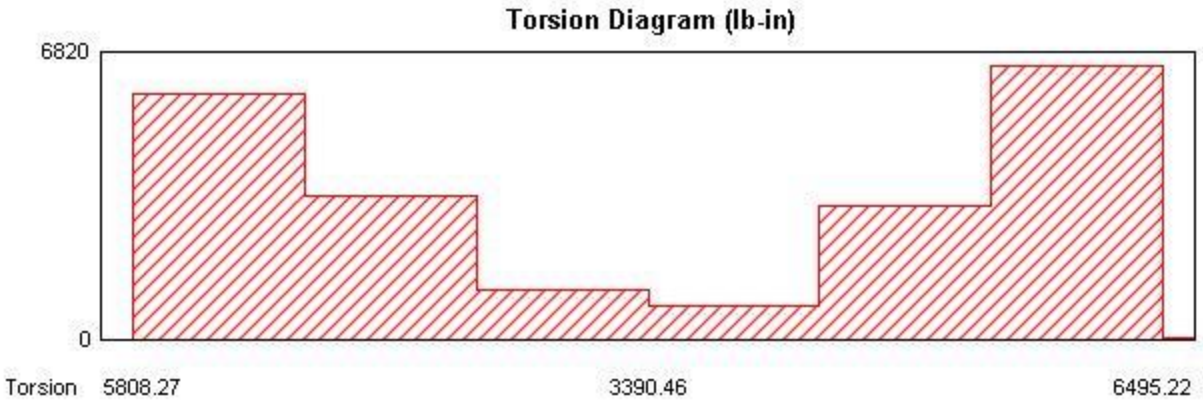


Shear	8273.73	6659.71	10781.36

Transverse Reinforcement (in2/ft)



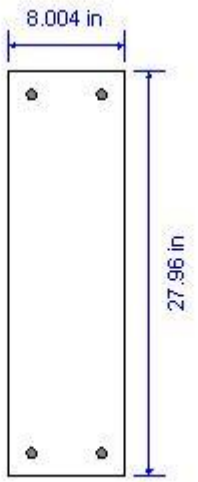
Asv/s reqd	0	0	8.004E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
6820
0
Torsion
5808.27
3390.46
6495.22

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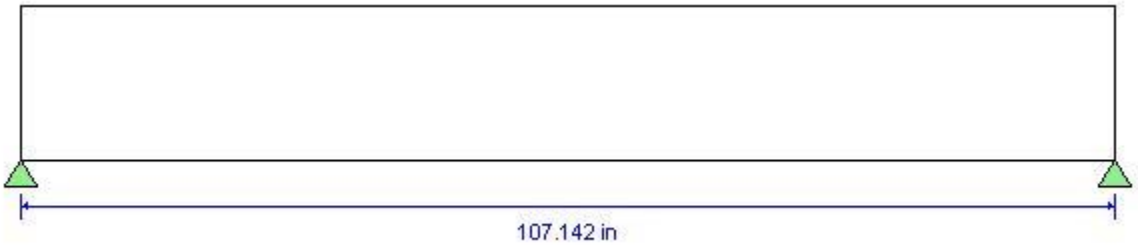
Geometric Properties

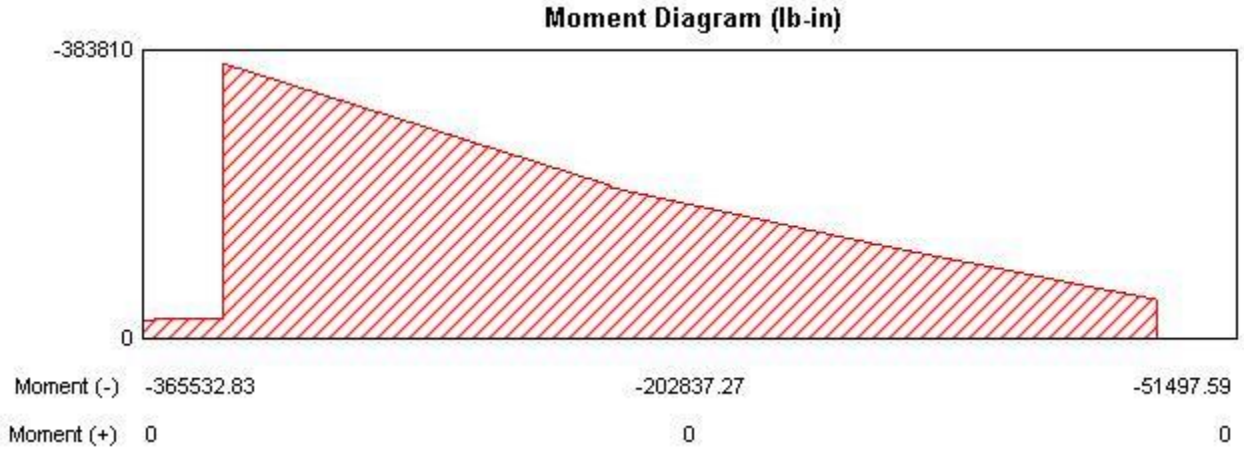
Combination = Overall Envelope
Beam Label = 102
Section Property = B8-8x28-4 ksi
Length = 107.142 in
Section Width = 8.004 in
Section Depth = 27.96 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



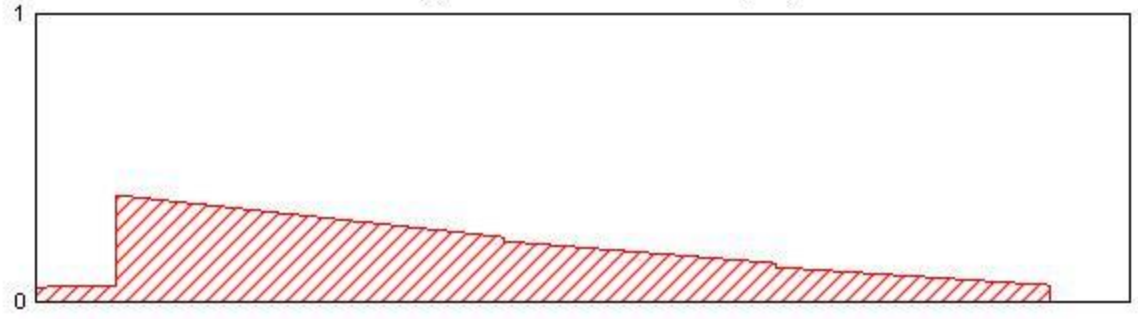
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



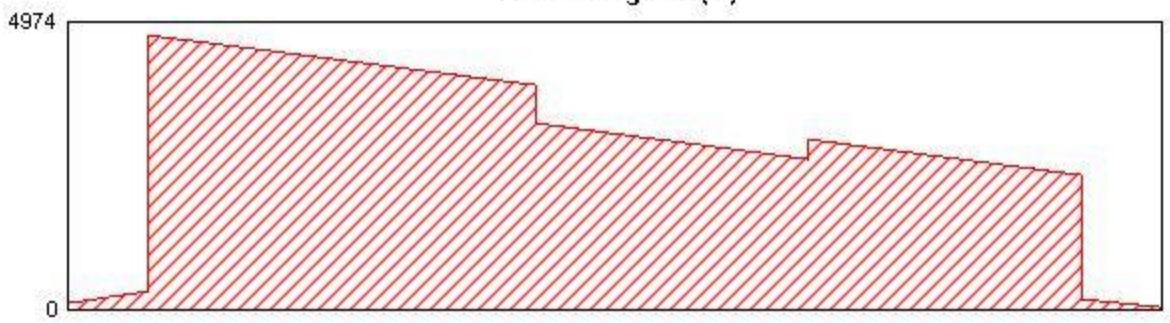


Longitudinal Reinforcement (in2)



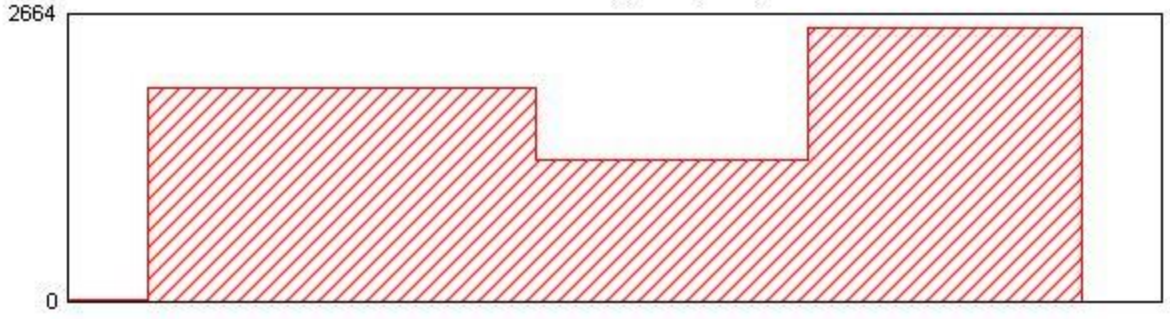
As (top)	0.28	0.16	4.252E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	4736.71	3853.7	2309.34

Torsion Diagram (lb-in)

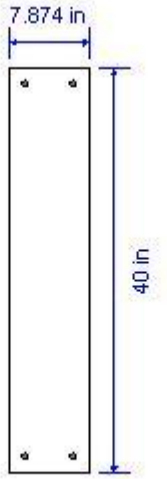


Torsion	1971.71	2536.4	2536.4

ACI 318-14 Concrete Beam Design

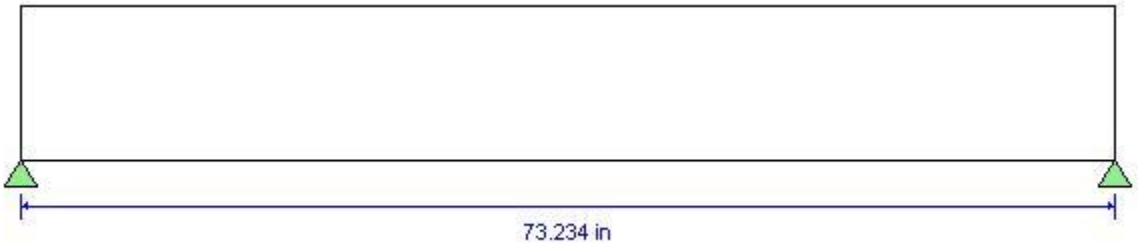
Geometric Properties

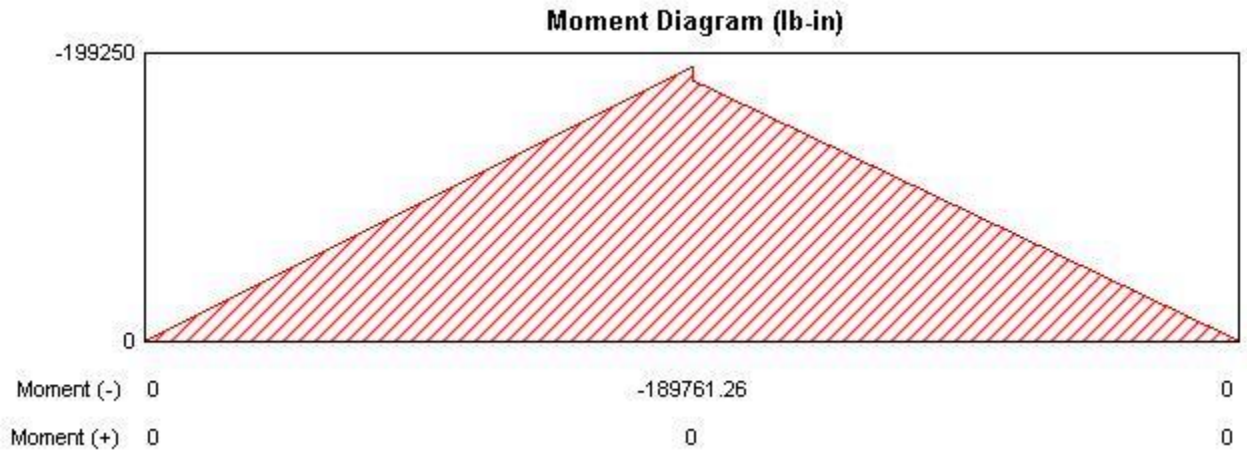
Combination = Overall Envelope
Beam Label = 105
Section Property = B10-8x40-4 ksi
Length = 73.234 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



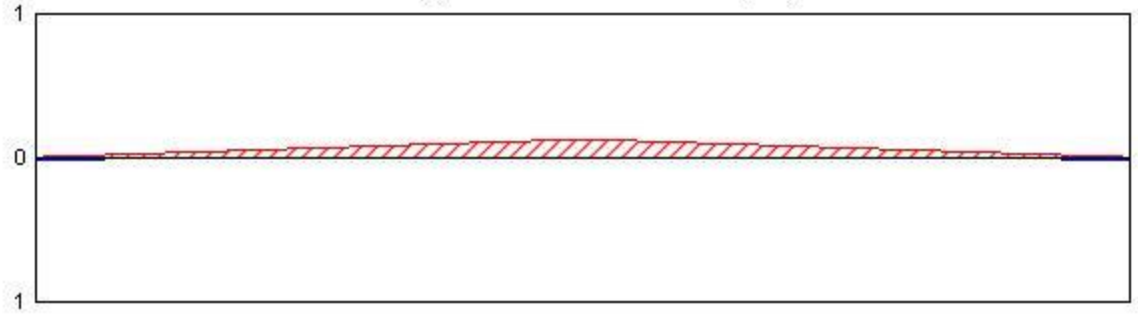
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



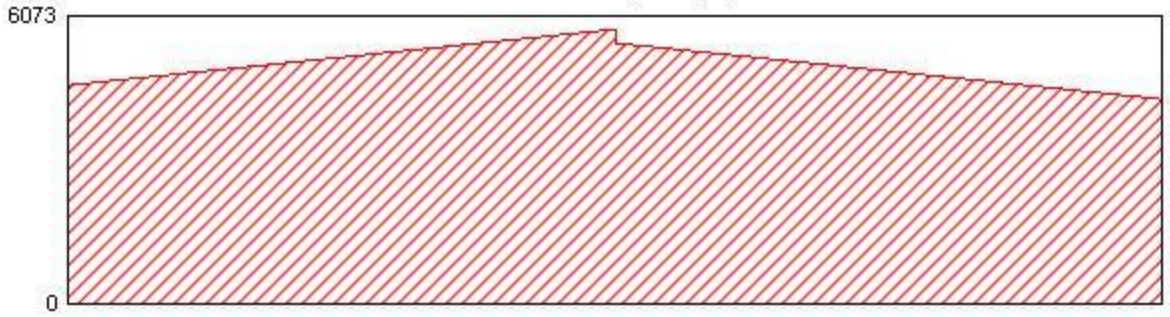


Longitudinal Reinforcement (in2)



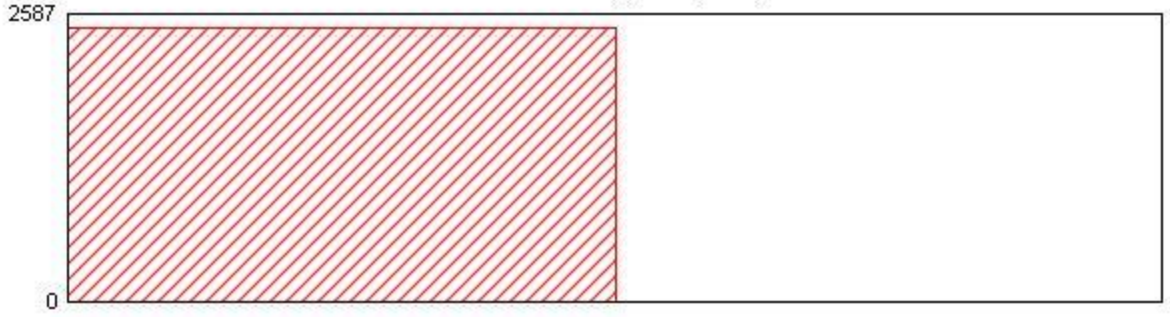
As (top)	0	0.1	0
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	4581.7	5783.03	4298.75

Torsion Diagram (lb-in)

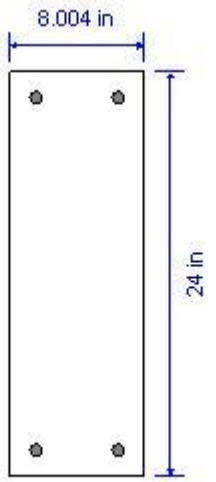


Torsion	2462.9	2462.9	0

ACI 318-14 Concrete Beam Design

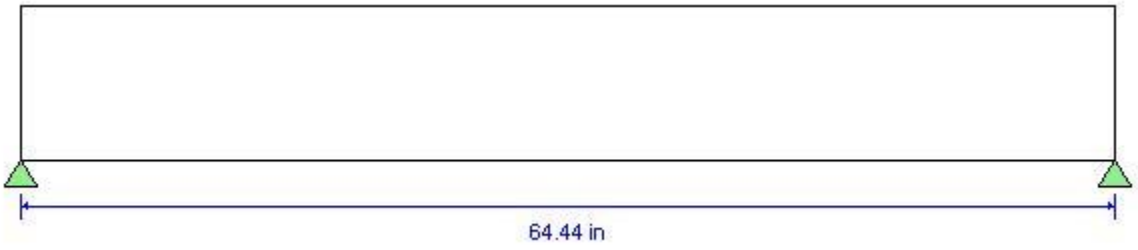
Geometric Properties

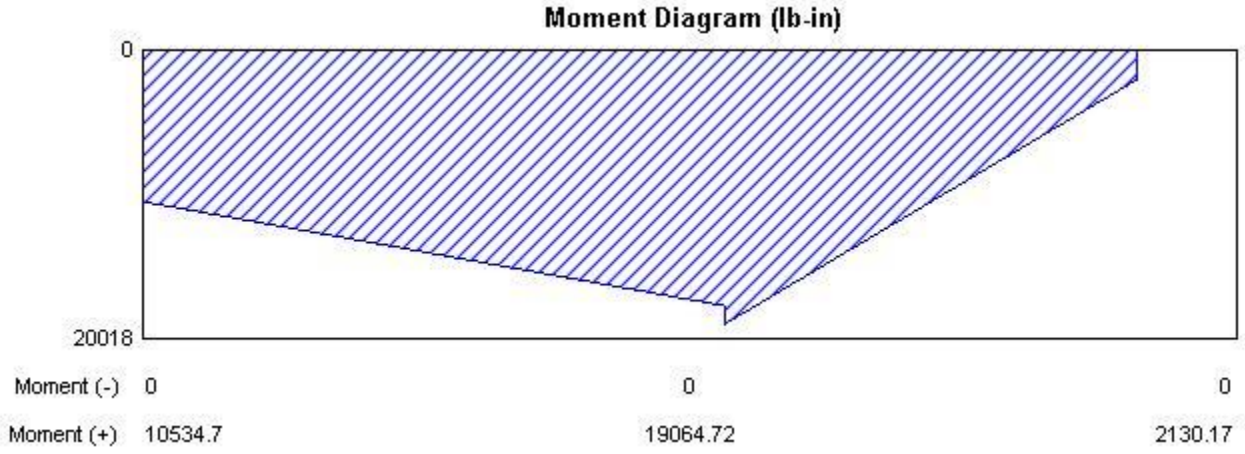
Combination = Overall Envelope
Beam Label = 111
Section Property = B6-8x24-4ksi
Length = 64.44 in
Section Width = 8.004 in
Section Depth = 24 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



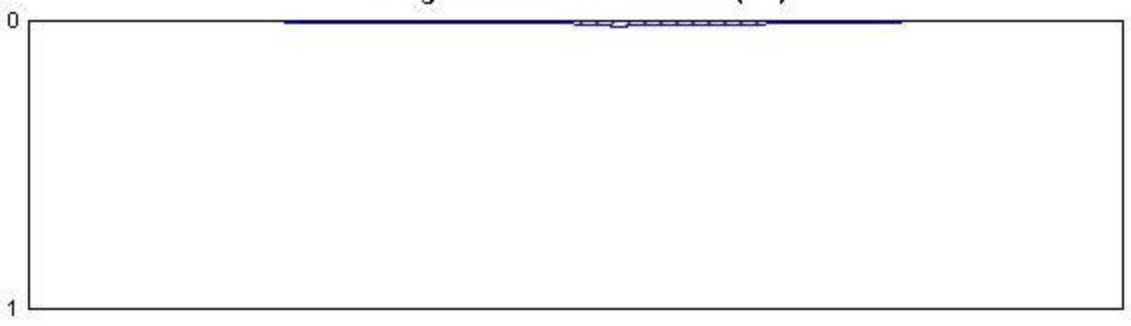
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



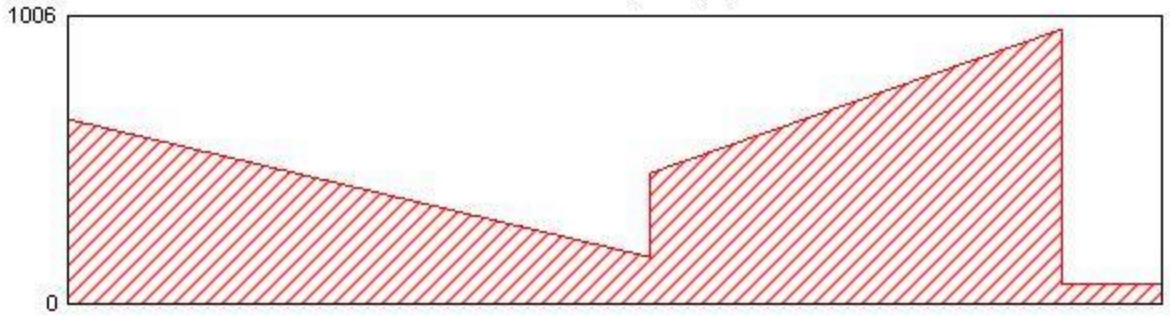


Longitudinal Reinforcement (in2)



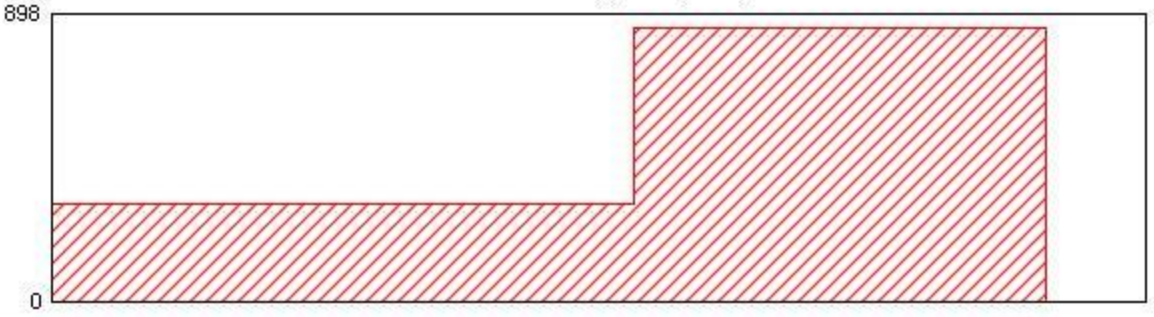
As (top)	0	0	0
Combo			
As (bot)	0	1.362E-002	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)



Shear	639.97	453.21	957.43

Torsion Diagram (lb-in)

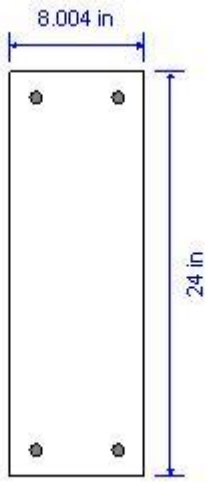


Torsion	304.72	854.47	854.47

ACI 318-14 Concrete Beam Design

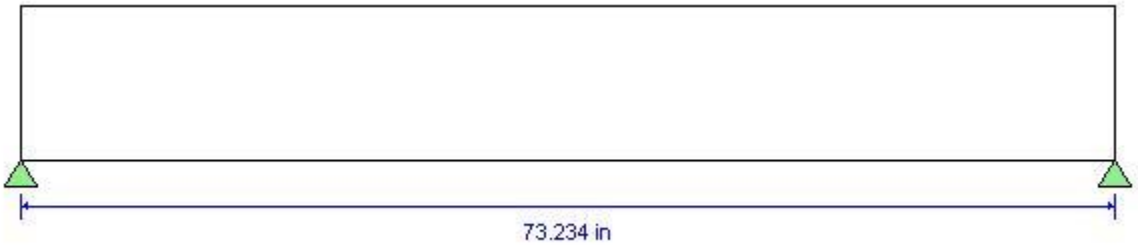
Geometric Properties

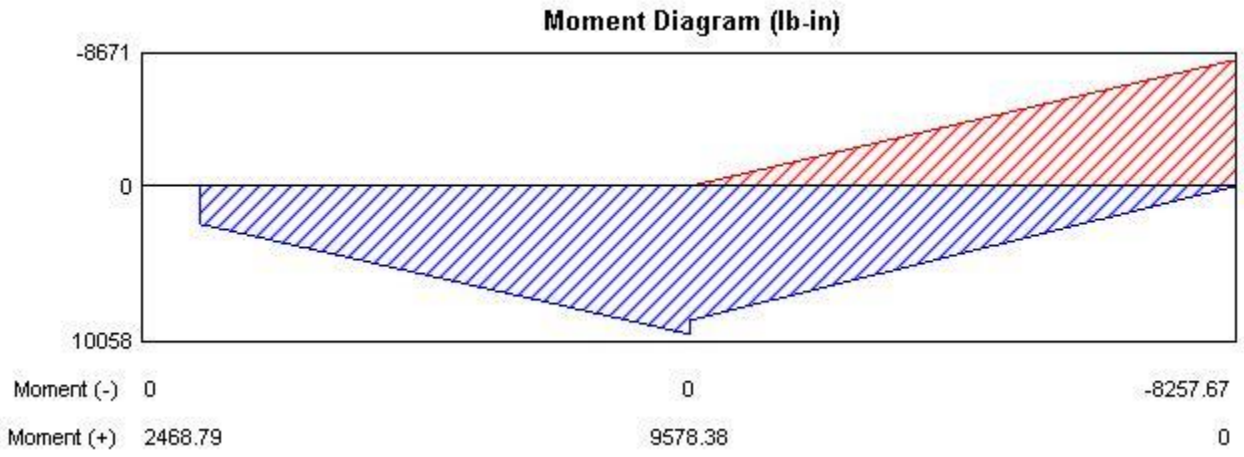
Combination = Overall Envelope
Beam Label = 113
Section Property = B6-8x24-4ksi
Length = 73.234 in
Section Width = 8.004 in
Section Depth = 24 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



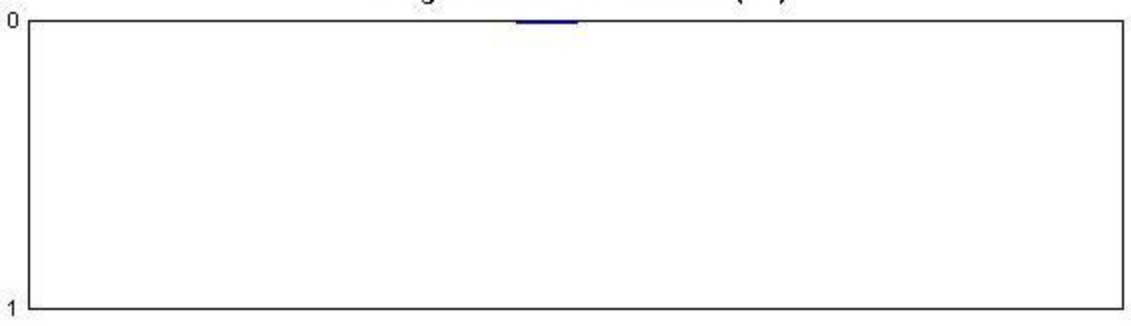
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



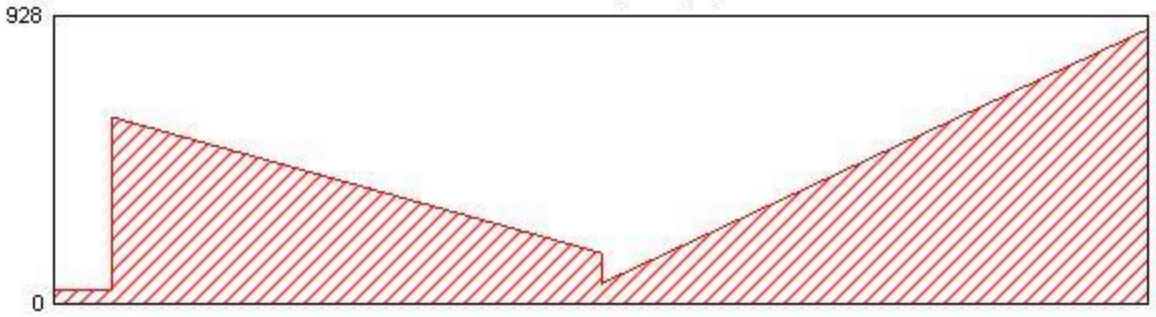


Longitudinal Reinforcement (in2)



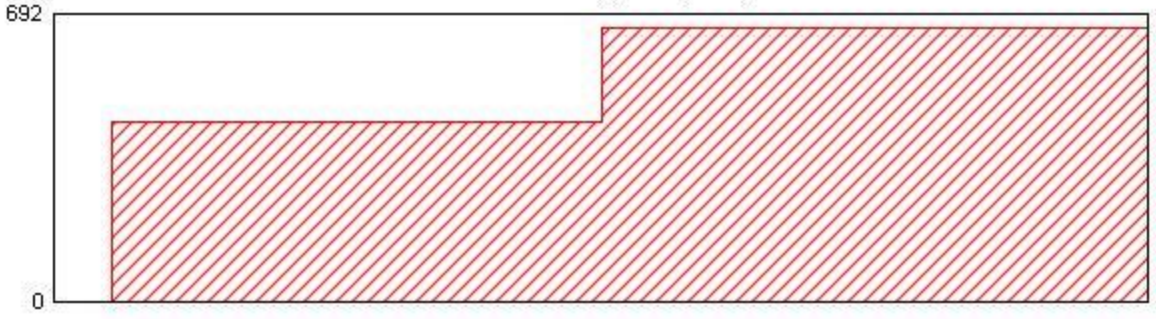
As (top)	0	0	0
Combo			Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	0	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	DCONU1	

Shear Diagram (lb)



Shear	600.24	162.66	883.61

Torsion Diagram (lb-in)

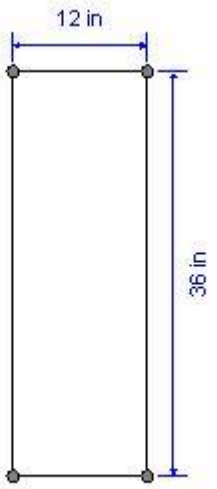


Torsion	431.59	658.95	658.95

ACI 318-14 Concrete Beam Design

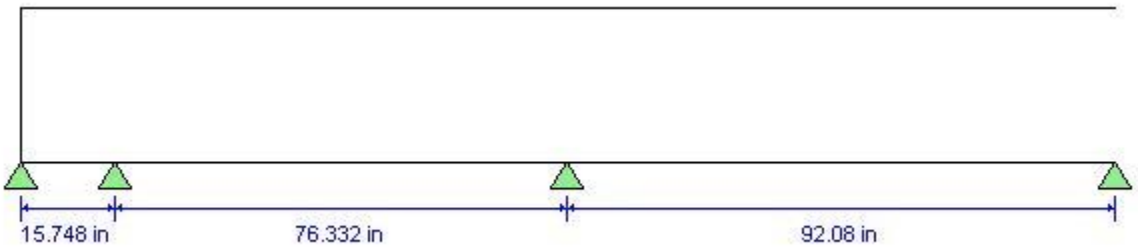
Geometric Properties

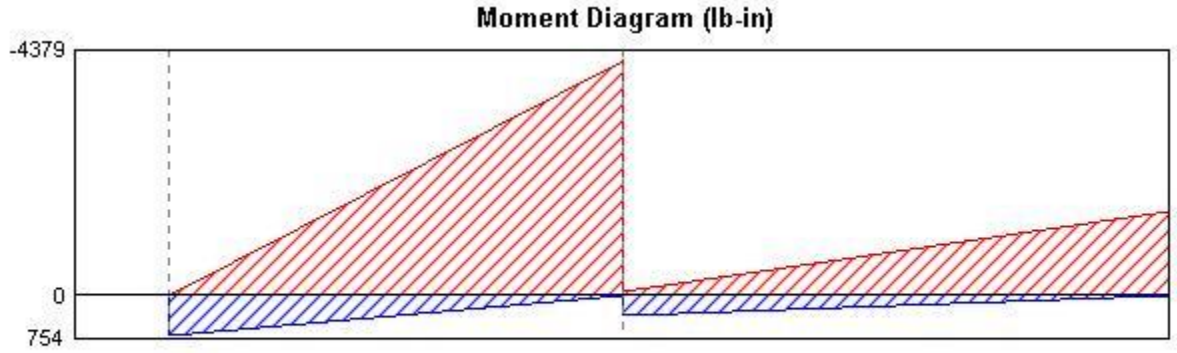
Combination = Overall Envelope
Beam Label = 5
Section Property =
Length = 276.24 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

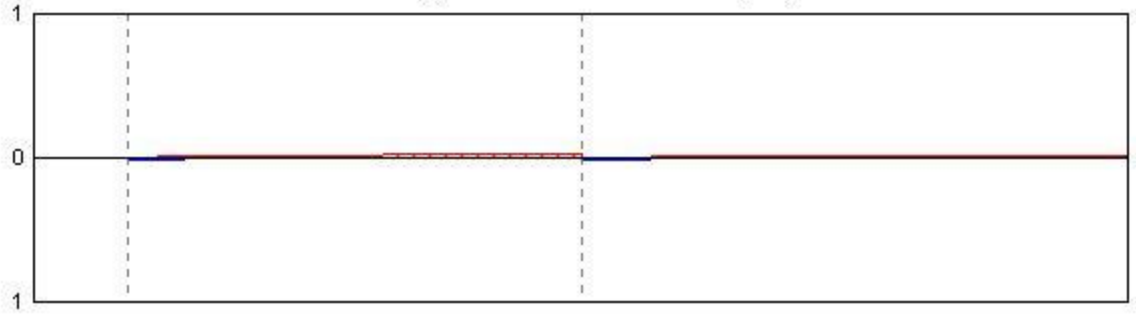
Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





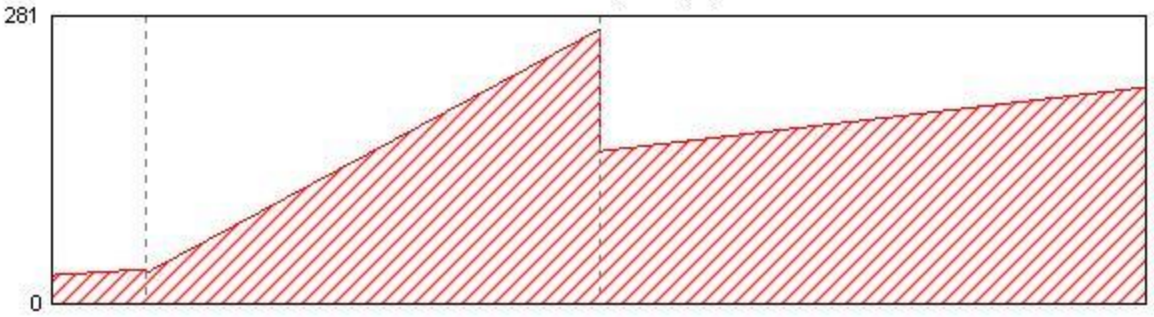
Moment (-)	0	-28.94	-28.94	0	-4169.64	-4169.64	0	-2083.83
Moment (+)	0	717.48	717.48	0	341	341	0	0

Longitudinal Reinforcement (in2)

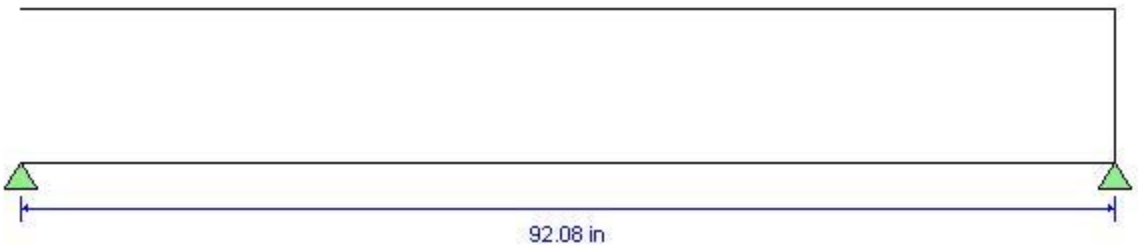


As (top)	5.065E-003	0	1.832E-002	1.832E-002	0	1.023E-002
Combo	DCONU2	DCONU2	DCONU2	DCONU2		DCONU1
As (bot)	3.794E-003	0	5.622E-003	5.622E-003	0	0
Combo	DCONU2	DCONU2	DCONU2	DCONU2		

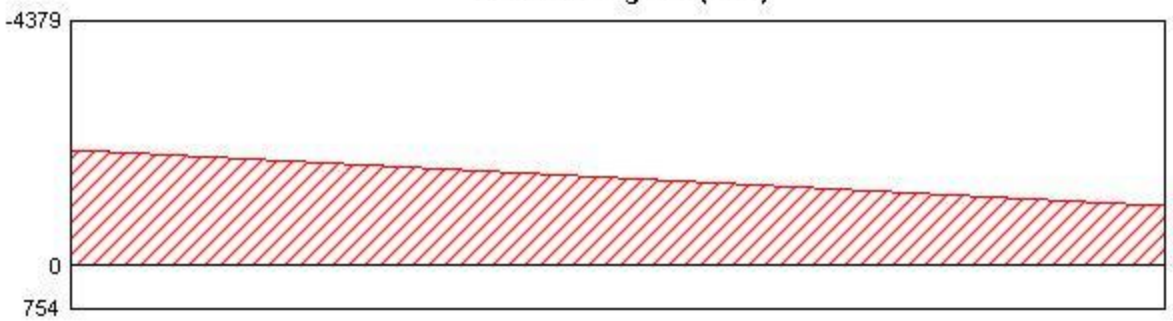
Shear Diagram (lb)



Shear	33.71	33.71	0	267.34	267.34	0	210.36

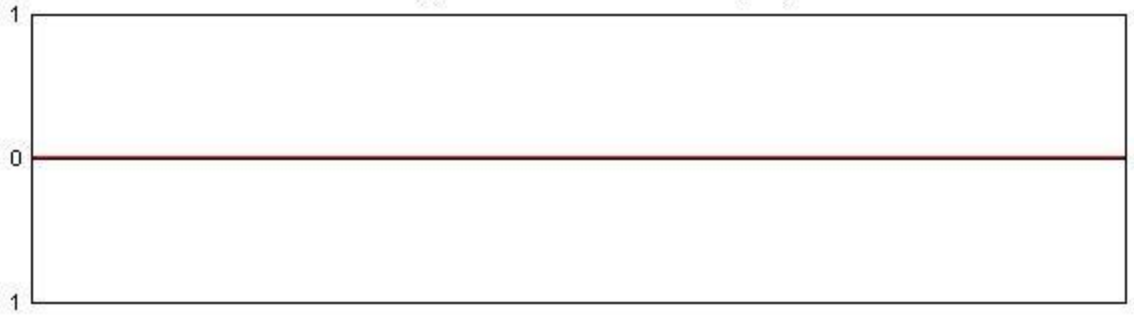


Moment Diagram (lb-in)



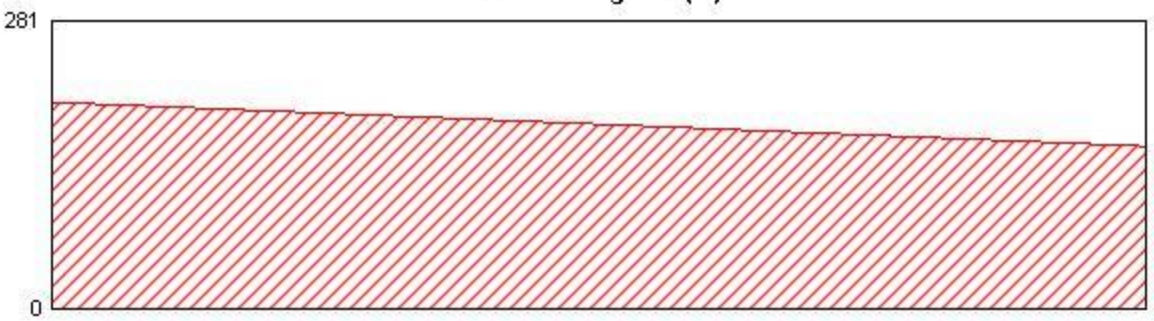
Moment (-)	-2083.83	0	-1074.75
Moment (+)	0	0	0

Longitudinal Reinforcement (in2)



As (top)	1.023E-002	0	7.321E-003
Combo	DCONU1		DCONU1
As (bot)	0	0	0
Combo			

Shear Diagram (lb)

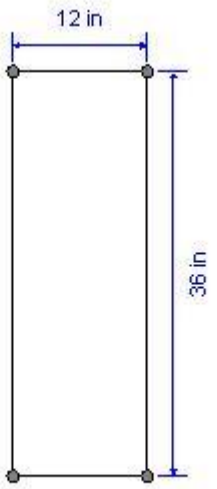


Shear	210.36	0	157.13

ACI 318-14 Concrete Beam Design

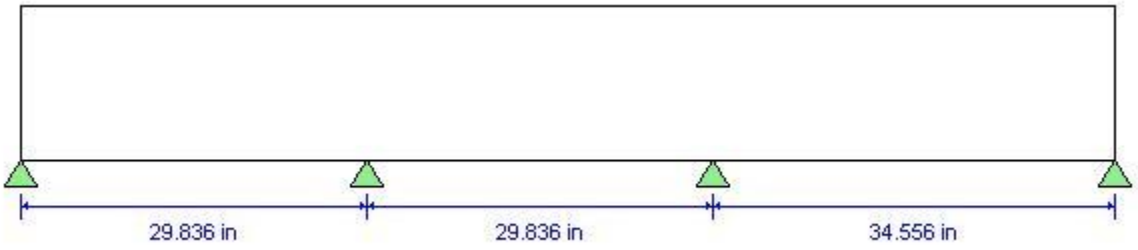
Geometric Properties

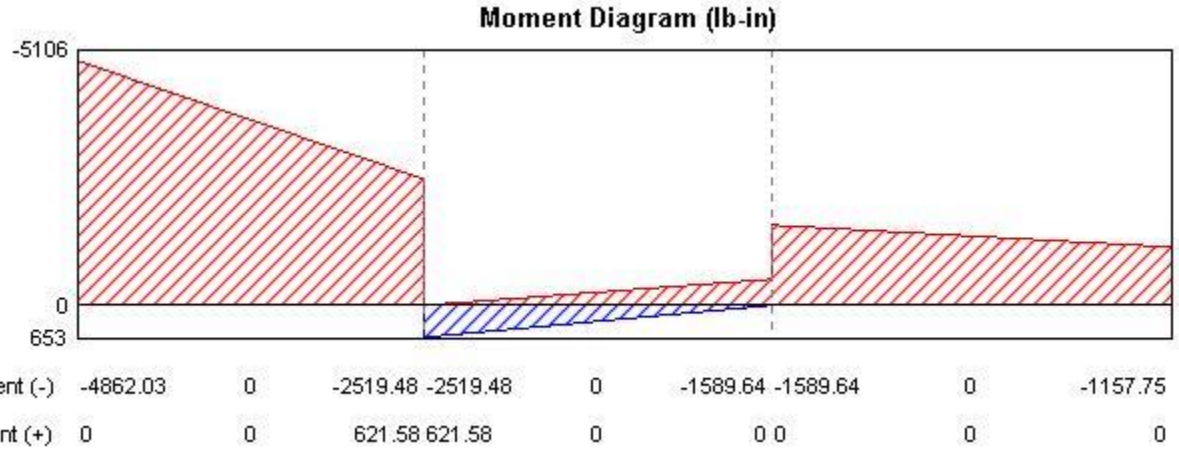
Combination = Overall Envelope
Beam Label = 51
Section Property =
Length = 94.228 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

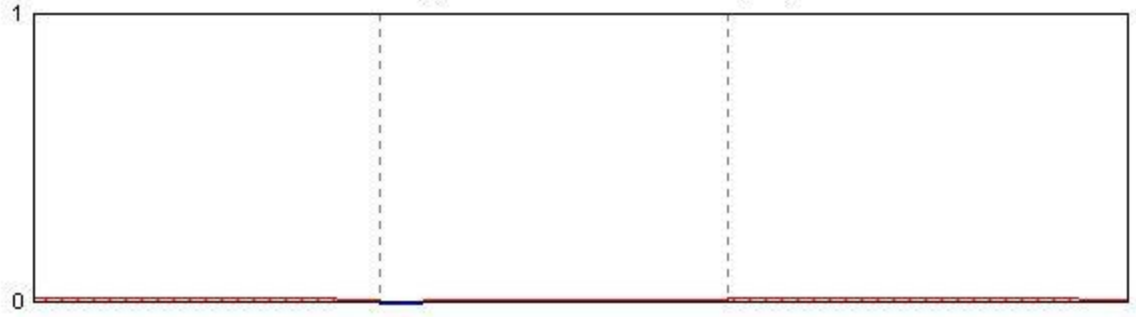
Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





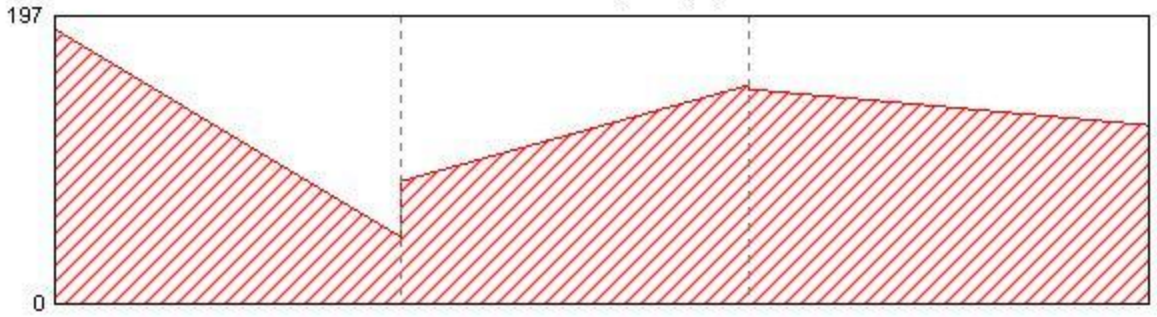
Moment (-)	-4862.03	0	-2519.48	-2519.48	0	-1589.64	-1589.64	0	-1157.75
Moment (+)	0	0	621.58	621.58	0	0	0	0	0

Longitudinal Reinforcement (in2)



As (top)	1.292E-002	0	6.146E-003	6.146E-003	0	8.995E-003	8.995E-003	0	7.522E-003
Combo	DCONU2		DCONU2	DCONU2		DCONU2	DCONU2		DCONU1
As (bot)	0	0	3.462E-003	3.462E-003	0	0	0	0	0
Combo			DCONU2	DCONU2					

Shear Diagram (lb)

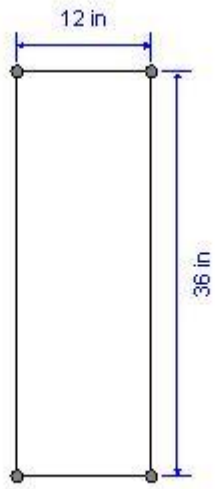


Shear	186.99	0	82.86	82.86	0	149.19	149.19	0	122.35

ACI 318-14 Concrete Beam Design

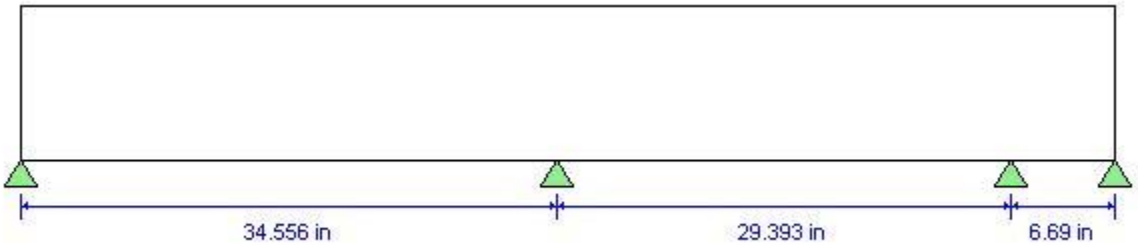
Geometric Properties

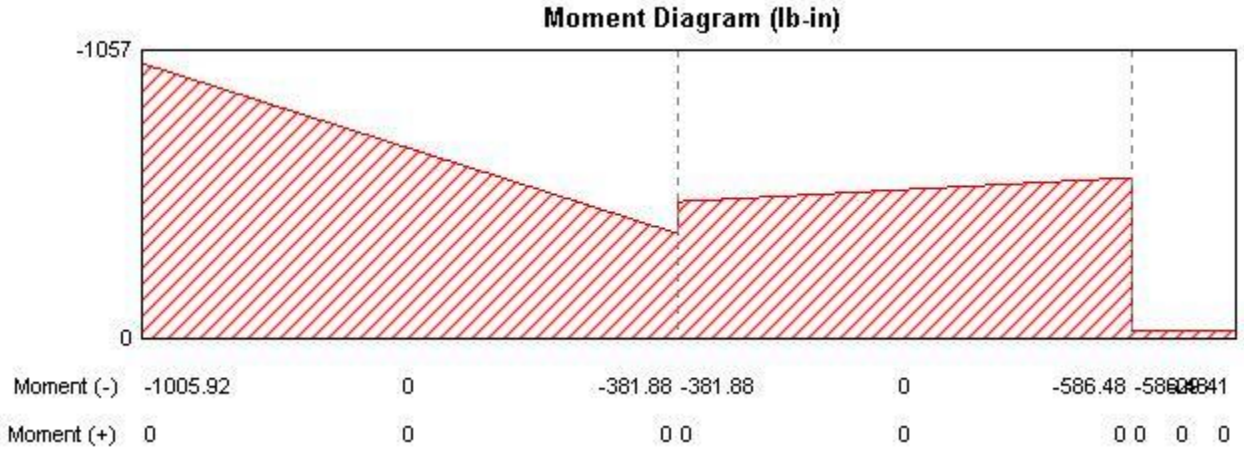
Combination = Overall Envelope
Beam Label = 57
Section Property =
Length = 70.639 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



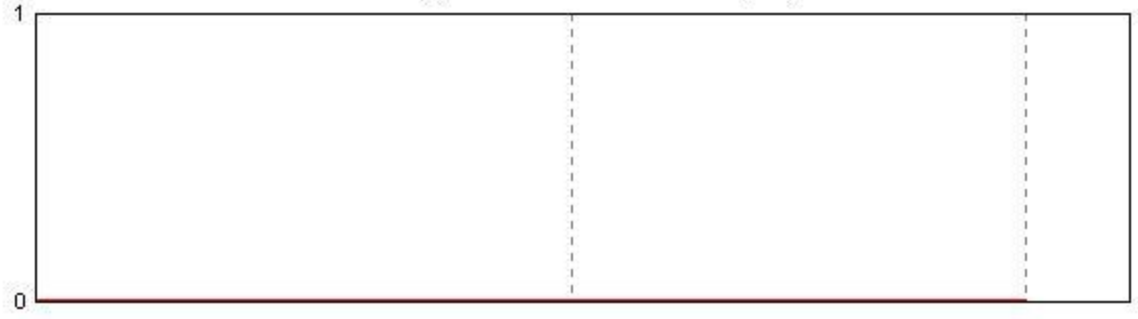
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2



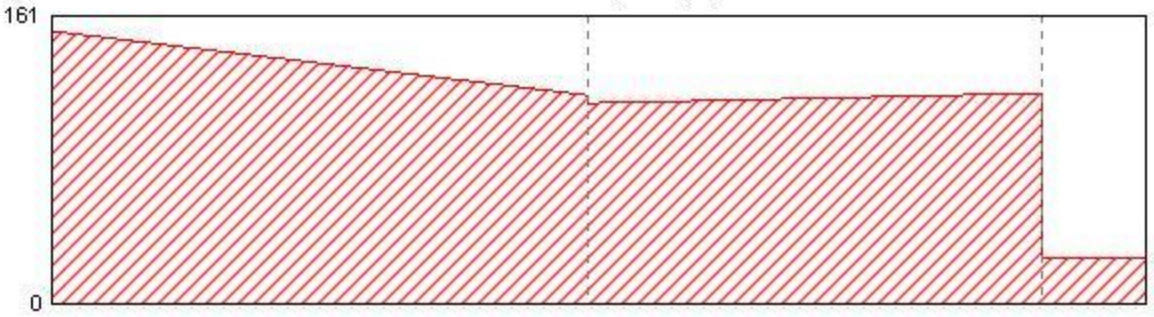


Longitudinal Reinforcement (in2)



As (top)	5.429E-003	0	3.49E-003	3.49E-003	0	2.875E-003	2.87[illegible]E-003
Combo	DCONU1		DCONU1	DCONU1		DCONU1	D[illegible]1
As (bot)	0	0	0	0	0	0	0 0 0
Combo							

Shear Diagram (lb)

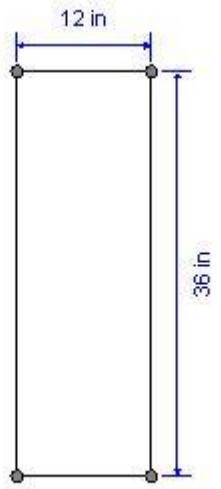


Shear	152.44	0	116.33	116.33	0	117.35	117.[illegible]6.12

ACI 318-14 Concrete Beam Design

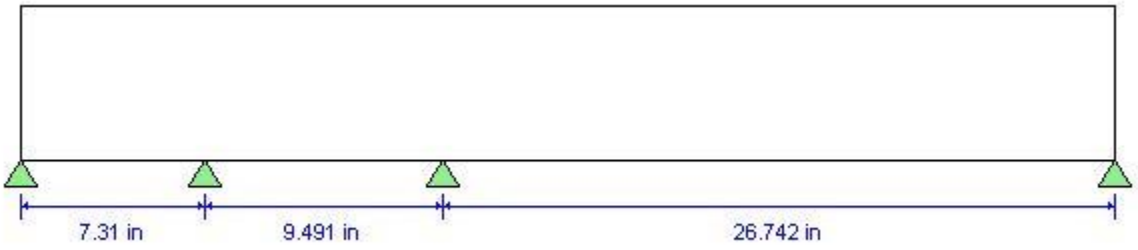
Geometric Properties

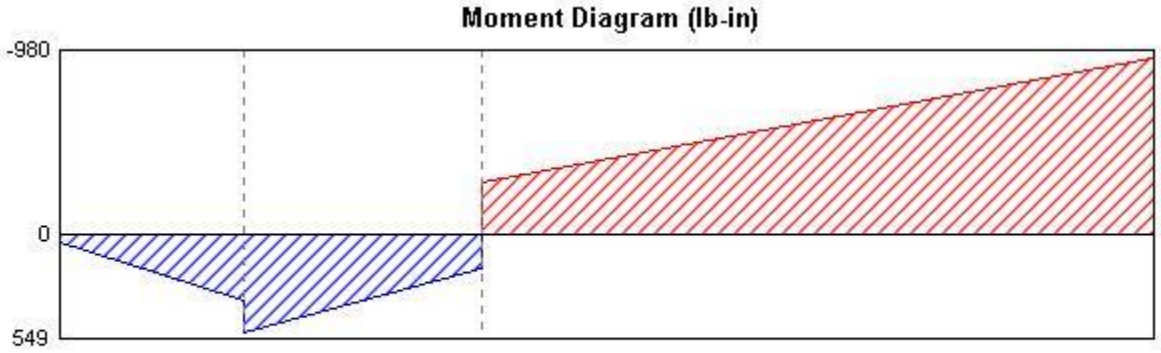
Combination = Overall Envelope
Beam Label = 67
Section Property =
Length = 43.543 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



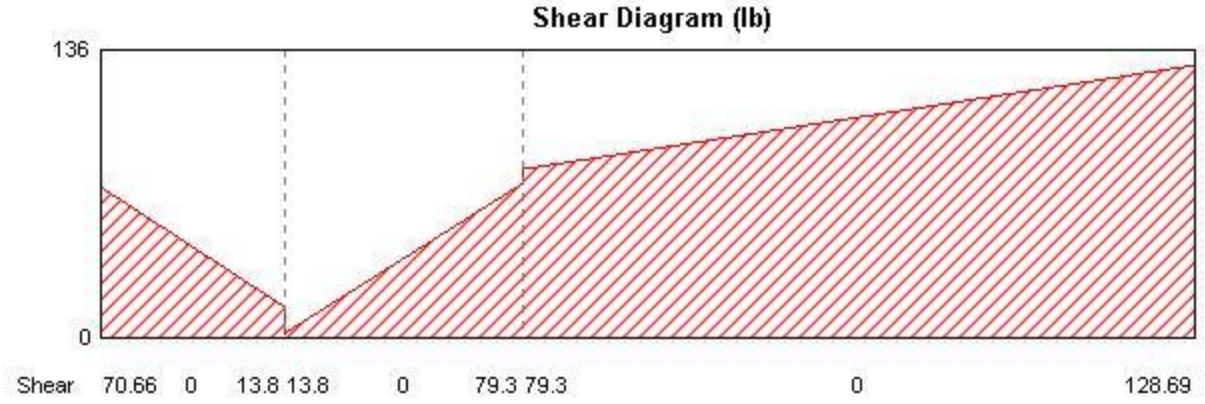
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





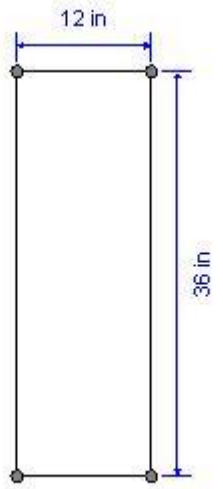
Moment (-)	0	0	0	0	0	-272.16	-272.16	0	-932.49
Moment (+)	39.96	0	522.75	522.75	0	184.88	184.88	0	0


Shear Diagram (lb)
136
0
Shear 70.66 0 13.8 13.8 0 79.3 79.3 0 128.69

ACI 318-14 Concrete Beam Design

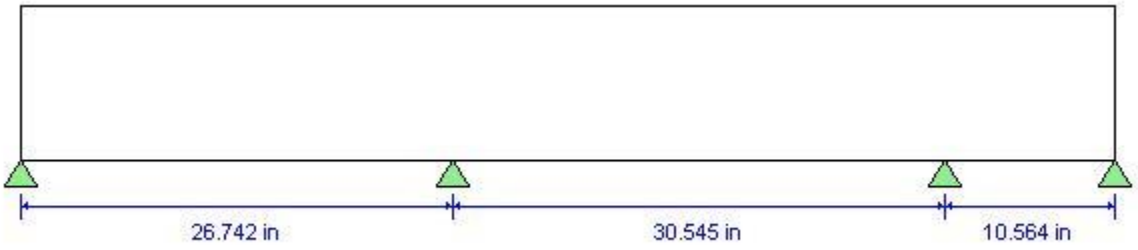
Geometric Properties

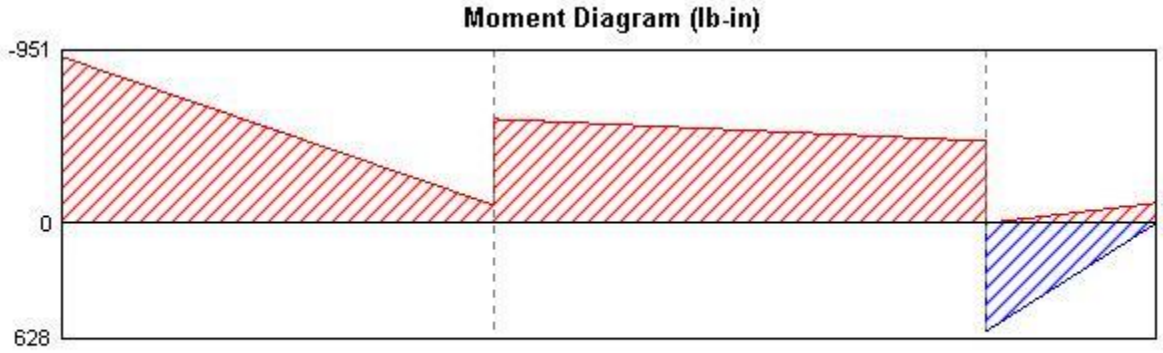
Combination = Overall Envelope
Beam Label = 68
Section Property =
Length = 67.851 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



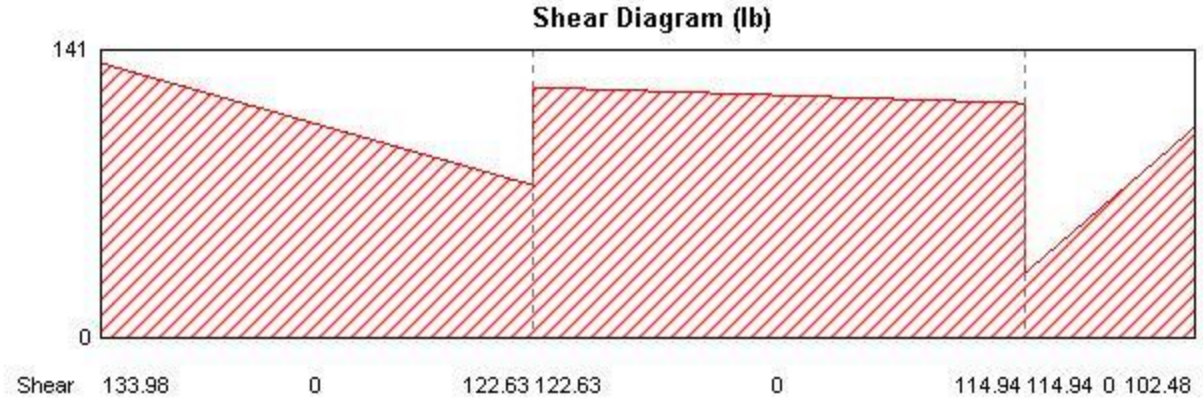
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





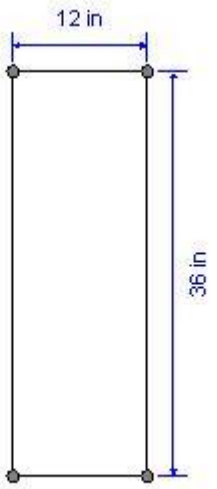
Moment (-)	-905.67	0	-565.69	-565.69	0	-448.22	-448.22	0	-109.91	
Moment (+)	0	0	0	0		0	597.35	597.35	0	0


Shear Diagram (lb)
141
0
Shear 133.98 0 122.63 122.63 0 114.94 114.94 0 102.48

ACI 318-14 Concrete Beam Design

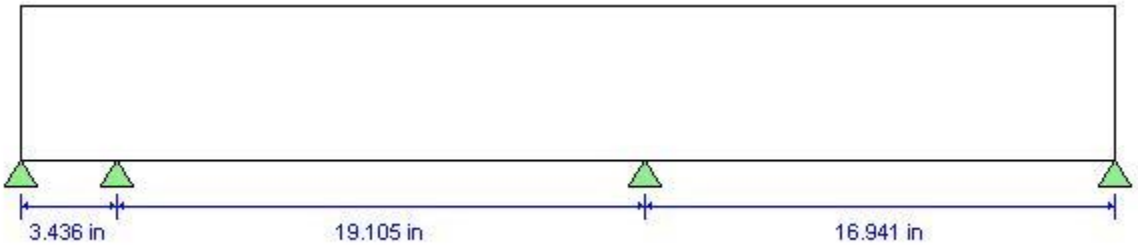
Geometric Properties

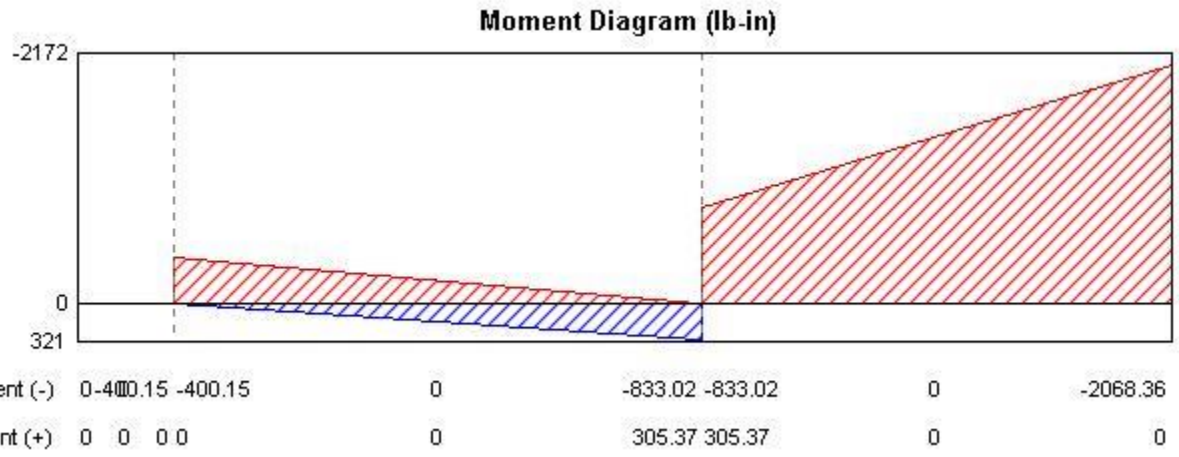
Combination = Overall Envelope
Beam Label = 70
Section Property =
Length = 39.483 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





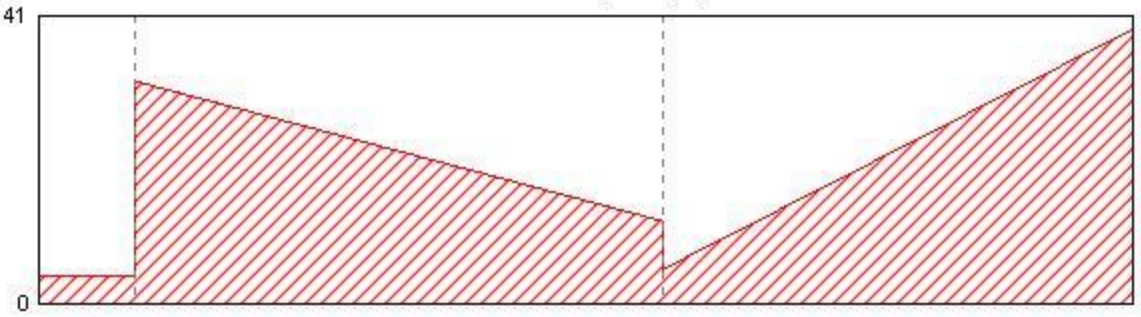
Moment (-)	0	-400.15	-400.15	0	-833.02	-833.02	0	-2068.36
Moment (+)	0	0	0	0	0	305.37	305.37	0	0

Longitudinal Reinforcement (in2)



As (top)	0	0	0	0	0	3.629E-003	3.629E-003	0	7.194E-003
Combo	DCONU1	DCONU1				DCONU2	DCONU2		DCONU2
As (bot)	0	0	0	0	0	0	0	0	0
Combo						DCONU2	DCONU2		

Shear Diagram (lb)

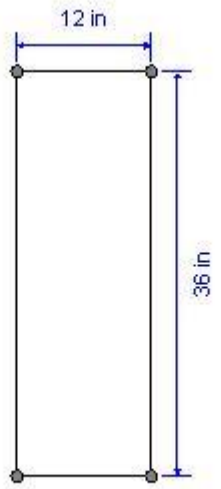


Shear	131	141	108.68	108.68	0	39.92	39.92	0	133.79

ACI 318-14 Concrete Beam Design

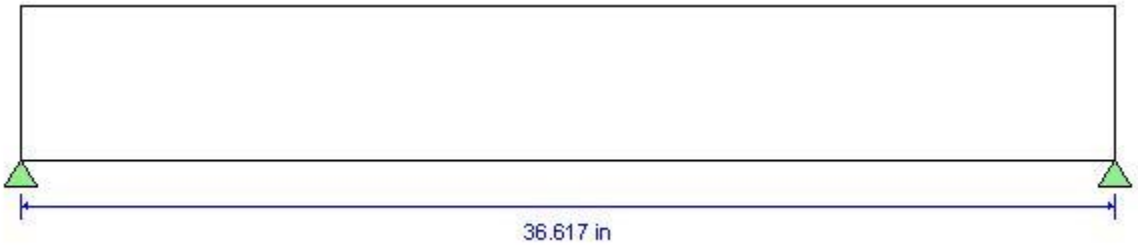
Geometric Properties

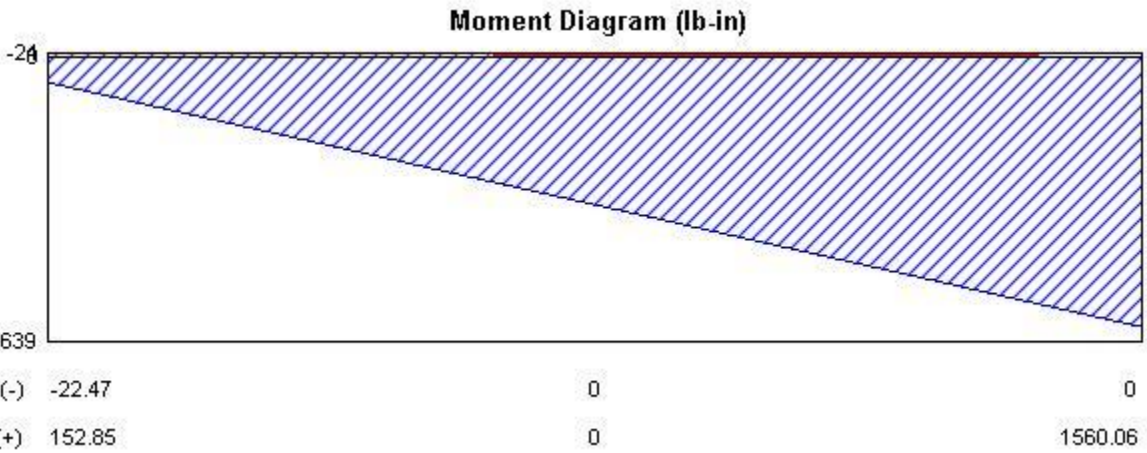
Combination = Overall Envelope
Beam Label = 89
Section Property =
Length = 36.617 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2





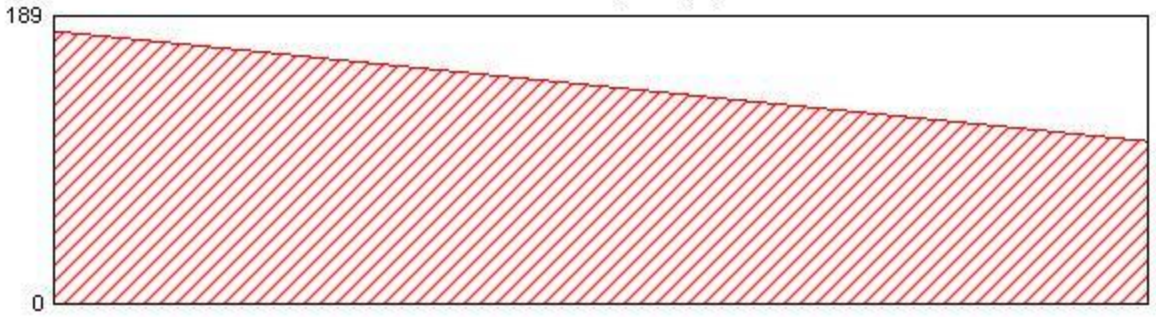
Moment (-)	-22.47	0	0
Moment (+)	152.85	0	1560.06

Longitudinal Reinforcement (in2)



As (top)	0	0	0
Combo	DCONU1		
As (bot)	0	0	4.121E-003
Combo	DCONU2		DCONU2

Shear Diagram (lb)

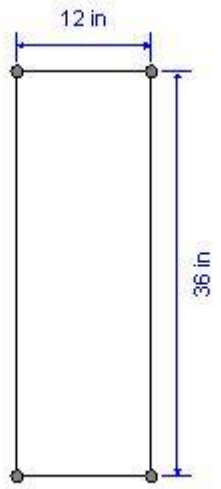


Shear	179.08	0	105.72

ACI 318-14 Concrete Beam Design

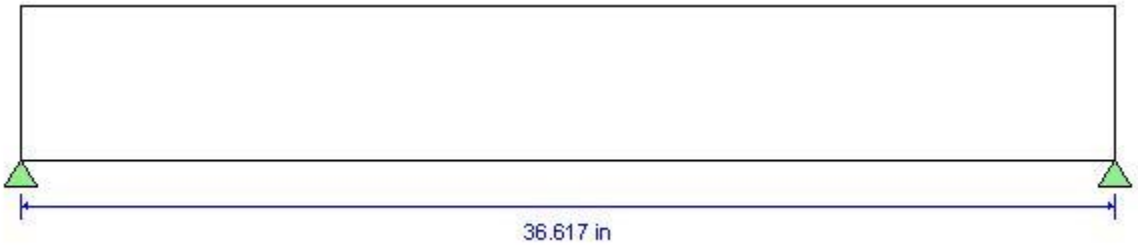
Geometric Properties

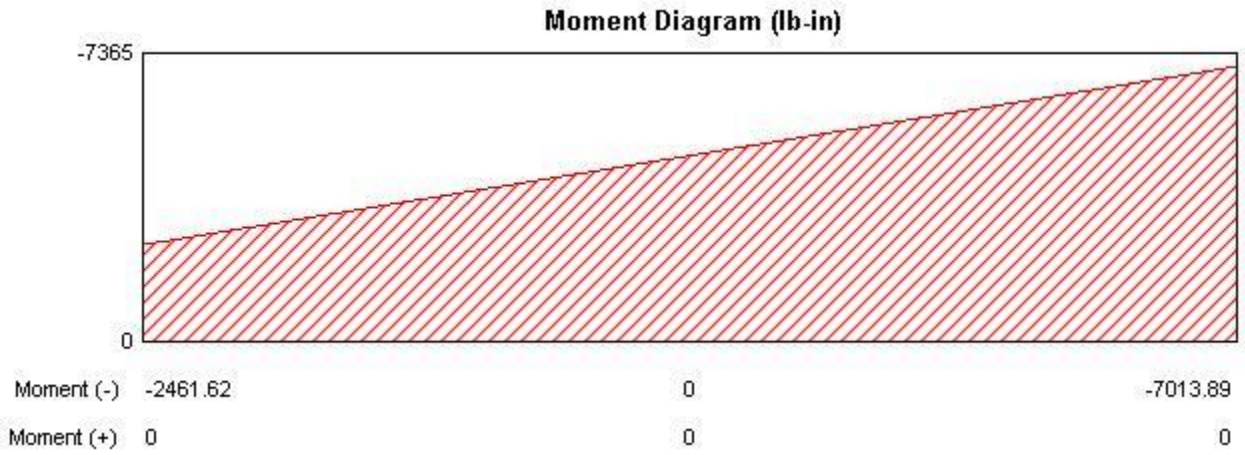
Combination = Overall Envelope
Beam Label = 90
Section Property =
Length = 36.617 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



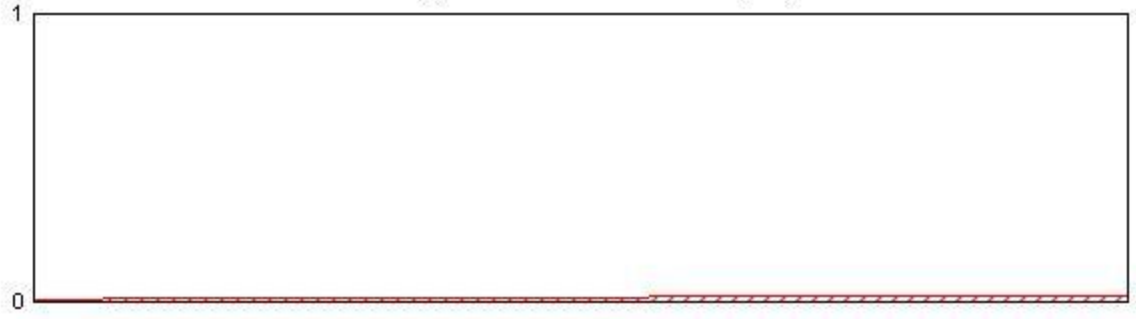
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2



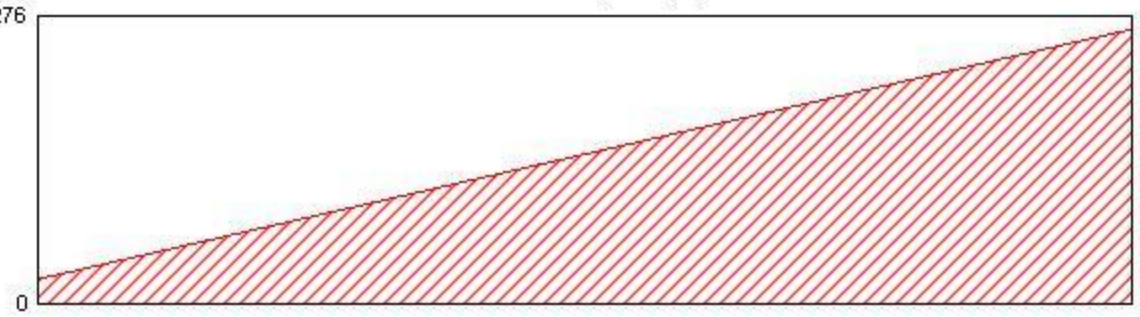


Longitudinal Reinforcement (in2)



As (top)	3.32E-003	0	1.613E-002
Combo	DCONU1		DCONU2
As (bot)	0	0	0
Combo			

Shear Diagram (lb)

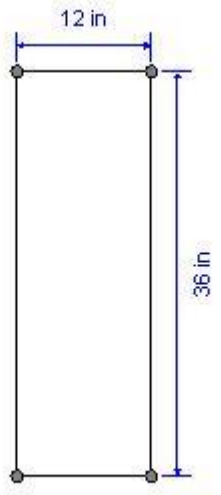


Shear	22.85	0	261.95

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 44
Section Property =
Length = 108.36 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



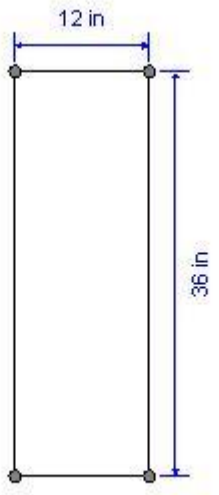
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 50
Section Property =
Length = 73.804 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



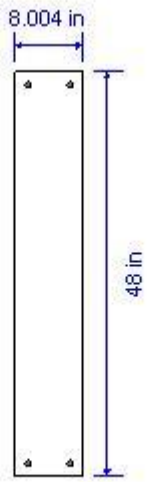
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

ACI 318-14 Concrete Beam Design

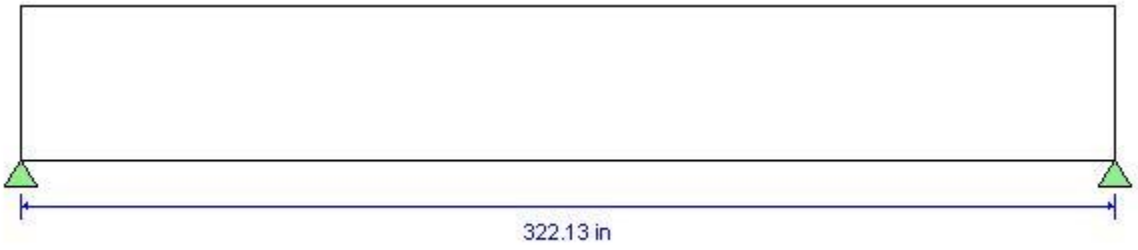
Geometric Properties

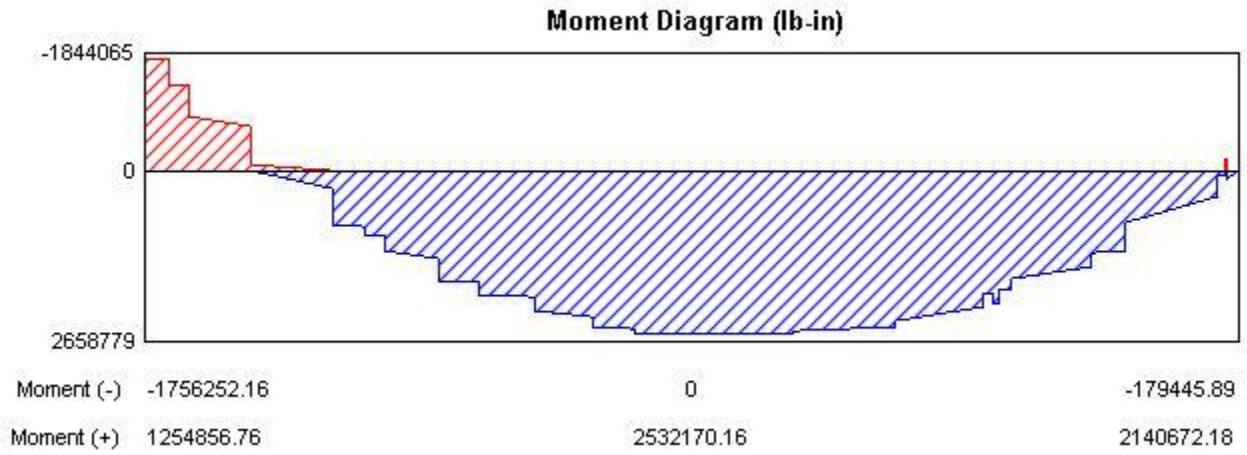
Combination = Overall Envelope
Beam Label = 99
Section Property = B14-8x48-fc 4ksi
Length = 322.13 in
Section Width = 8.004 in
Section Depth = 48 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



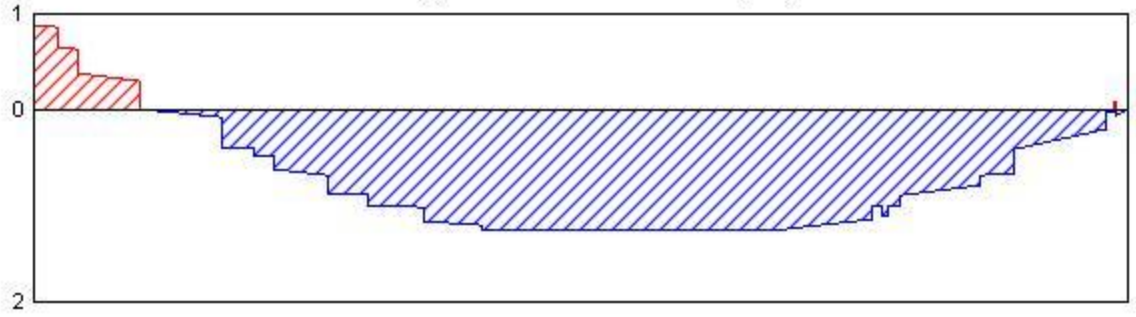
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



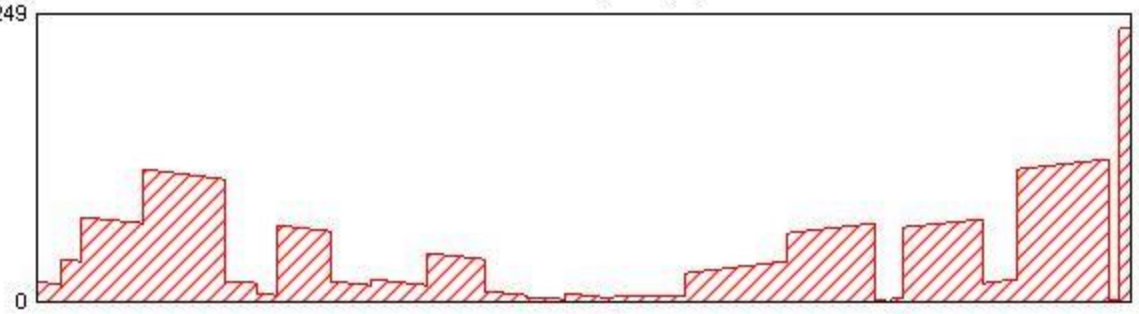


Longitudinal Reinforcement (in2)



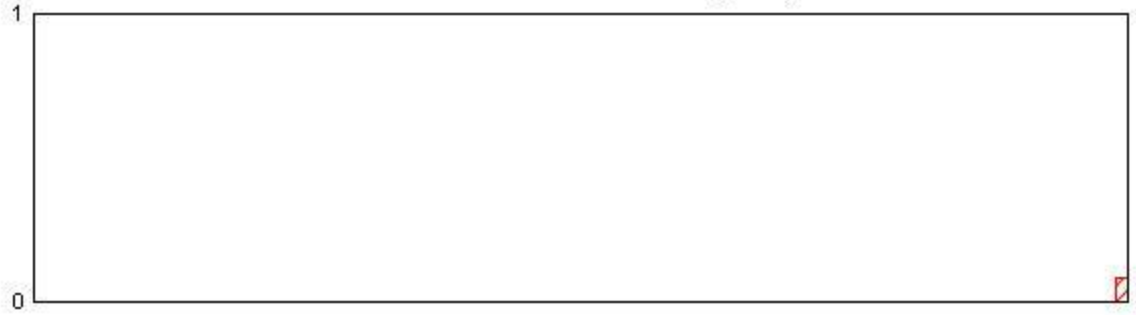
As (top)	0.65	0	6.191E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		DCONU1
As (bot)	0.47	1.02	0.86
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr

Shear Diagram (lb)

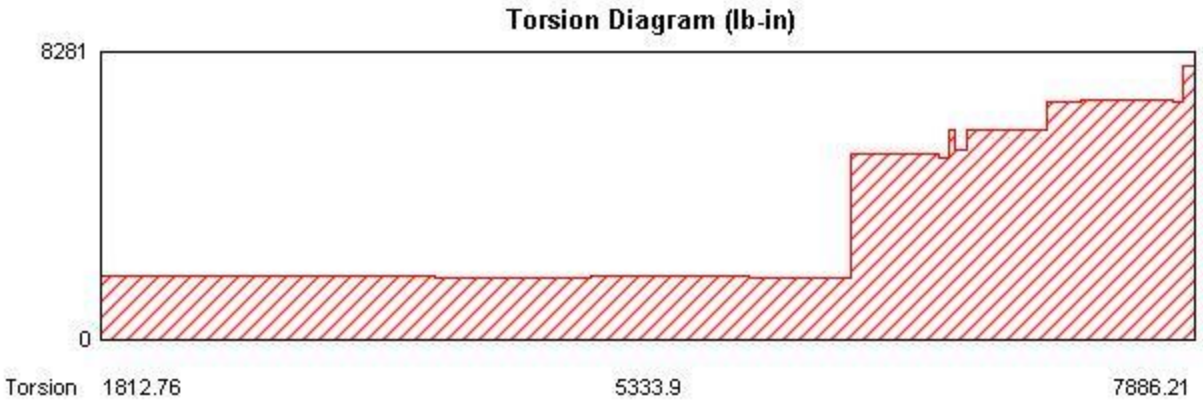


Shear	14708.96	7819.1	30712.68

Transverse Reinforcement (in2/ft)



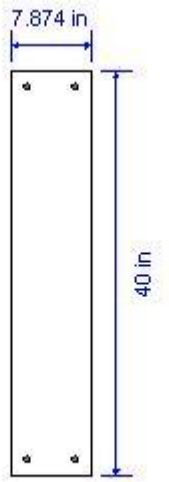
Asv/s reqd	0	0	8.004E-002
Combo	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr


Torsion Diagram (lb-in)
8281
0
Torsion
1812.76
5333.9
7886.21

ACI 318-14 Concrete Beam Design

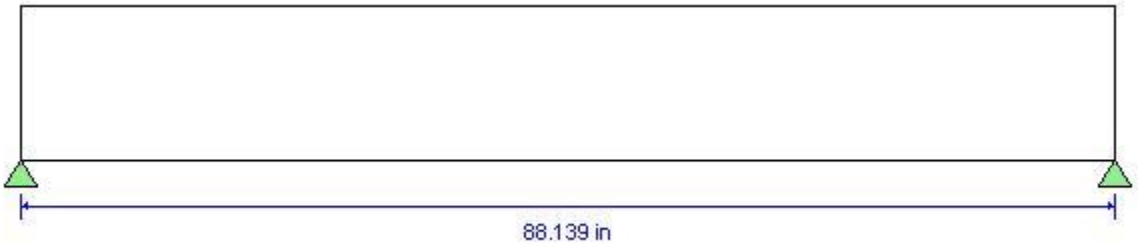
Geometric Properties

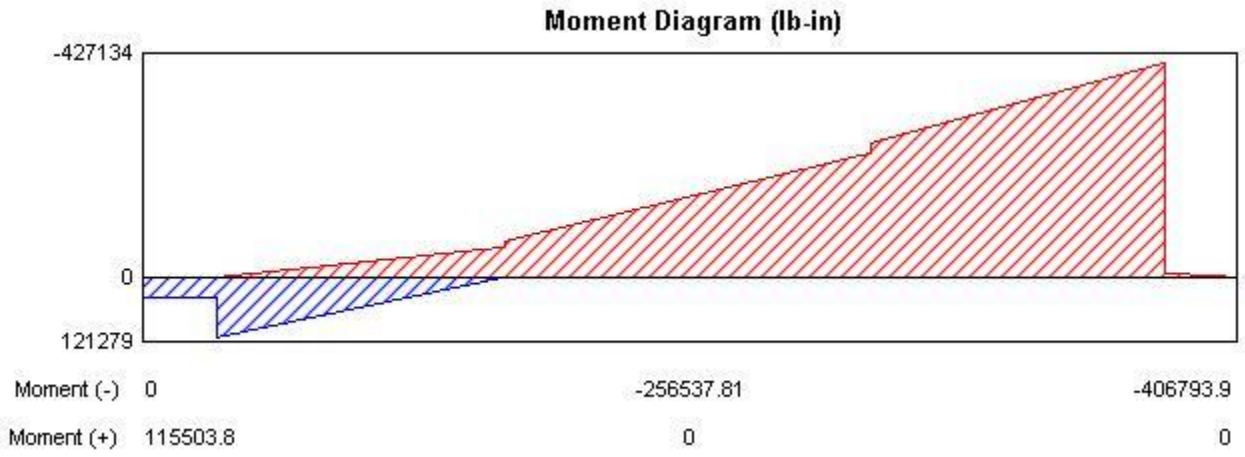
Combination = Overall Envelope
Beam Label = 117
Section Property = B10-8x40-4 ksi
Length = 88.139 in
Section Width = 7.874 in
Section Depth = 40 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



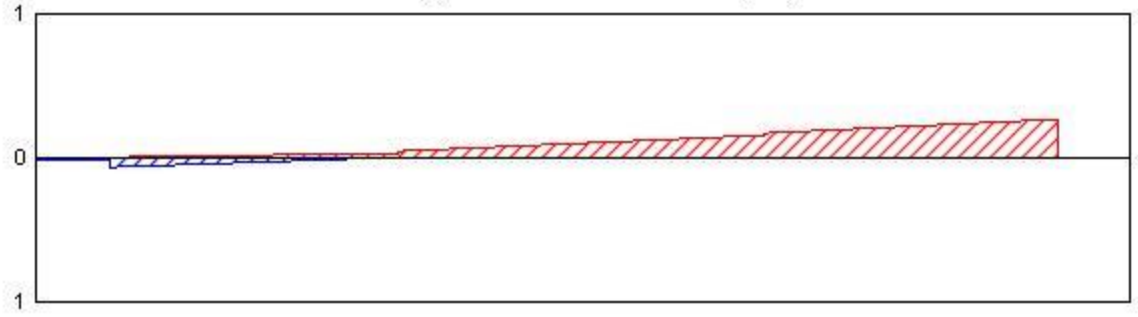
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



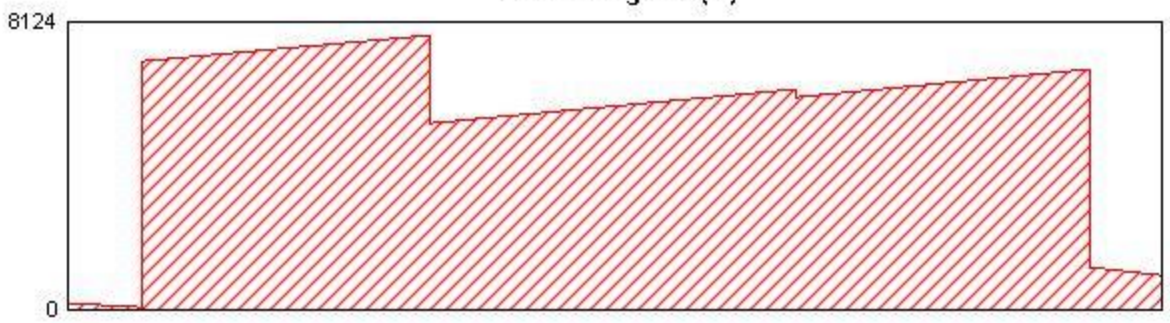


Longitudinal Reinforcement (in2)



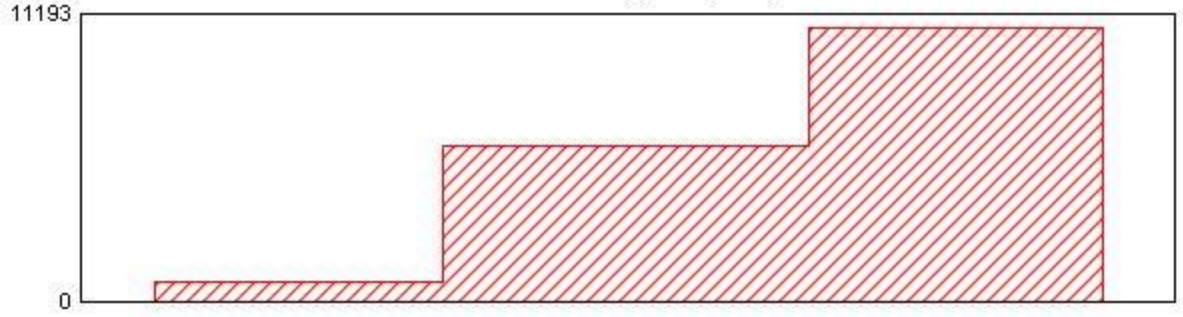
As (top)	0	0.13	0.2
Combo		Comb=1.2 D + 1.6 L +1.6 Lr	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	5.073E-002	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		

Shear Diagram (lb)



Shear	6972.1	7736.97	6765.48

Torsion Diagram (lb-in)

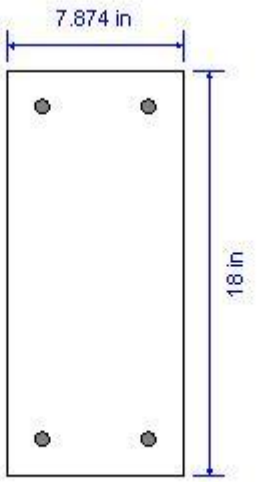


Torsion	782.31	10659.73	10659.73

ACI 318-14 Concrete Beam Design

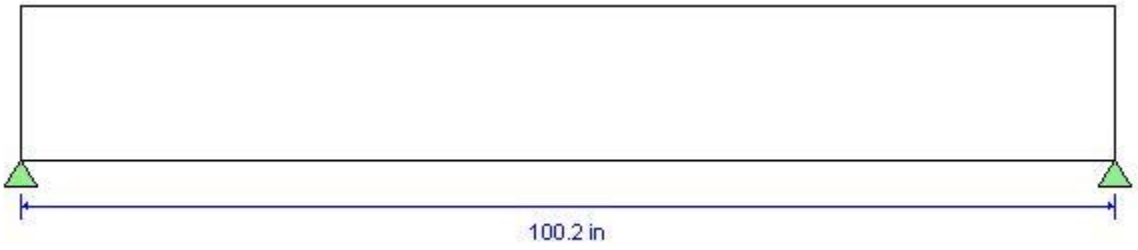
Geometric Properties

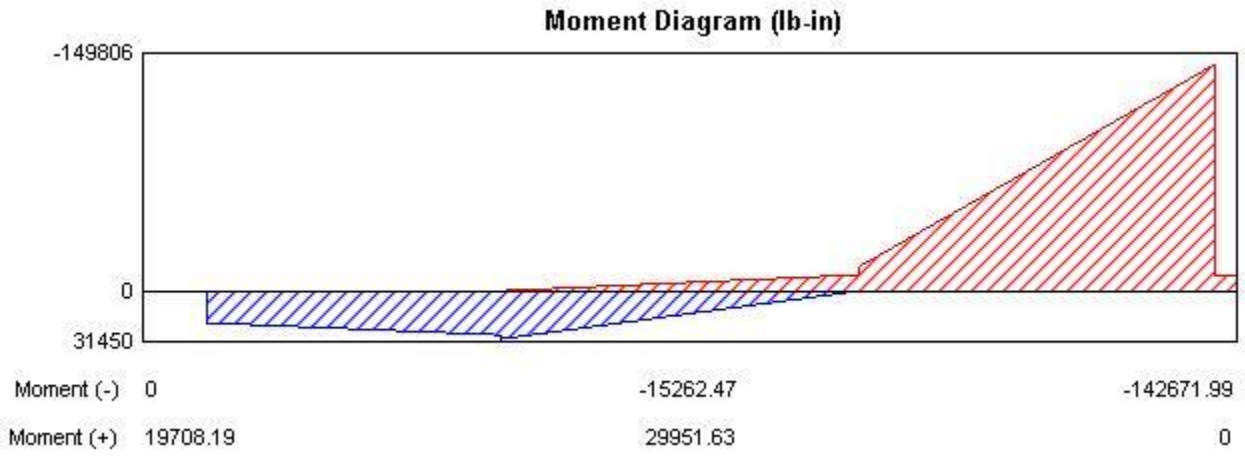
Combination = Overall Envelope
Beam Label = 119
Section Property = B7-8x18-4 ksi
Length = 100.2 in
Section Width = 7.874 in
Section Depth = 18 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



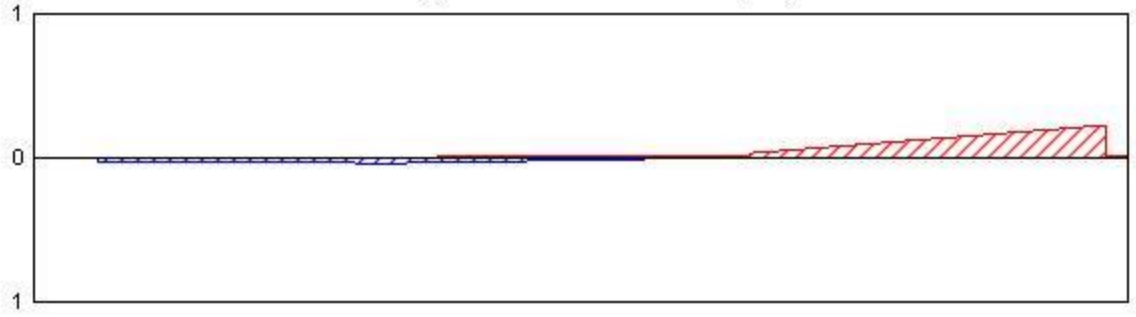
Material Properties

Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2



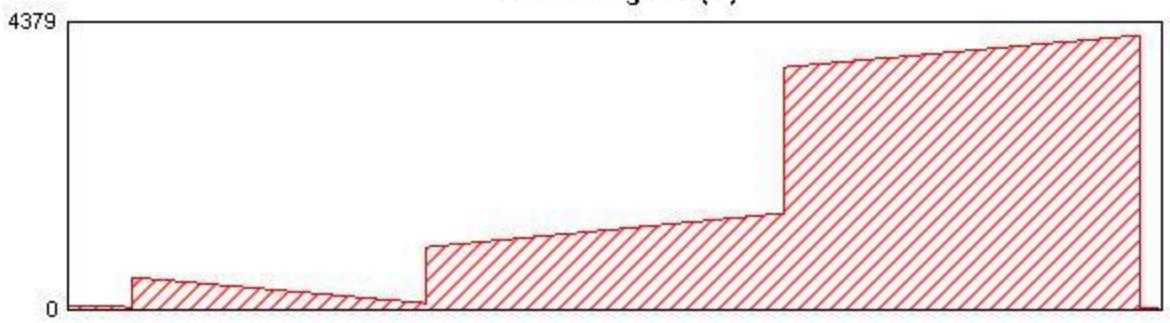


Longitudinal Reinforcement (in2)



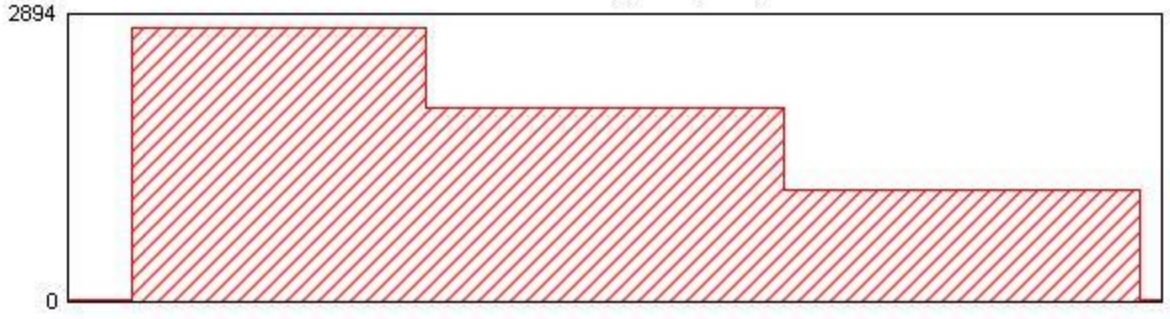
As (top)	0	1.668E-002	0.16
Combo		DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr
As (bot)	2.027E-002	3.28E-002	0
Combo	DCONU1	Comb=1.2 D + 1.6 L +1.6 Lr	

Shear Diagram (lb)



Shear	479.99	3686.35	4169.61

Torsion Diagram (lb-in)

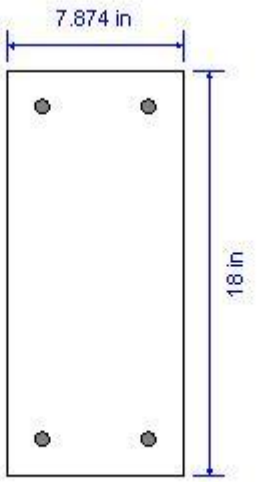


Torsion	2755.53	2755.53	1121.45

ACI 318-14 Concrete Beam Design

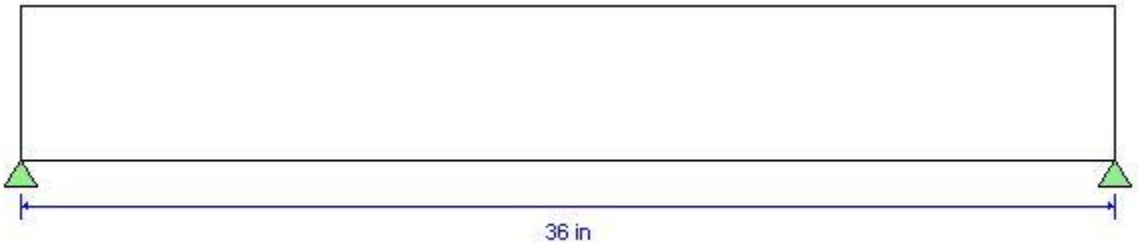
Geometric Properties

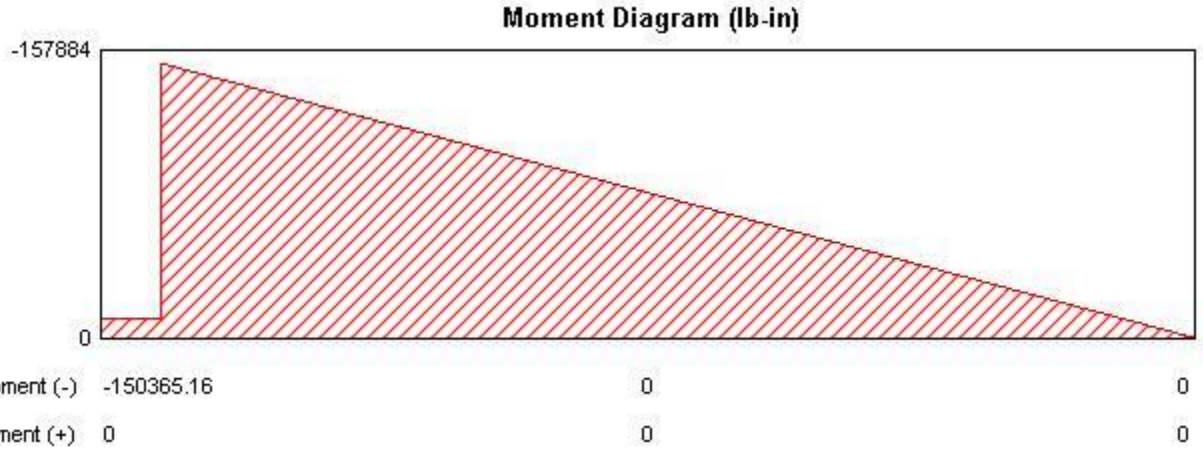
Combination = Overall Envelope
Beam Label = 120
Section Property = B7-8x18-4 ksi
Length = 36 in
Section Width = 7.874 in
Section Depth = 18 in
Distance to Top Rebar Center = 1.5748 in
Distance to Bot Rebar Center = 1.5748 in



Material Properties

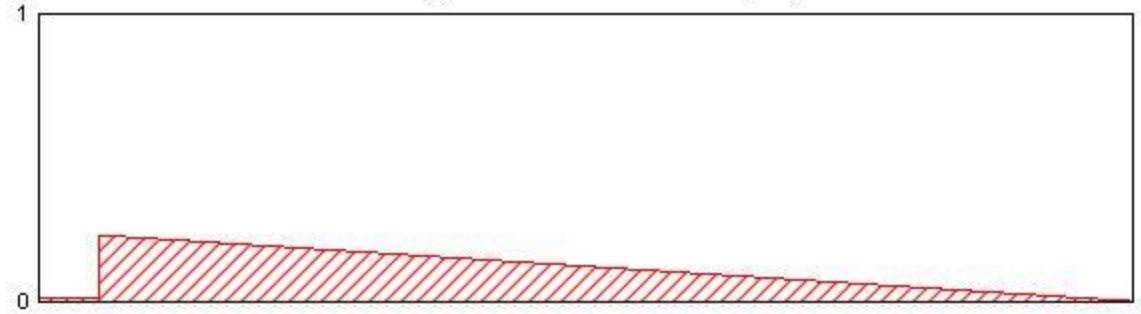
Concrete Comp. Strength = 4000 lb/in2
Concrete Modulus = 4030508.35 lb/in2
Longitudinal Rebar Yield = 60000 lb/in2
Shear Rebar Yield = 60000 lb/in2





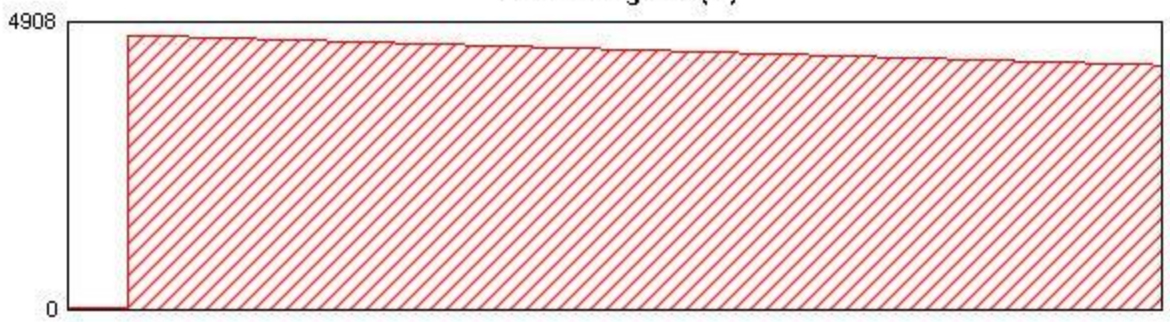
Moment (-)	-150365.16	0	0
Moment (+)	0	0	0

Longitudinal Reinforcement (in2)



As (top)	0.17	0	0
Combo	Comb=1.2 D + 1.6 L +1.6 Lr		
As (bot)	0	0	0
Combo			

Shear Diagram (lb)



Shear	4673.49	0	4171.52

Torsion Diagram (lb-in)

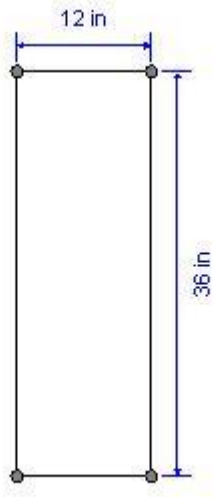


Torsion	21.57	0	0

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 66
Section Property =
Length = 87.6 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



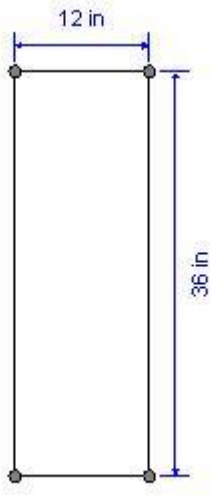
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 114
Section Property =
Length = 73.233 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



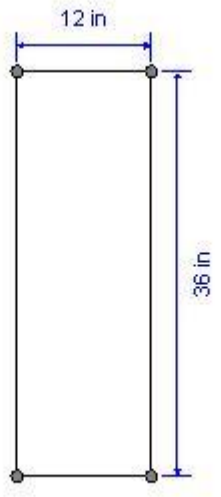
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 121
Section Property =
Length = 73.233 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



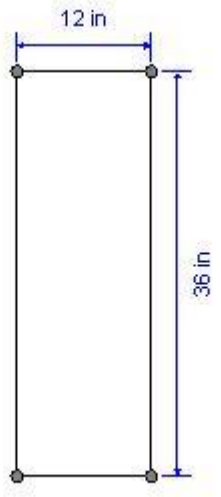
Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

ACI 318-14 Concrete Beam Design

Geometric Properties

Combination = Overall Envelope
Beam Label = 124
Section Property =
Length = 57.36 in
Section Width = 12 in
Section Depth = 36 in
Distance to Top Rebar Center = 3 in
Distance to Bot Rebar Center = 3 in



Material Properties

Concrete Comp. Strength = 4351.13 lb/in2
Concrete Modulus = 3867766.32 lb/in2
Longitudinal Rebar Yield = 58015.09 lb/in2
Shear Rebar Yield = 58015.09 lb/in2

6.5. Punching check/design

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 1 of 3

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 1 of 3

Point	GlobalX	GlobalY	Location	Perimeter	Depth	Status	Ratio
	in	in		in	in		
23	0.000	705.954				Not Calculated	
24	0.000	1017.320				Not Calculated	
60	133.920	705.954				Not Calculated	
67	340.920	705.954				Not Calculated	
128	215.760	1017.320				Not Calculated	
194	215.760	871.280				Not Calculated	
220	470.880	926.480				Not Calculated	
228	586.920	926.480				Not Calculated	
229	554.880	926.480				Not Calculated	
297	586.920	862.558				Not Calculated	
398	638.022	342.320				Not Calculated	
399	1014.102	342.320				Not Calculated	
407	758.040	862.558				Not Calculated	
413	758.040	926.480				Not Calculated	
424	758.040	779.758				Not Calculated	
431	758.040	705.954				Not Calculated	
441	906.960	779.758				Not Calculated	
446	906.960	818.278				Not Calculated	
447	966.000	818.278				Not Calculated	
448	1073.040	818.278				Not Calculated	
449	1132.080	818.278				Not Calculated	
477	1199.640	705.954				Not Calculated	
482	1199.640	906.920				Not Calculated	
496	1132.080	705.954				Not Calculated	
529	1395.600	906.920				Not Calculated	
535	1395.600	871.280				Not Calculated	
541	1395.600	779.758				Not Calculated	

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 1 of 3

Point	GlobalX	GlobalY	Location	Perimeter	Depth	Status	Ratio
	in	in		in	in		
580	1199.640	1043.120				Not Calculated	
586	1395.600	1007.120				Not Calculated	
599	1395.600	632.720				Not Calculated	
630	1331.160	445.760				Not Calculated	
667	906.960	445.760				Not Calculated	
691	41.880	-31.000				Not Calculated	
697	41.880	245.240				Not Calculated	
703	340.920	245.240				Not Calculated	
709	340.920	-31.000				Not Calculated	
727	41.880	278.360				Not Calculated	
766	1199.640	445.760				Not Calculated	
767	446.040	386.840				Not Calculated	
1236	1014.102	386.840				Not Calculated	
1287	340.920	278.360				Not Calculated	
1590	1331.160	386.840				Not Calculated	
1597	906.960	342.320				Not Calculated	
2373	446.040	278.360				Not Calculated	
2378	446.040	445.760				Not Calculated	
2379	446.040	357.622				Not Calculated	
2387	499.673	445.760				Not Calculated	
2394	618.000	612.440				Not Calculated	
2397	586.920	445.760				Not Calculated	
2405	638.022	445.760				Not Calculated	
2410	906.960	632.720				Not Calculated	
2411	1041.960	632.720				Not Calculated	
2412	1172.400	632.720	Interior	42.0002	6.5000	OK	0.713246
3302	340.920	589.674	Interior	82.1602	6.5000	OK	0.458298
3303	340.920	482.154	Interior	82.1602	6.5000	OK	0.612909
3315	586.920	743.240	Interior	42.0002	6.5000	OK	0.914766

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 2 of 3

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 2 of 3

Point	Combo	Vu	Gamma_v2	Gamma_v3	ShrStrMax	Mu2	ShrStrCap
		lb			lb/in2	lb-in	lb/in2
23							
24							
60							
67							
128							
194							
220							
228							
229							
297							
398							
399							
407							
413							
424							
431							
441							
446							
447							
448							
449							
477							
482							
496							
529							
535							
541							
580							
586							
599							
630							
667							
691							
697							
703							
709							
727							
766							
767							
1236							
1287							
1590							
1597							
2373							
2378							
2379							
2387							
2394							
2397							
2405							
2410							
2411							
2412	DCONU2	36424.45	0.400000	0.400000	135.33	-4334.23	189.74

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 2 of 3

Point	Combo	Vu	Gamma_v2	Gamma_v3	ShrStrMax	Mu2	ShrStrCap
		lb			lb/in2	lb-in	lb/in2
3302	DCONU2	45894.40	0.400000	0.400000	86.96	1306.87	189.74
3303	DCONU2	60443.75	0.400000	0.400000	116.29	-979.84	189.74
3315	DCONU2	45595.68	0.400000	0.400000	173.56	-1395.88	189.74

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 3 of 3

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 3 of 3

Point	Mu3	ReinfType	UnbalMu2	UnbalMu3	NumRails	StudPerRail
	lb-in		lb-in	lb-in		
23		None				
24		None				
60		None				
67		None				
128		None				
194		None				
220		None				
228		None				
229		None				
297		None				
398		None				
399		None				
407		None				
413		None				
424		None				
431		None				
441		None				
446		None				
447		None				
448		None				
449		None				
477		None				
482		None				
496		None				
529		None				
535		None				
541		None				
580		None				
586		None				
599		None				
630		None				
667		None				
691		None				
697		None				
703		None				
709		None				
727		None				
766		None				
767		None				
1236		None				
1287		None				
1590		None				
1597		None				
2373		None				

Table 47: Concrete Slab Design 02 - Punching Shear Data, Part 3 of 3

Point	Mu3	ReinfType	UnbalMu2	UnbalMu3	NumRails	StudPerRail
	lb-in		lb-in	lb-in		
2378		None				
2379		None				
2387		None				
2394		None				
2397		None				
2405		None				
2410		None				
2411		None				
2412	658.52	None	-1733.69	263.41		
3302	8236.50	None	522.75	3294.60		
3303	28161.98	None	-391.94	11264.79		
3315	15747.33	None	-558.35	6298.93		

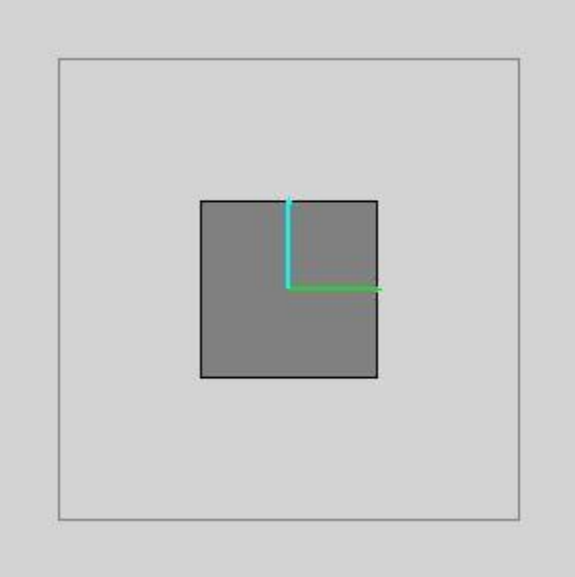
ACI 318-14 Punching Shear Check & Design

Geometric Properties

Combination = 1
Point Label = 2412
Column Shape = Rectangular
Column Location = Interior
Global X-Coordinate = 1172.4 in
Global Y-Coordinate = 632.72 in

Column Punching Check

Avg. Eff. Slab Thickness = 6.5 in
Eff. Punching Perimeter = 42.0002 in
Cover = 1.5 in
Conc. Comp. Strength = 4000 lb/in2
Reinforcement Ratio = 0.0000
Section Inertia I22 = 5497.07 in4
Section Inertia I33 = 5497.07 in4
Section Inertia I23 = 0 in4
Gamma_v2 = 0.4
Gamma_v3 = 0.4
Moment Mu2 = -4334.23 lb-in
Moment Mu3= 658.52 lb-in
Shear Force = 36424.45 lb
Unbalanced Moment Mu2 = -1733.69 lb-in



Column Punching Perimeter

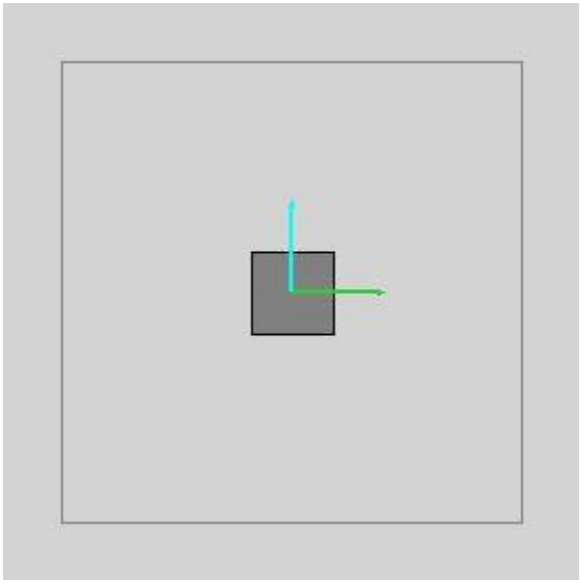
ACI 318-14 Punching Shear Check & Design

Geometric Properties

Combination = 1
Point Label = 3302
Column Shape = Rectangular
Column Location = Interior
Global X-Coordinate = 340.92 in
Global Y-Coordinate = 589.674 in

Column Punching Check

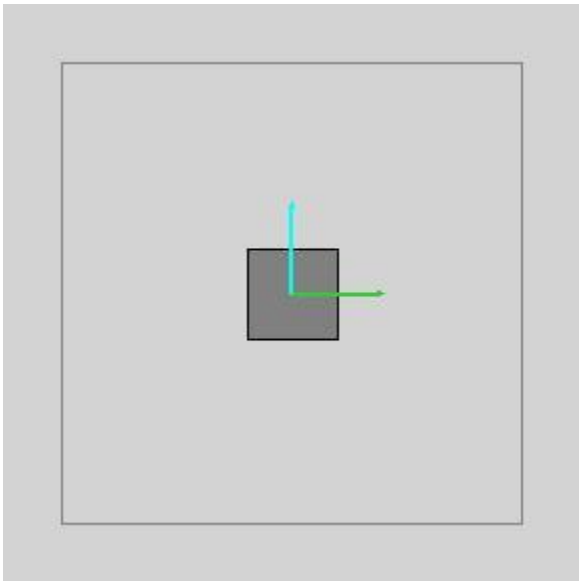
Avg. Eff. Slab Thickness = 18.5 in
Eff. Punching Perimeter = 90 in
Cover = 1.5 in
Conc. Comp. Strength = 3000 lb/in2
Reinforcement Ratio = 0.0000
Section Inertia I22 = 164227.97 in4
Section Inertia I33 = 164227.97 in4
Section Inertia I23 = 0 in4
Gamma_v2 = 0.4
Gamma_v3 = 0.4
Moment Mu2 = 1315.67 lb-in
Moment Mu3= 8236.5 lb-in
Shear Force = 45894.4 lb
Unbalanced Moment Mu2 = 526.27 lb-in



Column Punching Perimeter

Drop Panel Punching Check

Unbalanced Moment Mu3= 3294.6 lb-in
Max Design Shear Stress = 27.83 lb/in2
Conc. Shear Stress Capacity = 164.32 lb/in2
Punching Shear Ratio = 0.17
Avg. Eff. Slab Thickness = 6.5 in
Eff. Punching Perimeter = 82.1602 in
Cover = 1.5 in
Conc. Comp. Strength = 4000 lb/in2
Reinforcement Ratio = 0.000
Section Inertia I22 = 38491.76 in4
Section Inertia I33 = 38491.76 in4
Section Inertia I23 = 0 in4
Shear Force = 45894.4 lb
Unbalanced Moment Mu2 = 522.75 lb-in



Drop Panel Punching Perimeter

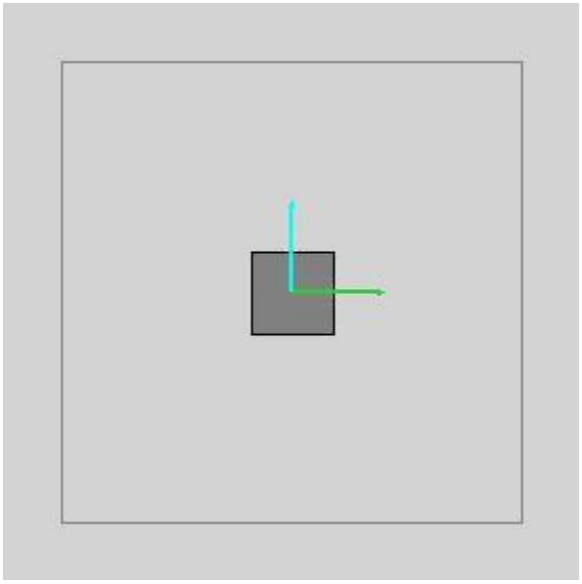
ACI 318-14 Punching Shear Check & Design

Geometric Properties

Combination = 1
Point Label = 3303
Column Shape = Rectangular
Column Location = Interior
Global X-Coordinate = 340.92 in
Global Y-Coordinate = 482.154 in

Column Punching Check

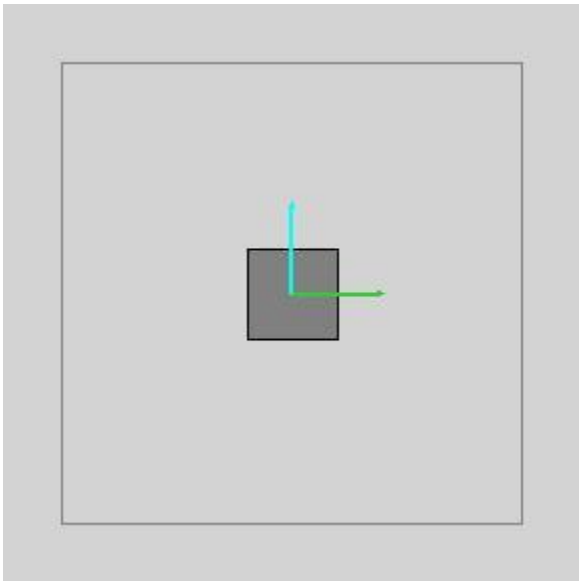
Avg. Eff. Slab Thickness = 18.5 in
Eff. Punching Perimeter = 90 in
Cover = 1.5 in
Conc. Comp. Strength = 3000 lb/in2
Reinforcement Ratio = 0.0000
Section Inertia I22 = 164227.97 in4
Section Inertia I33 = 164227.97 in4
Section Inertia I23 = 0 in4
Gamma_v2 = 0.4
Gamma_v3 = 0.4
Moment Mu2 = -968.25 lb-in
Moment Mu3= 28161.98 lb-in
Shear Force = 60443.75 lb
Unbalanced Moment Mu2 = -387.3 lb-in



Column Punching Perimeter

Drop Panel Punching Check

Unbalanced Moment Mu3= 11264.79 lb-in
Max Design Shear Stress = 37.1 lb/in2
Conc. Shear Stress Capacity = 164.32 lb/in2
Punching Shear Ratio = 0.23
Avg. Eff. Slab Thickness = 6.5 in
Eff. Punching Perimeter = 82.1602 in
Cover = 1.5 in
Conc. Comp. Strength = 4000 lb/in2
Reinforcement Ratio = 0.000
Section Inertia I22 = 38491.76 in4
Section Inertia I33 = 38491.76 in4
Section Inertia I23 = 0 in4
Shear Force = 60443.75 lb
Unbalanced Moment Mu2 = -391.94 lb-in



Drop Panel Punching Perimeter

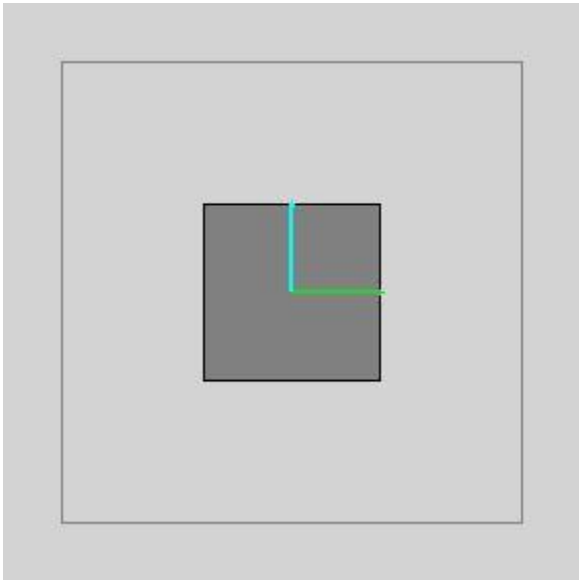
ACI 318-14 Punching Shear Check & Design

Geometric Properties

Combination = 1
Point Label = 3315
Column Shape = Rectangular
Column Location = Interior
Global X-Coordinate = 586.92 in
Global Y-Coordinate = 743.24 in

Column Punching Check

Avg. Eff. Slab Thickness = 6.5 in
Eff. Punching Perimeter = 42.0002 in
Cover = 1.5 in
Conc. Comp. Strength = 4000 lb/in2
Reinforcement Ratio = 0.0000
Section Inertia I22 = 5497.07 in4
Section Inertia I33 = 5497.07 in4
Section Inertia I23 = 0 in4
Gamma_v2 = 0.4
Gamma_v3 = 0.4
Moment Mu2 = -1395.88 lb-in
Moment Mu3= 15747.33 lb-in
Shear Force = 45595.68 lb
Unbalanced Moment Mu2 = -558.35 lb-in



Column Punching Perimeter

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

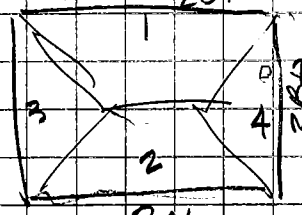
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Second Floor Slabs hand Verification

S9 – S10 (NOT same horiz plane)

S9



DL = 100 psf (8" THK)

AOL = 20 psf

LL = 30 psf

VTL = 200 psf

$\xi = \frac{L_y}{L_x} = \frac{12}{16} = 0.75$

$M_x = 0.0318 \times 0.200 \times 12^2 = 0.92$ k/f

$M_y = 0.0619 \times 0.200 \times 12^2 = 1.78$ k/f

$A_s = \frac{1.78}{4 \times 6} = 0.07$ in²/f

$A_{s_{min}} = 0.0018 \times 12 \times 8 = 0.17$

#4 @ 12" OK

$W_{d_1} = W_{d_2} = 0.3125 \times 16 \times 12/16 \times .120 = 0.45$ KLF

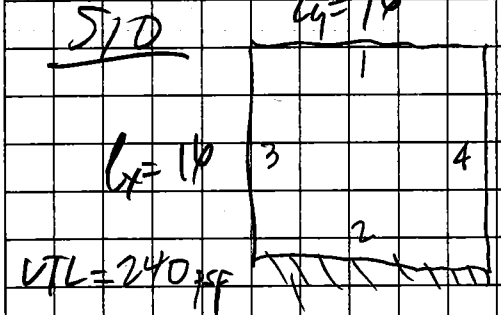
$W_{d_3} = W_{d_4} = 0.1875 \times .120 \times 16 \times 12/12 = 0.36$ KLF

$W_{L_1} = W_{L_2} = 0.3125 \times 0.03 \times 16 \times 2/16 = 0.113$ KLF

$W_{L_3} = W_{L_4} = 0.1875 \times 0.03 \times 16 \times 12/12 = 0.090$ KLF

A_{sb}

S10



$M_x = 0.0318 \times .240 \times 16^2 = 1.95$ k/f

$M_y = 0.0243 \times .240 \times 16^2 = 1.49$ k/f

$M_x^- = 0.0829 \times .240 \times 16^2 = 5.15$ k/f

$A_s = \frac{1.95}{4 \times 6} = 0.08$ in²/f

$A_s^- = \frac{5.15}{4 \times 6.5} = 0.20$ in²/f #4 @ 12" T

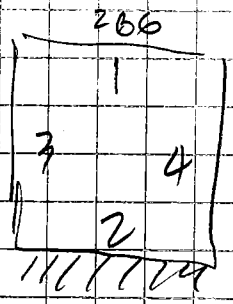
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Slab S-10 reactions



$W_{d_1} = 0.232 \times .140 \times 16 = 0.52$ KLF

$W_{d_2} = 0.402 \times .140 \times 16 = 0.91$ KLF

$W_{d_3} = 0.183 \times .140 \times 16 = 0.41$ KLF

$W_{d_4} = 0.183 \times .140 \times 16 = 0.41$ KLF

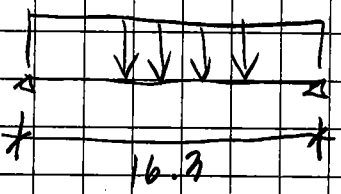
$W_{L_1} = 0.232 \times .04 \times 16 = 0.15$ KLF

$W_{L_2} = 0.402 \times .040 \times 16 = 0.26$ KLF

$W_{L_3} = 0.183 \times .04 \times 16 = 0.12$ KLF

$W_{L_4} = 0.183 \times .04 \times 16 = 0.12$ KLF

Beam 286: 8"x28"

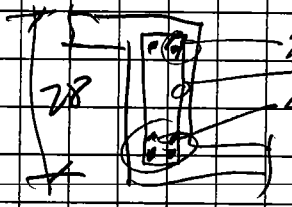


$R_D = 17^K$

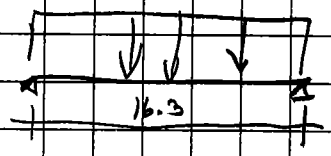
$R_L = 3^K$

$D = 0.52 + 0.45 + 0.80 = 1.77$ KLF

$L = 0.15 + 0.11 + 0.15 = 0.42$ KLF



Beam 287: 8x18

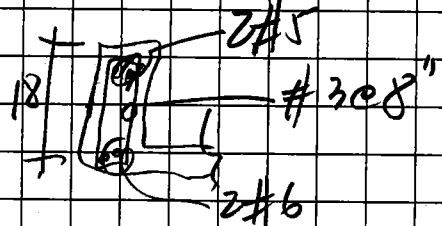


$R_D = 6^K$

$R_L = 2^K$

$D = 0.45$ KLF

$L = 0.11$ KLF



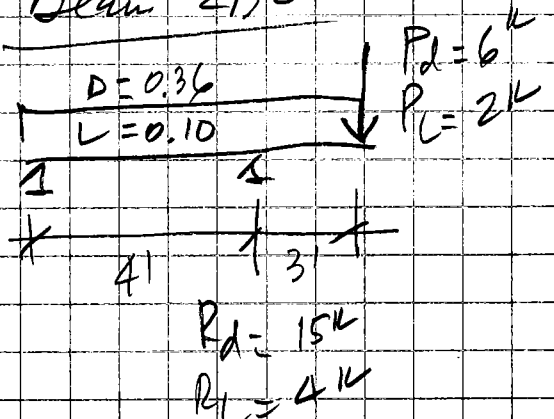
GF CONSULTING ENGINEERS, Inc.

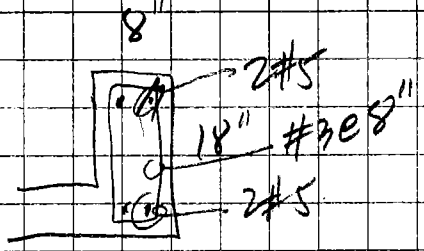
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

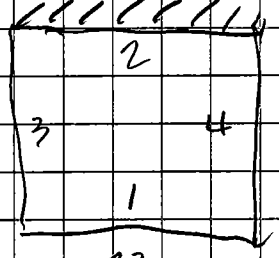
Beam 2B8





SLAB S4

DL = 120 LL = 30 UTL = 0.200 KSF



$M_x^e = -0.0839 \times .20 \times 23^2 = 8.88$ 'K/F

$M_x = 0.0318 \times .20 \times 23^2 = 3.36$ 'K/F

$M_y = 0.0243 \times .20 \times 23^2 = 2.57$ 'K/F

$A_{s+}^x = \frac{3.36}{4 \times 6.5} = 0.13$ in²/F #4@12" o/c

$A_s^- = \frac{8.88}{4 \times 6.5} = 0.34$ in²/F #5@10" Top.

$W_{D1} = 0.232 \times .120 \times 23 = 0.64$

$W_{D2} = 0.402 \times .120 \times 23 = 1.11$

$W_{D3} = W_{D4} = 0.183 \times .120 \times 23 = 0.51$

$W_{L1} = 0.232 \times 0.03 \times 23 = 0.16$

$W_{L2} = 0.402 \times 0.03 \times 23 = 0.28$

$W_{L3} = W_{L4} = 0.183 \times 0.03 \times 23 = 0.13$

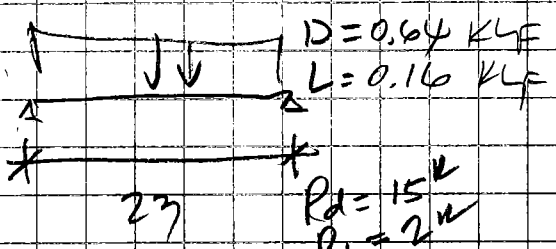
GF CONSULTING ENGINEERS, Inc.

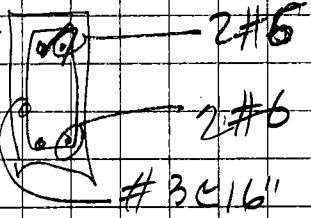
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

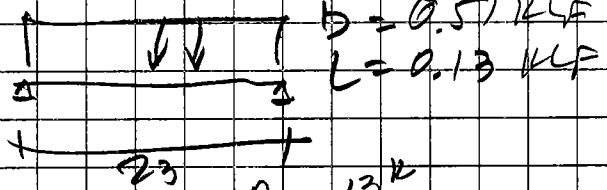
ENG: _____ SHEET: _____ OF _____

2B19 - 12x40



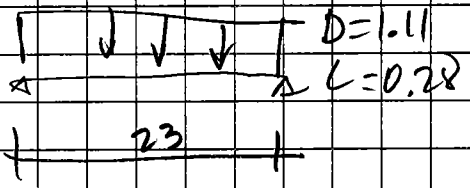


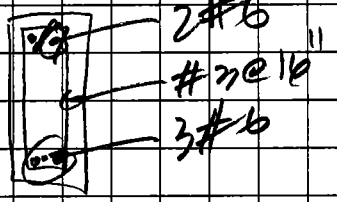
2B20: 12x40



2#6T
2#6B
#3@16"

2B18: 12x40





GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

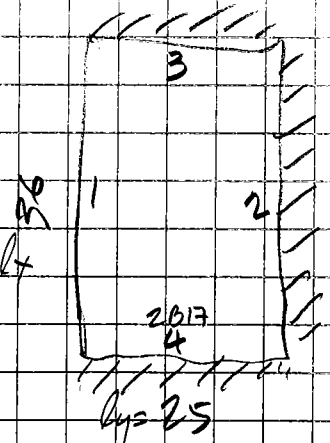
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Slab S3

DL = 100
SDL = 40
LL = 40

UTL = 240 psf



$\frac{l_y}{l_x} = \frac{25}{36} = 0.70$

$M_x^e = -0.0568 \times .240 \times 25^2 = 8.52$ k/f
$M_y^e = -0.0731 \times 150 = 10.97$ k/f
$M_x = 0.0114 \times 150 = 1.71$
$M_y = 0.0322 \times 150 = 4.83$ k/f

$A_s^y = \frac{4.83}{4 \times 6.5} = 0.19$ in²/f #4@12 ew B.tt.

$A_{sx}^- = \frac{8.52}{4 \times 6.5} = 0.33$ in²/f

$A_{sy}^- = \frac{10.97}{4 \times 6.5} = 0.42$ in²/f #5@8"

$W_{D1} = 0.204 \times .140 \times 25 = 0.71$ k/f
$W_{D2} = 0.354 \times .140 \times 25 = 1.24$
$W_{D3} = W_{D4} = 0.221 \times .140 \times 36 = 1.11$
$W_{L1} = 0.204 \times .040 \times 25 = 0.21$
$W_{L2} = 0.34 \times 0.04 \times 25 = 0.34$
$W_{L3} = W_{L4} = 0.221 \times 0.04 \times 36 = 0.32$

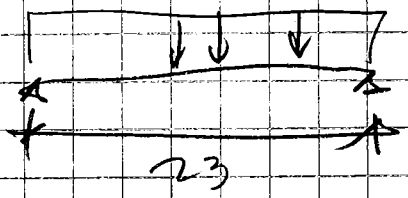
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Beam 2B17 8x44 upt



$R_d = 29^k$

$R_L = 4^k$

D = 1.11 + 0.80 = 1.91 KLF

L = 0.32 + 0.150 = 0.47 KLF

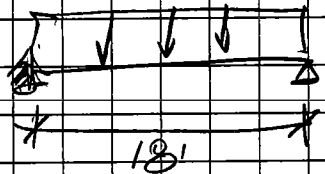
4#6B
2#6T
#3@16"

Beam 2B1 8x38 4pt

Load Fm S2: D = 140 x 6' = 0.84 KLF
L = 40 x 6' = 0.24

Load Fm S3: D = 1.11 KLF
L = 0.32

Load Fm Roof: D = 0.80 KLF
L = 0.15



D = 2.75 KLF
L = 0.75 KLF

4#6T&B
#3@12

$R_D = 37^k$ $R_D = 22^k$

$R_L = 7^k$ $R_L = 5^k$

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

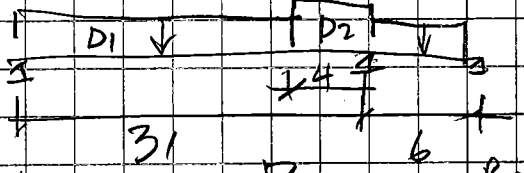
ENG: _____ SHEET: _____ OF _____

Beam 2B2A-B : 8x26

From S14: $D_1 = 140 \times 3' = 420 + 30 = 450$ plf

$L_1 = 40 \times 3' = 120 + 50 = 170$ plf

4#5B
4#6T
#3 @ 12"



$R_D = 10$ $R_L = 2$

$R_D = 33$ $R_L = 8$

$R_D = -11$ $R_L = -3$

$D_2 = 450 + 675 = 1125$

$L_2 = 170 + 180 = 352$

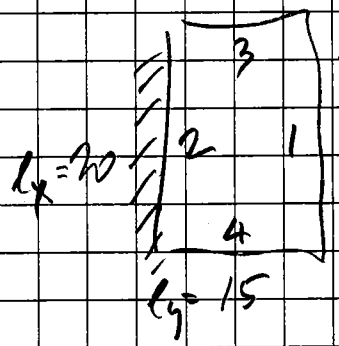
FROM STAIR

Slab S6

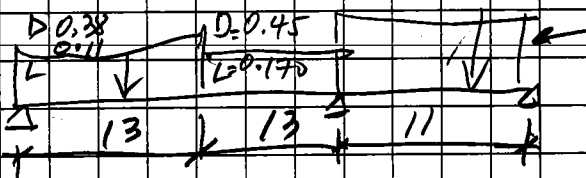
$W_{D_4} = 0.137 \times 0.140 \times 20 = 0.38$ klf

$W_{L_4} = 0.137 \times 0.04 \times 20 = 0.11$ klf

$\frac{l_y}{l_x} = \frac{15}{20} = 0.75$



Beam 2B3A-B 8x26



$D = 7.3$ k

$L = 1.7$ k

2.7 k
6 k

6 k
2 k

$D = 140 \times 11 = 1540$

$L = 40 \times 11 = 440$

4#5B
4#6T
#3 @ 12"

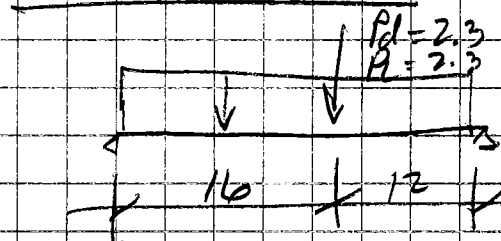
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Beam 2B12 8×52



From Roof
D = 1200
L = 150

From Floor
D = 140×11 = 1540
L = 40×11 = 440

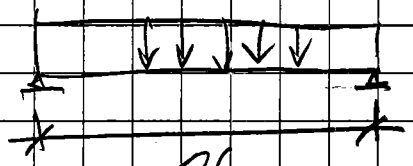
D 45k 42k
L 6k 7k

8" × 52"

4#7 B
4#6 T

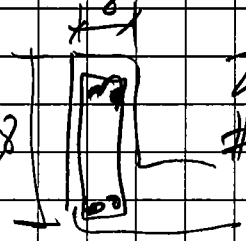
#3@16"

Beam 2B16 8×28



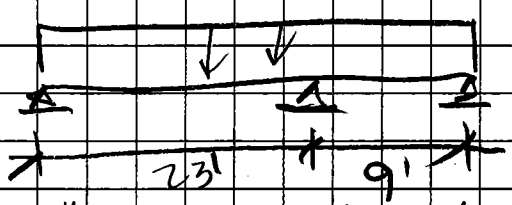
D = 2.5' × 100 =
L = 2.5 × 30 =

D 7k 7k
L 1k 1k



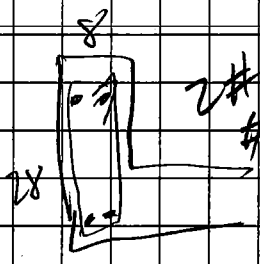
2#6 T&B
#3@12

Beam 2B13 – 2B14 8×28



D = 4.5 × 100
L = 4.5 × 30

D 7k 19k 1
L 2k 3k 1



2#6 T&B
#3@12

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

2B10 – 2B11 12×40

D=425
L=425

D = 40×140
L = 40×40

D=400
L

4.5×100 = D
4.5×30 = L

13 24

P 8k 13k
L 2k 5k 3

3#6 T&B
#3@12"

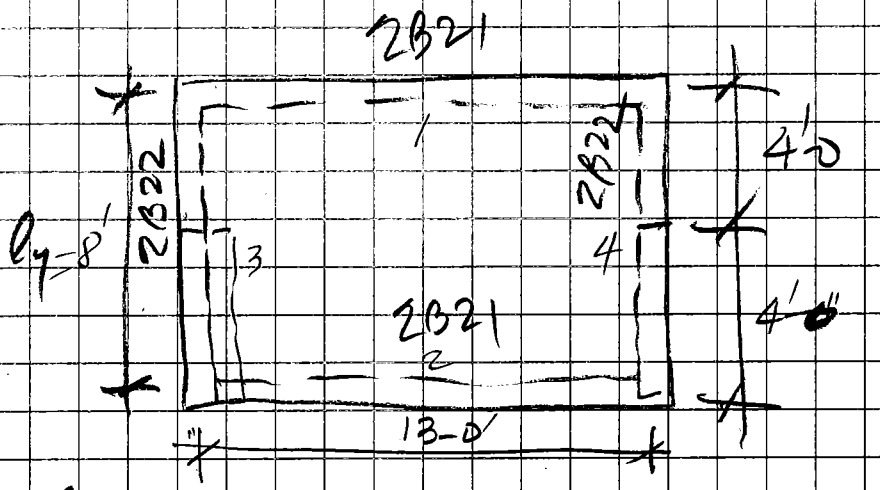
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Front Entrance Canopy (Low Canopy)



Slab Thickness

$$h = \frac{1.1 \times 13}{36 + 5(\frac{13}{8.3})} = 3.9''$$

Make it 5"

DL = 63 } 75 psf
ADL = 12 }
LL = 30 psf

$$\frac{l_y}{l_x} = \frac{8}{13} = 0.61$$

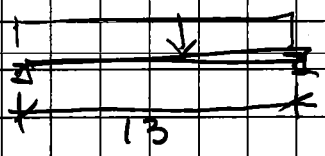
$W_{D1} = W_{D2} = 0.35 \times 0.075 \times 8 = 0.210$ K/F

$W_{D3} = W_{D4} = 0.150 \times 0.075 \times 13 = 0.15$

$W_{L1} = W_{L2} = 0.35 \times 0.030 \times 8 = 0.10$

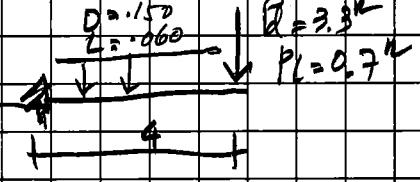
$W_{L3} = W_{L4} = 0.150 \times 0.030 \times 13 = 0.06$

Bm 1 & 2: 8x30"



2#5 T&B #3@12" o/c
R_d=3.3K R_L=0.7K
2#4 EF

Bm 3 & 4: 8x30'



D → 3.3K 10K

L → 0.7 2

2#6 T&B #3@12" o/c
2#4 EF

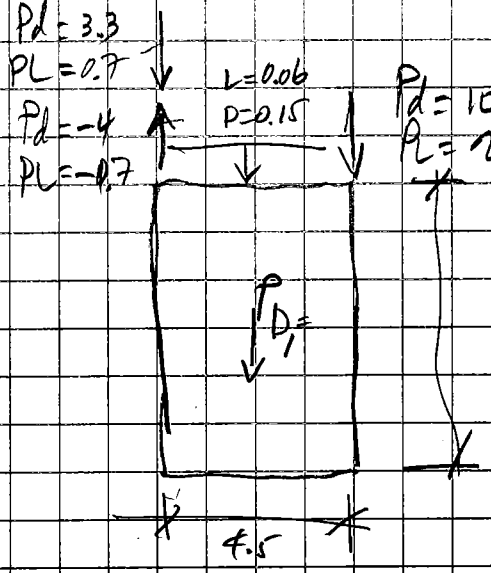
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PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Column CCS & Footings



$M_D = 10 \times 2.25' = 22.5^{'k}$

$M_L = 2 \times 2.25' = 4.5^{'k}$

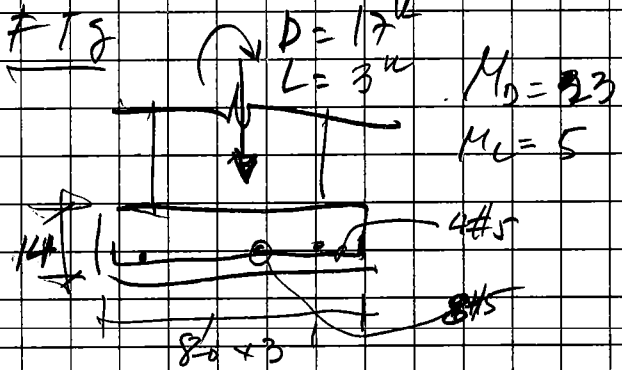
$P_{d_1} = 0.66 \times 4.5 \times 14 \times .15 = 6.25^{k}$

$\sum D = 10 + 6.25 + 4.5 \times 0.15 \cong 17^{k}$

$\sum L = 0.15 \times 4.5' + 2^{k} \cong 2.7^{k}$

~~8"x53" w/ 4#5 Long~~

FTG



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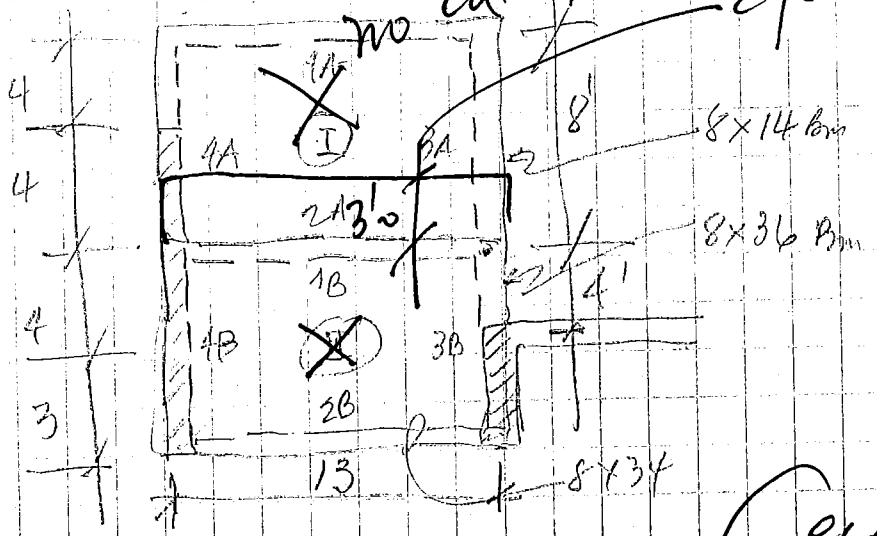
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Upper front Canopy

Upper canopy eliminated & Just eyebrow.



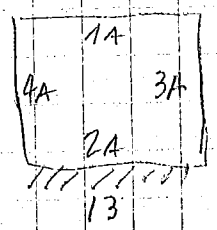
DL = 75 psf
LL = 30 psf

Uplift: -180.7 x .6
= 108 psf

Bm 3A-3B
= -140 x .6
= 84 psf

Load Distrib. (I)

no slabs



$W_{D1A} = 0.286 \times 75 \times 8 = 172$ plf

$W_{D2A} = 0.414 \times 75 \times 8 = 300$ plf

$W_{D3-4A} = 0.11 \times 75 \times 13 = 108$ plf

eyebrow

DL = 75 psf x 3 = 225 plf

LL = W_{L1A} = 30 psf x 3 = 90 plf

$W_{L1A} = 0.286 \times 30 \times 8 = 70$ plf

$W_{L2A} = 0.414 \times 30 \times 8 = 120$ plf

$W_{L3-4A} = 0.11 \times 30 \times 13 = 45$ plf

Load Distrib (II) Mirror (I)

Beam 2A-1B : 8x27 (RB2)

D = 400 #
L = 40 psf

D = 225 + 170 = 395 plf

L = 240 + 170 = 410 plf

90

260 plf

from roof

13'

5k
3k

5k
3k

8x27 2#6 T&B
#3 @ 12"
#3 @ 6"
3 #4 ef

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Beam 3A : 3D

RB4

$\Delta_{LT} = 0.78$

$\frac{L}{480} = \frac{144}{480} = 0.30$

$P_D = 110^k$
$P_L = 50$

$P_D = 8.4^k$
$P_L = 0.4^k$

$P_D = 5^k$
$P_L = 2^k$

NG

32

3 4

with 14"

$\Delta_{LT} = 0.40$ w/14" $\quad \Delta_{LT} = 0.19$ w/18"D

$M_D = 110 \times 8 \div 2 + 2.4 \times 8 + 150 \times 8 \div 2 = 27.5^{'k}$

$M_L = 50 \times 8 \div 2 + 0.4 \times 8 = 4.8^{'k}$

$D = 2.4 + .150 \times 8 = 3.6^k$
$L = 0.4 + .050 \times 8 = 0.8^k$

$\text{Max } \Delta_{LT} = \frac{L}{480} = \frac{144}{480} = 0.30$

$P_D = 3.6 + 8 = 11.6^k$
$P_L = 0.4 + 3 = 3.4^k$

4 4

$\Delta_{LT} = 0.03$" $\quad \Delta_{LT} = 0.04$"

TOTAL $\Delta_{LT} = 0.40 + 0.03 = 0.43$ NG.

with 18" Trial:
$\Delta = 0.19 + 0.04$
$= 0.23$
$\delta = \frac{L}{626} > \frac{L}{480}$ OK

14"D

Trial Beam needs $\text{Max } \Delta_{LT} = 0.27$

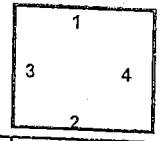
Make it $8 \times 18 \rightarrow \Delta_{LT} = 0.19$

LOSAS RECTANGULARES APOYADAS EN SU PERÍMETRO

CARGA UNIFORMEMENTE REPARTIDA TOTAL

DESCARGAS TOTALES SOBRE LOS APOYO.

P total = coeficiente . p . Lx . Ly



$\varepsilon = \frac{Ly}{Lx}$

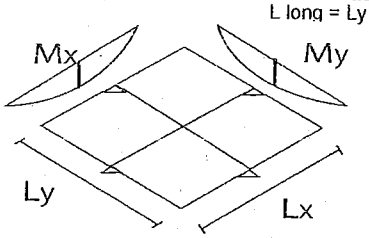
		0,50	0,55	0,60	0,65	0,70	0,75	0,80	0,85	0,90	0,95	1,00	1,10	1,20	1,30	1,40	1,50	1,60	1,70	1,80	1,90	2,00
Ly/Lx ①	1	0,375	0,3625	0,350	0,3375	0,325	0,3125	0,300	0,2875	0,275	0,2625	0,250	0,227	0,208	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	2	0,375	0,3625	0,350	0,3375	0,325	0,3125	0,300	0,2875	0,275	0,2625	0,250	0,227	0,208	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	3	0,125	0,1375	0,150	0,1625	0,175	0,1875	0,200	0,2125	0,225	0,2375	0,250	0,273	0,292	0,308	0,321	0,333	0,344	0,353	0,361	0,368	0,375
	4	0,125	0,1375	0,150	0,1625	0,175	0,1875	0,200	0,2125	0,225	0,2375	0,250	0,273	0,292	0,308	0,321	0,333	0,344	0,353	0,361	0,368	0,375
Ly/Lx ②	1	0,299	0,292	0,286	0,279	0,272	0,266	0,259	0,252	0,245	0,239	0,232	0,219	0,205	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	2	0,517	0,506	0,494	0,483	0,472	0,460	0,449	0,436	0,425	0,413	0,402	0,379	0,355	0,332	0,309	0,289	0,272	0,255	0,241	0,228	0,217
	3	0,092	0,101	0,110	0,119	0,128	0,137	0,146	0,156	0,165	0,174	0,183	0,201	0,220	0,238	0,256	0,272	0,286	0,299	0,310	0,320	0,329
	4	0,092	0,101	0,110	0,119	0,128	0,137	0,146	0,156	0,165	0,174	0,183	0,201	0,220	0,238	0,256	0,272	0,286	0,299	0,310	0,320	0,329
Ly/Lx ③	1	0,428	0,421	0,413	0,406	0,399	0,392	0,385	0,377	0,370	0,363	0,356	0,341	0,327	0,312	0,298	0,283	0,269	0,255	0,241	0,228	0,217
	2	0,428	0,421	0,413	0,406	0,399	0,392	0,385	0,377	0,370	0,363	0,356	0,341	0,327	0,312	0,298	0,283	0,269	0,255	0,241	0,228	0,217
	3	0,072	0,079	0,087	0,094	0,101	0,108	0,115	0,123	0,130	0,137	0,144	0,159	0,173	0,188	0,202	0,217	0,231	0,245	0,259	0,272	0,283
	4	0,072	0,079	0,087	0,094	0,101	0,108	0,115	0,123	0,130	0,137	0,144	0,159	0,173	0,188	0,202	0,217	0,231	0,245	0,259	0,272	0,283
Ly/Lx ④	1	0,275	0,265	0,256	0,247	0,238	0,229	0,220	0,210	0,201	0,192	0,183	0,166	0,153	0,141	0,131	0,122	0,114	0,108	0,102	0,096	0,092
	2	0,475	0,460	0,444	0,428	0,412	0,396	0,380	0,365	0,349	0,333	0,317	0,288	0,264	0,244	0,226	0,211	0,198	0,186	0,176	0,167	0,158
	3	0,158	0,174	0,190	0,206	0,222	0,238	0,254	0,269	0,285	0,301	0,317	0,346	0,370	0,390	0,408	0,423	0,436	0,448	0,458	0,467	0,475
	4	0,092	0,101	0,110	0,119	0,128	0,137	0,146	0,156	0,165	0,174	0,183	0,200	0,214	0,225	0,235	0,244	0,252	0,258	0,264	0,270	0,275
Ly/Lx ⑤	1	0,250	0,238	0,227	0,215	0,204	0,192	0,180	0,170	0,160	0,152	0,144	0,131	0,120	0,112	0,103	0,097	0,090	0,085	0,081	0,076	0,073
	2	0,434	0,414	0,393	0,373	0,354	0,334	0,312	0,294	0,278	0,264	0,250	0,227	0,208	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	3	0,158	0,174	0,190	0,206	0,221	0,237	0,254	0,268	0,281	0,292	0,303	0,321	0,336	0,348	0,359	0,368	0,377	0,384	0,390	0,396	0,401
	4	0,158	0,174	0,190	0,206	0,221	0,237	0,254	0,268	0,281	0,292	0,303	0,321	0,336	0,348	0,359	0,368	0,377	0,384	0,390	0,396	0,401
Ly/Lx ⑥	1	0,375	0,3625	0,350	0,3375	0,325	0,3125	0,300	0,2875	0,275	0,2625	0,250	0,227	0,208	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	2	0,375	0,3625	0,350	0,3375	0,325	0,3125	0,300	0,2875	0,275	0,2625	0,250	0,227	0,208	0,192	0,179	0,167	0,156	0,147	0,139	0,132	0,125
	3	0,125	0,1375	0,150	0,1625	0,175	0,1875	0,200	0,2125	0,225	0,2375	0,250	0,273	0,292	0,308	0,321	0,333	0,344	0,353	0,361	0,368	0,375
	4	0,125	0,1375	0,150	0,1625	0,175	0,1875	0,200	0,2125	0,225	0,2375	0,250	0,273	0,292	0,308	0,321	0,333	0,344	0,353	0,361	0,368	0,375

Moments in two way slabs

Mx, My, Mx^e, My^e = coeff. $q \cdot L^2$ (shorter)

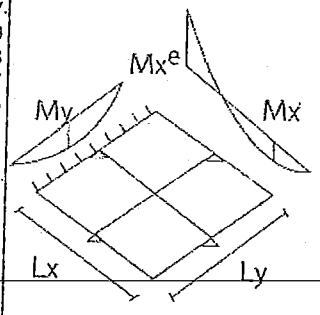
Panel type 1: Lx or Ly any edge

		Mx	My
L short = Lx, L long = Ly	0.50	0.0965	0.0174
	0.55	0.0892	0.0210
	0.60	0.0820	0.0243
	0.65	0.0750	0.0273
	0.70	0.0683	0.0298
	0.75	0.0619	0.0318
	0.80	0.0560	0.0334
	0.85	0.0506	0.0348
	0.90	0.0456	0.0359
	0.95	0.0410	0.0365
L short = Ly, L long = Lx	1.00	0.0368	0.0368
	0.95	0.0365	0.0410
	0.90	0.0359	0.0456
	0.85	0.0348	0.0506
	0.80	0.0334	0.0560
	0.75	0.0318	0.0619
	0.70	0.0298	0.0683
	0.65	0.0273	0.0750
	0.60	0.0243	0.0820
	0.55	0.0210	0.0892
	0.50	0.0174	0.0965



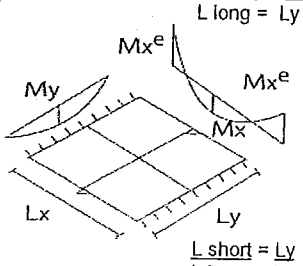
Panel type 2: Ly is always continuous edge

		Mx^e	Mx	My
L short = Lx, L long = Ly	0.50	-0.1214	0.0584	0.0060
	0.55	-0.1188	0.0562	0.0083
	0.60	-0.1159	0.0538	0.0105
	0.65	-0.1126	0.0512	0.0127
	0.70	-0.1089	0.0485	0.0149
	0.75	-0.1050	0.0457	0.0168
	0.80	-0.1008	0.0428	0.0187
	0.85	-0.0965	0.0400	0.0205
	0.90	-0.0922	0.0372	0.0221
	0.95	-0.0880	0.0345	0.0234
L short = Ly, L long = Lx	1.00	-0.0839	0.0318	0.0243
	0.95	-0.0881	0.0327	0.0282
	0.90	-0.0924	0.0330	0.0323
	0.85	-0.0967	0.0328	0.0369
	0.80	-0.1011	0.0324	0.0423
	0.75	-0.1055	0.0315	0.0485
	0.70	-0.1096	0.0309	0.0553
	0.65	-0.1133	0.0292	0.0627
	0.60	-0.1165	0.0269	0.0707
	0.55	-0.1192	0.0240	0.0792
	0.50	-0.1215	0.0204	0.0880



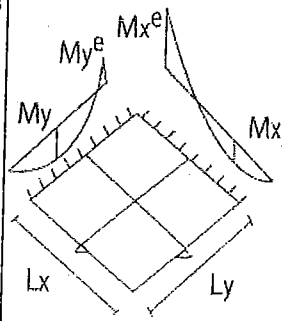
Panel type 3: Ly to be continuous edge

		Mx^e	Mx	My
L short = Lx, L long = Ly	0.50	-0.0845	0.0414	0.0017
	0.55	-0.0843	0.0408	0.0029
	0.60	-0.0837	0.0400	0.0013
	0.65	-0.0828	0.0391	0.0058
	0.70	-0.0816	0.0380	0.0073
	0.75	-0.0801	0.0366	0.0088
	0.80	-0.0784	0.0350	0.0103
	0.85	-0.0765	0.0335	0.0119
	0.90	-0.0744	0.0319	0.0134
	0.95	-0.0722	0.0302	0.0147
L short = Ly, L long = Lx	1.00	-0.0698	0.0285	0.0158
	0.95	-0.0745	0.0297	0.0187
	0.90	-0.0796	0.0307	0.0225
	0.85	-0.0849	0.0314	0.0267
	0.80	-0.0902	0.0318	0.0316
	0.75	-0.0957	0.0320	0.0374
	0.70	-0.1011	0.0319	0.0442
	0.65	-0.1063	0.0310	0.0519
	0.60	-0.1111	0.0292	0.0604
	0.55	-0.1154	0.0266	0.0697
	0.50	-0.1191	0.0234	0.0799



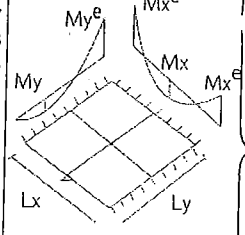
Panel type 4: Lx or Ly any edge

		Mx^e	My^e	Mx	My
L short = Lx, L long = Ly	0.50	-0.1177	-0.0782	0.0560	0.0079
	0.55	-0.1136	-0.0779	0.0529	0.0105
	0.60	-0.1093	-0.0776	0.0496	0.0130
	0.65	-0.1047	-0.0773	0.0462	0.0153
	0.70	-0.0996	-0.0768	0.0426	0.0171
	0.75	-0.0940	-0.0759	0.0390	0.0188
	0.80	-0.0882	-0.0746	0.0355	0.0203
	0.85	-0.0825	-0.0731	0.0322	0.0216
	0.90	-0.0773	-0.0714	0.0291	0.0226
	0.95	-0.0724	-0.0696	0.0262	0.0232
L short = Ly, L long = Lx	1.00	-0.0677	-0.0677	0.0234	0.0234
	0.95	-0.0696	-0.0724	0.0232	0.0262
	0.90	-0.0714	-0.0773	0.0226	0.0291
	0.85	-0.0731	-0.0825	0.0216	0.0322
	0.80	-0.0746	-0.0882	0.0203	0.0355
	0.75	-0.0759	-0.0940	0.0188	0.0390
	0.70	-0.0768	-0.0996	0.0171	0.0426
	0.65	-0.0773	-0.1047	0.0153	0.0462
	0.60	-0.0776	-0.1093	0.0130	0.0496
	0.55	-0.0779	-0.1136	0.0105	0.0529
	0.50	-0.0782	-0.1177	0.0079	0.0560



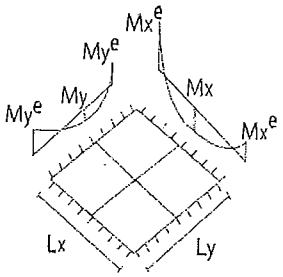
Panel type 5: Lx to be free edge

		Mx^e	My^e	Mx	My
L short = Lx, L long = Ly	0.50	-0.1836	-0.0563	0.0409	0.0028
	0.55	-0.1826	-0.0564	0.0398	0.0041
	0.60	-0.1813	-0.0566	0.0385	0.0059
	0.65	-0.1796	-0.0569	0.0370	0.0075
	0.70	-0.0774	-0.0572	0.0352	0.0091
	0.75	-0.0748	-0.0571	0.0333	0.0107
	0.80	-0.0720	-0.0568	0.0313	0.0123
	0.85	-0.0691	-0.0564	0.0292	0.0138
	0.90	-0.0660	-0.0560	0.0270	0.0151
	0.95	-0.0628	-0.0556	0.0249	0.0161
L short = Ly, L long = Lx	1.00	-0.0596	-0.0551	0.0228	0.0167
	0.95	-0.0626	-0.0599	0.0230	0.0193
	0.90	-0.0655	-0.0652	0.0231	0.0222
	0.85	-0.0682	-0.0710	0.0229	0.0254
	0.80	-0.0706	-0.0773	0.0224	0.0289
	0.75	-0.0727	-0.0839	0.0214	0.0327
	0.70	-0.0743	-0.0907	0.0198	0.0368
	0.65	-0.0755	-0.0978	0.0177	0.0411
	0.60	-0.0765	-0.1046	0.0153	0.0452
	0.55	-0.0774	-0.1101	0.0127	0.0492
	0.50	-0.0782	-0.1140	0.0098	0.0535

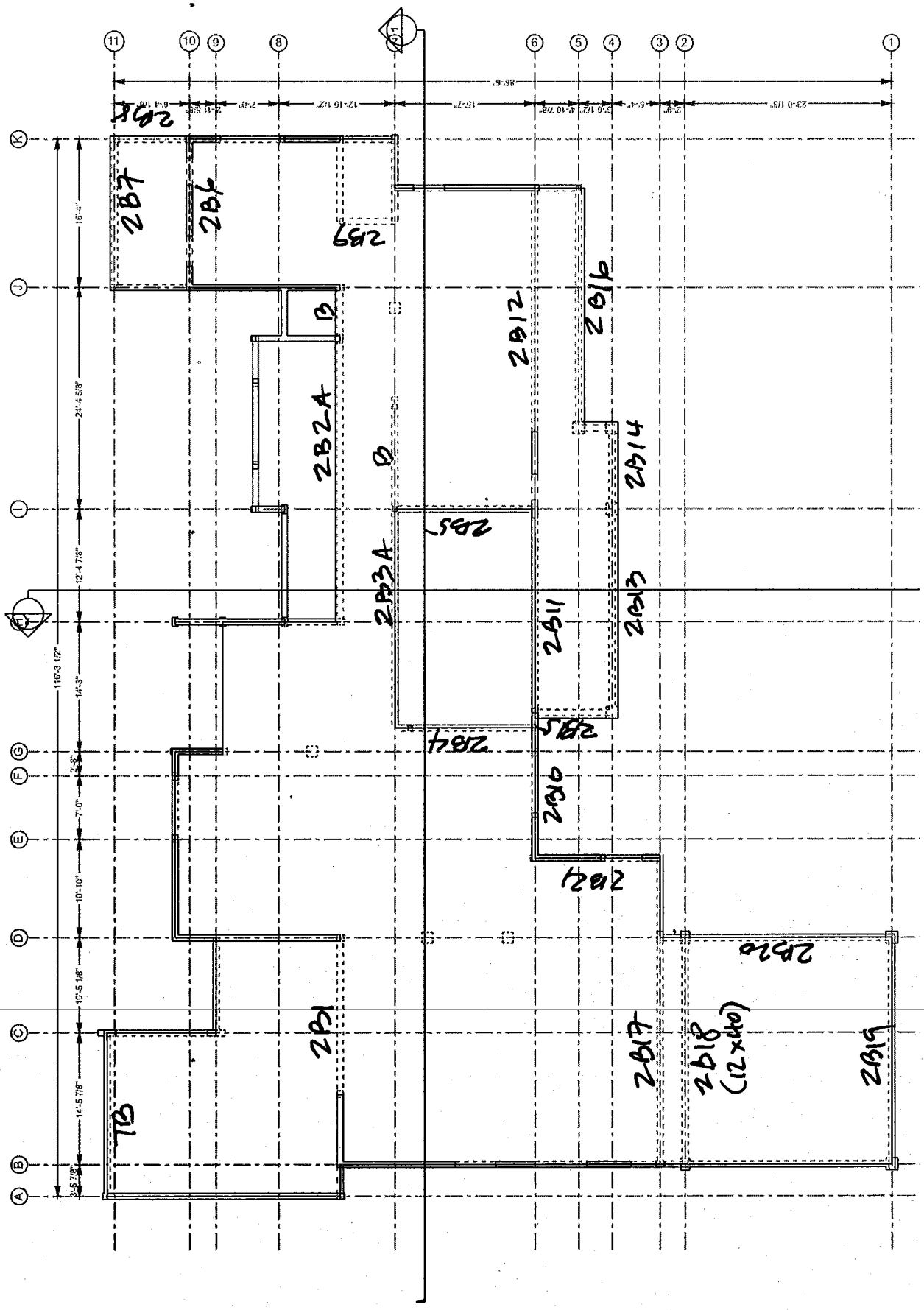


Panel type 6: Lx or Ly any edge

		Mx^e	My^e	Mx	My
L short = Lx, L long = Ly	0.50	-0.0826	-0.0560	0.0401	0.0038
	0.55	-0.0806	-0.0561	0.0385	0.0055
	0.60	-0.0784	-0.0562	0.0367	0.0076
	0.65	-0.0759	-0.0565	0.0346	0.0096
	0.70	-0.0731	-0.0568	0.0322	0.0114
	0.75	-0.0698	-0.0564	0.0297	0.0129
	0.80	-0.0661	-0.0558	0.0271	0.0143
	0.85	-0.0620	-0.0550	0.0246	0.0156
	0.90	-0.0580	-0.0540	0.0222	0.0167
	0.95	-0.0543	-0.0527	0.0198	0.0173
L short = Ly, L long = Lx	1.00	-0.0511	-0.0511	0.0176	0.0176
	0.95	-0.0527	-0.0543	0.0173	0.0198
	0.90	-0.0540	-0.0580	0.0167	0.0222
	0.85	-0.0550	-0.0620	0.0156	0.0246
	0.80	-0.0558	-0.0661	0.0143	0.0271
	0.75	-0.0564	-0.0698	0.0129	0.0297
	0.70	-0.0568	-0.0731	0.0114	0.0322
	0.65	-0.0565	-0.0759	0.0096	0.0346
	0.60	-0.0562	-0.0784	0.0076	0.0367
	0.55	-0.0562	-0.0806	0.0055	0.0385
	0.50	-0.0560	-0.0826	0.0038	0.0401



Second Floor Beam & Lds



CONCRETE BEAM DESIGN

GEOMETRY

Type ...Inverted-T	Not deep beam		OK
Total Height h	38.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	12.0	in	OK
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	18.00	ft	1	Fixed
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	43.5	51.4	26.1	30.9	
2	38.4	55.7	23.1	33.4	
3	36.7	54.9	22.1	32.9	
4	43.1	50.2	25.8	30.1	
5	41.8	47.5	25.1	28.5	
6	36.7	46.8	22.1	28.1	
7	36.7	44.1	22.1	26.5	
8	43.1	48.3	25.8	29.0	
9	41.8	47.5	25.1	28.5	
10	41.8	47.5	25.1	28.5	
11	22.0	33.1	13.2	19.8	
12	22.0	33.1	13.2	19.8	

LOAD COMBINATIONS

	SERVICE	FACTORED
1	D+L	1.4D
2	D+Lr	1.2D+1.6L+0.5Lr
3	D+S	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	1.2D+0.5L+1.6S
6	D+0.6W	1.2D+1.6Lr+0.5W
7	D+0.7E	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	0.9D+W
12	0.6D+0.7E	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	-185	104	0	OK
2	-201	113	0	OK
3	-198	111	0	OK
4	-181	102	0	OK
5	-171	96	0	OK
6	-168	95	0	OK
7	-159	89	0	OK
8	-174	98	0	OK
9	-171	96	0	OK
10	-171	96	0	OK
11	-119	67	0	OK
12	-119	67	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	34.6	-14.0	OK
2	37.5	-15.2	OK
3	36.9	-15.0	OK
4	33.7	-13.7	OK
5	31.9	-12.9	OK
6	31.5	-12.8	OK
7	29.7	-12.0	OK
8	32.5	-13.2	OK
9	31.9	-12.9	OK
10	31.9	-12.9	OK
11	22.2	-9.0	OK
12	22.2	-9.0	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		2#6x16.00 ft		
d	1.8	3.5	1.8	in
+As	0.88	1.76	0.88	in²
c/d	0.08	0.09	0.06	
+ϕMn	144	267	139	
Top		2#6 Cont		
Top	2#6		0#5	
d'	4.0	2.0	2.0	in
-As	1.76	0.88	0.88	in²
c/d	0.04	0.04	0.03	
-ϕMn	-274	-154	-145	
As min	0.97	0.97	0.00	in²

DEFLECTIONS

Comb		
D	0.02	in
L	0.01	in
Lr, S	0.00	in
LS	0.00	in
LT	0.04	in
Ieff D	42,590	in4
Ieff L	39,464	in4
Ieff LS	41,680	in4
λ	1.73	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	27.5		-27.5	
ϕVs	29.9		-29.9	
ϕVn	57.3		-57.3	
Av/s	0.018		0.018	OK
Min Av/s	0.007		N.A.	
s	12.0	11.0	12.0	OK
s max	18.0	18.0	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.55	OK
Max. Steel Area As max Ratio	0.29	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.24	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	0.67	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.01	OK
C	Long-Term	4824	480	0.10	OK
D	Long-Term	4824	240	0.05	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

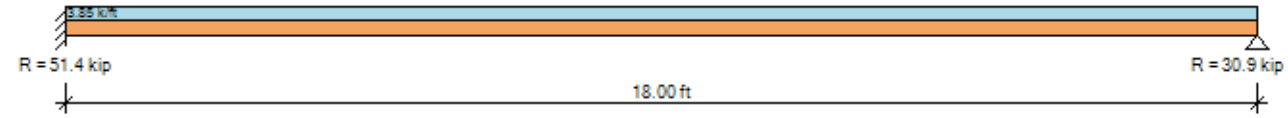
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



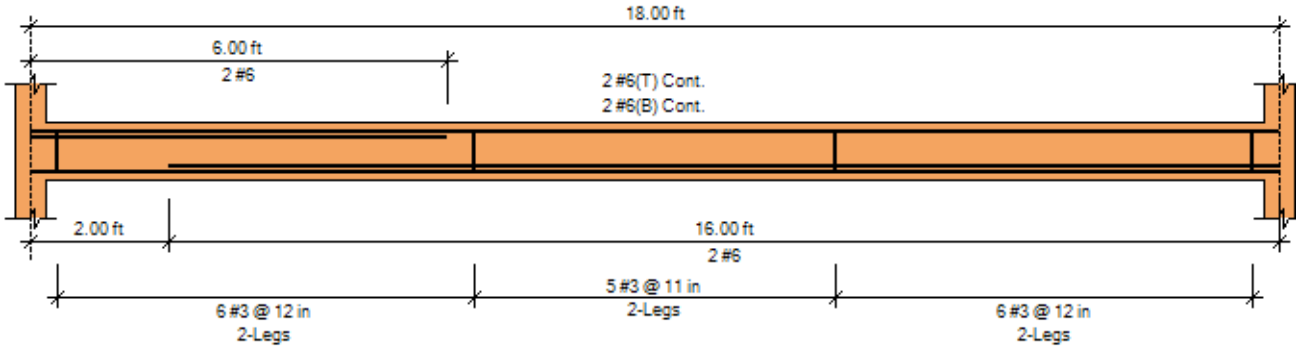
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Deep beam		NG
Total Height h	26.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	12.0	in	NG
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	31.00	ft	1	Pinned
2	6.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	11.5	13.2	40.4	46.1	-13.9	-15.8
2	2	9.4	14.7	33.0	51.5	-11.3	-17.6
3	3	9.4	14.7	33.0	51.5	-11.3	-17.6
4	4	11.0	12.4	38.5	43.3	-13.2	-14.8
5	5	11.0	12.4	38.5	43.3	-13.2	-14.8
6	6	9.4	11.3	33.0	39.5	-11.3	-13.6
7	7	9.4	11.3	33.0	39.5	-11.3	-13.6
8	8	11.0	12.4	38.5	43.3	-13.2	-14.8
9	9	11.0	12.4	38.5	43.3	-13.2	-14.8
10	10	11.0	12.4	38.5	43.3	-13.2	-14.8
11	11	5.7	8.5	19.8	29.7	-6.8	-10.2
12	12	5.7	8.5	19.8	29.7	-6.8	-10.2

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	0	81	-114	0	-52	OK
2	0	90	-127	0	-58	OK
3	0	90	-127	0	-58	OK
4	0	76	-107	0	-49	OK
5	0	76	-107	0	-49	OK
6	0	69	-98	0	-45	OK
7	0	69	-98	0	-45	OK
8	0	76	-107	0	-49	OK
9	0	76	-107	0	-49	OK
10	0	76	-107	0	-49	OK
11	0	52	-74	0	-34	OK
12	0	52	-74	0	-34	OK

SHEAR FORCES Vu (kip)

Comb					
1	10.7	-19.2	19.8	18.3	OK
2	11.9	-21.3	22.0	20.4	OK
3	11.9	-21.3	22.0	20.4	OK
4	10.0	-18.0	18.5	17.2	OK
5	10.0	-18.0	18.5	17.2	OK
6	9.2	-16.4	16.9	15.7	OK
7	9.2	-16.4	16.9	15.7	OK
8	10.0	-18.0	18.5	17.2	OK
9	10.0	-18.0	18.5	17.2	OK
10	10.0	-18.0	18.5	17.2	OK
11	6.9	-12.3	12.7	11.8	OK
12	6.9	-12.3	12.7	11.8	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		2#5 Cont		
Bot		2#5x27.00 ft		2#5x6.00 ft		
d	2.0	4.0	2.0	4.0	2.0	in
+As	0.62	1.24	0.62	1.24	0.62	in²
c/d	0.07	0.07	0.07	0.07	0.07	
+ϕMn	76	126	76	126	79	
Top		2#6 Cont		2#6 Cont		
Top	2#5		2#5		2#6	
d'	4.0	2.0	4.0	2.0	4.0	in
-As	1.50	0.88	1.50	0.88	1.76	in²
c/d	0.13	0.12	0.13	0.12	0.15	
-ϕMn	-146	-95	-146	-95	-169	
As min	0.00	0.64	1.28	0.00	0.77	in²

SHEAR STRENGTH (kip, in)

ϕVc	18.2	-18.2	18.2	-18.2			
ϕVs	19.8	-19.8	19.8	-19.8			
ϕVn	38.0	-38.0	38.0	-38.0			
Av/s	0.018	0.018	0.018	0.018			OK
Min Av/s	0.007	N.A.	0.007	0.007			
s	12.0	12.0	12.0	12.0	6.0	12.0	OK
s max	12.0	12.0	N.A.	12.0	N.A.	12.0	

DEFLECTIONS

Comb			
D	0.19	0.00	in
L	0.14	0.00	in
Lr, S	0.00	0.00	in
LS	0.04	0.00	in
LT	0.52	0.00	in
Ieff D	11,419	17,471	in4
Ieff L	8,105	17,394	in4
Ieff LS	10,129	17,441	in4
λ	1.63	1.63	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.85	OK
Max. Steel Area As max Ratio	0.35	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	1.00	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	2647	360	0.14	OK
C	Long-Term	714	480	0.67	OK
D	Long-Term	714	240	0.34	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

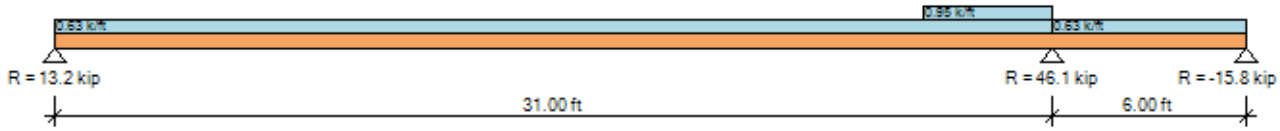


Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

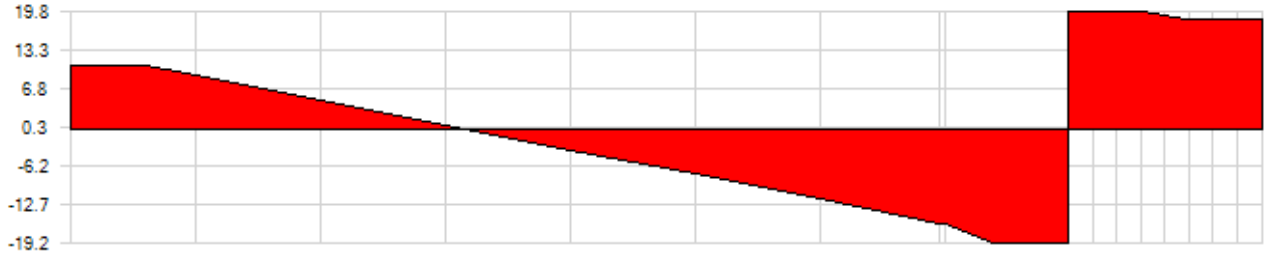
TORSION STRENGTH (k-ft)



At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



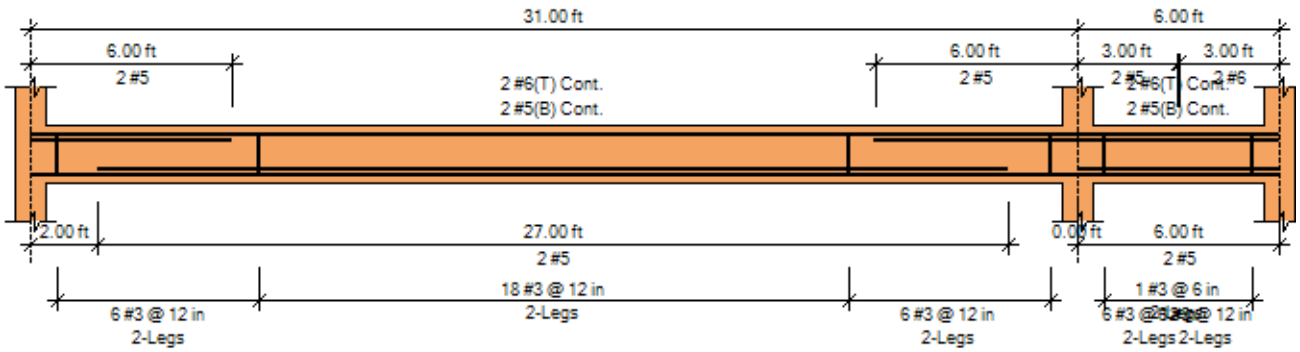
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	26.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	6.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	26.00	ft	1	Pinned
2	11.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	8.0	9.5	31.1	35.5	7.0	7.7
2	2	6.8	10.1	25.3	39.7	5.5	8.9
3	3	6.8	10.1	25.3	39.7	5.5	8.9
4	4	7.7	8.7	29.7	33.3	6.6	7.3
5	5	7.7	8.7	29.7	33.3	6.6	7.3
6	6	6.8	8.1	25.3	30.4	5.5	6.6
7	7	6.8	8.1	25.3	30.4	5.5	6.6
8	8	7.7	8.7	29.7	33.3	6.6	7.3
9	9	7.7	8.7	29.7	33.3	6.6	7.3
10	10	7.7	8.7	29.7	33.3	6.6	7.3
11	11	4.1	6.1	15.2	22.8	3.3	4.9
12	12	4.1	6.1	15.2	22.8	3.3	4.9

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	0	49	-76	12	0	OK
2	0	53	-82	14	0	OK
3	0	53	-82	14	0	OK
4	0	45	-70	11	0	OK
5	0	45	-70	11	0	OK
6	0	42	-65	10	0	OK
7	0	42	-65	10	0	OK
8	0	45	-70	11	0	OK
9	0	45	-70	11	0	OK
10	0	45	-70	11	0	OK
11	0	32	-49	7	0	OK
12	0	32	-49	7	0	OK

SHEAR FORCES Vu (kip)

Comb					
1	7.3	-13.0	14.2	-1.8	OK
2	7.8	-14.4	16.0	-2.2	OK
3	7.8	-14.4	16.0	-2.2	OK
4	6.8	-12.2	13.4	-1.8	OK
5	6.8	-12.2	13.4	-1.8	OK
6	6.3	-11.2	12.2	-1.5	OK
7	6.3	-11.2	12.2	-1.5	OK
8	6.8	-12.2	13.4	-1.8	OK
9	6.8	-12.2	13.4	-1.8	OK
10	6.8	-12.2	13.4	-1.8	OK
11	4.7	-8.4	9.1	-1.2	OK
12	4.7	-8.4	9.1	-1.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		2#5 Cont		
Bot		2#5x24.00 ft		2#5x11.00 ft		
d	2.0	4.0	2.0	4.0	2.0	in
+As	0.62	1.24	0.62	1.24	0.62	in²
c/d	0.09	0.08	0.09	0.08	0.09	
+ϕMn	76	124	76	124	76	
Top		2#6 Cont		2#6 Cont		
Top	2#6		2#6		2#6	
d'	4.0	2.0	4.0	2.0	4.0	in
-As	1.76	0.88	1.76	0.88	1.76	in²
c/d	0.15	0.12	0.15	0.12	0.15	
-ϕMn	-169	-95	-169	-95	-169	
As min	0.00	0.64	1.10	0.18	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	18.2		-18.2	18.2		-18.2	
ϕVs	19.8		-19.8	19.8		-19.8	
ϕVn	38.0		-38.0	38.0		-38.0	
Av/s	0.018		0.018	0.018		0.018	OK
Min Av/s	N.A.		N.A.	0.007		N.A.	
s	12.0	12.0	12.0	12.0	6.0	12.0	OK
s max	N.A.	12.0	N.A.	12.0	N.A.	N.A.	

DEFLECTIONS

Comb			
D	0.06	0.00	in
L	0.01	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.12	0.00	in
Ieff D	15,887	15,887	in4
Ieff L	15,539	15,539	in4
Ieff LS	15,752	15,752	in4
λ	1.63	1.63	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.62	OK
Max. Steel Area As max Ratio	0.41	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	1.00	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.02	OK
C	Long-Term	2650	480	0.18	OK
D	Long-Term	2650	240	0.09	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

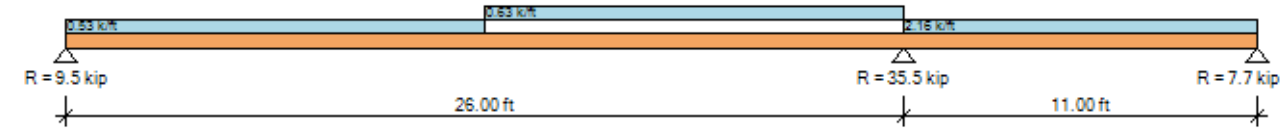
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



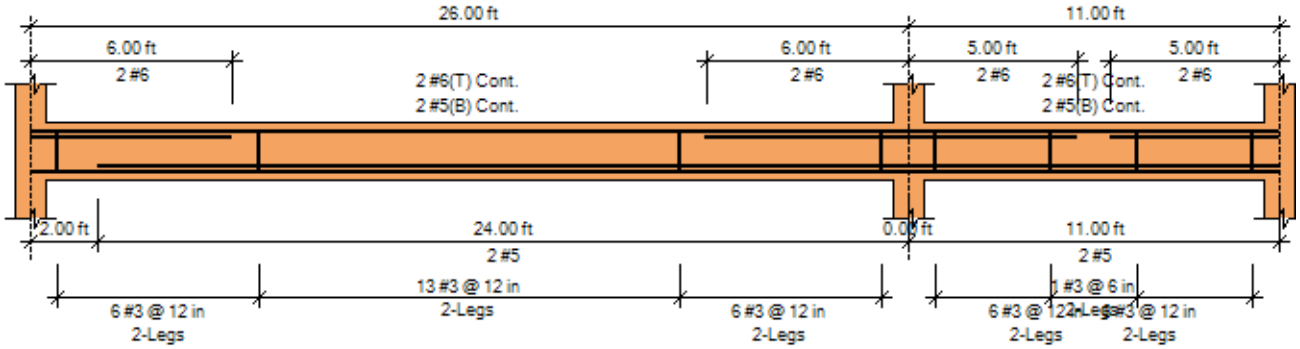
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	28.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	16.33	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	19.8	22.9	19.8	22.9	
2	16.4	25.1	16.4	25.1	
3	16.4	25.1	16.4	25.1	
4	18.9	21.3	18.9	21.3	
5	18.9	21.3	18.9	21.3	
6	16.4	19.6	16.4	19.6	
7	16.4	19.6	16.4	19.6	
8	18.9	21.3	18.9	21.3	
9	18.9	21.3	18.9	21.3	
10	18.9	21.3	18.9	21.3	
11	9.8	14.7	9.8	14.7	
12	9.8	14.7	9.8	14.7	

LOAD COMBINATIONS

	SERVICE	FACTORED
1	D+L	1.4D
2	D+Lr	1.2D+1.6L+0.5Lr
3	D+S	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	1.2D+0.5L+1.6S
6	D+0.6W	1.2D+1.6Lr+0.5W
7	D+0.7E	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	0.9D+W
12	0.6D+0.7E	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	93	0	OK
2	0	103	0	OK
3	0	103	0	OK
4	0	87	0	OK
5	0	87	0	OK
6	0	80	0	OK
7	0	80	0	OK
8	0	87	0	OK
9	0	87	0	OK
10	0	87	0	OK
11	0	60	0	OK
12	0	60	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	15.9	-15.9	OK
2	17.4	-17.4	OK
3	17.4	-17.4	OK
4	14.8	-14.8	OK
5	14.8	-14.8	OK
6	13.6	-13.6	OK
7	13.6	-13.6	OK
8	14.8	-14.8	OK
9	14.8	-14.8	OK
10	14.8	-14.8	OK
11	10.2	-10.2	OK
12	10.2	-10.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		
Bot		2#5x12.00 ft		
d	2.0	4.0	2.0	in
+As	0.62	1.24	0.62	in²
c/d	0.07	0.10	0.07	
+ϕMn	71	132	71	
Top		2#5 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.62	0.62	0.62	in²
c/d	0.07	0.10	0.07	
-ϕMn	-71	-76	-71	
As min	0.00	0.69	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	19.7		-19.7	
ϕVs	25.7		-25.7	
ϕVn	45.5		-45.5	
Av/s	0.022		0.022	OK
Min Av/s	0.007		N.A.	
s	10.0	10.0	10.0	OK
s max	13.0	N.A.	N.A.	

DEFLECTIONS

Comb		
D	0.11	in
L	0.06	in
Lr, S	0.00	in
LS	0.02	in
LT	0.28	in
Ieff D	8,222	in4
Ieff L	6,492	in4
Ieff LS	7,557	in4
λ	1.74	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.56	OK
Max. Steel Area As max Ratio	0.29	OK
Rebars Min Spacing Smin Ratio ..	0.50	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.30	OK
Stirrups Spacing s max Ratio	0.77	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	3406	360	0.11	OK
C	Long-Term	712	480	0.67	OK
D	Long-Term	712	240	0.34	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

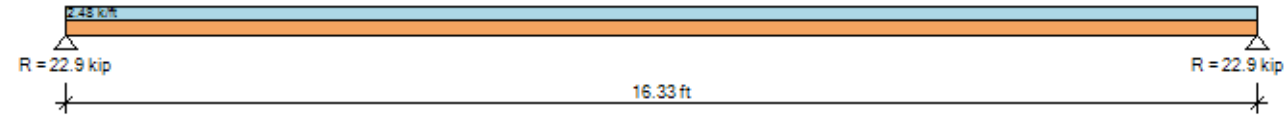
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

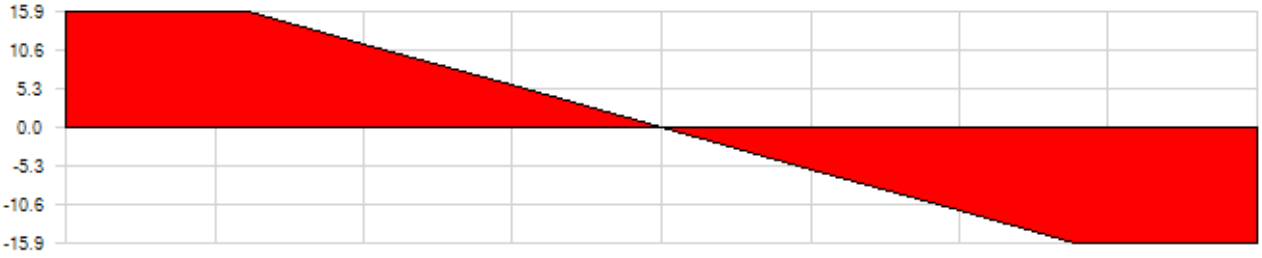
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



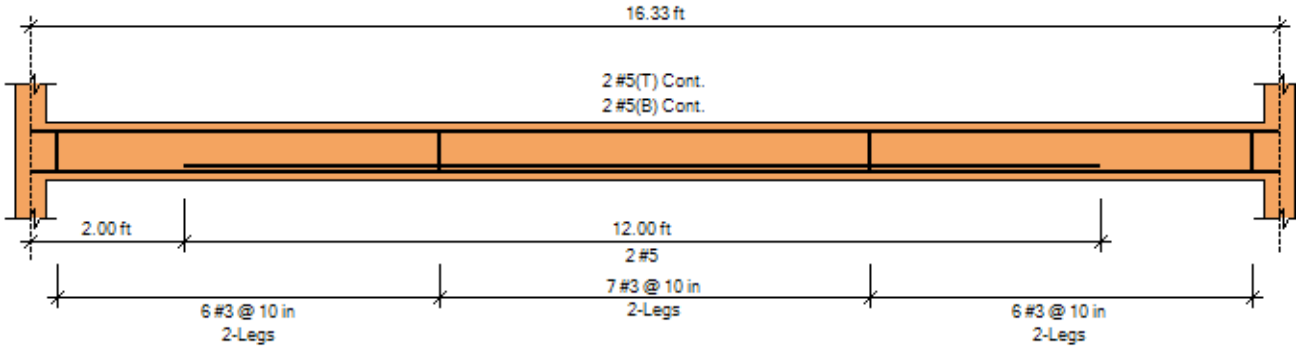
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...L-beam	Not deep beam		OK
Total Height h	18.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	16.33	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	5.7	8.0	5.7	8.0	
2	6.7	7.3	6.7	7.3	
3	5.7	6.9	5.7	6.9	
4	6.5	8.4	6.5	8.4	
5	5.7	6.9	5.7	6.9	
6	5.7	8.4	5.7	8.4	
7	5.7	6.9	5.7	6.9	
8	6.5	7.3	6.5	7.3	
9	5.7	6.9	5.7	6.9	
10	5.7	6.9	5.7	6.9	
11	3.4	5.1	3.4	5.1	
12	3.4	5.1	3.4	5.1	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	33	0	OK
2	0	30	0	OK
3	0	28	0	OK
4	0	34	0	OK
5	0	28	0	OK
6	0	34	0	OK
7	0	28	0	OK
8	0	30	0	OK
9	0	28	0	OK
10	0	28	0	OK
11	0	21	0	OK
12	0	21	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	6.4	-6.4	OK
2	5.8	-5.8	OK
3	5.5	-5.5	OK
4	6.7	-6.7	OK
5	5.5	-5.5	OK
6	6.7	-6.7	OK
7	5.5	-5.5	OK
8	5.8	-5.8	OK
9	5.5	-5.5	OK
10	5.5	-5.5	OK
11	4.1	-4.1	OK
12	4.1	-4.1	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x12.00 ft		
d	2.0	4.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.14	0.14	0.14	
+ϕMn	59	59	59	
Top		2#5 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.62	0.62	0.62	in²
c/d	0.08	0.08	0.08	
-ϕMn	-47	-47	-47	
As min	0.00	0.71	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	12.1		-12.1	
ϕVs	19.8		-19.8	
ϕVn	31.9		-31.9	
Av/s	0.028		0.028	OK
Min Av/s	0.007		N.A.	
s	8.0	8.0	8.0	OK
s max	8.0	N.A.	N.A.	

DEFLECTIONS

Comb		
D	0.10	in
L	0.00	in
Lr, S	0.05	in
LS	0.02	in
LT	0.19	in
Ieff D	3,030	in4
Ieff L	3,030	in4
Ieff LS	2,775	in4
λ	1.61	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.80	OK
Max. Steel Area As max Ratio	0.31	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.24	OK
Stirrups Spacing s max Ratio	1.00	OK
Stirrups Min. # of Legs Ratio	0.29	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	3698	180	0.05	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	1034	480	0.46	OK
D	Long-Term	1034	240	0.23	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

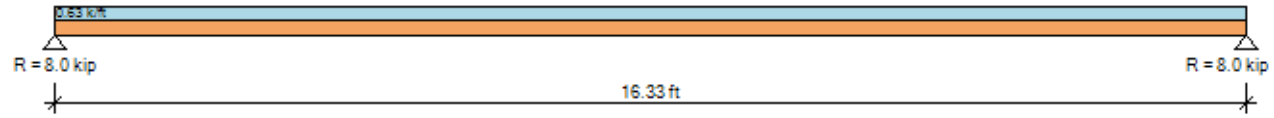
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

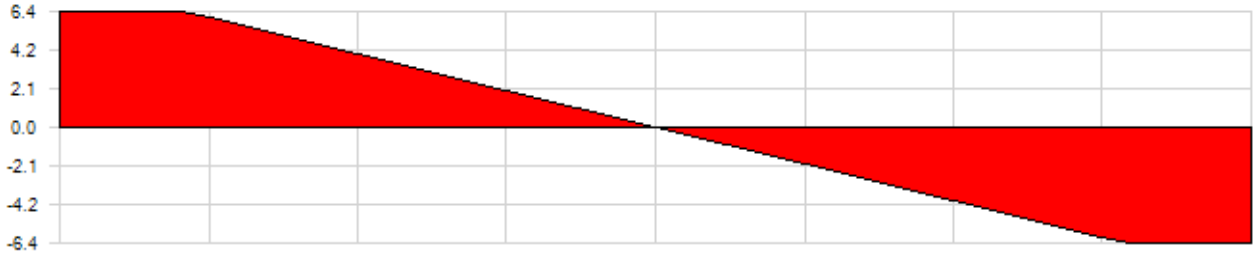
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



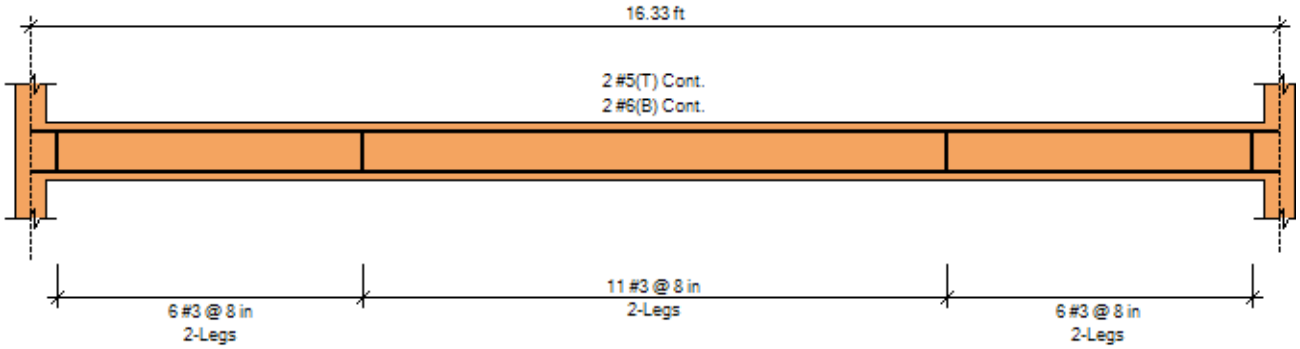
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...L-beam	Deep beam		NG
Total Height h	18.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	3.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	4.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	1	-4.0	-5.6	14.2	19.9
2	2	-5.4	-5.5	18.3	19.1
3	3	-4.0	-4.8	14.2	17.1
4	4	-5.0	-7.0	17.3	23.7
5	5	-4.0	-4.8	14.2	17.1
6	6	-4.0	-7.0	14.2	23.7
7	7	-4.0	-4.8	14.2	17.1
8	8	-5.0	-5.5	17.3	19.1
9	9	-4.0	-4.8	14.2	17.1
10	10	-4.0	-4.8	14.2	17.1
11	11	-2.4	-3.6	8.5	12.8
12	12	-2.4	-3.6	8.5	12.8

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	-9	0	-29	OK
2	-9	0	-28	OK
3	-8	0	-25	OK
4	-12	0	-35	OK
5	-8	0	-25	OK
6	-12	0	-35	OK
7	-8	0	-25	OK
8	-9	0	-28	OK
9	-8	0	-25	OK
10	-8	0	-25	OK
11	-6	0	-19	OK
12	-6	0	-19	OK

SHEAR FORCES Vu (kip)

Comb				
1	-7.0	-7.5	9.5	OK
2	-6.8	-7.3	9.2	OK
3	-6.0	-6.5	8.2	OK
4	-8.5	-9.1	11.6	OK
5	-6.0	-6.5	8.2	OK
6	-8.5	-9.1	11.6	OK
7	-6.0	-6.5	8.2	OK
8	-6.8	-7.3	9.2	OK
9	-6.0	-6.5	8.2	OK
10	-6.0	-6.5	8.2	OK
11	-4.5	-4.9	6.1	OK
12	-4.5	-4.9	6.1	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		
Bot		0#5x2.00 ft		
d	2.0	4.0	2.0	in
+As	0.62	0.62	0.62	in²
c/d	0.12	0.12	0.12	
+ϕMn	43	43	43	
Top		2#5 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.62	0.62	0.62	in²
c/d	0.08	0.08	0.08	
-ϕMn	-47	-47	-47	
As min	0.22	0.00	0.43	in²

DEFLECTIONS

Comb			
D	0.00	0.01	in
L	0.00	0.00	in
Lr, S	0.00	0.01	in
LS	0.00	0.00	in
LT	0.00	0.02	in
Ieff D	4,642	6,490	in4
Ieff L	4,642	6,490	in4
Ieff LS	4,546	5,850	in4
λ	1.61	2.00	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	12.1		-12.1	12.1	
ϕVs	19.8		-19.8	26.4	
ϕVn	31.9		-31.9	38.5	
Av/s	0.028		0.028	0.037	OK
Min Av/s	N.A.		N.A.	0.007	
s	8.0	6.0	8.0	6.0	OK
s max	N.A.	N.A.	N.A.	8.0	

REINFORCEMENT

Min. Steel Area As min Ratio	0.69	OK
Max. Steel Area As max Ratio	0.22	OK
Rebars Min Spacing Smin Ratio ..	0.50	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.24	OK
Stirrups Spacing s max Ratio	1.00	OK
Stirrups Min. # of Legs Ratio	0.29	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	4820	90	0.02	OK
B	Live Load	9999	180	0.00	OK
C	Long-Term	2386	240	0.10	OK
D	Long-Term	2386	120	0.05	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

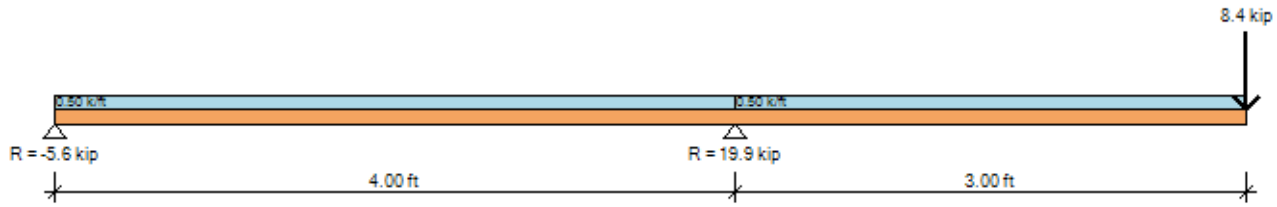
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

Comb				
1	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)

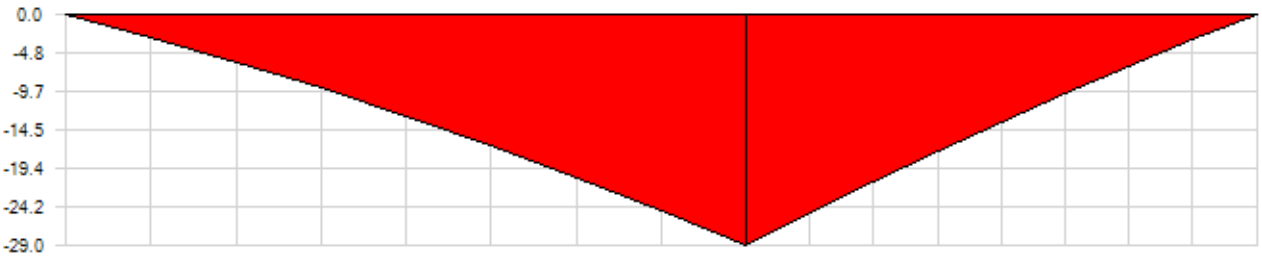
At/s (in)	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	OK



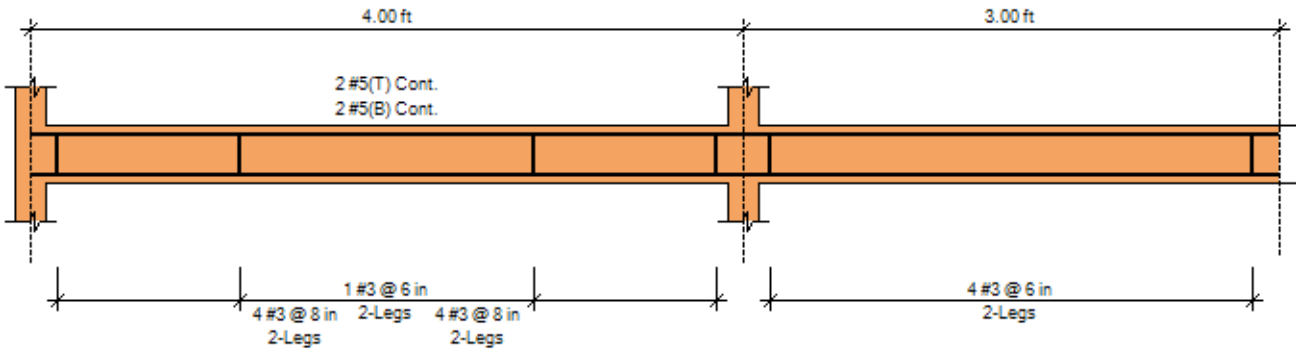
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Deep beam		NG
Total Height h	40.0	in	
Beam Width b	12.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	13.00	ft	1	Pinned
2	24.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	8.5	10.5	40.2	54.7	12.9	18.1
2	2	9.5	11.5	44.6	51.5	14.2	16.0
3	3	7.5	10.5	39.1	48.8	12.9	15.4
4	4	9.7	12.7	44.1	56.3	13.8	17.5
5	5	8.2	9.5	40.0	47.5	12.9	15.5
6	6	7.5	12.3	39.1	55.7	12.9	17.5
7	7	7.5	9.0	39.1	46.9	12.9	15.5
8	8	9.7	10.5	44.1	50.2	13.8	16.1
9	9	8.2	9.5	40.0	47.5	12.9	15.5
10	10	8.2	9.5	40.0	47.5	12.9	15.5
11	11	4.5	6.8	23.4	35.2	7.8	11.6
12	12	4.5	6.8	23.4	35.2	7.8	11.6

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	0	19	-110	86	0	OK
2	0	22	-101	76	0	OK
3	0	20	-96	73	0	OK
4	0	24	-110	83	0	OK
5	0	17	-95	74	0	OK
6	0	23	-109	83	0	OK
7	0	16	-94	74	0	OK
8	0	19	-100	77	0	OK
9	0	17	-95	74	0	OK
10	0	17	-95	74	0	OK
11	0	12	-71	56	0	OK
12	0	12	-71	56	0	OK

SHEAR FORCES Vu (kip)

Comb					
1	-0.7	-16.2	20.0	-10.9	OK
2	0.2	-15.7	18.0	-9.6	OK
3	0.0	-14.7	17.2	-9.2	OK
4	0.2	-17.2	19.6	-10.4	OK
5	-0.4	-14.2	17.2	-9.3	OK
6	0.1	-16.9	19.6	-10.4	OK
7	-0.6	-13.9	17.2	-9.3	OK
8	-0.2	-15.1	17.9	-9.6	OK
9	-0.4	-14.2	17.2	-9.3	OK
10	-0.4	-14.2	17.2	-9.3	OK
11	-0.4	-10.4	12.9	-7.0	OK
12	-0.4	-10.4	12.9	-7.0	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		3#6 Cont		3#6 Cont		
Bot		0#5x6.00 ft		0#5x6.00 ft		
d	2.0	2.0	2.0	2.0	2.0	in
+As	1.32	1.32	1.32	1.32	1.32	in²
c/d	0.06	0.06	0.06	0.06	0.06	
+ϕMn	220	220	220	220	220	
Top		3#6 Cont		3#6 Cont		
Top	0#5		0#5		0#5	
d'	2.0	2.0	2.0	2.0	2.0	in
-As	1.32	1.32	1.32	1.32	1.32	in²
c/d	0.06	0.06	0.06	0.06	0.06	
-ϕMn	-220	-220	-220	-220	-220	
As min	0.00	0.19	0.87	0.68	0.00	in²

DEFLECTIONS

Comb			
D	0.00	0.02	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.00	0.04	in
Ieff D	70,025	70,025	in4
Ieff L	70,025	70,025	in4
Ieff LS	70,025	70,025	in4
λ	1.75	1.75	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	43.3		-43.3	43.3		-43.3	
ϕVs	31.4		-31.4	31.4		-31.4	
ϕVn	74.6		-74.6	74.6		-74.6	
Av/s	0.018		0.018	0.018		0.018	OK
Min Av/s	N.A.		N.A.	N.A.		N.A.	
s	12.0	6.0	12.0	12.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.66	OK
Max. Steel Area As max Ratio	0.14	OK
Rebars Min Spacing Smin Ratio ..	0.40	OK
Rebars Max Spacing Smax Ratio	0.31	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	7821	480	0.06	OK
D	Long-Term	7821	240	0.03	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

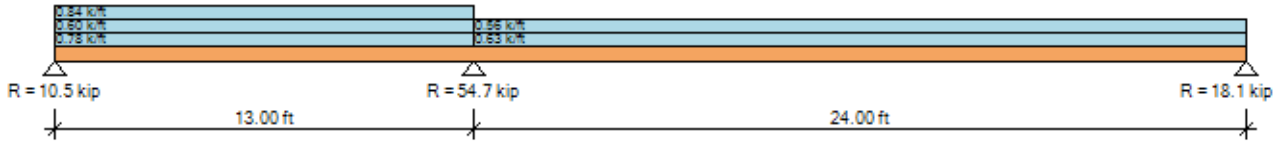


Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)



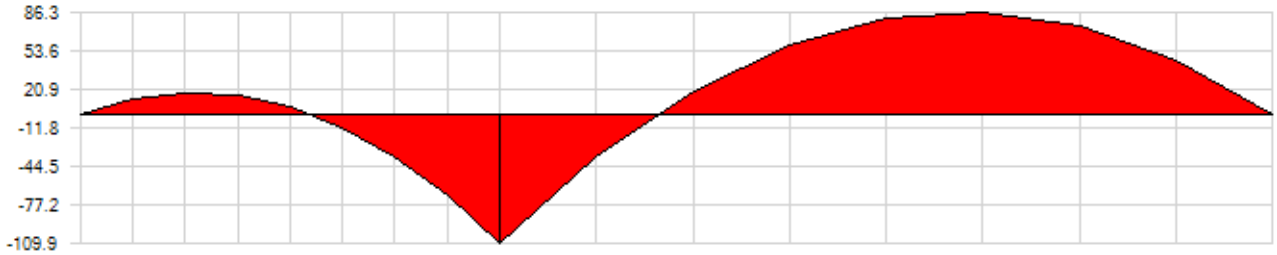
At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



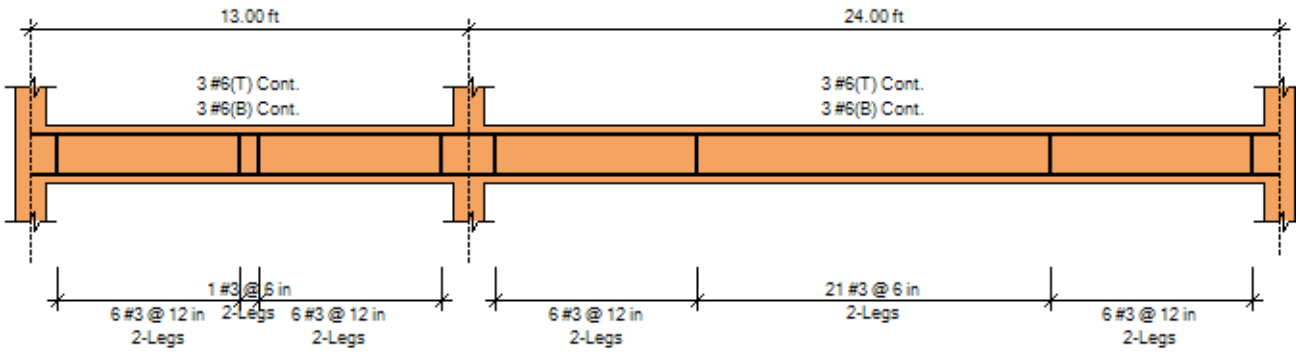
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	52.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	28.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	50.5	62.1	48.1	58.8	
2	47.5	64.7	45.4	61.9	
3	44.4	63.1	42.0	60.2	
4	51.3	61.3	49.2	58.9	
5	49.0	56.3	46.6	53.4	
6	44.4	58.2	42.0	55.8	
7	44.4	53.3	42.0	50.4	
8	51.3	57.9	49.2	55.2	
9	49.0	56.3	46.6	53.4	
10	49.0	56.3	46.6	53.4	
11	26.6	39.9	25.2	37.8	
12	26.6	39.9	25.2	37.8	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	435	0	OK
2	0	456	0	OK
3	0	441	0	OK
4	0	440	0	OK
5	0	394	0	OK
6	0	418	0	OK
7	0	372	0	OK
8	0	408	0	OK
9	0	394	0	OK
10	0	394	0	OK
11	0	279	0	OK
12	0	279	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	41.4	-40.6	OK
2	43.3	-42.8	OK
3	42.1	-41.4	OK
4	41.4	-41.2	OK
5	37.5	-36.9	OK
6	39.3	-39.2	OK
7	35.5	-34.8	OK
8	38.7	-38.2	OK
9	37.5	-36.9	OK
10	37.5	-36.9	OK
11	26.6	-26.1	OK
12	26.6	-26.1	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#7 Cont		
Bot		2#7x28.00 ft		
d	2.0	4.0	2.0	in
+As	1.20	2.40	1.20	in²
c/d	0.06	0.09	0.06	
+ϕMn	266	508	266	
Top		2#6 Cont		
Top	2#6		2#6	
d'	4.0	2.0	4.0	in
-As	1.76	0.88	1.76	in²
c/d	0.06	0.06	0.06	
-ϕMn	-376	-200	-376	
As min	0.00	1.33	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	37.9		-37.9	
ϕVs	41.3		-41.3	
ϕVn	79.2		-79.2	
Av/s	0.018		0.018	OK
Min Av/s	0.007		N.A.	
s	12.0	12.0	12.0	OK
s max	24.0	24.0	N.A.	

DEFLECTIONS

Comb		
D	0.29	in
L	0.07	in
Lr, S	0.05	in
LS	0.03	in
LT	0.65	in
Ieff D	42,158	in4
Ieff L	38,757	in4
Ieff LS	40,204	in4
λ	1.80	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.56	OK
Max. Steel Area As max Ratio	0.30	OK
Rebars Min Spacing Smin Ratio ..	0.57	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	0.50	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	7404	180	0.02	OK
B	Live Load	4892	360	0.07	OK
C	Long-Term	520	480	0.92	OK
D	Long-Term	520	240	0.46	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

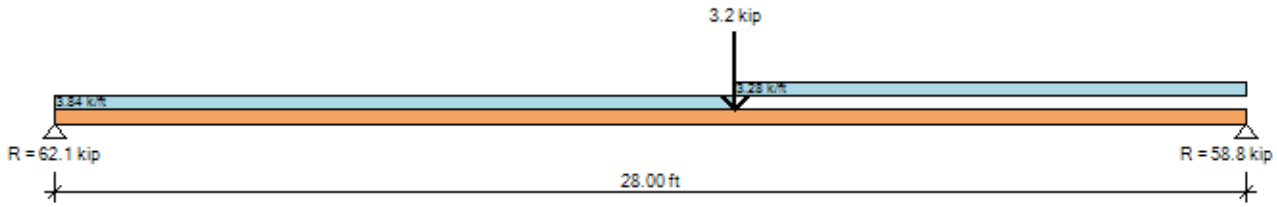
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



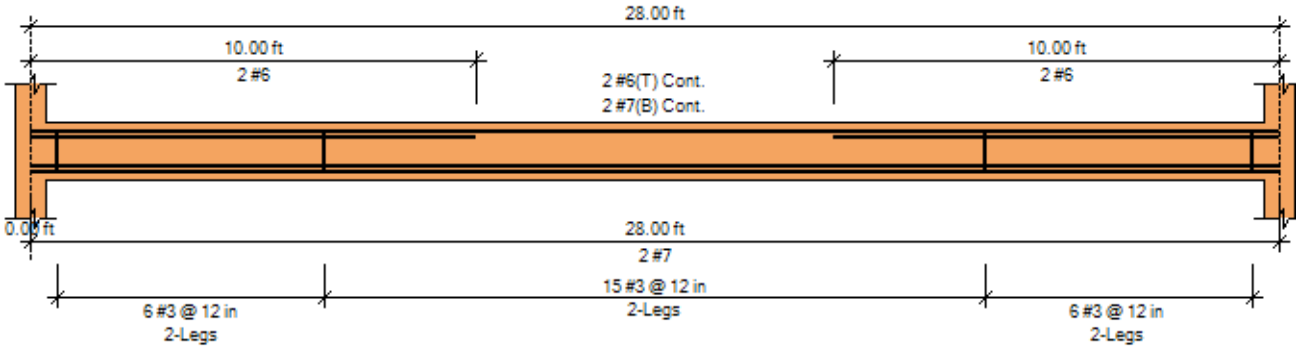
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Deep beam		NG
Total Height h	28.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 3

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	9.00	ft	2	Pinned
3	N.A.		3	Pinned
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb							
1	1	7.6	8.9	19.5	22.8	-0.9	-1.0
2	2	6.4	9.6	16.3	24.6	-0.7	-1.1
3	3	6.4	9.6	16.3	24.6	-0.7	-1.1
4	4	7.3	8.3	18.7	21.1	-0.9	-1.0
5	5	7.3	8.3	18.7	21.1	-0.9	-1.0
6	6	6.4	7.6	16.3	19.5	-0.7	-0.9
7	7	6.4	7.6	16.3	19.5	-0.7	-0.9
8	8	7.3	8.3	18.7	21.1	-0.9	-1.0
9	9	7.3	8.3	18.7	21.1	-0.9	-1.0
10	10	7.3	8.3	18.7	21.1	-0.9	-1.0
11	11	3.8	5.7	9.8	14.6	-0.4	-0.7
12	12	3.8	5.7	9.8	14.6	-0.4	-0.7

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb						
1	0	41	-55	0	-17	OK
2	0	45	-58	0	-17	OK
3	0	45	-58	0	-17	OK
4	0	38	-51	0	-15	OK
5	0	38	-51	0	-15	OK
6	0	35	-47	0	-14	OK
7	0	35	-47	0	-14	OK
8	0	38	-51	0	-15	OK
9	0	38	-51	0	-15	OK
10	0	38	-51	0	-15	OK
11	0	27	-36	0	-11	OK
12	0	27	-36	0	-11	OK

SHEAR FORCES Vu (kip)

Comb					
1	6.5	-10.7	7.3	3.4	OK
2	7.1	-11.6	7.9	3.7	OK
3	7.1	-11.6	7.9	3.7	OK
4	6.0	-9.9	6.7	3.2	OK
5	6.0	-9.9	6.7	3.2	OK
6	5.6	-9.2	6.2	2.9	OK
7	5.6	-9.2	6.2	2.9	OK
8	6.0	-9.9	6.7	3.2	OK
9	6.0	-9.9	6.7	3.2	OK
10	6.0	-9.9	6.7	3.2	OK
11	4.2	-6.9	4.7	2.2	OK
12	4.2	-6.9	4.7	2.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		2#6 Cont		
Bot		0#5x21.00 ft		0#5x7.00 ft		
d	2.0	4.0	2.0	4.0	2.0	in
+As	0.88	0.88	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	0.08	0.08	
+ϕMn	99	99	99	99	99	
Top		2#6 Cont		2#6 Cont		
Top	0#5		0#5		0#5	
d'	4.0	2.0	4.0	2.0	4.0	in
-As	0.88	0.88	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	0.08	0.08	
-ϕMn	-99	-99	-99	-99	-99	
As min	0.00	0.54	0.69	0.00	0.21	in²

SHEAR STRENGTH (kip, in)

ϕVc	19.7	-19.7	19.7	-19.7			
ϕVs	21.5	-21.5	21.5	-21.5			
ϕVn	41.2	-41.2	41.2	-41.2			
Av/s	0.018	0.018	0.018	0.018			OK
Min Av/s	N.A.	N.A.	N.A.	N.A.			
s	12.0	12.0	12.0	12.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	

DEFLECTIONS

Comb			
D	0.04	0.00	in
L	0.01	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.08	0.00	in
Ieff D	16,420	16,420	in4
Ieff L	16,420	16,420	in4
Ieff LS	16,420	16,420	in4
λ	1.65	1.65	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.79	OK
Max. Steel Area As max Ratio	0.19	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	0.27	OK
Min. Stirrups Area Av/s Ratio	0.36	OK
Stirrups Spacing s max Ratio	0.92	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.01	OK
C	Long-Term	3594	480	0.13	OK
D	Long-Term	3594	240	0.07	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

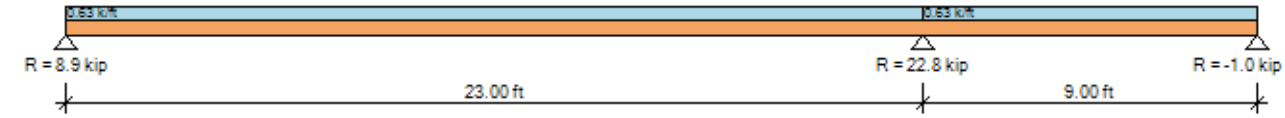


Comb					
1	0.0	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)



At/s (in)	0.0000	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	N.A.	OK



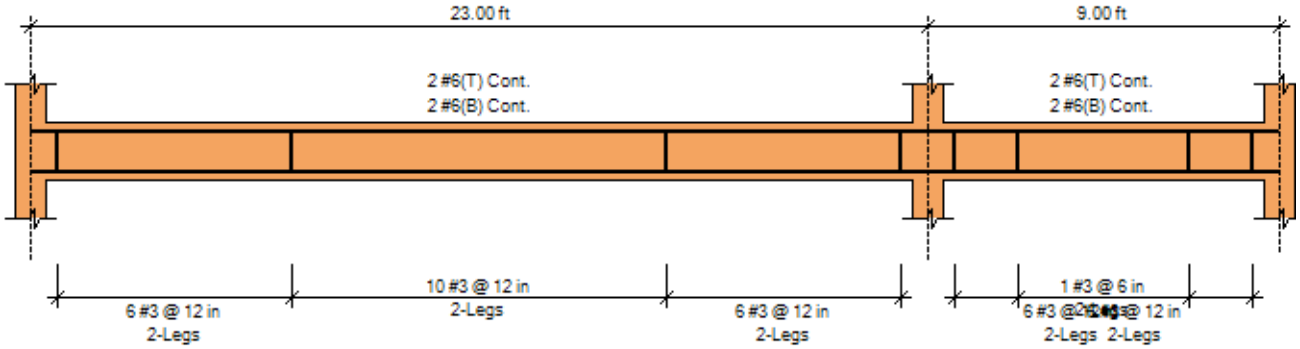
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Rectangular	Not deep beam		OK
Total Height h	28.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	N.A.		
Left Flange Width bfl	N.A.		
Right Flange Width bfr	N.A.		
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	26.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	6.3	8.8	6.3	8.8	
2	7.3	8.0	7.3	8.0	
3	6.3	7.5	6.3	7.5	
4	7.0	9.1	7.0	9.1	
5	6.3	7.5	6.3	7.5	
6	6.3	9.1	6.3	9.1	
7	6.3	7.5	6.3	7.5	
8	7.0	8.0	7.0	8.0	
9	6.3	7.5	6.3	7.5	
10	6.3	7.5	6.3	7.5	
11	3.8	5.7	3.8	5.7	
12	3.8	5.7	3.8	5.7	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	57	0	OK
2	0	52	0	OK
3	0	49	0	OK
4	0	59	0	OK
5	0	49	0	OK
6	0	59	0	OK
7	0	49	0	OK
8	0	52	0	OK
9	0	49	0	OK
10	0	49	0	OK
11	0	37	0	OK
12	0	37	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	7.1	-7.1	OK
2	6.5	-6.5	OK
3	6.1	-6.1	OK
4	7.3	-7.4	OK
5	6.1	-6.1	OK
6	7.3	-7.4	OK
7	6.1	-6.1	OK
8	6.5	-6.5	OK
9	6.1	-6.1	OK
10	6.1	-6.1	OK
11	4.6	-4.6	OK
12	4.6	-4.6	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	
+ϕMn	99	99	99	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	
-ϕMn	-99	-99	-99	
As min	0.00	0.69	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	19.7		-19.7	
ϕVs	21.5		-21.5	
ϕVn	41.2		-41.2	
Av/s	0.018		0.018	OK
Min Av/s	N.A.		N.A.	
s	12.0	12.0	12.0	OK
s max	N.A.	N.A.	N.A.	

DEFLECTIONS

Comb		
D	0.08	in
L	0.00	in
Lr, S	0.02	in
LS	0.00	in
LT	0.15	in
Ieff D	16,420	in4
Ieff L	16,420	in4
Ieff LS	16,420	in4
λ	1.65	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.78	OK
Max. Steel Area As max Ratio	0.19	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.01	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	2151	480	0.22	OK
D	Long-Term	2151	240	0.11	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

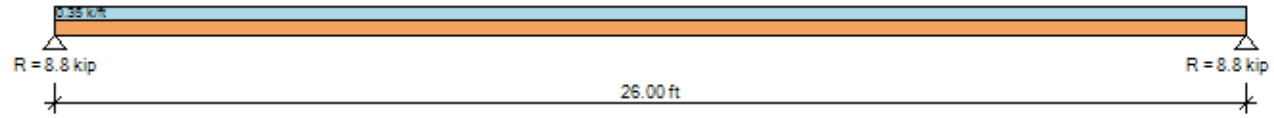
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

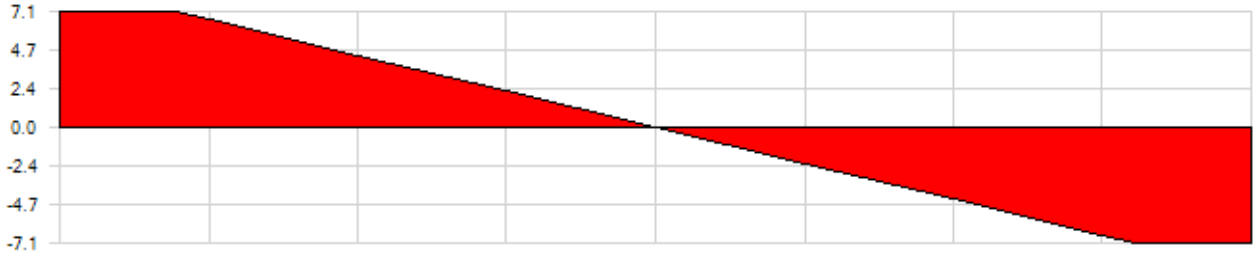
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



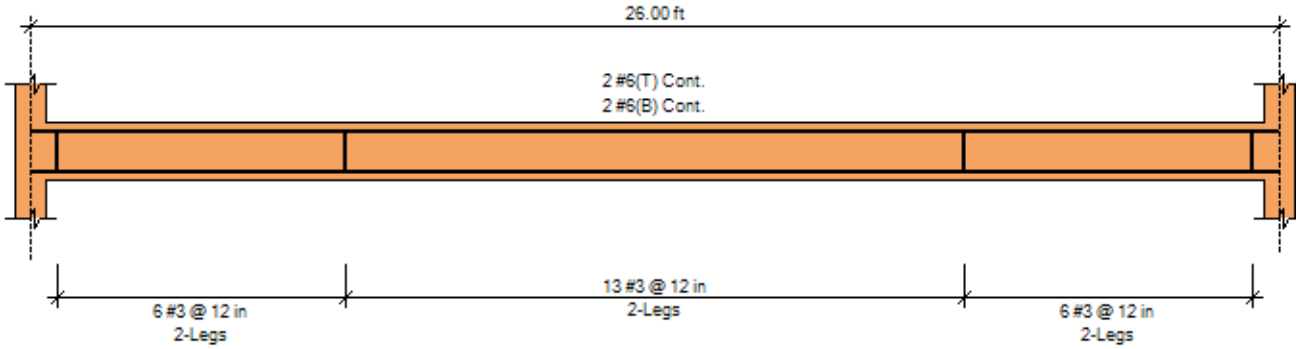
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Inverted-T	Not deep beam		OK
Total Height h	44.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	12.0	in	OK
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	32.2	39.9	32.2	39.9	
2	30.2	40.9	30.2	40.9	
3	28.5	40.1	28.5	40.1	
4	32.5	38.8	32.5	38.8	
5	31.2	36.0	31.2	36.0	
6	28.5	36.9	28.5	36.9	
7	28.5	34.2	28.5	34.2	
8	32.5	36.9	32.5	36.9	
9	31.2	36.0	31.2	36.0	
10	31.2	36.0	31.2	36.0	
11	17.1	25.6	17.1	25.6	
12	17.1	25.6	17.1	25.6	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	229	0	OK
2	0	235	0	OK
3	0	230	0	OK
4	0	223	0	OK
5	0	207	0	OK
6	0	212	0	OK
7	0	197	0	OK
8	0	212	0	OK
9	0	207	0	OK
10	0	207	0	OK
11	0	147	0	OK
12	0	147	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	25.4	-25.4	OK
2	26.0	-26.0	OK
3	25.5	-25.5	OK
4	24.7	-24.7	OK
5	22.9	-22.9	OK
6	23.5	-23.5	OK
7	21.7	-21.7	OK
8	23.5	-23.5	OK
9	22.9	-22.9	OK
10	22.9	-22.9	OK
11	16.3	-16.3	OK
12	16.3	-16.3	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		2#6x19.00 ft		
d	1.8	3.5	1.8	in
+As	0.88	1.76	0.88	in²
c/d	0.05	0.08	0.05	
+ϕMn	163	315	163	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.03	0.03	0.03	
-ϕMn	-169	-177	-169	
As min	0.00	1.75	0.00	in²

SHEAR STRENGTH (kip, in)

ϕVc	32.0		-32.0	
ϕVs	26.1		-26.1	
ϕVn	58.1		-58.1	
Av/s	0.014		0.014	OK
Min Av/s	0.007		N.A.	
s	16.0	16.0	16.0	OK
s max	21.0	N.A.	N.A.	

DEFLECTIONS

Comb		
D	0.13	in
L	0.04	in
Lr, S	0.02	in
LS	0.02	in
LT	0.31	in
Ieff D	33,175	in4
Ieff L	28,230	in4
Ieff LS	30,698	in4
λ	1.77	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

REINFORCEMENT

Min. Steel Area As min Ratio	0.99	OK
Max. Steel Area As max Ratio	0.26	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.24	OK
Shear Vs/Vs max Ratio	0.54	OK
Min. Stirrups Area Av/s Ratio	0.48	OK
Stirrups Spacing s max Ratio	0.76	OK
Stirrups Min. # of Legs Ratio	0.19	OK

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.01	OK
B	Live Load	6473	360	0.06	OK
C	Long-Term	902	480	0.53	OK
D	Long-Term	902	240	0.27	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

MATERIALS

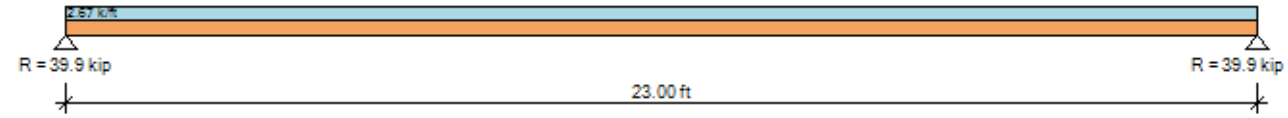
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

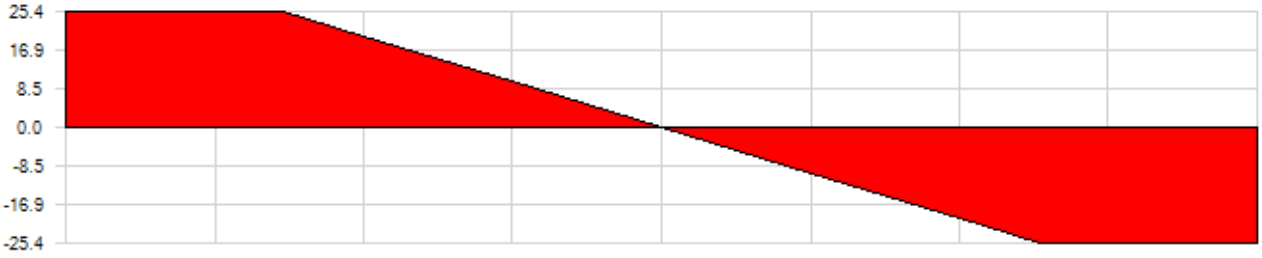
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



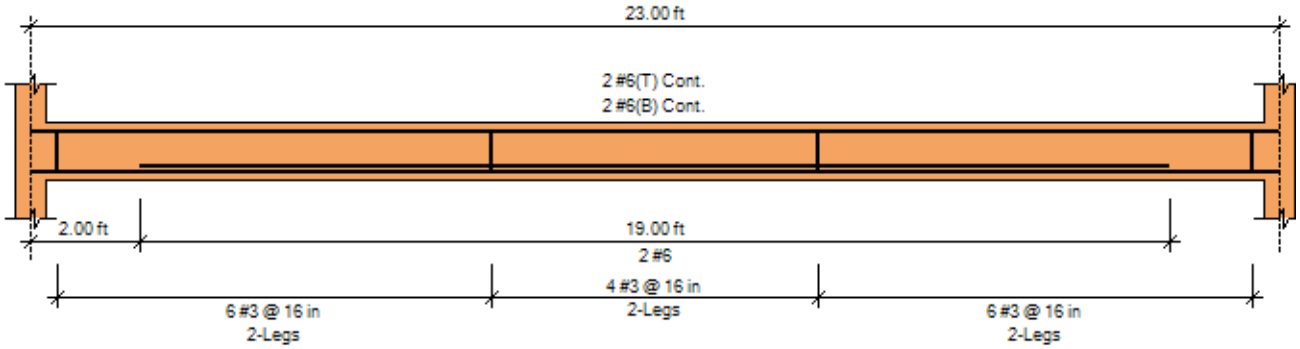
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	40.0	in	
Beam Width b	12.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	1	19.7	27.5	19.7	27.5
2	2	22.9	25.2	22.9	25.2
3	3	19.7	23.6	19.7	23.6
4	4	22.1	28.7	22.1	28.8
5	5	19.7	23.6	19.7	23.6
6	6	19.7	28.7	19.7	28.8
7	7	19.7	23.6	19.7	23.6
8	8	22.1	25.2	22.1	25.2
9	9	19.7	23.6	19.7	23.6
10	10	19.7	23.6	19.7	23.6
11	11	11.8	17.7	11.8	17.7
12	12	11.8	17.7	11.8	17.7

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	158	0	OK
2	0	145	0	OK
3	0	136	0	OK
4	0	165	0	OK
5	0	136	0	OK
6	0	165	0	OK
7	0	136	0	OK
8	0	145	0	OK
9	0	136	0	OK
10	0	136	0	OK
11	0	102	0	OK
12	0	102	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	18.4	-18.4	OK
2	16.8	-16.8	OK
3	15.7	-15.7	OK
4	19.2	-19.2	OK
5	15.7	-15.7	OK
6	19.2	-19.2	OK
7	15.7	-15.7	OK
8	16.8	-16.8	OK
9	15.7	-15.7	OK
10	15.7	-15.7	OK
11	11.8	-11.8	OK
12	11.8	-11.8	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		3#6 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	1.32	1.32	1.32	in²
c/d	0.04	0.04	0.04	
+ϕMn	224	224	224	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.05	0.05	0.05	
-ϕMn	-148	-148	-148	
As min	0.00	1.31	0.00	in²

DEFLECTIONS

Comb		
D	0.03	in
L	0.00	in
Lr, S	0.01	in
LS	0.00	in
LT	0.06	in
Ieff D	90,551	in4
Ieff L	90,551	in4
Ieff LS	90,551	in4
λ	1.82	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	43.3		-43.3	
ϕVs	23.5		-23.5	
ϕVn	66.8		-66.8	
Av/s	0.014		0.014	OK
Min Av/s	N.A.		N.A.	
s	16.0	16.0	16.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	1.00	OK
Max. Steel Area As max Ratio	0.15	OK
Rebars Min Spacing Smin Ratio ..	0.40	OK
Rebars Max Spacing Smax Ratio	0.62	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	4373	480	0.11	OK
D	Long-Term	4373	240	0.05	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

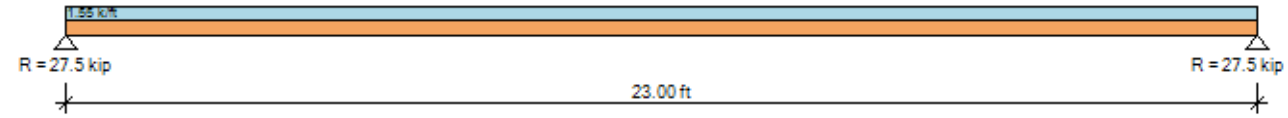
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

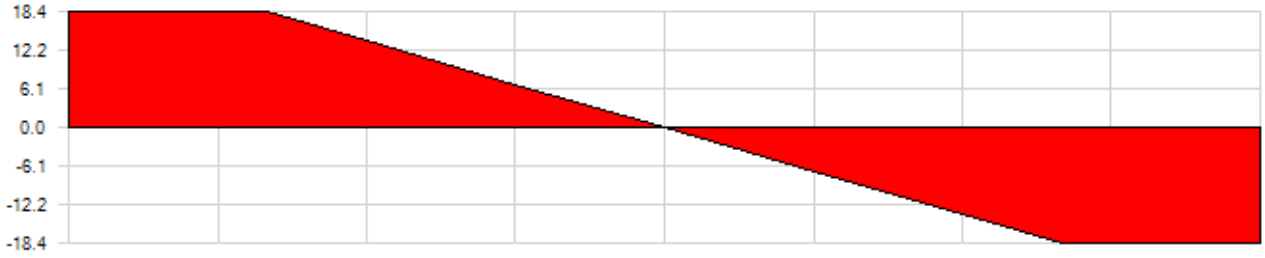
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



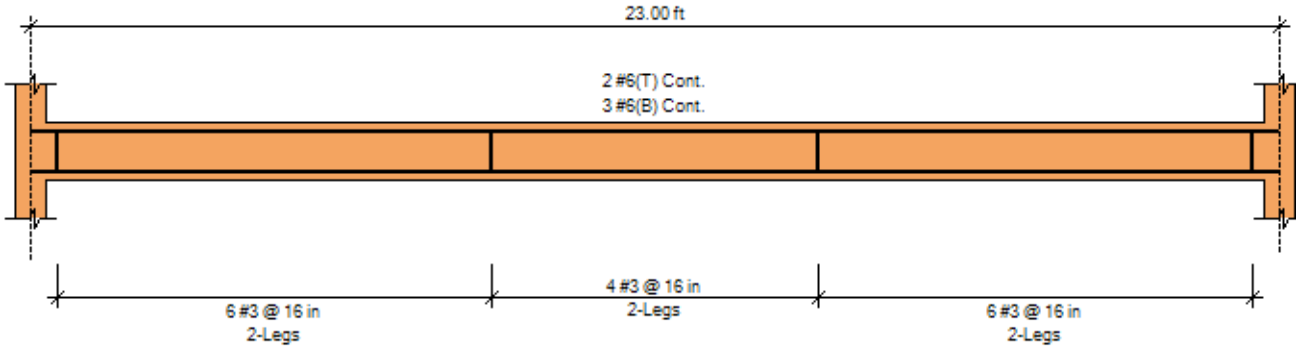
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	40.0	in	
Beam Width b	12.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	6.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	13.7	19.2	13.7	19.2	
2	15.5	17.3	15.5	17.3	
3	13.7	16.4	13.7	16.4	
4	15.1	19.4	15.1	19.4	
5	13.7	16.4	13.7	16.4	
6	13.7	19.4	13.7	19.4	
7	13.7	16.4	13.7	16.4	
8	15.1	17.3	15.1	17.3	
9	13.7	16.4	13.7	16.4	
10	13.7	16.4	13.7	16.4	
11	8.2	12.3	8.2	12.3	
12	8.2	12.3	8.2	12.3	

LOAD COMBINATIONS

	SERVICE	FACTORED
1	D+L	1.4D
2	D+Lr	1.2D+1.6L+0.5Lr
3	D+S	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	1.2D+0.5L+1.6S
6	D+0.6W	1.2D+1.6Lr+0.5W
7	D+0.7E	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	0.9D+W
12	0.6D+0.7E	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	110	0	OK
2	0	100	0	OK
3	0	94	0	OK
4	0	111	0	OK
5	0	94	0	OK
6	0	111	0	OK
7	0	94	0	OK
8	0	100	0	OK
9	0	94	0	OK
10	0	94	0	OK
11	0	71	0	OK
12	0	71	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	12.8	-12.8	OK
2	11.6	-11.6	OK
3	10.9	-10.9	OK
4	12.9	-12.9	OK
5	10.9	-10.9	OK
6	12.9	-12.9	OK
7	10.9	-10.9	OK
8	11.6	-11.6	OK
9	10.9	-10.9	OK
10	10.9	-10.9	OK
11	8.2	-8.2	OK
12	8.2	-8.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.04	0.04	0.04	
+ϕMn	150	150	150	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.05	0.05	0.05	
-ϕMn	-148	-148	-148	
As min	0.00	0.88	0.00	in²

DEFLECTIONS

Comb		
D	0.03	in
L	0.00	in
Lr, S	0.00	in
LS	0.00	in
LT	0.05	in
Ieff D	79,495	in4
Ieff L	79,495	in4
Ieff LS	79,495	in4
λ	1.82	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	43.3		-43.3	
ϕVs	23.5		-23.5	
ϕVn	66.8		-66.8	
Av/s	0.014		0.014	OK
Min Av/s	N.A.		N.A.	
s	16.0	16.0	16.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	1.00	OK
Max. Steel Area As max Ratio	0.10	OK
Rebars Min Spacing Smin Ratio ..	0.17	OK
Rebars Max Spacing Smax Ratio	0.62	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	5563	480	0.09	OK
D	Long-Term	5563	240	0.04	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

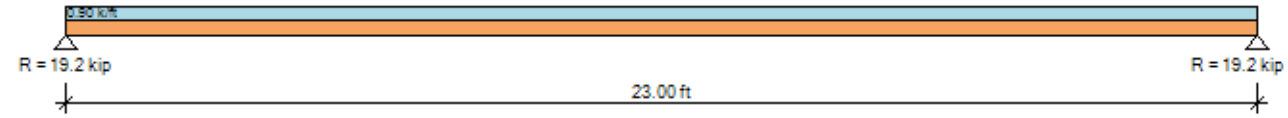
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

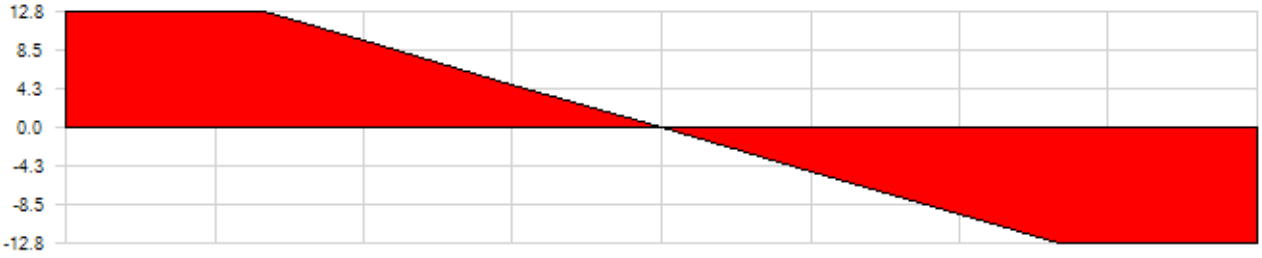
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



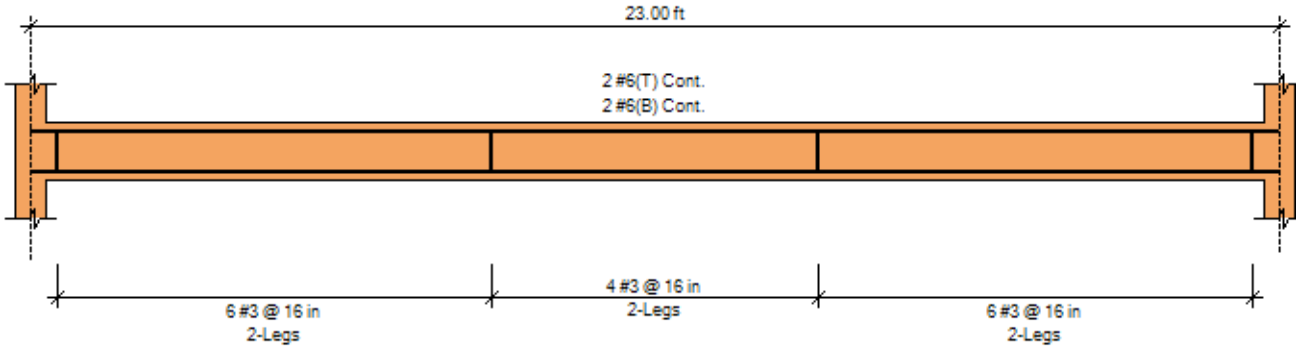
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	40.0	in	
Beam Width b	12.0	in	
Flange Thickness tf	8.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	12.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	23.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	12.8	17.9	12.8	17.9	
2	14.3	16.1	14.3	16.1	
3	12.8	15.3	12.8	15.3	
4	13.9	17.7	13.9	17.7	
5	12.8	15.3	12.8	15.3	
6	12.8	17.7	12.8	17.7	
7	12.8	15.3	12.8	15.3	
8	13.9	16.1	13.9	16.1	
9	12.8	15.3	12.8	15.3	
10	12.8	15.3	12.8	15.3	
11	7.7	11.5	7.7	11.5	
12	7.7	11.5	7.7	11.5	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	103	0	OK
2	0	92	0	OK
3	0	88	0	OK
4	0	102	0	OK
5	0	88	0	OK
6	0	102	0	OK
7	0	88	0	OK
8	0	92	0	OK
9	0	88	0	OK
10	0	88	0	OK
11	0	66	0	OK
12	0	66	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	11.9	-11.9	OK
2	10.7	-10.7	OK
3	10.2	-10.2	OK
4	11.8	-11.8	OK
5	10.2	-10.2	OK
6	11.8	-11.8	OK
7	10.2	-10.2	OK
8	10.7	-10.7	OK
9	10.2	-10.2	OK
10	10.2	-10.2	OK
11	7.7	-7.7	OK
12	7.7	-7.7	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.04	0.04	0.04	
+ϕMn	152	152	152	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.05	0.05	0.05	
-ϕMn	-148	-148	-148	
As min	0.00	0.81	0.00	in²

DEFLECTIONS

Comb		
D	0.02	in
L	0.00	in
Lr, S	0.00	in
LS	0.00	in
LT	0.04	in
Ieff D	89,185	in4
Ieff L	89,185	in4
Ieff LS	89,185	in4
λ	1.82	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	43.3		-43.3	
ϕVs	23.5		-23.5	
ϕVn	66.8		-66.8	
Av/s	0.014		0.014	OK
Min Av/s	N.A.		N.A.	
s	16.0	16.0	16.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.92	OK
Max. Steel Area As max Ratio	0.10	OK
Rebars Min Spacing Smin Ratio ..	0.17	OK
Rebars Max Spacing Smax Ratio	0.62	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	6725	480	0.07	OK
D	Long-Term	6725	240	0.04	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

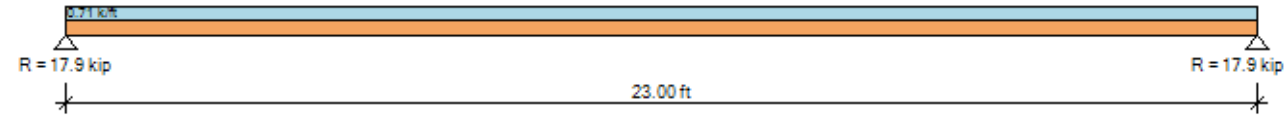
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

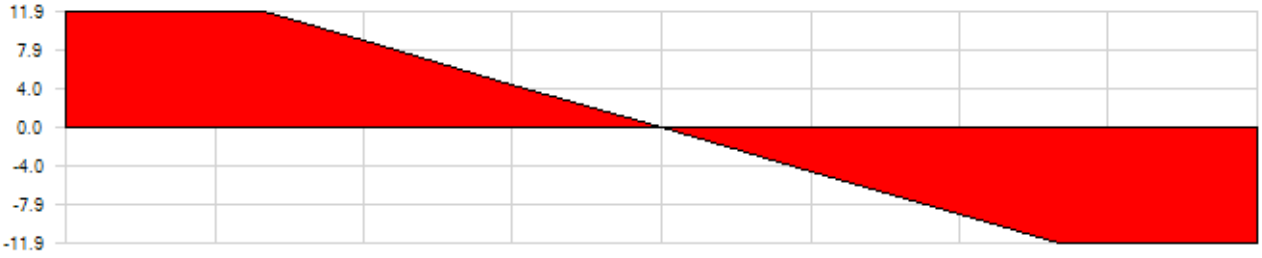
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



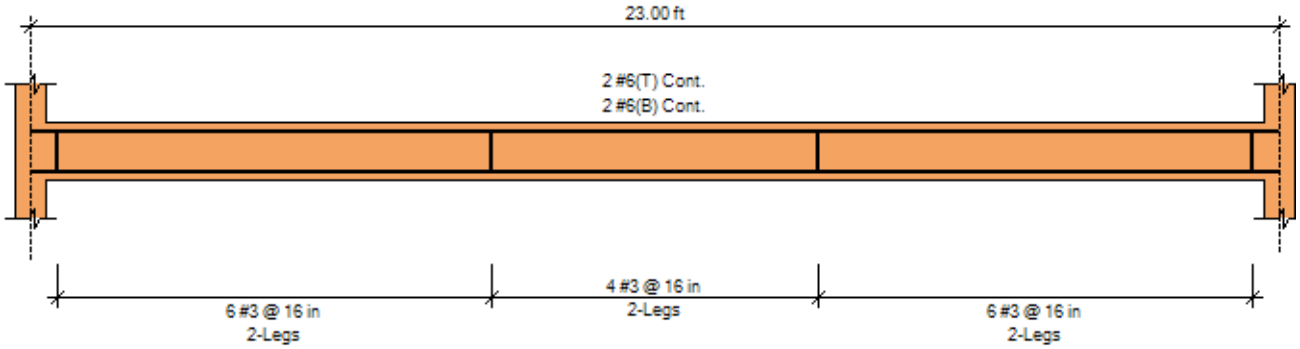
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Not deep beam		OK
Total Height h	30.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	5.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	8.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	13.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	1	3.3	4.6	3.3	4.6
2	2	3.9	4.2	3.9	4.2
3	3	3.3	3.9	3.3	3.9
4	4	3.7	5.0	3.7	5.0
5	5	3.3	3.9	3.3	3.9
6	6	3.3	5.0	3.3	5.0
7	7	3.3	3.9	3.3	3.9
8	8	3.7	4.2	3.7	4.2
9	9	3.3	3.9	3.3	3.9
10	10	3.3	3.9	3.3	3.9
11	11	2.0	2.9	2.0	2.9
12	12	2.0	2.9	2.0	2.9

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	0	15	0	OK
2	0	14	0	OK
3	0	13	0	OK
4	0	16	0	OK
5	0	13	0	OK
6	0	16	0	OK
7	0	13	0	OK
8	0	14	0	OK
9	0	13	0	OK
10	0	13	0	OK
11	0	10	0	OK
12	0	10	0	OK

SHEAR FORCES Vu (kip)

Comb			
1	2.7	-2.7	OK
2	2.5	-2.5	OK
3	2.3	-2.3	OK
4	2.9	-2.9	OK
5	2.3	-2.3	OK
6	2.9	-2.9	OK
7	2.3	-2.3	OK
8	2.5	-2.5	OK
9	2.3	-2.3	OK
10	2.3	-2.3	OK
11	1.7	-1.7	OK
12	1.7	-1.7	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#5 Cont		
Bot		0#5x6.00 ft		
d	2.0	2.0	2.0	in
+As	0.62	0.62	0.62	in²
c/d	0.05	0.05	0.05	
+ϕMn	79	79	79	
Top		2#5 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.62	0.62	0.62	in²
c/d	0.07	0.07	0.07	
-ϕMn	-76	-76	-76	
As min	0.00	0.17	0.00	in²

DEFLECTIONS

Comb		
D	0.00	in
L	0.00	in
Lr, S	0.00	in
LS	0.00	in
LT	0.01	in
Ieff D	24,959	in4
Ieff L	24,959	in4
Ieff LS	24,959	in4
λ	1.76	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	21.3		-21.3	
ϕVs	23.1		-23.1	
ϕVn	44.4		-44.4	
Av/s	0.018		0.018	OK
Min Av/s	N.A.		N.A.	
s	12.0	6.0	12.0	OK
s max	N.A.	N.A.	N.A.	

REINFORCEMENT

Min. Steel Area As min Ratio	0.28	OK
Max. Steel Area As max Ratio	0.13	OK
Rebars Min Spacing Smin Ratio ..	0.50	OK
Rebars Max Spacing Smax Ratio	0.26	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	180	0.00	OK
B	Live Load	9999	360	0.00	OK
C	Long-Term	9999	480	0.02	OK
D	Long-Term	9999	240	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

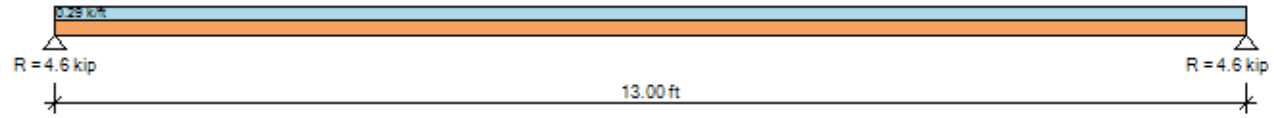
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

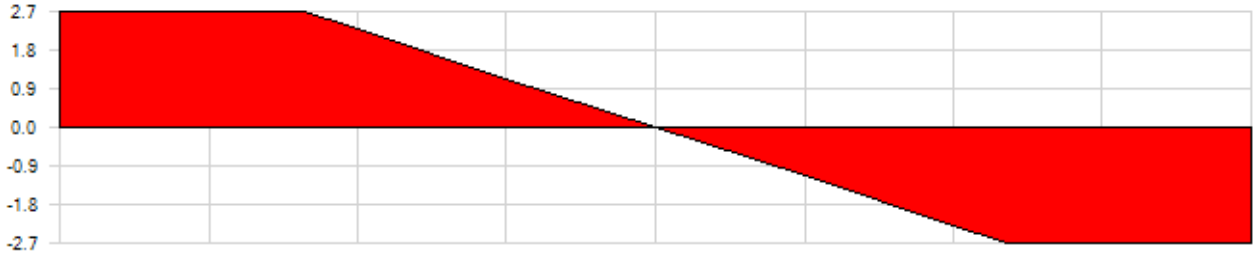
Comb			
1	0.0	0.0	OK
2	0.0	0.0	OK
3	0.0	0.0	OK
4	0.0	0.0	OK
5	0.0	0.0	OK
6	0.0	0.0	OK
7	0.0	0.0	OK
8	0.0	0.0	OK
9	0.0	0.0	OK
10	0.0	0.0	OK
11	0.0	0.0	OK
12	0.0	0.0	OK

TORSION STRENGTH (k-ft)

At/s (in)	0.0000	0.0000	
Al (in²)	0.00	0.00	
ϕTn (kip)	0.0	0.0	
Stress (psi)	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	OK



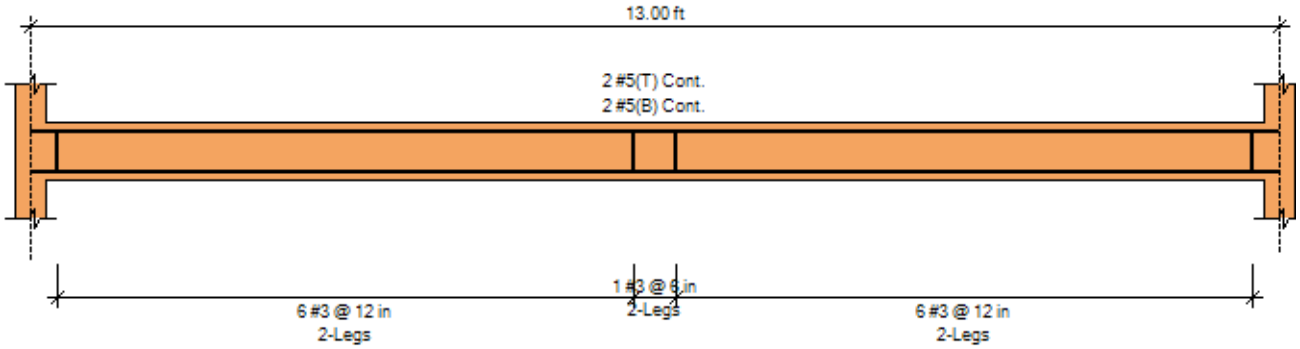
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Deep beam		NG
Total Height h	30.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	5.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	6.0	in	NG
Left Cantilever	0.00	ft	
Right Cantilever	4.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	4.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	-3.3	-4.6	10.1	14.1	
2	-4.0	-4.3	11.9	13.0	
3	-3.3	-4.0	10.1	12.1	
4	-3.8	-5.1	11.5	15.1	
5	-3.3	-4.0	10.1	12.1	
6	-3.3	-5.1	10.1	15.1	
7	-3.3	-4.0	10.1	12.1	
8	-3.8	-4.3	11.5	13.0	
9	-3.3	-4.0	10.1	12.1	
10	-3.3	-4.0	10.1	12.1	
11	-2.0	-3.0	6.0	9.0	
12	-2.0	-3.0	6.0	9.0	

LOAD COMBINATIONS

	SERVICE		FACTORED
1	D+L	1	1.4D
2	D+Lr	2	1.2D+1.6L+0.5Lr
3	D+S	3	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	4	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	5	1.2D+0.5L+1.6S
6	D+0.6W	6	1.2D+1.6Lr+0.5W
7	D+0.7E	7	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	8	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	9	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	10	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	11	0.9D+W
12	0.6D+0.7E	12	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	-8	0	-23	OK
2	-7	0	-22	OK
3	-7	0	-20	OK
4	-8	0	-25	OK
5	-7	0	-20	OK
6	-8	0	-25	OK
7	-7	0	-20	OK
8	-7	0	-22	OK
9	-7	0	-20	OK
10	-7	0	-20	OK
11	-5	0	-15	OK
12	-5	0	-15	OK

SHEAR FORCES Vu (kip)

Comb				
1	-6.6	-5.0	5.0	OK
2	-6.1	-4.7	4.7	OK
3	-5.7	-4.3	4.3	OK
4	-7.1	-5.5	5.5	OK
5	-5.7	-4.3	4.3	OK
6	-7.1	-5.5	5.5	OK
7	-5.7	-4.3	4.3	OK
8	-6.1	-4.7	4.7	OK
9	-5.7	-4.3	4.3	OK
10	-5.7	-4.3	4.3	OK
11	-4.3	-3.2	3.2	OK
12	-4.3	-3.2	3.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x2.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.06	0.06	0.06	
+ϕMn	109	109	109	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	
-ϕMn	-107	-107	-107	
As min	0.09	0.00	0.27	in²

DEFLECTIONS

Comb			
D	0.00	0.00	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.00	0.00	in
Ieff D	23,719	20,095	in4
Ieff L	23,719	20,095	in4
Ieff LS	23,719	20,095	in4
λ	1.67	2.00	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	21.3		-21.3	21.3	
ϕVs	23.1		-23.1	23.1	
ϕVn	44.4		-44.4	44.4	
Av/s	0.018		0.018	0.018	OK
Min Av/s	N.A.		N.A.	N.A	
s	12.0	6.0	12.0	12.0	OK
s max	N.A.	N.A.	N.A.	N.A	

REINFORCEMENT

Min. Steel Area As min Ratio	0.31	OK
Max. Steel Area As max Ratio	0.18	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	90	0.00	OK
B	Live Load	9999	180	0.00	OK
C	Long-Term	9709	240	0.02	OK
D	Long-Term	9709	120	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

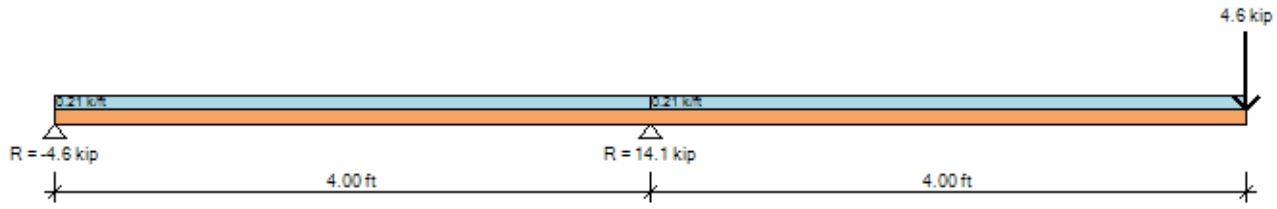
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

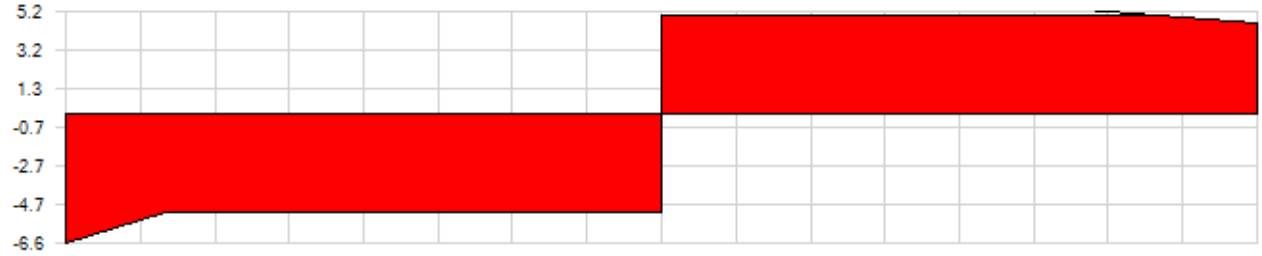
Comb				
1	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)

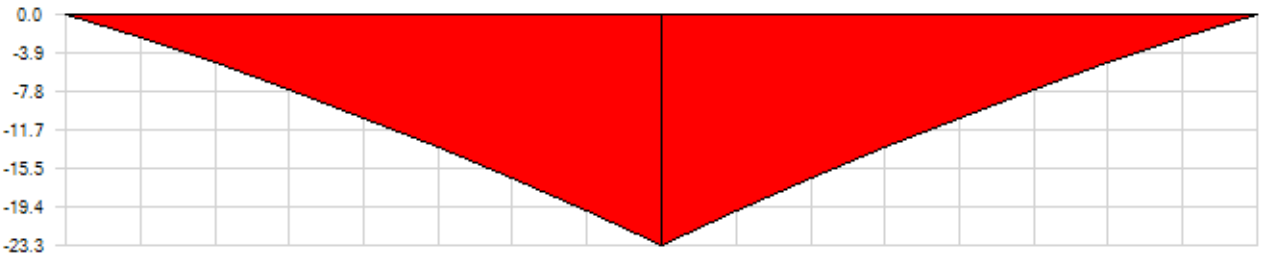
At/s (in)	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	OK



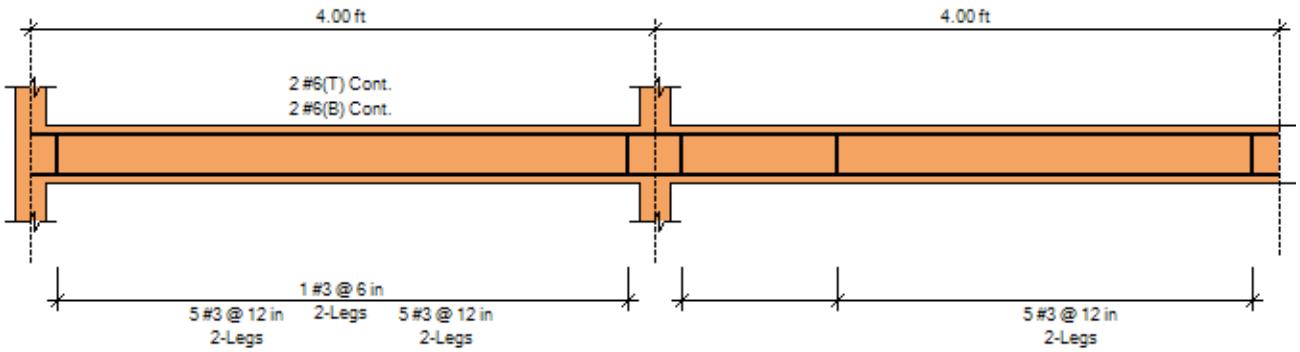
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION

CONCRETE BEAM DESIGN

GEOMETRY

Type ...Spandrel	Deep beam		NG
Total Height h	30.0	in	
Beam Width b	8.0	in	
Flange Thickness tf	5.0	in	
Left Flange Width bfl	N.A.		
Right Flange Width bfr	4.0	in	OK
Left Cantilever	0.00	ft	
Right Cantilever	4.00	ft	

BEAM SUPPORTS

Number of Supports 2

Span	Length		Support	Type
1	4.00	ft	1	Pinned
2	N.A.		2	Pinned
3	N.A.		3	N.A.
4	N.A.		4	N.A.
5	N.A.		5	N.A.
			6	N.A.

SUPPORT REACTIONS (kip)

Comb					
1	-3.3	-4.6	10.0	14.0	
2	-4.0	-4.3	11.8	12.9	
3	-3.3	-4.0	10.0	12.0	
4	-3.8	-5.1	11.4	15.0	
5	-3.3	-4.0	10.0	12.0	
6	-3.3	-5.1	10.0	15.0	
7	-3.3	-4.0	10.0	12.0	
8	-3.8	-4.3	11.4	12.9	
9	-3.3	-4.0	10.0	12.0	
10	-3.3	-4.0	10.0	12.0	
11	-2.0	-3.0	6.0	9.0	
12	-2.0	-3.0	6.0	9.0	

LOAD COMBINATIONS

	SERVICE	FACTORED
1	D+L	1.4D
2	D+Lr	1.2D+1.6L+0.5Lr
3	D+S	1.2D+1.6L+0.5S
4	D+0.75L+0.75Lr	1.2D+0.5L+1.6Lr
5	D+0.75L+0.75S	1.2D+0.5L+1.6S
6	D+0.6W	1.2D+1.6Lr+0.5W
7	D+0.7E	1.2D+1.6S+0.5W
8	D+0.75L+0.75Lr+0.45W	1.2D+0.5L+0.5Lr+W
9	D+0.75L+0.75S+0.45W	1.2D+0.5L+0.5S+W
10	D+0.75L+0.75S+0.525E	1.2D+0.5L+0.2S+E
11	0.6D+0.6W	0.9D+W
12	0.6D+0.7E	0.9D+E

BENDING MOMENTS Mu (k-ft)

Comb				
1	-8	0	-23	OK
2	-7	0	-22	OK
3	-7	0	-20	OK
4	-8	0	-25	OK
5	-7	0	-20	OK
6	-8	0	-25	OK
7	-7	0	-20	OK
8	-7	0	-22	OK
9	-7	0	-20	OK
10	-7	0	-20	OK
11	-5	0	-15	OK
12	-5	0	-15	OK

SHEAR FORCES Vu (kip)

Comb				
1	-6.6	-5.0	5.0	OK
2	-6.1	-4.7	4.7	OK
3	-5.6	-4.3	4.3	OK
4	-7.1	-5.5	5.5	OK
5	-5.6	-4.3	4.3	OK
6	-7.1	-5.5	5.5	OK
7	-5.6	-4.3	4.3	OK
8	-6.1	-4.7	4.7	OK
9	-5.6	-4.3	4.3	OK
10	-5.6	-4.3	4.3	OK
11	-4.2	-3.2	3.2	OK
12	-4.2	-3.2	3.2	OK

CONCRETE BEAM DESIGN

BENDING STRENGTH (k-ft)

Bot		2#6 Cont		
Bot		0#5x2.00 ft		
d	2.0	2.0	2.0	in
+As	0.88	0.88	0.88	in²
c/d	0.06	0.06	0.06	
+ϕMn	109	109	109	
Top		2#6 Cont		
Top	0#5		0#5	
d'	2.0	2.0	2.0	in
-As	0.88	0.88	0.88	in²
c/d	0.08	0.08	0.08	
-ϕMn	-107	-107	-107	
As min	0.09	0.00	0.27	in²

DEFLECTIONS

Comb			
D	0.00	0.00	in
L	0.00	0.00	in
Lr, S	0.00	0.00	in
LS	0.00	0.00	in
LT	0.00	0.00	in
Ieff D	22,596	20,095	in4
Ieff L	22,596	20,095	in4
Ieff LS	22,596	20,095	in4
λ	1.67	2.00	

LS = 0.30(L+Lr) LT = L+Lr+λ(D+LS)

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-10/16

SHEAR STRENGTH (kip, in)

ϕVc	21.3		-21.3	21.3	
ϕVs	23.1		-23.1	23.1	
ϕVn	44.4		-44.4	44.4	
Av/s	0.018		0.018	0.018	OK
Min Av/s	N.A.		N.A.	N.A	
s	12.0	6.0	12.0	12.0	OK
s max	N.A.	N.A.	N.A.	N.A	

REINFORCEMENT

Min. Steel Area As min Ratio	0.31	OK
Max. Steel Area As max Ratio	0.18	OK
Rebars Min Spacing Smin Ratio ..	0.53	OK
Rebars Max Spacing Smax Ratio	0.25	OK
Shear Vs/Vs max Ratio	N.A.	
Min. Stirrups Area Av/s Ratio	N.A.	
Stirrups Spacing s max Ratio	N.A.	
Stirrups Min. # of Legs Ratio	N.A.	

MAX. DEFLECTIONS L/δ

Type	Load	L/δ	Allow.	Ratio	
A	Lr, S, W	9999	90	0.00	OK
B	Live Load	9999	180	0.00	OK
C	Long-Term	9772	240	0.02	OK
D	Long-Term	9772	120	0.01	OK

A - Roofs not attached to non-structural elements.
B - Floors not attached to non-structural elements.
C - Attached to elements likely to be damaged.
D - Attached to elements not likely to be damaged.

MATERIALS

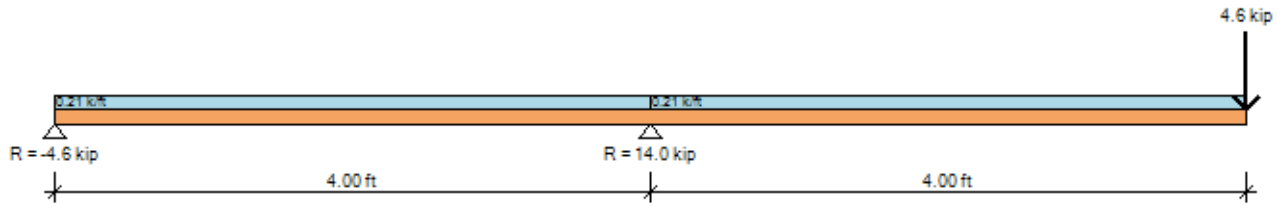
Concrete Strength f'c	4.0	ksi
Compression Strain Limit	0.003	
Concrete density	150	pcf
Concrete Stress-Strain	Hognestad	
Rebar Steel Strength fy	60.0	ksi
Steel Stress-Strain	Elasto-plastic	

TORSION MOMENTS Tu (k-ft)

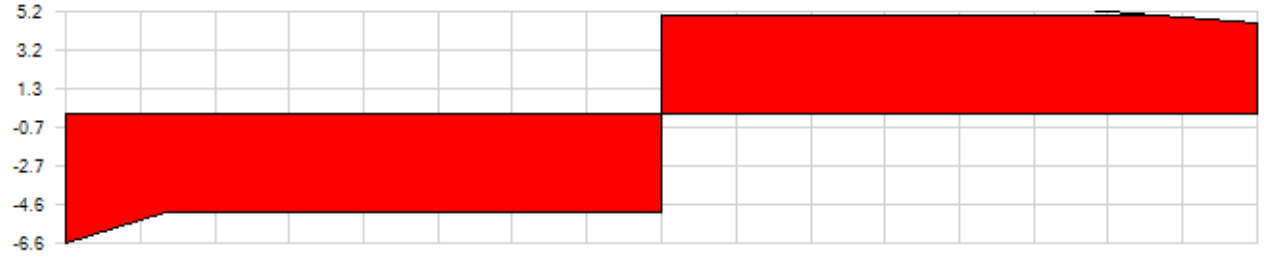
Comb				
1	0.0	0.0	0.0	OK
2	0.0	0.0	0.0	OK
3	0.0	0.0	0.0	OK
4	0.0	0.0	0.0	OK
5	0.0	0.0	0.0	OK
6	0.0	0.0	0.0	OK
7	0.0	0.0	0.0	OK
8	0.0	0.0	0.0	OK
9	0.0	0.0	0.0	OK
10	0.0	0.0	0.0	OK
11	0.0	0.0	0.0	OK
12	0.0	0.0	0.0	OK

TORSION STRENGTH (k-ft)

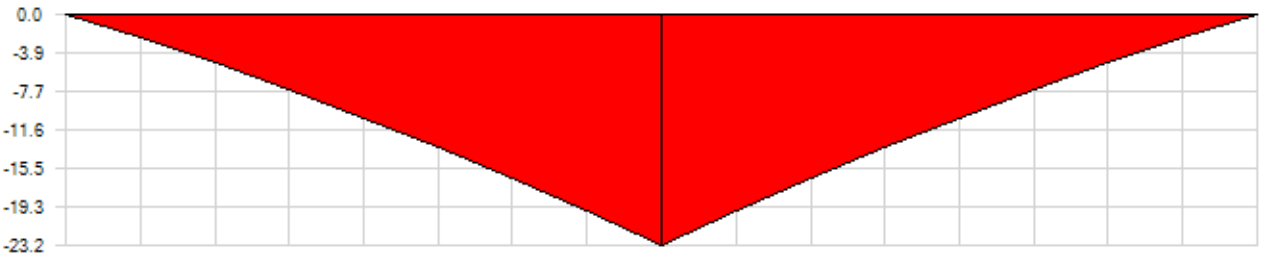
At/s (in)	0.0000	0.0000	0.0000	
Al (in²)	0.00	0.00	0.00	
ϕTn (kip)	0.0	0.0	0.0	
Stress (psi)	N.A.	N.A.	N.A.	OK
Total At/s (in)	N.A.	N.A.	N.A.	OK



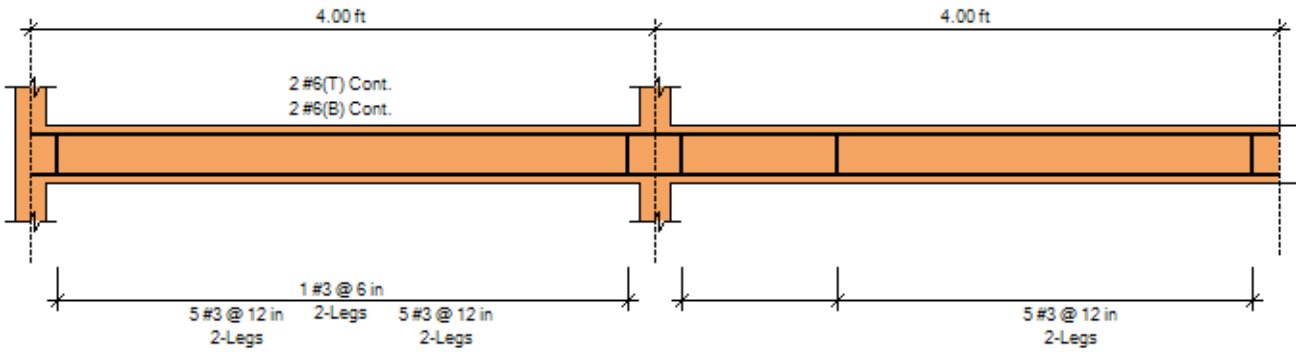
Comb: 1.4D



SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)



BEAM ELEVATION



Steel Columns Calculations

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

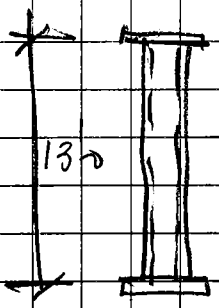
ENG: _____ SHEET: _____ OF _____

Steel Column (A)

$P_d = 30 + 5.8 = 35.8^k$

$P_L = 8 + 5.8 = 13.8^k$

↑ From Roof Truss Girder



HSS 4x4x3/8

Steel Column (B)

$P_d = 37^k$

$P_L = 11^k$

Same HSS 4x4x3/8

Steel Column (C)

$P_d = 28^k + 3.9^k = 31.9^k$

$P_L = 8^k + 3.9^k = 12^k$

↑ From Roof Girder

Same HSS 4x4x3/8

Columns (D), (E), (F) & (G) All have lower total load than (A), thus use same column.

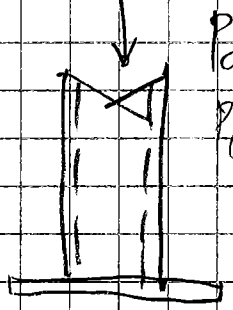
GF CONSULTING ENGINEERS, Inc.

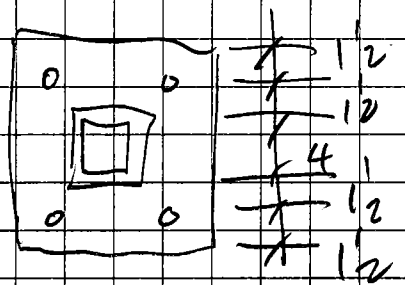
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Steel Columns base Plate





Base PL
3/4 x 10 x 10
w/ (4) 3/4" Ø x F1554-36 AB

see comp. Print out

ASDIP Steel 5.6.2.3

BASE PLATE / ANCHORAGE DESIGN

GEOMETRY

Column Section HSS4X4X3/8

		Width		Length		
Column		4.0		4.0	in	
Plate		12.0		12.0	in	OK
Concrete Support	Wp1	8.0	Lp1	8.0	in	OK
	Wp2	8.0	Lp2	8.0	in	OK
Rod Edge		2.0		2.0	in	OK
Thickness of Grout				0.0	in	

MATERIALS

Plate Steel Strength Fy	36.0	ksi
Pier Concrete Strength f'c	4.0	ksi

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Vertical Load P	35.0	12.0	0.0	0.0	0.0	0.0	kip
Bending Moment Mx ...	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Bending Moment Mz ...	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Horizontal Load Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Horizontal Load Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

AXIALLY LOADED PLATES (Comb. 1.2D+1.6L+0.5Lr)

Pu = 1.2 * 35.0 + 1.6 * 12.0 + 0.5 * 0.0 = 61.2 kip

Bearing stress $fp = P / (W * L) =$ 61.2 / (12.0 * 12.0) = 0.4 ksi

Bearing strength $Fp = 0.85 * f'c \sqrt{\frac{A2}{A1}}$ $= 0.85 * 4.0 * \sqrt{\frac{256}{144}}$ = 4.5 ksi — ACI 22.8.3.2

Bearing under-strength factor ϕ = 0.65 — AISC J8

Design bearing strength $Fp = \phi Fpn =$ 0.65 * 4.5 = 2.9 ksi

Bearing strength ratio = $\frac{fp}{Fp} = \frac{0.4}{2.9}$ = 0.14 < 1.0 OK

Critical section m = $0.5 * (L - 0.95 * d) =$ 0.5 * (12.0 - 0.95 *4.0) = 4.1 in — AISC (14-3)

Critical section n = $0.5 * (W - 0.95 * b) =$ 0.5 * (12.0 - 0.95 *4.0) = 4.1 in

Controlling section $k = Max (m, n) =$ Max (4.1, 4.1) = 4.1 in

Moment due to tension between flanges Mt = 0.0 k-in/in

Moment due to bearing $Mb = fp * k^2 / 2 =$ 0.4 * 4.1² / 2 = 3.6 k-in/in

Flexure under-strength factor ϕ = 0.90 — AISC F1

Plate thickness $t = \sqrt{\frac{4 * Mc}{Phi * fy}}$ $= \sqrt{\frac{4 * 3.6}{0.9 * 36}}$ = 0.66 in — AISC-DG#1 3.1.2

BASE PLATES WITH MOMENT (Comb. 1.2D+1.6L+0.5Lr)

Mxu = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 = 0.0 k-ft — *ez = Mx / P =* 0.0 / 61.2 = 0.0 in

Mzu = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 = 0.0 k-ft — *ex = Mz / P =* 0.0 / 61.2 = 0.0 in

Max bearing stress fp = 0.4 ksi (See plan)

Bearing strength ratio = $\frac{fp}{Fp} = \frac{0.4}{2.9} = 0.14$ < 1.0 OK

Max rod tension = 0.0 kip (See plan)

- Moments calculated as the bearing stress times the lever arm to the critical section

X-Moment due to bearing Mbx = 3.6 k-in/in (Controls)

Z-Moment due to bearing Mbz = 3.6 k-in/in

X-Moment due to tension *Mtx = T * (n - Edge) / Min (L, 2 * (n - Edge))*

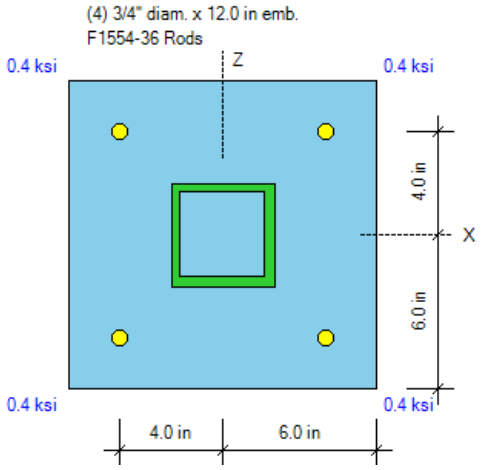
Mtx = 0.0 * (4.1 - 2.0) / Min (12.0, 2 * (4.1 - 2.0) = 0.0 k-in/in

Z-Moment due to tension *Mtz = T * (m - Edge) / Min (W, 2 * (m - Edge))*

Mtz = 0.0 * (4.1 - 2.0) / Min (12.0, 2 * (4.1 - 2.0) = 0.0 k-in/in

- Plate forced to remain in the elastic range to ensure strain compatibility (Mn = My)

Plate thickness $t = \sqrt{\frac{6 * Mc}{Phi * fy}} = \sqrt{\frac{6 * 3.6}{0.9 * 36}} = 0.81$ in — AISC-DG#1 3.1.2



DESIGN CODES

Steel Design	AISC 360-16
Anchorage Design	ACI 318-19
Load Combinations	ASCE 7-10/16

ANCHORAGE DESIGN

Rod Material Specification ..... F1554-36 , Use (4) Rods , fya = 36.0 ksi, futa = 58.0 ksi

Anchor Rod Size ....3/4" diam. x 12.0 in emb. , Ase = 0.33 in² , Abrg = 0.91 in²

- Tension Analysis (Comb. 1.4D) — ACI 17.6

Total tension force Nu = 0.0 kip , # of tension rods = 0 , Tension force per rod Niu = 0.0 kip

- Steel strength of anchors in tension — ACI Eq. (17.6.1.1)

Steel strength $Nsa = Ase * futa =$ 0.33 * 58.0 = 19.4 kip — ACI Eq. (17.6.1.2)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Steel strength ratio = $\frac{Niu}{\phi\, Nsa} = \frac{0.0}{0.75 * 19.4} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete breakout strength of anchors in tension — ACI 17.6.2

Anchor reinforcement: Use 2 bars #5 per rod

Bar strength $Nrg = Area * Bars\ per\ rod * Tension\ rods * fy =$ 0.31 * 2.0 * 0 * 60 = 0.0 kip

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Bar strength ratio = $\frac{Nu}{\phi\, Nrg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Anchor effective embedment $hef =$ 12.0 in — ACI 17.6.2.1.2

Anchor group area Anc = 0.0 in² (See the tension breakout area plan) — ACI 17.6.2.1.1

Single anchor area $Anco = 9\, hef^2 =$ 9 * (12.0)² = 1296.0 in² — ACI Eq. (17.6.2.1.4)

Single anchor strength $Nb = 24 \sqrt{f'c} * hef^{1.5} = 24 \sqrt{4000} * 12^{1.5}$ = 63.1 kip — ACI Eq. (17.6.2.2.1)

Eccentricity factor $\Psi ec =$ 1.00 (No eccentric load) — ACI 17.6.2.3

Edge effects factor $\Psi ed = 0.7 + 0.3 \frac{Ca,min}{1.5\, hef} = 0.7 + 0.3 \frac{0.0}{1.5 * 12.0} = 0.70$ — ACI 17.6.2.4.1

Cracking factor $\Psi cn =$ 1.00 (Cracked concrete at service load level) — ACI 17.6.2.5.1

Breakout strength $Ncbg = \frac{Anc}{Anco}\, \Psi ec\, \Psi ed\, \Psi cn\, Nb$

$Ncbg = \frac{0.0}{1296.0}$ 1.00 * 0.70 * 1.00 * 63.1 = 0.0 kip — ACI Eq. (17.6.2.1b)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Breakout strength ratio = $\frac{Nu}{\phi\, Ncbg} = \frac{0.0}{0.75 * 0.0} = 0.00$ — ACI 17.5.2

Breakout strength ratio controls (0.00 < 0.00) 0.00 < 1.0 OK — ACI 17.5.2.1

- Concrete pullout strength of anchors in tension — ACI 17.6.3

Single anchor strength $Np = 8\, Abrg\, f'c =$ 8 * 0.91 * 4.0 = 29.2 kip — ACI Eq. (17.6.3.2.2a)

Cracking factor $\Psi cp =$ 1.00 (Cracked concrete at service load level) — ACI 17.6.3.3

Pullout strength $Npn = \Psi cp\, Np =$ 1.00 * 29.2 = 29.2 kip — ACI Eq. (17.6.3.1)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Pullout strength ratio = $\frac{Niu}{\phi\, Npn} = \frac{0.0}{0.75 * 29.2} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete side-face blowout strength of anchors in tension — ACI 17.6.4

$Nsb = 160\ Ca_1\ \sqrt{(Abrg * f'c)} * Edge =$ 160 * 0.0 * √(0.91 * 4.0) * 0.50 = 0.0 kip — ACI Eq. (17.6.4.1)

Side-face blowout $Nsbg = (1 + \frac{S}{6\ Ca_1})\ Nsb =$ $(1 + \frac{8.0}{6 * 0.0})$ 0.0 / 0.50 = 0.0 kip — ACI Eq. (17.6.4.2)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Side-face blowout ratio $= \frac{Nu}{\phi\ Nsbg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Controlling tension limit state: Steel Strength

Tension Design Ratio $= \frac{Nu}{\phi\ Nn} = 0.00$ < 1.0 OK — ACI 17.5.2

- Shear Analysis (Comb. 1.4D) — ACI 17.7.1

Shear resisted by Anchor Rods only. (washers are not welded to the base plate)

Total shear force Vu = 0.0 kip , Shear per rod Vi = 0.0 kip , (only front rods are effective)

- Steel strength of anchor rods in shear

Steel strength $Vsa = 0.6 * Ase * futa * grout\ factor =$ 0.6 * 0.33 * 58.0 * 1.00 = 11.6 kip — ACI 17.7.1.2

Under-strength factor $\phi = 0.65$ — ACI 17.5.3

Steel strength ratio $= \frac{Vi}{\phi\ Vsa} = \frac{0.0}{0.65 * 11.6} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete breakout strength of anchors in shear — ACI 17.7.2

Anchor reinforcement: Use hairpins #4 per rod (2 legs)

X-Bar strength $Vrg = 2 * Area * End\ rods * fy =$ 2 * 0.20 * 2 * 60.0 = 48.0 kip

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

X-Bar strength ratio $= \frac{Vux}{\phi\ Vrg} = \frac{0.0}{0.75 * 48.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Z-Bar strength $Vrg = 2 * Area * Eff\ rods * fy =$ 2 * 0.20 * 2 * 60.0 = 48.0 kip

Z-Bar strength ratio $= \frac{Vuz}{\phi\ Vrg} = \frac{0.0}{0.75 * 48.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Anchor group area $Avc = (1.5 * Ca_1) * (Ca_2 + S_2 + Cb_2)$

Avcx = (1.5 * 4.0) * (4.0 + 8.0 + 4.0) = 96.0 in² — ACI 17.7.2.1.1

Avcz = (1.5 * 4.0) * (4.0 + 8.0 + 4.0) = 96.0 in²

Single anchor area $Avcox = 4.5 * Ca_1^2 =$ 4.5 * (4.0)² = 72.0 in² — ACI Eq. (17.7.2.1.3)

Single anchor area $Avcoz = 4.5 * Ca_1^2 =$ 4.5 * (4.0)² = 72.0 in²

Single anchor strength $Vb = 7 * (\frac{le}{da})^{0.2} * \sqrt{da} * \sqrt{f'c} * (Ca1)^{1.5}$

Vbx = $7 * (\frac{6}{0.8})^{0.2} * \sqrt{0.8} * \sqrt{4000} * (4)^{1.5}$ Vbx = 4.6 kip Vbz = 4.6 kip — ACI Eq. (17.7.2.2.1a)

Eccentricity factor $\Psi ec = 1.00$ (No eccentric load) ACI 17.5.2.5

Edge effects factor $\Psi ed = 0.7 + 0.3\frac{Ca_2}{1.5\,Ca_1} = 0.7 + 0.3\frac{4.00}{1.5 * 4.0} = 0.90$ ACI 17.5.2.6

$\Psi edz = 0.7 + 0.3\frac{4.00}{1.5 * 4.0} = 0.90$ ACI 17.5.2.6

Cracking factor $\Psi cv = 1.20$ (Cracked concrete at service load level) ACI 17.7.2.5

Thickness factor $\Psi hvx = 1.0$ ACI 17.7.2.6

Thickness factor $\Psi hvz = 1.0$

Breakout strength $Vcbg = \frac{Avc}{Avco}\,\Psi ec\,\Psi ed\,\Psi cv\,\Psi hv\,Vb$ ACI Eq. (17.7.2.1b)

$Vcbgx = \frac{96.0}{72.0}$ 1.00 * 0.90 * 1.20 * 1.00 * 4.6 = 6.6 kip

$Vcbgz = \frac{96.0}{72.0}$ 1.00 * 0.90 * 1.20 * 1.00 * 4.6 = 6.6 kip

Under-strength factor $\phi = 0.75$ ACI 17.5.3

X-Breakout strength ratio $= \frac{Vux}{\phi\,Vcbgx} = \frac{0.0}{0.75 * 6.6} = 0.00$ ACI 17.5.2

Z-Breakout strength ratio $= \frac{Vuz}{\phi\,Vcbgz} = \frac{0.0}{0.75 * 6.6} = 0.00$

Breakout strength ratio = (X-ratio² + Z-ratio²)½ = (0.00² + 0.00²)½ = 0.00

Breakout strength ratio controls (0.00 < 0.00) 0.00 < 1.0 OK ACI 17.5.2.1

- Concrete pryout strength of anchors in shear

Pryout strength $Vcpg = 2.0 * Ncbg =$ 2.0 * 0.0 = 0.0 kip ACI 17.7.3

Under-strength factor $\phi = 0.75$ ACI 17.5.3

Pryout strength ratio $= \frac{Vu}{\phi\,Vcpg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK ACI 17.5.2

Controlling shear limit state: Steel Strength

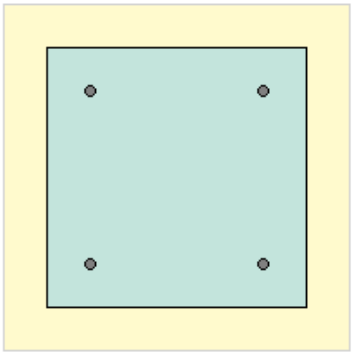
Shear Design Ratio $= \frac{Vu}{\phi\,Vn} = 0.00$ < 1.0 OK ACI 17.5.2

- Tension-Shear Interaction (Comb. 1.4D)

Combined Stress Ratio = $[(Tension\ Ratio)^{1.67} + (Shear\ Ratio)^{1.67}]^{0.6}$ ACI R17.8

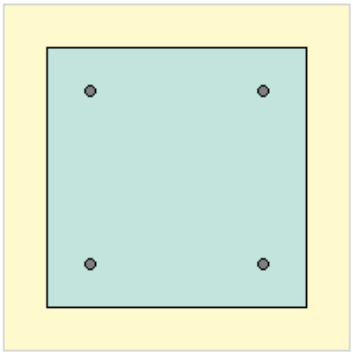
Combined Stress Ratio = $[(0.00)^{1.67} + (0.00)^{1.67}]^{0.6} = 0.00$ < 1.0 OK

Anchorage design is ductile

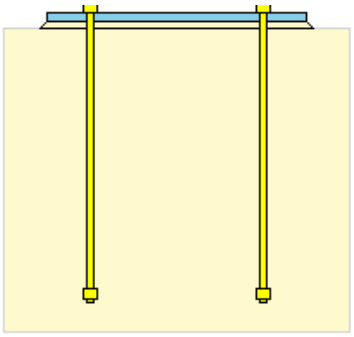


Area Anc = 0 (No anchors in tension)

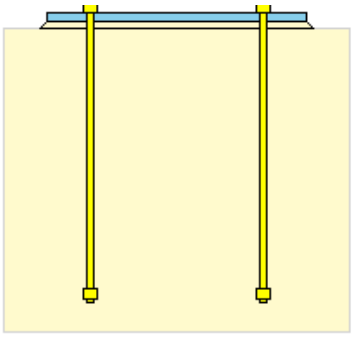
PLAN



Area Avc = 0 (No anchors in shear)



Tension Breakout



Shear Breakout

STEEL COLUMN DESIGN

GEOMETRY

Column Designation	HSS4X4X3/8	
Steel Yield Strength Fy	46.0	ksi
Modulus of Elasticity Es	29000	ksi
Member Length L	13.00	ft
Effective Length Kx-factor	1.00	
Effective Length Ky-factor	1.00	
Unbraced Length Lb	N.A.	ft

PROPERTIES

Weight	17	lb/ft	Sx ...	5.1	in³
Area .	4.8	in²	Zx ...	6.4	in³
Depth	4.0	in	rx	1.47	in
Width	4.00	in	Iy	10.3	in⁴
t nom .	0.38	in	Sy ...	5.1	in³
t des .	0.35	in	Zy ...	6.4	in³
Ix	10.3	in⁴	ry	1.47	in
			J	17.50	in⁴

UNFACTORED LOADS (Elastic Second-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	36.0	8.0	6.0	0.0	0.0	0.0	kip
Mx at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Mx at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft

FACTORED LOADS

Controlling load combination: 1.4D

Axial load Pr = 1.4 * 36.0 + 0.0 * 8.0 + 0.0 * 6.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 50.4 kip

Mx gravity bot = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral bot = 0.0 k-ft

Mx gravity top = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral top = 0.0 k-ft

My gravity bot = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral bot = 0.0 k-ft

My gravity top = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral top = 0.0 k-ft

M1x nt = 0.0 k-ft M1x lt = 0,0 k-ft M2x nt = 0.0 k-ft M2x lt = 0.0 k-ft

M1y nt = 0.0 k-ft M1y lt = 0.0 k-ft M2y nt = 0.0 k-ft M2y lt = 0.0 k-ft

Mrx = M2x nt + M2x lt = 0.0 + 0.0 = 0.0 k-ft Mry = M2y nt + M2y lt = 0.0 + 0.0 = 0.0 k-ft

DESIGN FOR COMPRESSION

Slender unstiffened Qs = 1.00 , Slender stiffened Qa = 1.00 , Q = Qs * Qa = 1.00 — AISC E7

Slenderness ratio $Kx\,L / rx$ = 1.00 * 13.00 / 1.47 = 106.1

Slenderness ratio $Ky\,L / ry$ = 1.00 * 13.00 / 1.47 = 106.1

Maximum slenderness ratio = Max (106.1, 106.1) = 106.1 — AISC E3

- Flexural Buckling

Elastic buckling $Fe = \frac{\pi^2 * E}{(K\,L / r)^2} = \frac{3.14^2 * 29000}{106.1^2}$ = 25.4 ksi — AISC Eq. E3-4

$Fe >= 0.44 * Q * Fy$ (25.4 >= 0.44 * 1.00 * 46.0 = 20.2)

Flexural buckling stress $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

Fcr = 1.00 * 46.0 * (0.658)$^{1.00 * 46.0 / 25.4}$ = 21.6 ksi

- Torsional Buckling

$Fe = [\ \frac{\pi^2 * E * Cw}{(Ky\,L)^2} + G * J\ ]\ \frac{1}{Ix + Iy}$ — AISC Eq. E4-4

$= [\ \frac{\pi^2 * 29000 * 9.1}{(1.00 * 13.0 * 12)^2} + 11200 * 17.5\ ]\ \frac{1}{10.3 + 10.3}$ = 9519.8 ksi

$Fe >= 0.44 * Q * Fy$ (9519.8 >= 0.44 * 1.00 * 46.0 = 20.2)

Torsional buckling $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

Fcr = 1.00 * 46.0 * (0.658)$^{1.00 * 46.0 / 9519.8}$ = 45.9 ksi

Compressive strength $Pn = Fcr * Ag$ = 21.6 * 4.8 = 103.1 kip — AISC Eq. E3-1

Controlling limit state: Flexural Buckling

Compressive design ratio = $\frac{Pu}{\phi\,Pn} = \frac{59.0}{0.90 * 103.1}$ = 0.64 < 1.0 OK — AISC E1

DESIGN FOR FLEXURE

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* — AISC Eq F1-1

= 12.5 * 0.0 * 1.0 / (2.5 * 0.0 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0 = 1.00

- Yielding

Plastic moment *Mpx = Fy * Zx* = 46.0 * 6.4 = 293.9 k-in

Nominal strength *Mnx = Mpx* = 293.9 / 12 = 24.5 k-ft

Plastic moment *Mpy = Fy * Zy* = 46.0 * 6.4 = 293.9 k-in — AISC Eq. F7-1

Nominal strength *Mny = Mpy* = 293.9 / 12 = 24.5 k-ft

- Flange Local Buckling — AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) — AISC F7.2(a)

- Web Local Buckling — AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) — AISC F7.3(a)

X - Controlling limit state: Yielding

X - Flexural design ratio = $\frac{Mx}{\phi\, Mnx} = \frac{0.0}{0.90 * 24.5} = 0.00$ < 1.0 OK — AISC F1

Y - Controlling limit state: Yielding

Y - Flexural design ratio = $\frac{My}{\phi\, Mny} = \frac{0.0}{0.90 * 24.5} = 0.00$ < 1.0 OK — AISC F1

DESIGN FOR COMBINED FORCES

Design axial strength $Pc = \phi Pn =$ 0.90 * 103.1 = 92.8 kip — AISC E1

Design flexural strength $Mcx = \phi Mnx =$ 0.90 * 24.5 = 22.0 kip — AISC F1

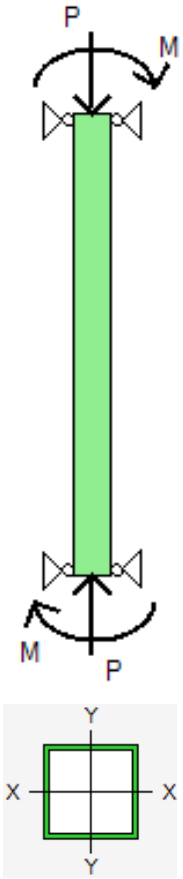
Design flexural strength $Mcy = \phi Mny =$ 0.90 * 24.5 = 22.0 kip — AISC F1

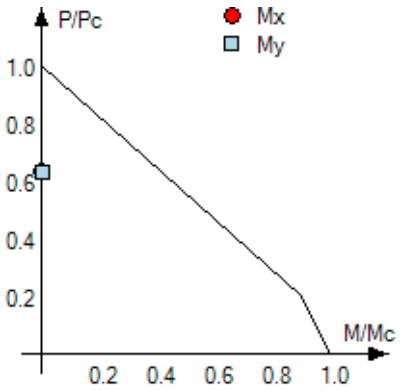
Combined forces ratio = $\frac{P}{Pc} + 8/9 \left[\frac{Mx}{Mcx} + \frac{My}{Mcy}\right]$ — AISC Eq. H1-1a

$= \frac{59.0}{92.8} + 8/9 \left[\frac{0.0}{22.0} + \frac{0.0}{22.0}\right] = 0.64 < 1.0$ OK

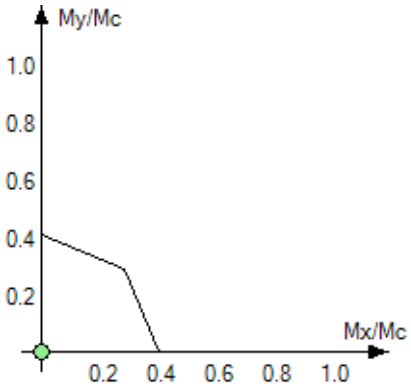
DESIGN CODES

Steel Design	AISC 360-16
Load Combinations	ASCE 7-10/16





INTERACTION DIAGRAM



BIAXIAL BENDING DIAGRAM

ASDIP Steel 5.6.2.3

BASE PLATE / ANCHORAGE DESIGN

GEOMETRY

Column Section HSS4X4X3/8

		Width		Length		
Column		4.0		4.0	in	
Plate		10.0		10.0	in	OK
Concrete Support	Wp1	8.0	Lp1	8.0	in	OK
	Wp2	8.0	Lp2	8.0	in	OK
Rod Edge		2.0		2.0	in	OK
Thickness of Grout				1.5	in	

MATERIALS

Plate Steel Strength Fy	36.0	ksi
Pier Concrete Strength f'c	4.0	ksi

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Vertical Load P	35.0	12.0	0.0	0.0	0.0	0.0	kip
Bending Moment Mx ...	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Bending Moment Mz ...	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Horizontal Load Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Horizontal Load Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

AXIALLY LOADED PLATES (Comb. 1.2D+1.6L+0.5Lr)

Pu = 1.2 * 35.0 + 1.6 * 12.0 + 0.5 * 0.0 = 61.2 kip

Bearing stress $fp = P / (W * L) =$ 61.2 / (10.0 * 10.0) = 0.6 ksi

Bearing strength $Fp = 0.85 * f'c \sqrt{\frac{A2}{A1}}$ $= 0.85 * 4.0 * \sqrt{\frac{256}{100}}$ = 5.4 ksi — ACI 22.8.3.2

Bearing under-strength factor ϕ = 0.65 — AISC J8

Design bearing strength $Fp = \phi Fpn =$ 0.65 * 5.4 = 3.5 ksi

Bearing strength ratio = $\frac{fp}{Fp} = \frac{0.6}{3.5}$ = 0.17 < 1.0 OK

Critical section m = $0.5 * (L - 0.95 * d) =$ 0.5 * (10.0 - 0.95 *4.0) = 3.1 in — AISC (14-3)

Critical section n = $0.5 * (W - 0.95 * b) =$ 0.5 * (10.0 - 0.95 *4.0) = 3.1 in

Controlling section $k = Max (m, n) =$ Max (3.1, 3.1) = 3.1 in

Moment due to tension between flanges Mt = 0.0 k-in/in

Moment due to bearing $Mb = fp * k^2 / 2 =$ 0.6 * 3.1² / 2 = 2.9 k-in/in

Flexure under-strength factor ϕ = 0.90 — AISC F1

Plate thickness $t = \sqrt{\frac{4 * Mc}{Phi * fy}}$ $= \sqrt{\frac{4 * 2.9}{0.9 * 36}}$ = 0.60 in — AISC-DG#1 3.1.2

BASE PLATE / ANCHORAGE DESIGN

BASE PLATES WITH MOMENT (Comb. 1.2D+1.6L+0.5Lr)

Mxu = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 = 0.0 k-ft *ez = Mx / P =* 0.0 / 61.2 = 0.0 in

Mzu = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 = 0.0 k-ft *ex = Mz / P =* 0.0 / 61.2 = 0.0 in

Max bearing stress fp = 0.6 ksi (See plan)

Bearing strength ratio = $\frac{fp}{Fp} = \frac{0.6}{3.5}$ = 0.17 < 1.0 OK

Max rod tension = 0.0 kip (See plan)

- Moments calculated as the bearing stress times the lever arm to the critical section

X-Moment due to bearing Mbx = 2.9 k-in/in (Controls)

Z-Moment due to bearing Mbz = 2.9 k-in/in

X-Moment due to tension *Mtx = T * (n - Edge) / Min (L, 2 * (n - Edge))*

Mtx = 0.0 * (3.1 - 2.0) / Min (10.0, 2 * (3.1 - 2.0) = 0.0 k-in/in

Z-Moment due to tension *Mtz = T * (m - Edge) / Min (W, 2 * (m - Edge))*

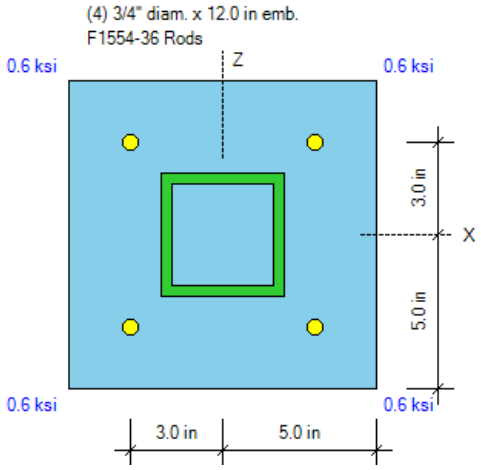
Mtz = 0.0 * (3.1 - 2.0) / Min (10.0, 2 * (3.1 - 2.0) = 0.0 k-in/in

- Plate forced to remain in the elastic range to ensure strain compatibility (Mn = My)

Plate thickness $t = \sqrt{\frac{6 * Mc}{Phi * fy}} = \sqrt{\frac{6 * 2.9}{0.9 * 36}}$ = 0.74 in AISC-DG#1 3.1.2

DESIGN CODES

Steel Design	AISC 360-16
Anchorage Design	ACI 318-19
Load Combinations	ASCE 7-10/16



ANCHORAGE DESIGN

Rod Material Specification ..... F1554-36 , Use (4) Rods , fya = 36.0 ksi, futa = 58.0 ksi

Anchor Rod Size ....3/4" diam. x 12.0 in emb. , Ase = 0.33 in² , Abrg = 0.91 in²

- Tension Analysis (Comb. 1.4D) — ACI 17.6

Total tension force Nu = 0.0 kip , # of tension rods = 0 , Tension force per rod Niu = 0.0 kip

- Steel strength of anchors in tension — ACI Eq. (17.6.1.1)

Steel strength $Nsa = Ase * futa =$ 0.33 * 58.0 = 19.4 kip — ACI Eq. (17.6.1.2)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Steel strength ratio = $\frac{Niu}{\phi\, Nsa} = \frac{0.0}{0.75 * 19.4} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete breakout strength of anchors in tension — ACI 17.6.2

Anchor reinforcement: Use 2 bars #5 per rod

Bar strength $Nrg = Area * Bars\ per\ rod * Tension\ rods * fy =$ 0.31 * 2.0 * 0 * 60 = 0.0 kip

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Bar strength ratio = $\frac{Nu}{\phi\, Nrg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Anchor effective embedment $hef =$ 12.0 in — ACI 17.6.2.1.2

Anchor group area Anc = 0.0 in² (See the tension breakout area plan) — ACI 17.6.2.1.1

Single anchor area $Anco = 9\, hef^2 =$ 9 * (12.0)² = 1296.0 in² — ACI Eq. (17.6.2.1.4)

Single anchor strength $Nb = 24 \sqrt{f'c} * hef^{1.5} = 24 \sqrt{4000} * 12^{1.5}$ = 63.1 kip — ACI Eq. (17.6.2.2.1)

Eccentricity factor $\Psi ec =$ 1.00 (No eccentric load) — ACI 17.6.2.3

Edge effects factor $\Psi ed = 0.7 + 0.3 \frac{Ca,min}{1.5\, hef} = 0.7 + 0.3 \frac{0.0}{1.5 * 12.0} = 0.70$ — ACI 17.6.2.4.1

Cracking factor $\Psi cn =$ 1.00 (Cracked concrete at service load level) — ACI 17.6.2.5.1

Breakout strength $Ncbg = \frac{Anc}{Anco}\, \Psi ec\, \Psi ed\, \Psi cn\, Nb$

$Ncbg = \frac{0.0}{1296.0}$ 1.00 * 0.70 * 1.00 * 63.1 = 0.0 kip — ACI Eq. (17.6.2.1b)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Breakout strength ratio = $\frac{Nu}{\phi\, Ncbg} = \frac{0.0}{0.75 * 0.0} = 0.00$ — ACI 17.5.2

Breakout strength ratio controls (0.00 < 0.00) 0.00 < 1.0 OK — ACI 17.5.2.1

- Concrete pullout strength of anchors in tension — ACI 17.6.3

Single anchor strength $Np = 8\, Abrg\, f'c =$ 8 * 0.91 * 4.0 = 29.2 kip — ACI Eq. (17.6.3.2.2a)

Cracking factor $\Psi cp =$ 1.00 (Cracked concrete at service load level) — ACI 17.6.3.3

Pullout strength $Npn = \Psi cp\, Np =$ 1.00 * 29.2 = 29.2 kip — ACI Eq. (17.6.3.1)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Pullout strength ratio = $\frac{Niu}{\phi\, Npn} = \frac{0.0}{0.75 * 29.2} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete side-face blowout strength of anchors in tension — ACI 17.6.4

$Nsb = 160\ Ca_1\ \sqrt{(Abrg * f'c)} * Edge =$ 160 * 0.0 * √(0.91 * 4.0) * 0.50 = 0.0 kip — ACI Eq. (17.6.4.1)

Side-face blowout $Nsbg = (1 + \frac{S}{6\ Ca_1})\ Nsb =$ $(1 + \frac{6.0}{6 * 0.0})$ 0.0 / 0.50 = 0.0 kip — ACI Eq. (17.6.4.2)

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

Side-face blowout ratio $= \frac{Nu}{\phi\ Nsbg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Controlling tension limit state: Steel Strength

Tension Design Ratio $= \frac{Nu}{\phi\ Nn} = 0.00$ < 1.0 OK — ACI 17.5.2

- Shear Analysis (Comb. 1.4D) — ACI 17.7.1

Shear resisted by Anchor Rods only. (washers are not welded to the base plate)

Total shear force Vu = 0.0 kip , Shear per rod Vi = 0.0 kip , (only front rods are effective)

- Steel strength of anchor rods in shear

Steel strength $Vsa = 0.6 * Ase * futa * grout\ factor =$ 0.6 * 0.33 * 58.0 * 0.80 = 9.3 kip — ACI 17.7.1.2

Under-strength factor $\phi = 0.65$ — ACI 17.5.3

Steel strength ratio $= \frac{Vi}{\phi\ Vsa} = \frac{0.0}{0.65 * 9.3} = 0.00$ < 1.0 OK — ACI 17.5.2

- Concrete breakout strength of anchors in shear — ACI 17.7.2

Anchor reinforcement: Use hairpins #4 per rod (2 legs)

X-Bar strength $Vrg = 2 * Area * End\ rods * fy =$ 2 * 0.20 * 2 * 60.0 = 48.0 kip

Under-strength factor $\phi = 0.75$ — ACI 17.5.3

X-Bar strength ratio $= \frac{Vux}{\phi\ Vrg} = \frac{0.0}{0.75 * 48.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Z-Bar strength $Vrg = 2 * Area * Eff\ rods * fy =$ 2 * 0.20 * 2 * 60.0 = 48.0 kip

Z-Bar strength ratio $= \frac{Vuz}{\phi\ Vrg} = \frac{0.0}{0.75 * 48.0} = 0.00$ < 1.0 OK — ACI 17.5.2

Anchor group area $Avc = (1.5 * Ca_1) * (Ca_2 + S_2 + Cb_2)$

Avcx = (1.5 * 5.0) * (5.0 + 6.0 + 5.0) = 120.0 in² — ACI 17.7.2.1.1

Avcz = (1.5 * 5.0) * (5.0 + 6.0 + 5.0) = 120.0 in²

Single anchor area $Avcox = 4.5 * Ca_1^2 =$ 4.5 * (5.0)² = 112.5 in² — ACI Eq. (17.7.2.1.3)

Single anchor area $Avcoz = 4.5 * Ca_1^2 =$ 4.5 * (5.0)² = 112.5 in²

Single anchor strength $Vb = 7 * (\frac{le}{da})^{0.2} * \sqrt{da} * \sqrt{f'c} * (Ca1)^{1.5}$

Vbx = $7 * (\frac{6}{0.8})^{0.2} * \sqrt{0.8} * \sqrt{4000} * (5)^{1.5}$ Vbx = 6.4 kip Vbz = 6.4 kip — ACI Eq. (17.7.2.2.1a)

Eccentricity factor $\Psi ec = 1.00$ (No eccentric load) ACI 17.5.2.5

Edge effects factor $\Psi ed = 0.7 + 0.3\frac{Ca_2}{1.5\,Ca_1} = 0.7 + 0.3\frac{5.00}{1.5 * 5.0} = 0.90$ ACI 17.5.2.6

$\Psi edz = 0.7 + 0.3\frac{5.00}{1.5 * 5.0} = 0.90$ ACI 17.5.2.6

Cracking factor $\Psi cv = 1.20$ (Cracked concrete at service load level) ACI 17.7.2.5

Thickness factor $\Psi hvx = 1.0$ ACI 17.7.2.6

Thickness factor $\Psi hvz = 1.0$

Breakout strength $Vcbg = \frac{Avc}{Avco}\,\Psi ec\,\Psi ed\,\Psi cv\,\Psi hv\,Vb$ ACI Eq. (17.7.2.1b)

$Vcbgx = \frac{120.0}{112.5}$ 1.00 * 0.90 * 1.20 * 1.00 * 6.4 = 7.3 kip

$Vcbgz = \frac{120.0}{112.5}$ 1.00 * 0.90 * 1.20 * 1.00 * 6.4 = 7.3 kip

Under-strength factor $\phi = 0.75$ ACI 17.5.3

X-Breakout strength ratio $= \frac{Vux}{\phi\,Vcbgx} = \frac{0.0}{0.75 * 7.3} = 0.00$ ACI 17.5.2

Z-Breakout strength ratio $= \frac{Vuz}{\phi\,Vcbgz} = \frac{0.0}{0.75 * 7.3} = 0.00$

Breakout strength ratio = (X-ratio² + Z-ratio²)½ = (0.00² + 0.00²)½ = 0.00

Breakout strength ratio controls (0.00 < 0.00) 0.00 < 1.0 OK ACI 17.5.2.1

- Concrete pryout strength of anchors in shear

Pryout strength $Vcpg = 2.0 * Ncbg =$ 2.0 * 0.0 = 0.0 kip ACI 17.7.3

Under-strength factor $\phi = 0.75$ ACI 17.5.3

Pryout strength ratio $= \frac{Vu}{\phi\,Vcpg} = \frac{0.0}{0.75 * 0.0} = 0.00$ < 1.0 OK ACI 17.5.2

Controlling shear limit state: Steel Strength

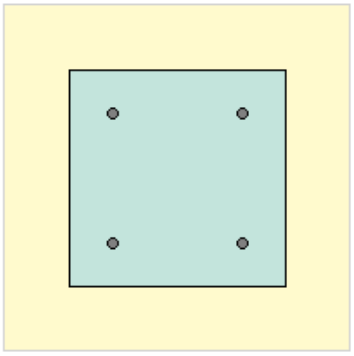
Shear Design Ratio $= \frac{Vu}{\phi\,Vn} = 0.00$ < 1.0 OK ACI 17.5.2

- Tension-Shear Interaction (Comb. 1.4D)

Combined Stress Ratio = $[(Tension\ Ratio)^{1.67} + (Shear\ Ratio)^{1.67}]^{0.6}$ ACI R17.8

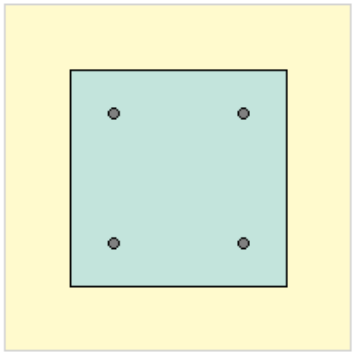
Combined Stress Ratio = $[(0.00)^{1.67} + (0.00)^{1.67}]^{0.6} = 0.00$ < 1.0 OK

Anchorage design is ductile

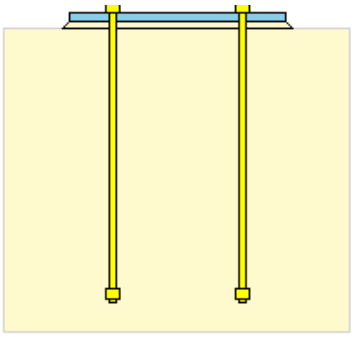


Area Anc = 0 (No anchors in tension)

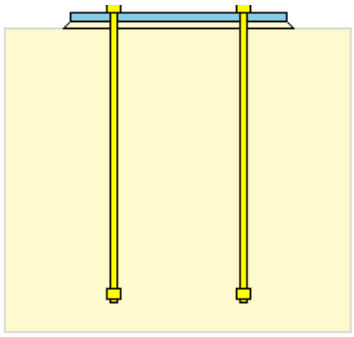
PLAN



Area Avc = 0 (No anchors in shear)



Tension Breakout



Shear Breakout

STEEL COLUMN DESIGN

GEOMETRY

Column Designation	HSS4X4X3/8	
Steel Yield Strength Fy	46.0	ksi
Modulus of Elasticity Es	29000	ksi
Member Length L	13.00	ft
Effective Length Kx-factor	1.00	
Effective Length Ky-factor	1.00	
Unbraced Length Lb	N.A.	ft

PROPERTIES

Weight	17	lb/ft	Sx ...	5.1	in³
Area .	4.8	in²	Zx ...	6.4	in³
Depth	4.0	in	rx	1.47	in
Width	4.00	in	Iy	10.3	in⁴
t nom .	0.38	in	Sy ...	5.1	in³
t des .	0.35	in	Zy ...	6.4	in³
Ix	10.3	in⁴	ry	1.47	in
			J	17.50	in⁴

UNFACTORED LOADS (Elastic Second-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	37.0	11.0	0.0	0.0	0.0	0.0	kip
Mx at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Mx at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Bottom	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
My at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft

FACTORED LOADS

Controlling load combination: 1.4D

Axial load Pr = 1.4 * 37.0 + 0.0 * 11.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 51.8 kip

Mx gravity bot = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral bot = 0.0 k-ft

Mx gravity top = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

Mx lateral top = 0.0 k-ft

My gravity bot = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral bot = 0.0 k-ft

My gravity top = 1.4 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.0 * 0.0 + 0.00 * 0.0 = 0.0 k-ft

My lateral top = 0.0 k-ft

M1x nt = 0.0 k-ft M1x lt = 0,0 k-ft M2x nt = 0.0 k-ft M2x lt = 0.0 k-ft

M1y nt = 0.0 k-ft M1y lt = 0.0 k-ft M2y nt = 0.0 k-ft M2y lt = 0.0 k-ft

Mrx = M2x nt + M2x lt = 0.0 + 0.0 = 0.0 k-ft Mry = M2y nt + M2y lt = 0.0 + 0.0 = 0.0 k-ft

DESIGN FOR COMPRESSION

Slender unstiffened Qs = 1.00 , Slender stiffened Qa = 1.00 , Q = Qs * Qa = 1.00 — AISC E7

Slenderness ratio $Kx\,L / rx$ = 1.00 * 13.00 / 1.47 = 106.1

Slenderness ratio $Ky\,L / ry$ = 1.00 * 13.00 / 1.47 = 106.1

Maximum slenderness ratio = Max (106.1, 106.1) = 106.1 — AISC E3

- Flexural Buckling

Elastic buckling $Fe = \frac{\pi^2 * E}{(K\,L / r)^2} = \frac{3.14^2 * 29000}{106.1^2}$ = 25.4 ksi — AISC Eq. E3-4

$Fe >= 0.44 * Q * Fy$ (25.4 >= 0.44 * 1.00 * 46.0 = 20.2)

Flexural buckling stress $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

Fcr = 1.00 * 46.0 * (0.658)$^{1.00 * 46.0 / 25.4}$ = 21.6 ksi

- Torsional Buckling

$Fe = [\ \frac{\pi^2 * E * Cw}{(Ky\,L)^2} + G * J\ ]\ \frac{1}{Ix + Iy}$ — AISC Eq. E4-4

$= [\ \frac{\pi^2 * 29000 * 9.1}{(1.00 * 13.0 * 12)^2} + 11200 * 17.5\ ]\ \frac{1}{10.3 + 10.3}$ = 9519.8 ksi

$Fe >= 0.44 * Q * Fy$ (9519.8 >= 0.44 * 1.00 * 46.0 = 20.2)

Torsional buckling $Fcr = Q * Fy * (0.658)^{Q * Fy / Fe}$ — AISC Eq. E3-2

Fcr = 1.00 * 46.0 * (0.658)$^{1.00 * 46.0 / 9519.8}$ = 45.9 ksi

Compressive strength $Pn = Fcr * Ag$ = 21.6 * 4.8 = 103.1 kip — AISC Eq. E3-1

Controlling limit state: Flexural Buckling

Compressive design ratio = $\frac{Pu}{\phi\,Pn} = \frac{62.0}{0.90 * 103.1}$ = 0.67 < 1.0 OK — AISC E1

DESIGN FOR FLEXURE

*Cb = Min (3.0, 12.5 Mmax * Rm / (2.5 Mmax + 3 Ma + 4 Mb + 3 Mc))* — AISC Eq F1-1

= 12.5 * 0.0 * 1.0 / (2.5 * 0.0 + 3 * 0.0 + 4 * 0.0 + 3 * 0.0 = 1.00

- Yielding

Plastic moment *Mpx = Fy * Zx* = 46.0 * 6.4 = 293.9 k-in

Nominal strength *Mnx = Mpx* = 293.9 / 12 = 24.5 k-ft

Plastic moment *Mpy = Fy * Zy* = 46.0 * 6.4 = 293.9 k-in — AISC Eq. F7-1

Nominal strength *Mny = Mpy* = 293.9 / 12 = 24.5 k-ft

- Flange Local Buckling — AISC F7.2

Nominal strength *Mnx = N.A.* (compact flanges) — AISC F7.2(a)

- Web Local Buckling — AISC F7.3

Nominal strength *Mnx = N.A.* (compact webs) — AISC F7.3(a)

X - Controlling limit state: Yielding

X - Flexural design ratio = $\frac{Mx}{\phi\, Mnx} = \frac{0.0}{0.90 * 24.5} = 0.00$ < 1.0 OK — AISC F1

Y - Controlling limit state: Yielding

Y - Flexural design ratio = $\frac{My}{\phi\, Mny} = \frac{0.0}{0.90 * 24.5} = 0.00$ < 1.0 OK — AISC F1

DESIGN FOR COMBINED FORCES

Design axial strength $Pc = \phi Pn =$ 0.90 * 103.1 = 92.8 kip — AISC E1

Design flexural strength $Mcx = \phi Mnx =$ 0.90 * 24.5 = 22.0 kip — AISC F1

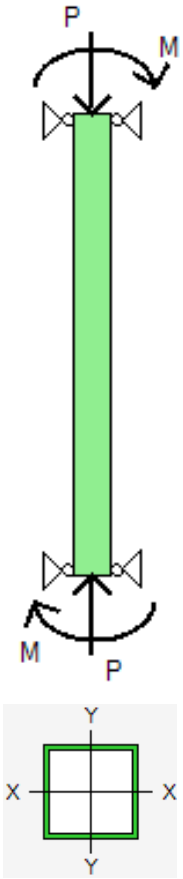
Design flexural strength $Mcy = \phi Mny =$ 0.90 * 24.5 = 22.0 kip — AISC F1

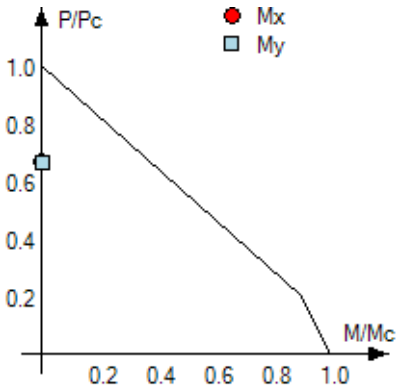
Combined forces ratio = $\frac{P}{Pc} + 8/9 \left[\frac{Mx}{Mcx} + \frac{My}{Mcy}\right]$ — AISC Eq. H1-1a

$= \frac{62.0}{92.8} + 8/9 \left[\frac{0.0}{22.0} + \frac{0.0}{22.0}\right] = 0.67 < 1.0$ OK

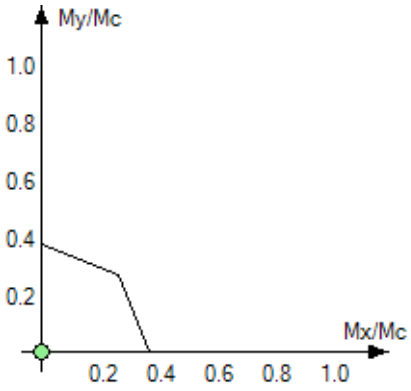
DESIGN CODES

Steel Design	AISC 360-16
Load Combinations	ASCE 7-10/16





INTERACTION DIAGRAM



BIAXIAL BENDING DIAGRAM

ASDIP Concrete 5.2.1.6 **CONCRETE COLUMN DESIGN** *www.asdipsoft.com*

GEOMETRY

Width ..... 8.0 in , Height .... 52.0 in
Use 14-#5 Longitudinal Bars , ρ = 0.010 OK
Use Ties #3 @ 8.0 in d' = 2.2 in OK
Concrete Clear Cover ..... 1.5 in

SLENDERNESS

Clear Member Length Lu ..... 12.00 ft
Effective Length Kx-factor 1.00
Effective Length Ky-factor 1.00
Lateral Stability ..... *Sway Column*
Slenderness Ratio = 62 > Limit = 22, Slender

UNFACTORED LOADS (Elastic First-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	17.0	3.0	0.0	0.0	0.0	0.0	kip
Mx at Bottom	22.5	4.5	0.0	0.0	0.0	0.0	k-ft
Mx at Top	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Story Axial Force ΣP ..	17895.0	1991.0	270.0	0.0			kip
Story Shear Force Vsy					302.6	0.0	kip

PROPERTIES

Concrete f'c = 4.0 ksi Max. strain = 0.0030 Stress-strain = Hognestad
Young's Modulus $Ec = 57 * (f'c * 1000)^{½}$ $= 57 * (4000)^{½}$ = 3605 ksi ACI 19.2.2.1
Rupture $fr = 7.5 * (f'c * 1000)^{½} / 1000 = 7.5 * (4000)^{½} / 1000$ = 0.47 ksi ACI Eq. (19.2.3.1)
Steel fy = 60.0 ksi Stress-strain = Elasto-plastic Es = 29000 ksi ACI 20.2.2.2
Radius of gyration rx = *Height / √12* = 15.0 in Radius of gyration ry = *Width / √12* = 2.3 in
Slenderness ratio x = *kx * Lu / rx* = 1.00 * 12.00 * 12 / 15.0 = 10 ACI 6.2.5
Slenderness ratio y = *ky * Lu / ry* = 1.00 * 12.00 * 12 / 2.3 = 62 (governs)
Slenderness limit = 22 > 10, Short ACI 6.2.5
Max. tie spacing = Min(48 dt, 16 db, width) = Min(48 * 0.38, 16 * 0.63, 8.0) = 8.0 in ACI 9.7.6.4.3

DESIGN CODES

Concrete Design ..... ACI 318-19
Load Combinations ..... ASCE 7-10/16

AMPLIFIED FACTORED LOADS (Sway Column by Approx. 2nd-Order Method)

Controlling load combination: 1.2D+1.6L+0.5Lr

Axial load Pu = 1.2 * 17.0 + 1.6 * 3.0 + 0.5 * 0.0 + 0 * 0.0 + 0 * 0.0 + 0 * 0.0 = 25.2 kip

Mux gravity bot = 1.2 * 22.5 + 1.6 * 4.5 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-ft

Mux lateral bot = 0 * 0.0 + 0 * 0.0 = 0.0 k-ft

Mux gravity top = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-ft

Mux lateral top = 0 * 0.0 + 0 * 0.0 = 0.0 k-ft

Muy gravity bot = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-ft

Muy lateral bot = 0 * 0.0 + 0 * 0.0 = 0.0 k-ft

Muy gravity top = 1.2 * 0.0 + 1.6 * 0.0 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-ft

Muy lateral top = 0 * 0.0 + 0 * 0.0 = 0.0 k-ft

M1x grav = 0.0 k-ft | M1x lat = 0.0 k-ft | M2x grav = 34.2 k-ft | M2x lat = 0.0 k-ft

M1x = 0.0 + 0.0 = 0.0 k-ft | M2x = 34.2 + 0.0 = 34.2 k-ft (M2 is always positive)

M1y grav = 0.0 k-ft | M1y lat = 0.0 k-ft | M2y grav = 0.0 k-ft | M2y lat = 0.0 k-ft

M1y = 0.0 + 0.0 = 0.0 k-ft | M2y = 0.0 + 0.0 = 0.0 k-ft (M2 is always positive)

ΣP Dead = 17895 kip | ΣP Live = 1991 kip | ΣP Lr = 270 kip | ΣP Snow = 0 kip

Vsy Wind = 303 kip | Vsy Seismic = 0 kip | Δoy Wind = 0.28 in | Δoy Seismic = 0.00 in

Vsx Wind = 0 kip | Vsx Seismic = 0 kip | Δox Wind = 0.00 in | Δox Seismic = 0.00 in

ΣPu = 1.2 * 17895.0 + 1.6 * 1991.0 + 0.5 * 270.0 + 0 * 0.0 = 24794.6 kip

Vusy = 0 * 302.6 + 0 * 0.0 = 0.0 kip | Vusx = 0 * 0.0 + 0 * 0.0 = 0.0 kip

Δoy = 0 * 0.28 + 0 * 0.00 = 0.00 in | Δox = 0 * 0.00 + 0 * 0.00 = 0.00 in

$$\frac{\Sigma Pu * \Delta oy}{Max(1, Vusy * Lc)} = \frac{24794.6 * 0.00}{\text{Max}(1, 0.0 * 0.0 * 12)} = 0.000$$ ACI Eq. (6.6.4.4.1)

Stability index Qx = $$\frac{\Sigma Pu * \Delta ox}{Max(1, Vusx * Lc)} = \frac{24794.6 * 0.00}{\text{Max}(1, 0.0 * 0.0 * 12)} = 0.000$$ ACI Eq. (6.6.4.4.1)

EIx = $0.2 * Ec * Icx + Es * Isx$ ACI Eq. (6.6.4.4.4b)

EIx = 0.2 * 3605 * 93739 + 29000 * 1094 = 9.93E+007 kip-in²

Ely = $0.2 * Ec * Icy + Es * Isy$

Ely = 0.2 * 3605 * 2219 + 29000 * 14 = 2.01E+006 kip-in²

Pcx = $\pi^2 * EIx / (kx * L)^2 =$ π² * 9.93E+007 / (1.00 * 12.0 * 12)² = 47265 kip ACI Eq. (6.6.4.4.2)

Pcy = $\pi^2 * Ely / (ky * L)^2 =$ π² * 2.01E+006 / (1.00 * 12.0 * 12)² = 958 kip

ΣPx = $\Sigma Pu / \Sigma Pcx = Pu / Pcx =$ 25.2 / 47265.3 = 0.00

ΣPy = $\Sigma Pu / \Sigma Pcy = Pu / Pcy =$ 25.2 / 958.2 = 0.03

δsx = $\frac{1}{1 - Qy} >= 1.0 = \frac{1}{1 - 0.000} = 1.00$ < 1.4 OK ACI Eq. (6.6.4.6.2)

δsy = $\frac{1}{1 - Qx} >= 1.0 = \frac{1}{1 - 0.000} = 1.00$ < 1.4 OK ACI Eq. (6.6.4.6.2)

Sway amplified moments control

Amplified moment Mcx = *M2nsx + δsx * M2sx =* 34.2 + 1.00 * 0.0 = 34.2 k-ft ACI Eq. (6.6.4.6.1b)

Amplified moment Mcy = 0.0 k-ft (not a biaxial column)

Amplified moment Mu = $\sqrt{(Mcx^2 + Mcy^2)}$ = $\sqrt{(34.2^2 + 0.0^2)}$ = 34.2 k-ft

Rotation angle $\theta = Arc\ Tan\ (Mcy / Mcx) =$ Arc Tan (0.0 / 34.2) = 0° from X-axis

COLUMN STRENGTH AT θ = 0°

- Check the bending capacity for Pu = 25.2 kip and Mu = 34.2 k-ft

By trial and error, neutral axis depth $k = c / d =$ 0.17

Strain1 = *(k * d - d1) / (k * d) * Max strain =* (0.17 * 49.8 - 2.2) / (0.17 * 49.8) * 0.0030 = 0.0022

As (in²)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (kip)	Ms = Fs(h/2-d) (k-ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.31	2.2	0.0022	60.0	3.5	17.5	34.8	175.8
As2 = 0.31	2.2	0.0022	60.0	3.3	17.5	34.8	175.8
As3 = 0.31	49.8	-0.0147	60.0	3.4	-18.6	36.9	175.8
As4 = 0.31	49.8	-0.0147	60.0	3.4	-18.6	36.9	175.8
As5 = 0.31	41.9	-0.0119	60.0	3.5	-18.6	24.6	78.1
As6 = 0.31	33.9	-0.0091	60.0	3.5	-18.6	12.3	19.5
As7 = 0.31	26.0	-0.0063	60.0	3.6	-18.6	0.0	0.0
As8 = 0.31	18.1	-0.0034	60.0	3.6	-18.6	-12.3	19.5
As9 = 0.31	10.1	-0.0006	17.6	3.6	-5.4	-7.2	78.1
As10 = 0.31	41.9	-0.0119	60.0	3.4	-18.6	24.6	78.1
As11 = 0.31	33.9	-0.0091	60.0	3.3	-18.6	12.3	19.5
As12 = 0.31	26.0	-0.0063	60.0	3.0	-18.6	0.0	0.0
As13 = 0.31	18.1	-0.0034	60.0	2.6	-18.6	-12.3	19.5
As14 = 0.31	10.1	-0.0006	17.6	2.2	-5.4	-7.2	78.1
					T = -161.9	Ms = 178.1	Is = 1093.7

Conc. force Fc = *C = Sum (Fci) = Sum (fci * Aci)* =189.9 kip

Conc. moment Mc = *Sum (Mci) = Sum (Fci * (h / 2 - di))* = 355.5 k-ft

Nominal flexural strength Mn = *Mc + Ms =* 355.5 + 178.1 = 533.6 k-ft

ϕ-factor slope = *(0.65 - 0.90) / (0.65 * Pn (balanced) - 0.9 * Pn (Es tension)*

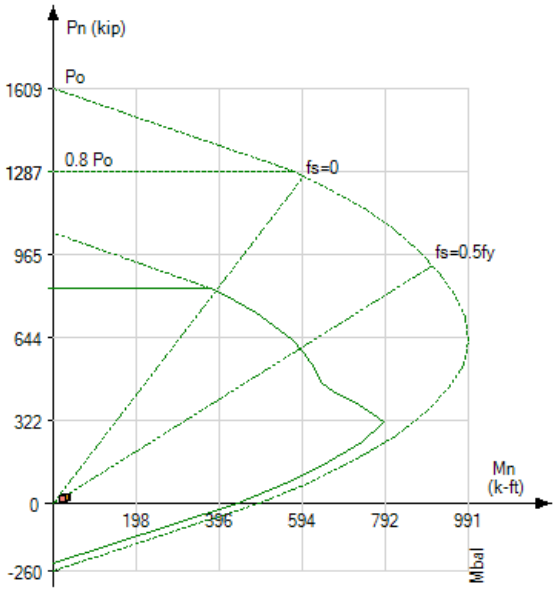
ϕ-factor slope = (0.65 - 0.90) / (0.65 * 689.0 - 0.9 * 342.4) = -0.0018 ACI 21.2.2

ϕ-factor = *Min (0.9, Max (0.65, slope * (Pu - 0.9 * Pn (Es tension)) + 0.9))*

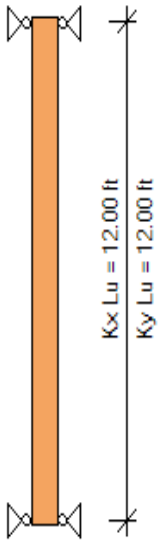
ϕ-factor = Min (0.9, Max (0.65, -0.0018 * (25.2 - 0.9 * 342.4) + 0.9)) = 0.90

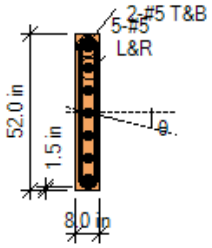
Design flexural strength ϕMn = 0.90 * 533.6 = 480.2 k-ft ACI 21.2.2

Flexural ratio = *Mu / ϕMn =* 34.2 / 480.2 = 0.07 < 1.0 OK ACI 4.4.4



INTERACTION DIAGRAM AT θ = 0 deg





Company:	GFCE	Date:	5/31/2023
Engineer:	FEH	Page:	1/6
Project:	LAMAS RESIDENCE		
Address:	13170 SW 128 ST		
Phone:	305-971-GFCE		
E-mail:	INFO@GFCE.NET		

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 4.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.38
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 6.00 x 0.50
Yield stress: 36000 psi

Profile type/size: HSS5X3X1/2

Recommended Anchor
Anchor Name: Headed Stud - 5/8"Ø AWS Type A Headed Stud

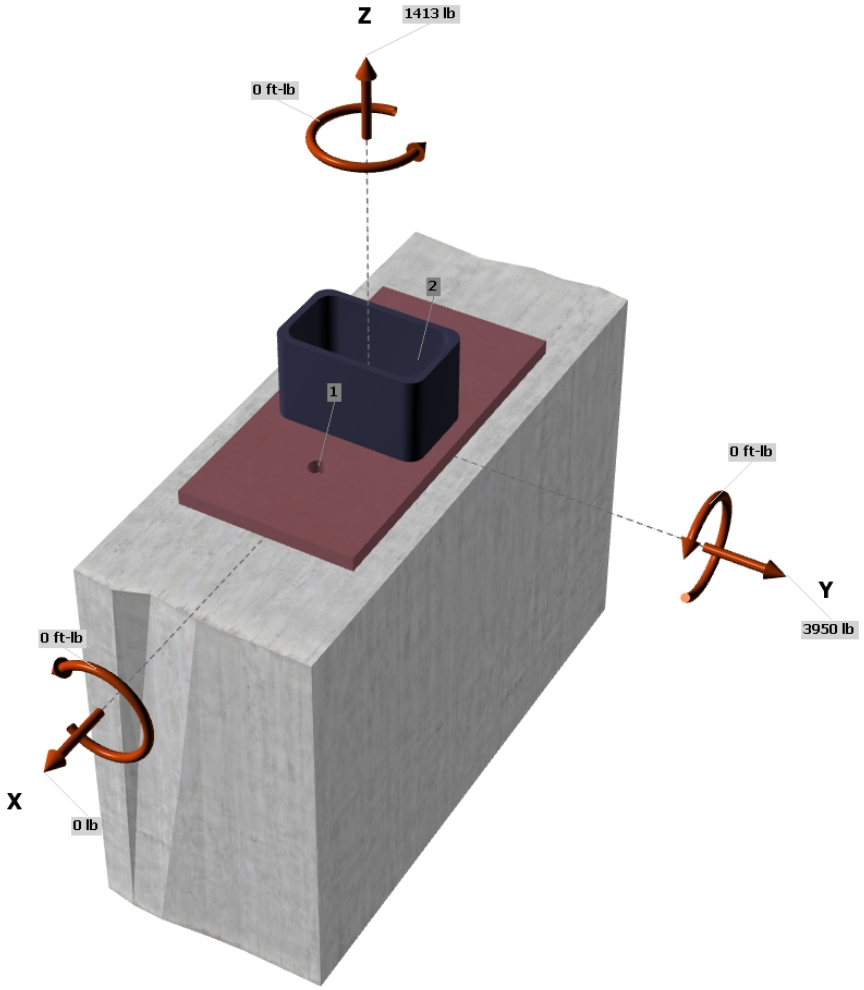


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

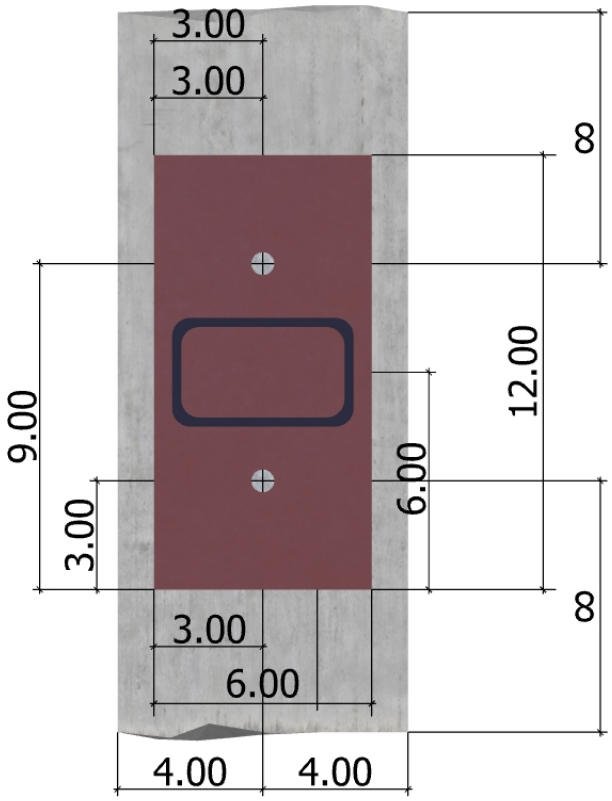
Strength level loads:

N_{ua} [lb]: 1413
V_{uax} [lb]: 0
V_{uay} [lb]: 3950
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

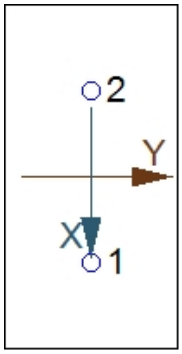


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	706.5	0.0	1975.0	1975.0
2	706.5	0.0	1975.0	1975.0
Sum	1413.0	0.0	3950.0	3950.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1413
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
18715	0.75	14036

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	4.000	12143

$\phi N_{cbg} = \phi \, (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
144.00	144.00	4.00	1.000	0.900	1.00	1.000	12143	0.70	7650

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.92	4000	0.70	20608

8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
18715	1.0	0.65	12165

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
4.00	0.625	1.00	4000	4.00	4059

$\phi V_{cbgy} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}\,V_{by}$ (Sec. 17.5.1.2 & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
108.00	72.00	1.000	1.000	1.000	1.000	4059	0.70	4262

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	144.00	144.00	1.000	0.900	1.000	1.000	12143	0.70	15300

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	707	14036	0.05	Pass
Concrete breakout	**1413**	**7650**	**0.18**	**Pass (Governs)**
Pullout	707	20608	0.03	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	1975	12165	0.16	Pass
T Concrete breakout y+	**3950**	**4262**	**0.93**	**Pass (Governs)**
Pryout	3950	15300	0.26	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.8.2	0.00	0.93	92.7%	1.0	Pass

5/8"Ø AWS Type A Headed Stud with hef = 4.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.125 inch

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

GF CONSULTING ENGINEERS INC.
13170 SW 128 STREET #204
MIAMI, FLORIDA 33186
PH: 305-971-GFCE (4323)
FAX: 305-971-4332

Title : LAMAS RESIDENCE **Job #** 23.-08
Dsgnr: FH **Date:** 8:22AM, 31 MAY 23
Description :

Scope : NEW TWO STORY SINGLE FAMILY RESIDENCE

Description

STRUCTURAL MULLION H1

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section : HSS5X3X1/2

		Pinned-Pinned	Fy	42.00	ksi
Center Span	13.00	ft	Bm Wt. Added to Loads	Load Duration Factor	1.00	
Left Cant.	0.00	ft	LL & ST Act Together	Elastic Modulus	29,000.0	ksi
Right Cant	0.00	ft				
Lu : Unbraced Length	13.00	ft				

Distributed Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	0.010							k/ft
LL	0.010							k/ft
ST	0.340							k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Short Term Load Case Governs Stress

Using: HSS5X3X1/2 section, Span = 13.00ft, Fy = 42.0ksi
End Fixity = Pinned-Pinned, Lu = 13.00ft, LDF = 1.000

	Actual		Allowable	
Moment	8.037	k-ft	15.177	k-ft
fb : Bending Stress	14.679	ksi	27.720	ksi
fb / Fb	**0.530 : 1**			
Shear	2.473	k	78.120	k
fv : Shear Stress	0.532	ksi	16.800	ksi
fv / Fv	**0.032 : 1**			

Max. Deflection	-0.514 in
Length/DL Defl	3,791.9 : 1
Length/(DL+LL Defl)	303.5 : 1

Force & Stress Summary

<<-- These columns are Dead + Live Load placed as noted -->>

	Maximum		DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max. M +	8.04	k-ft	0.64	0.85	8.04			k-ft
Max. M -								k-ft
Max. M @ Left								k-ft
Max. M @ Right								k-ft
Shear @ Left	2.47	k	0.20	0.26	2.47			k
Shear @ Right	2.47	k	0.20	0.26	2.47			k
Center Defl.	-0.514	in	-0.041	-0.055	-0.514	0.000	0.000	in
Left Cant Defl	0.000	in	0.000	0.000	0.000	0.000	0.000	in
Right Cant Defl	0.000	in	0.000	0.000	0.000	0.000	0.000	in
...Query Defl @	0.000	ft	0.000	0.000	0.000	0.000	0.000	in
Reaction @ Left	2.47		0.20	0.26	2.47			k
Reaction @ Rt	2.47		0.20	0.26	2.47			k

Fa calc'd per Eq. E2-2, K*L/r > Cc

Section Properties — HSS5X3X1/2

Depth	5.000 in	Weight	20.45 #/ft
Web Thick	0.465 in	Ixx	16.400 in4
Width	3.000 in	Iyy	7.180 in4
Flange Thick	0.465 in	Sxx	6.570 in3
Area	6.02 in2	Syy	4.780 in3
Rt	1.500 in	R-xx	1.650 in
Values for LRFD Design....		R-yy	1.090 in
J	17.600 in4	Zx	8.830 in3
Cw	10.30 in6	Zy	6.100 in3

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13170 SW 128 STREET #204
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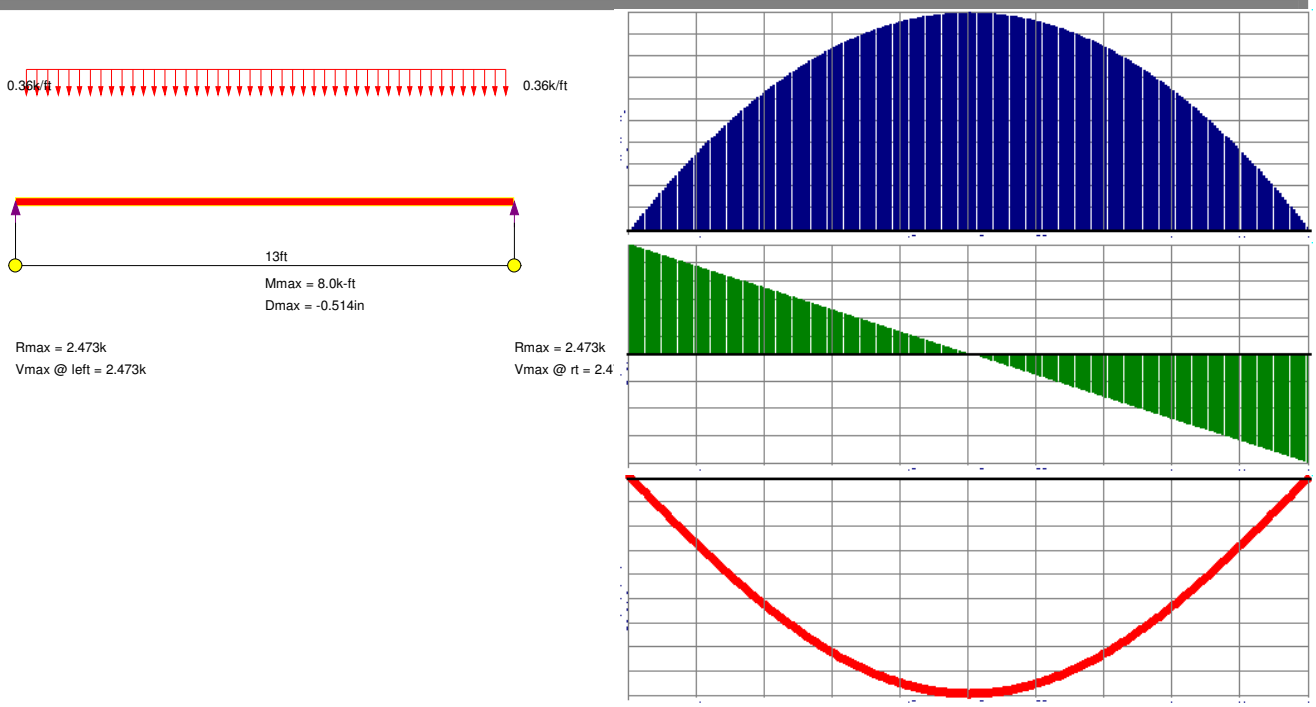
Title : LAMAS RESIDENCE **Job #** 23.-08
Dsgnr: FH **Date:** 8:22AM, 31 MAY 23
Description :

Scope : NEW TWO STORY SINGLE FAMILY RESIDENCE

Description

STRUCTURAL MULLION H1

Sketch & Diagram

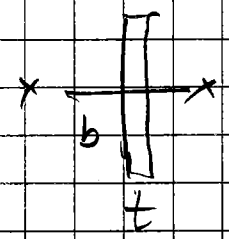


Front Curtain Wall Mullion #1 (cont.)

Try embed Plate 8 x 1/2

$$S_{xx} = \frac{t b^2}{6} = \frac{0.5 \times 8^2}{6} = 5.33 \text{ in}^3$$

$$M_R = 0.6 \times 36 \times 5.33 = 115.2 \text{ k-in} = 9.6 \text{ 'k} < M_F$$



$$I_{min} = \frac{5}{384} \frac{w l^4}{E \Delta}$$

$$= \frac{5}{384} \times \frac{.340 \times (13 \times 12)^4}{12 \times 25000 \times 0.65} = 11.6 \text{ in}^4$$

Plate 8 x 1/2:

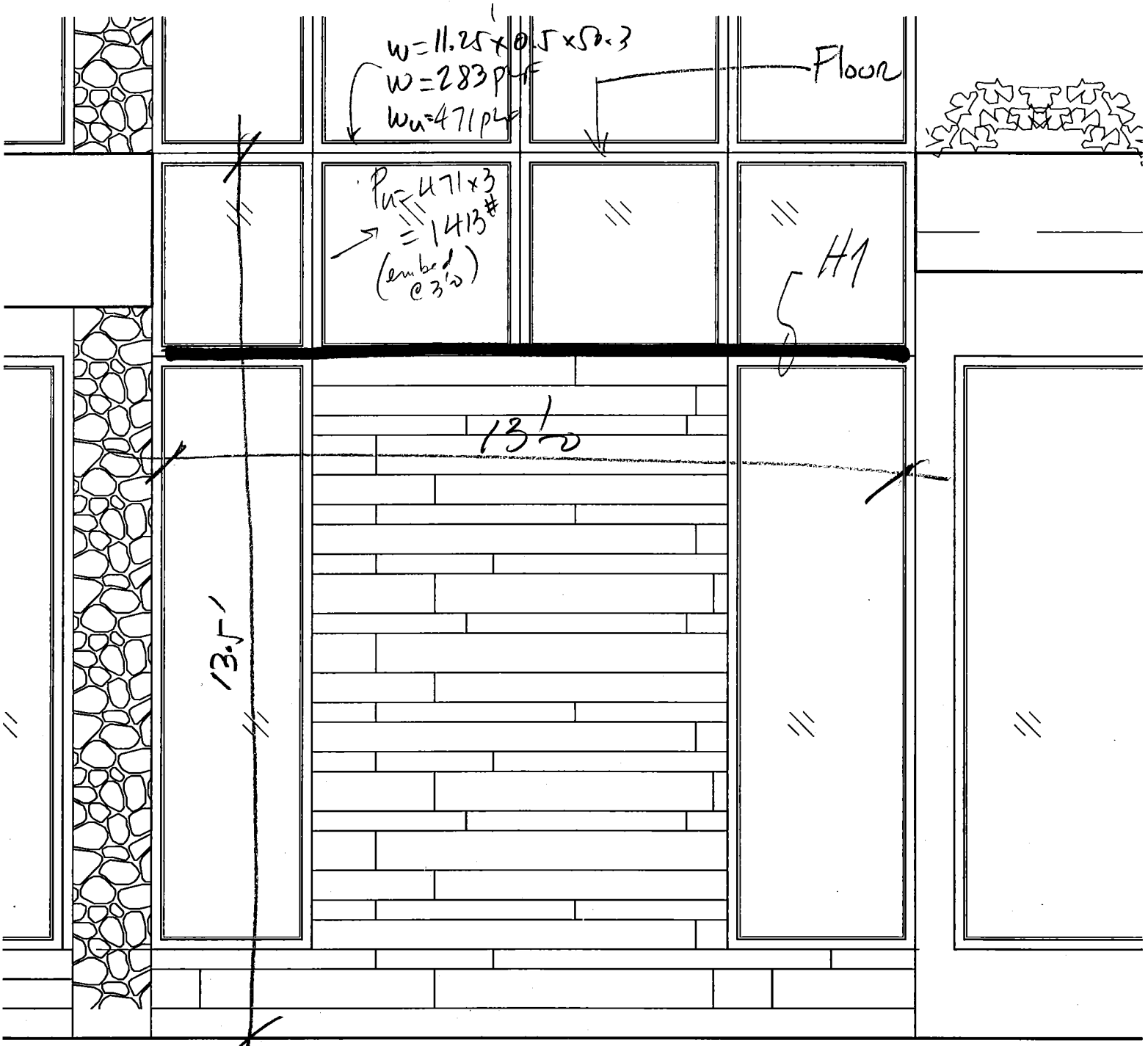
$$I_{xx} = \frac{b h^3}{12} = \frac{0.5 \times 8^3}{12} = 21.33 \text{ in}^4$$

8 x 3/4 $\rightarrow$ $S_{xx} = \frac{0.75 \times 8^2}{6} = 8 \text{ in}^3$

$M_R = 0.6 \times 36 \times 8 = 172.8 \text{ in-k} = 14.4 \text{ 'k} < M_F$

8 x 1" $\rightarrow$ $S_{xx} = 10.66 \text{ in}^3$

$M_R = 0.6 \times 36 \times 10.66 = 230.4 \text{ in-k} = 19.2 \text{ 'k} > M_F$



wind load: -83.8 x 0.6 = 50.3 psf

w = 50.3 x 13.5 x 0.5 = 340 plf

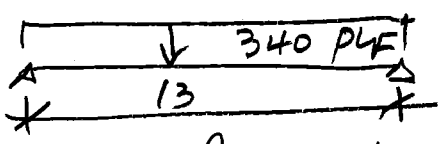
TA = 13.5 x 0.5 x 13
= 87 sf

w = -74.8 x 0.6 = 44.9

w = 44.9 x 13.5 x 0.5

w = 303 plf

H1:



$M_f = \frac{.340 \times 13^2}{8} = 7.18'k$

$\Delta_{max} = \frac{l}{240} = \frac{13 \times 12}{240} = 0.65"$

use HSS 5 x 3 x 1/2

$M_R = 15.18'k > M_f$

$\Delta = 0.51" < \Delta_{max}$

$R = 2.47^k$

Company:	GFCE	Date:	5/31/2023
Engineer:	FEH	Page:	1/5
Project:	LAMAS RESIDENCE		
Address:	13170 SW 128 ST		
Phone:	305-971-GFCE		
E-mail:	INFO@GFCE.NET		

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type B
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 6.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.25
C_{min} (inch): 1.25
S_{min} (inch): 2.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 6.00 x 0.50

Recommended Anchor
Anchor Name: Headed Stud - 1/2"Ø AWS Type B Headed Stud

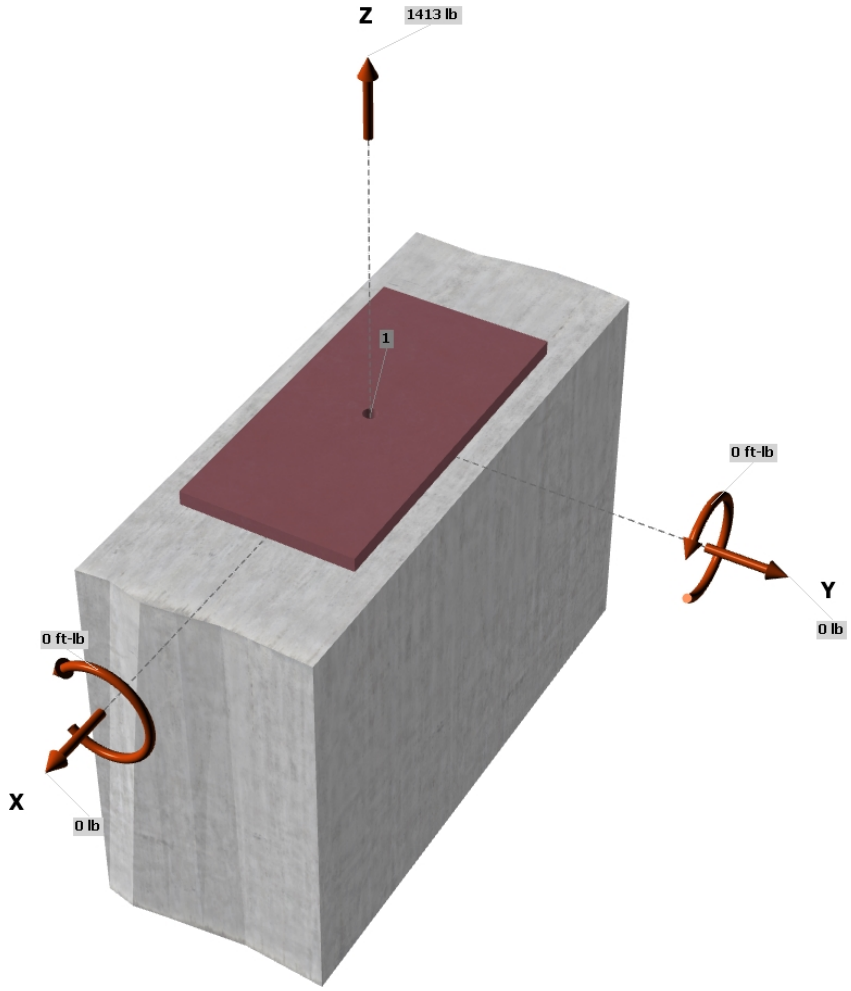


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

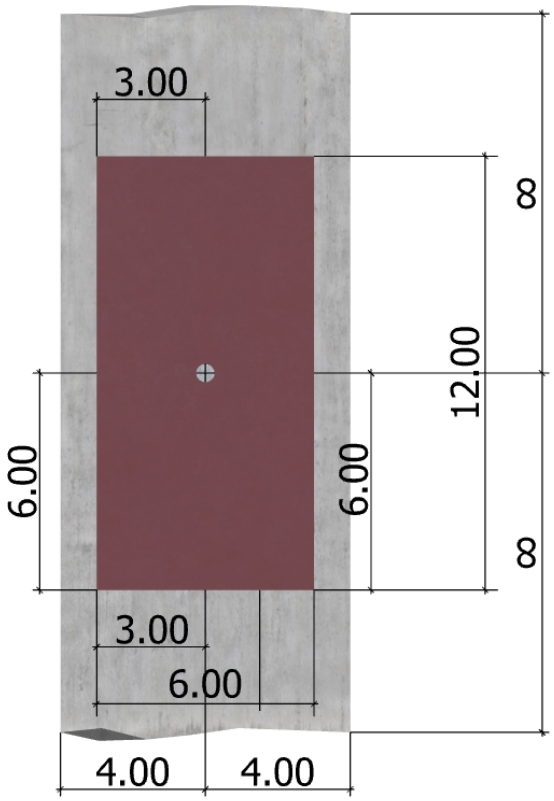
Strength level loads:

N_{ua} [lb]: 1413
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



<Figure 2>

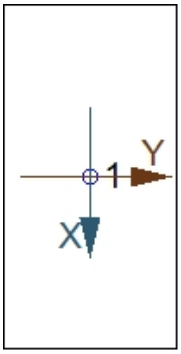


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1413.0	0.0	0.0	0.0
Sum	1413.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1413
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
12765	0.75	9574

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	6.000	22308

$\phi N_{cb} = \phi \, (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in²)	A_{Nco} (in²)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
144.00	324.00	4.00	0.833	1.00	1.000	22308	0.70	5784

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in²)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.59	4000	0.70	13194

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	1413	9574	0.15	Pass
Concrete breakout	**1413**	**5784**	**0.24**	**Pass (Governs)**
Pullout	1413	13194	0.11	Pass

1/2"Ø AWS Type B Headed Stud with hef = 6.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Stair Calculations

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET

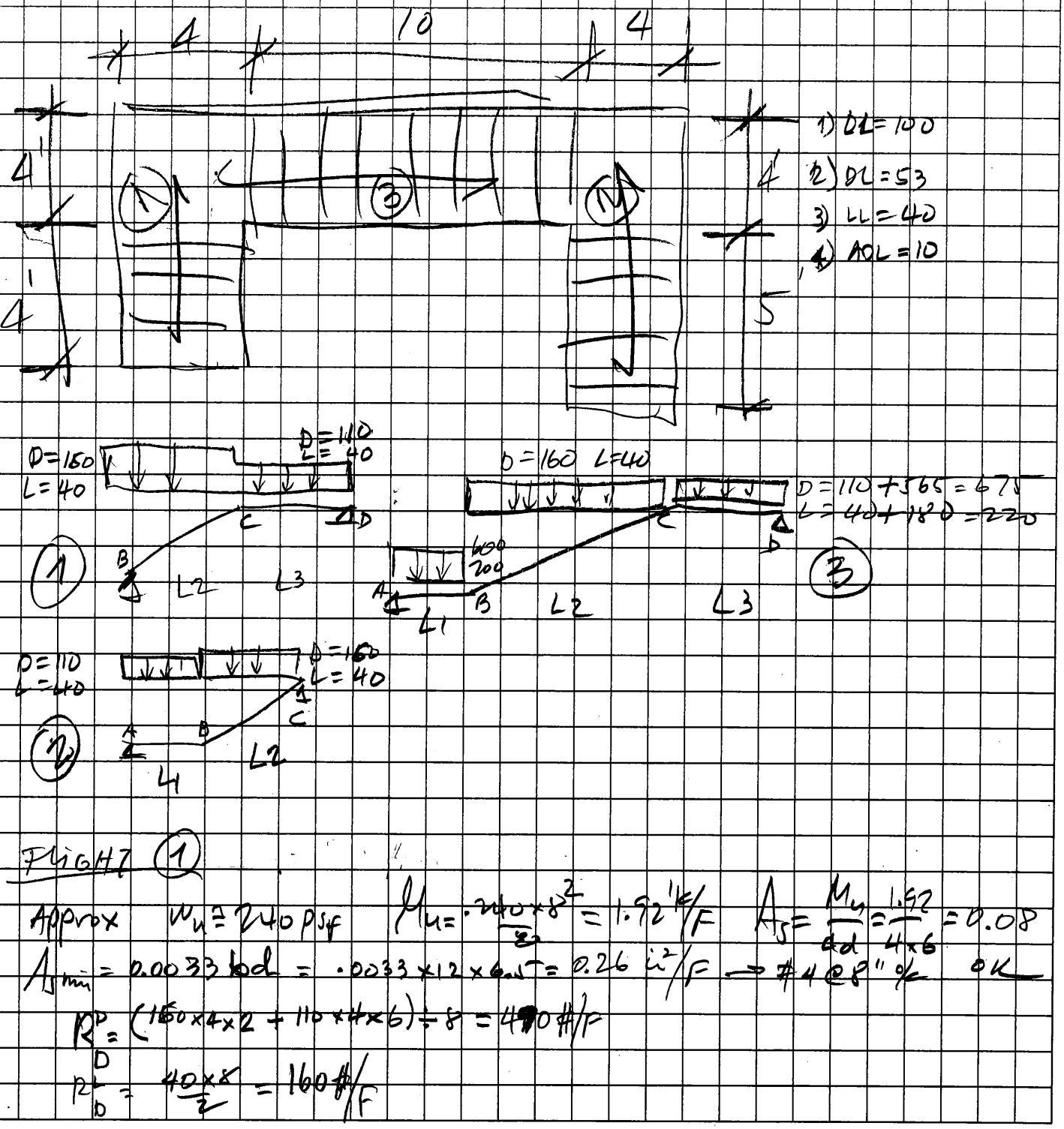
GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

STAIRS



FLIGHT (1)

Approx $w_u = 240$ psf $\quad M_u = \frac{.240 \times 8^2}{8} = 1.92$ 'k/F $\quad A_s = \frac{M_u}{\phi d} = \frac{1.52}{.4 \times 6} = 0.08$

$A_{s\,min} = 0.0033\,bd = .0033 \times 12 \times 6 = 0.26$ in²/F → #4 @ 8" o/c OK

$R_D = (150 \times 4 \times 2 + 110 \times 4 \times 6) \div 8 = 490$ #/F

$p^D_D = \frac{40 \times 8}{2} = 160$ #/F

GF CONSULTING ENGINEERS, Inc.

PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Flight (2)

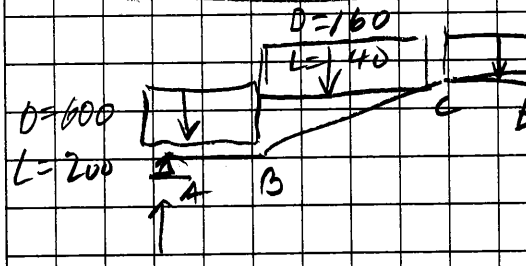
$M_u = \frac{.240 \times 9^2}{8} = 2.43 \text{ k/F}$ $\quad A_s = \frac{M_u}{\phi d} = \frac{2.43}{4 \times 6} = 0.10 \text{ in}^2/\text{F}$

use $A_{s\,min}$ #4@8"

$R_A^D = (160 \times 5 \times 2.5 + 110 \times 4 \times 7) \div 9 \approx 565 \text{ \#/F}$

$R_A^L = 40 \times 9 \div 2 = 180 \text{ \#/F}$

Flight (3)



$M_u = 18 \text{ k/F}$ $\quad A_s = \frac{18}{4 \times 5} = 0.60 \text{ in}^2/\text{F}$

#6@6" OK

$20 M_u = bd^2$

$d = \sqrt{\frac{20 \times 18}{12}} = 5.48 \text{ in}$

$M_u = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \left(\frac{f_y}{f_c'}\right)\right)$

$\rho = 0.75 \rho_b = 0.75 \times 0.021 = 0.0158$

$M_u = \phi A_s f_y (d - a/2) = \phi \rho b d f_y (d - \frac{a}{2})$

$a = A_s f_y / 0.85 f_c' b = \rho d f_y / 0.85 f_c'$

$M_u / \phi b d^2 = \rho f_y \left(1 - \frac{0.5 \rho f_y}{0.85 f_c'}\right) = R_n$

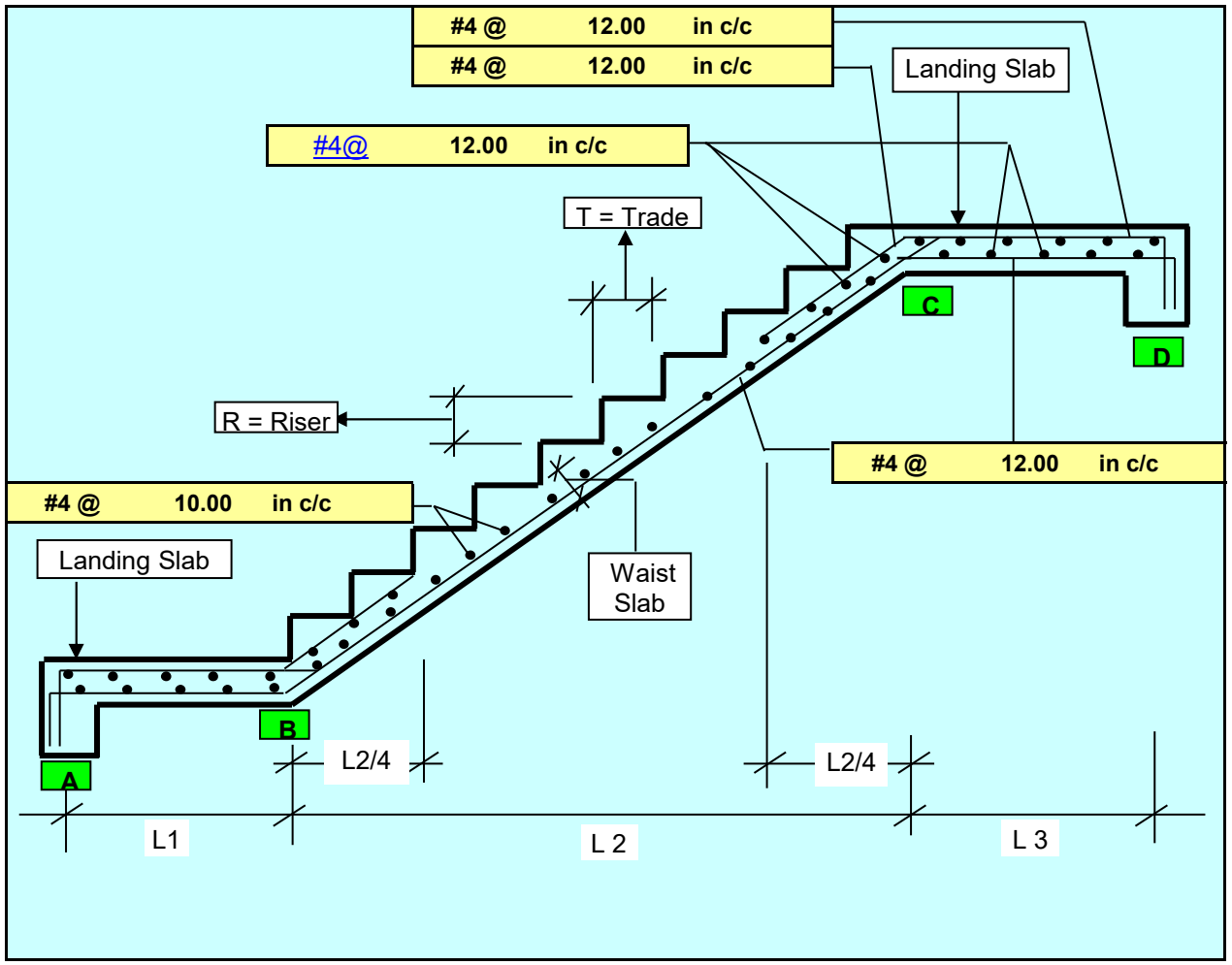
$R_n = 0.0158 \times 60000 \left(1 - \frac{0.5 \times 0.0158 \times 60}{0.85 \times 4}\right)$

$= 816 \text{ psi}$

$M_{un} = \phi b d^2 R_n$

$bd^2 = \frac{M_u}{\phi R_n} = \frac{M_u \times 12 \times 1000}{0.9 \times 816} \rightarrow bd^2 = 16 M_u \rightarrow d = \sqrt{\frac{16 \times 18}{12}} = 4.9 \text{ in}$

$< 8"$ OK



Design Data

Riser, R =	7	in
Trade, T =	11	in
f'_c =	4,000	psi

f_y = **60,000** psi

Effective Span Length

Horizontal Span, L =	L1+L2+L3	
=	8.00	ft

L1 =	**0.00**	**ft**
L2 =	**4.00**	**ft**
L3 =	**4.00**	**ft**

Thickness of Waist Slab

Thickness, t =	Span / 20	
=	4.80	in
Say, t =	**8.00**	**in**
So, d =	**7.00**	**in**

Loads on Slab

(1) D.L. of Slab on slope with landing slab = (t/12)x150 psf
= 100.00 psf

(2) D.L. due to Riser = Load on slope x $\sqrt{(R^2+T^2) / T^2}$

= 53.00 psf

(3) L.L. = **40.00** psf

(4) F.F. = **10.00** psf

Total D.L.= 173.00 psf

Total L.L.= 40.00 psf

Ultimate Load, w = 1.2 D.L. + 1.6 L.L.

= 272.00 psf

Moment

Moment in the Mid Span, $M_{u(+)}$ = $w*L^2/12$

= 1,450.67 lb-ft/ft

= 17,408.00 lb-in/ft

Checking Thickness

P = 0.75 P_b = 0.75x0.85x0.85x(f'c/fy)x(87000/(87000+fy))

= 0.021

$M_u = ØPfybd^2(1-0.59P(fy/f'c)$

d = 1.245 in

t = 2.245 in < 8.00 in, So, OK

So, t = 8.00 in

and d = 7.00 in

Main Reinfrocement

A_s = Mu / ((Øfy(d-a/2))

= 0.048 in^2 / ft

Let, a= 0.66 in

Check,a= $A_s fy / 0.85f'_c b$

= 0.071 in = 0.66 in, So OK

Spacing, S = $0.2*12/A_s$

= 49.66 in

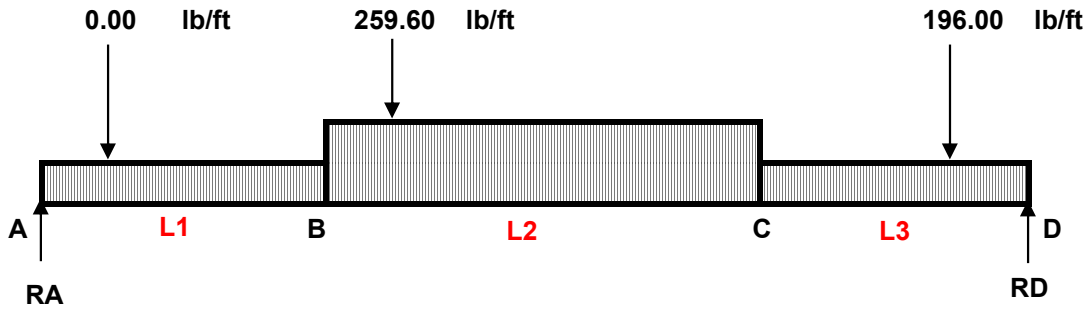
so, #4@ 12.00 in c/c

Distribution Reinforcement (Temp & Shrin. Reinforcement)

Distribution Rod = 0.15% of the Cross-sectonal area		
=	**0.13**	**in^2/ ft**

Spacing, S = 0.11*12/A_s	
= 10.48 in	**so, #4 @ 10.00 in c/c**

Reinforcement in the Landing Slab



Load Between A-B = 1.2x(Load (1) + Load (4)) + 1.6x Load (3)

= 0.00 psf

= 0.00 lb/ft

Load Between C-D = 1.2x(Load (1)+Load (4)) + 1.6x Load (3)

= 196.00 psf

= 196.00 lb/ft

Load Between B-C = 1.2x(Load (1) + Load (2) + Load (4)) + 1.6x Load (3)

= 259.60 psf

= 259.60 lb/ft

Reaction at A, R_A = 974.80 lb

Moment at A, MA(-)= $W*L^2/24$ = 725.33 lb-ft/ft

Moment at A, M_A = R_A*L1 - Load (A-B)*L1*L1/2 - Moment at A

= -725.33 lb-ft/ft

= -8,704.00 lb-in/ft

Reinforcement (Main): -

$A_s = M_A / ((\emptyset fy(d-a/2))$

= -0.024 in^2 / ft

Let, a= 0.67 in

Check,a= **$A_s fy / 0.85f'_c b$**

= -0.04 in = 0.67 in, So OK

Spacing, S = $0.11*12/A_s$

= -54.58 in

so, #6 @ -55.00 in c/c

Reinforcement (Temp): -

The minimum reinforcement required to control the shrinkage & temperature cracking: -

A_s = 0.0018bt in^2	for, fy=60,000
= 0.002bt in^2	for, fy=40,000
= 0.173 in^2	for, fy=60,000
= 0.192 in^2	for, fy=40,000

Spacing, S = 0.11*12/A_s					
=	7.64	in	so,#4@	8.00	in c/c for, fy=60,000

Spacing, S = 0.11*12/A_s					
=	6.88	in	so, #4@	7.00	in c/c for, fy=40,000

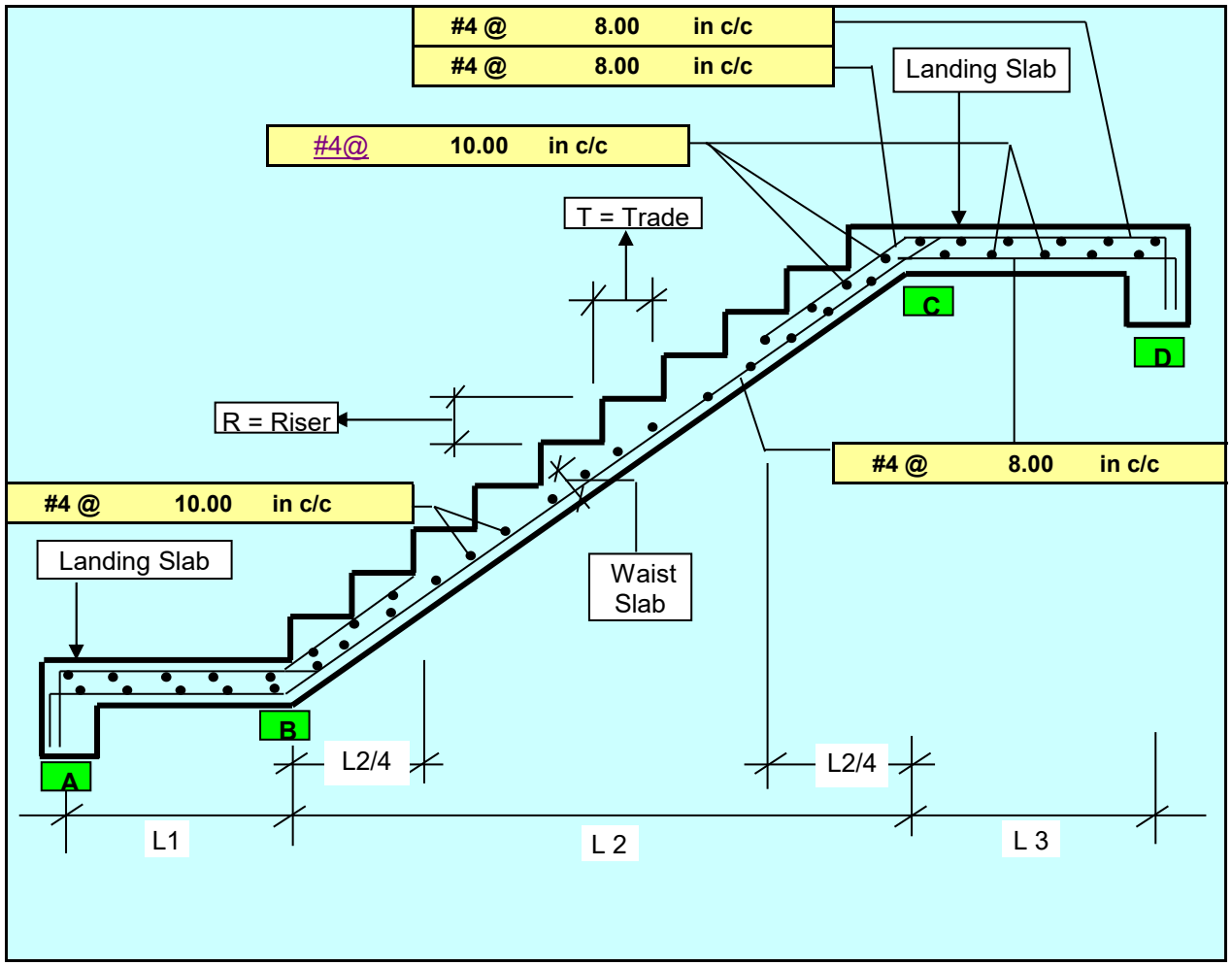
Shear Check

The factored shear force at a distance "d" from the face of the support :-

Shear force, V = 15% greater of wL/2 - wd/12
= 1.15*wL/2 - wd/12
= 1,092.53 lb

Nominal shear strength of the slab, Vc = $0.85*2\sqrt{f'c}*b*d$

= 9,031.46 lb > 1,092.53 lb
So, OK



Design Data

Riser, R =	7	in
Trade, T =	11	in
f'_c =	4,000	psi

f_y = **60,000** psi

Effective Span Length

Horizontal Span, L =	L1+L2+L3	
=	9.00	ft

L1 =	**4.00**	**ft**
L2 =	**5.00**	**ft**
L3 =	**0.00**	**ft**

Thickness of Waist Slab

Thickness, t =	Span / 20	
=	5.40	in
Say, t =	**8.00**	**in**
So, d =	**7.00**	**in**

Loads on Slab

(1) D.L. of Slab on slope with landing slab = (t/12)x150 psf
= 100.00 psf

(2) D.L. due to Riser = Load on slope x $\sqrt{(R^2+T^2) / T^2}$

= 53.00 psf

(3) L.L. = **40.00** psf

(4) F.F. = **10.00** psf

Total D.L.= 173.00 psf

Total L.L.= 40.00 psf

Ultimate Load, w =	1.2 D.L. + 1.6 L.L.

= 272.00 psf

Moment

Moment in the Mid Span, $M_{u(+)}$ = $w*L^2/12$

= 1,836.00 lb-ft/ft

= 22,032.00 lb-in/ft

Checking Thickness

P = 0.75 P_b = 0.75x0.85x0.85x(f'c/fy)x(87000/(87000+fy))

= 0.021

$M_u = ØPfybd^2(1-0.59P(fy/f'c)$

d = 1.400 in

t = 2.400 in < 8.00 in, So, OK

So, t = 8.00 in

and d = 7.00 in

Main Reinfrocement

A_s = Mu / ((Øfy(d-a/2))

= 0.061 in^2 / ft

Let, a= 0.66 in

Check,a= $A_s fy / 0.85f'_c b$

= 0.090 in = 0.66 in, So OK

Spacing, S = $0.2*12/A_s$

= 39.24 in

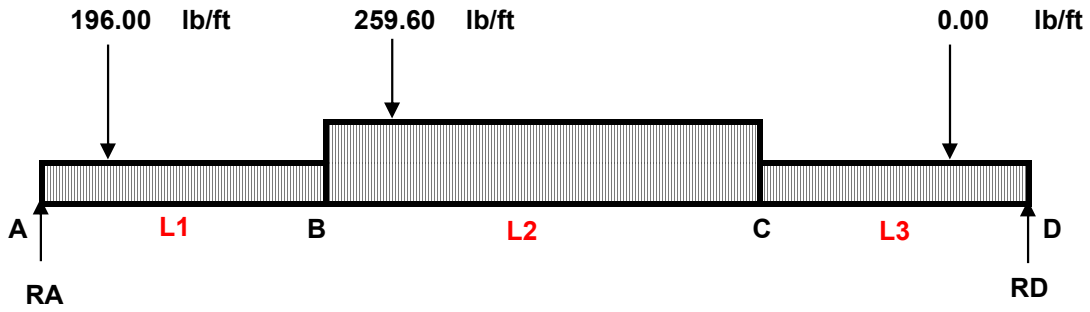
so,#4 @ 39.00 in c/c

Distribution Reinforcement (Temp & Shrin. Reinforcement)

Distribution Rod = 0.15% of the Cross-sectonal area		
=	**0.13**	**in^2/ ft**

Spacing, S = 0.11*12/A_s	
= 10.48 in	**so, #4 @ 10.00 in c/c**

Reinforcement in the Landing Slab



Load Between A-B = 1.2x(Load (1) + Load (4)) + 1.6x Load (3)
= 196.00 psf
= 196.00 lb/ft

Load Between C-D = 1.2x(Load (1) + Load (4)) + 1.6x Load (3)
= 0.00 psf
= 0.00 lb/ft

Load Between B-C = 1.2x(Load (1) + Load (2) + Load (4)) + 1.6x Load (3)

= 259.60 psf

= 259.60 lb/ft

Reaction at A, R_A = 970.33 lb

Moment at A, MA(-)= $W*L^2/24$ = 918.00 lb-ft/ft

Moment at A, M_A = R_A*L1 - Load (A-B)*L1*L1/2 - Moment at A

= 1,395.33 lb-ft/ft

= 16,744.00 lb-in/ft

Reinforcement (Main): -

$A_s = M_A / ((\emptyset fy(d-a/2))$

= 0.047 in^2 / ft

Let, a= 0.67 in

Check,a= **$A_s fy / 0.85f'_c b$**

= 0.07 in = 0.67 in, So OK

Spacing, S = $0.11*12/A_s$

= 28.37 in

so,#4 @ 28.00 in c/c

Reinforcement (Temp): -

The minimum reinforcement required to control the shrinkage & temperature cracking: -

A_s = 0.0018bt in^2	for, fy=60,000
= 0.002bt in^2	for, fy=40,000
= 0.173 in^2	for, fy=60,000
= 0.192 in^2	for, fy=40,000

Spacing, S = 0.11*12/A_s					
=	7.64	in	so,#4 @	8.00	in c/c

for, fy=60,000

Spacing, S = 0.11*12/A_s					
=	6.88	in	so, #4 @	7.00	in c/c

for, fy=40,000

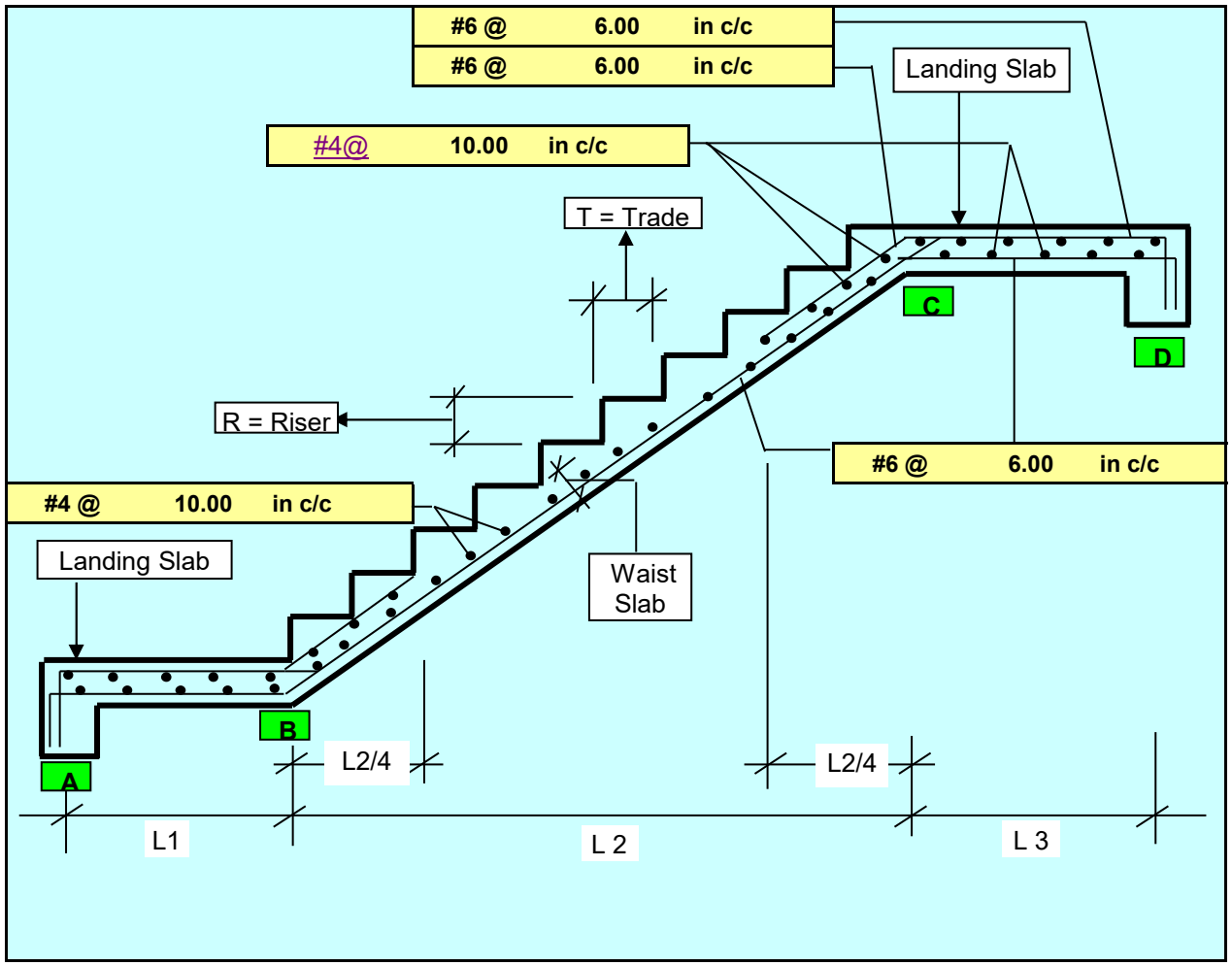
Shear Check

The factored shear force at a distance "d" from the face of the support :-

Shear force, V = 15% greater of wL/2 - wd/12
= 1.15*wL/2 - wd/12
= 1,248.93 lb

Nominal shear strength of the slab, Vc = $0.85*2\sqrt{f'c}*b*d$

= 9,031.46 lb > 1,248.93 lb
So, OK



Design Data

Riser, R =	7	in
Trade, T =	11	in
f'_c =	4,000	psi

f_y = **60,000** psi

Effective Span Length

Horizontal Span, L =	L1+L2+L3	
=	18.00	ft

L1 =	**4.00**	**ft**
L2 =	**10.00**	**ft**
L3 =	**4.00**	**ft**

Thickness of Waist Slab

Thickness, t =	Span / 20	
=	10.80	in
Say, t =	**8.00**	**in**
So, d =	**7.00**	**in**

Loads on Slab

(1) D.L. of Slab on slope with landing slab = (t/12)x150 psf
= 100.00 psf

(2) D.L. due to Riser = Load on slope x $\sqrt{(R^2+T^2) / T^2}$

= 53.00 psf

(3) L.L. = **40.00** psf

(4) F.F. = **10.00** psf

Total D.L.= 173.00 psf

Total L.L.= 40.00 psf

Ultimate Load, w =	1.2 D.L. + 1.6 L.L.

= 272.00 psf

Moment

Moment in the Mid Span, $M_{u(+)}$ = $w*L^2/12$

= 7,344.00 lb-ft/ft

= 88,128.00 lb-in/ft

Checking Thickness

P = 0.75 P_b = 0.75x0.85x0.85x(f'c/fy)x(87000/(87000+fy))

= 0.021

$M_u = ØPfybd^2(1-0.59P(fy/f'c)$

d = 2.801 in

t = 3.801 in < 8.00 in, So, OK

So, t = 8.00 in

and d = 7.00 in

Main Reinfrocement

A_s = Mu / ((Øfy(d-a/2))

= 0.245 in^2 / ft

Let, a= 0.66 in

Check,a= $A_s fy / 0.85f'_c b$

= 0.360 in = 0.66 in, So OK

Spacing, S = $0.2*12/A_s$

= 9.81 in

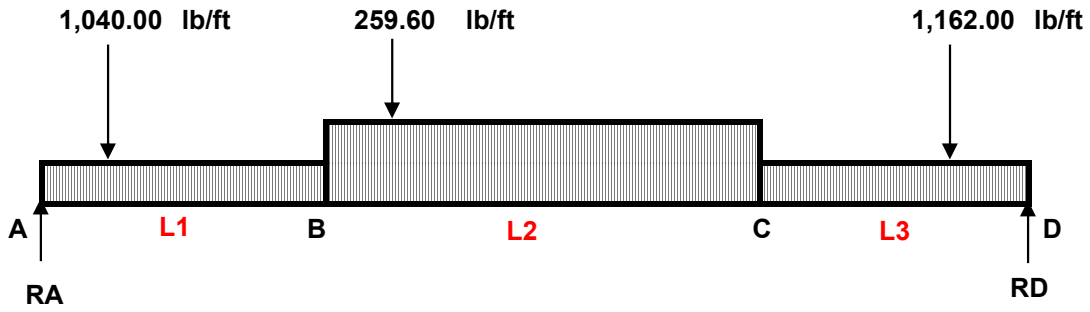
so,#4 @ 10.00 in c/c

Distribution Reinforcement (Temp & Shrin. Reinforcement)

Distribution Rod = 0.15% of the Cross-sectonal area		
=	**0.13**	**in^2/ ft**

Spacing, S = 0.11*12/A_s	
= 10.48 in	**so, #4 @ 10.00 in c/c**

Reinforcement in the Landing Slab



Load Between A-B = 1.2x(Load (1) + Load (4)) + 1.6x Load (3)
= 1,040.00 psf
= 1,040.00 lb/ft

Load Between C-D = 1.2x(Load (1) + Load (4)) + 1.6x Load (3)
= 1,162.00 psf
= 1,162.00 lb/ft

Load Between B-C = 1.2x(Load (1) + Load (2) + Load (4)) + 1.6x Load (3)

= 259.60 psf

= 259.60 lb/ft

Reaction at A, R_A = 5,512.22 lb

Moment at A, MA(-)= $W*L^2/24$ = 3,672.00 lb-ft/ft

Moment at A, M_A = R_A*L1 - Load (A-B)*L1*L1/2 - Moment at A

= 10,056.89 lb-ft/ft

= 120,682.67 lb-in/ft

Reinforcement (Main): -

$A_s = M_A / ((\emptyset fy(d-a/2))$

= 0.335 in^2 / ft

Let, a= 0.67 in

Check,a= **$A_s fy / 0.85f'_c b$**

= 0.49 in = 0.67 in, So OK

Spacing, S = $0.11*12/A_s$

= 3.94 in

so,#4 @ 4.00 in c/c

Reinforcement (Temp): -

The minimum reinforcement required to control the shrinkage & temperature cracking: -

A_s = 0.0018bt in^2	for, fy=60,000
= 0.002bt in^2	for, fy=40,000
= 0.173 in^2	for, fy=60,000
= 0.192 in^2	for, fy=40,000

Spacing, S = 0.11*12/A_s					
=	7.64	in	so,#4 @	8.00	in c/c

for, fy=60,000

Spacing, S = 0.11*12/A_s					
=	6.88	in	so, #4 @	7.00	in c/c

for, fy=40,000

Shear Check

The factored shear force at a distance "d" from the face of the support :-

Shear force, V = 15% greater of wL/2 - wd/12
= 1.15*wL/2 - wd/12
= 2,656.53 lb

Nominal shear strength of the slab, Vc = $0.85*2\sqrt{f'c}*b*d$

= 9,031.46 lb > 2,656.53 lb
So, OK

Reference	Calculations	Output

Design for Staircase

In this scenario the longitudinal reinforcement shall be used as the main reinforcement

Dimensions of Staircase

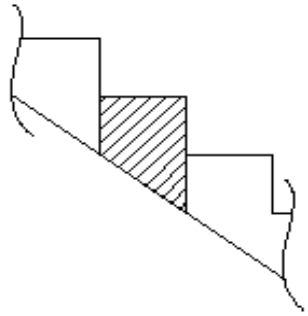
	Total Length of staircase	=	18 '	0 "	
	Total Height of staircase	=	6.5 '	0 "	
	Total Width of staircase	=	4 '	0 "	
	Height of riser	=		7 "	
ACI 318 - 08	Length of run	=		11 "	
Table 9.5 (a)	Minimum Thickness of Slab	=		8 "	

No. of risers = [(6.5 x 12) + 0] / 7 =
= 78 / 7 11.14 risers

No.of runs = [(18 x 12) + 0] / 11 =
= 216 / 11 19.64 runs

Calculations for Dead Weight

Volume of each step = Area of shaded region x Width of staircase



$\theta = angle of\ incline\ of\ staircase$

$$= tan^{-1} \frac{Height\ of\ staircase}{Horizontal\ Length\ of\ staircase}$$

= $\tan^{-1}$ [(6.5 x 12) + 0] / [(18 x 12) + 0]

= $\tan^{-1}$ (78 / 216) = 19.84

$$Sum\ of\ vertical\ sides = rise\ of\ step + \left[2 \times \left(\frac{thickness\ of\ slab}{\cos\theta}\right)\right]$$

= 7 + [2 x (8 / cos (19.84))]

= 24.01

Reference	Calculations	Output
	$Area\ of\ Shaded\ Region = \frac{1}{2} \times (sum\ of\ vertical\ sides) \times run$	
	= 1/2 x 24.01 x 11	
	= 132.1 in^2	
	$Dead\ Weight = Area\ of\ Shaded\ Region \times Unit\ Weight\ of\ Concrete$	
	= (132.055/144) x 150 = 138 lb/step	Dead Load
	= 151 lb/ft	151 psf
	Live Load	
Struc Conc. Thry & Des. **18.1**	Live Load = 40 psf	Live Load 40 psf
	Loading Combinations	
ACI 318 - 08 **9.2.1**	$Required\ Design\ Strength\ (W_u) = 1.2\ Dead\ Load + 1.6\ Live\ Load$	
	W_u = (1.2 x 151) + (1.6 x 40)	

W_u = 245.2 psf

W_u = 0.245 K.sf

Design Load

0.245 K.sf

Designing for Flexure

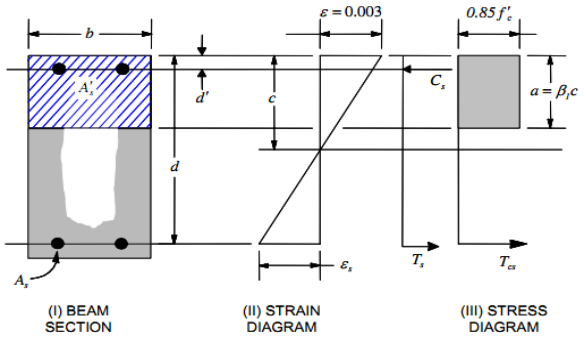
$$L = Length\ of\ staircase = \frac{Total\ Length\ of\ Staircase}{Cos\emptyset}$$

L = 18 / cos(19.84) = 19.14 ft

$$Maximum\ Bending\ Moment\ (M_u) = \frac{WL^2}{8}$$

M_u = (0.245 x 19.14 x 19.14) / 8

M_u = 11.22 K.ft



Reference	Calculations	Output
ACI 318 - 08 **7.7.1**	Using a rebar of diameter = 0.5 " and minimum cover = 1.5 " d = 8 - 1.5 - (0.5 x 0.5) = 6.25 "	
10.2.7.1	**Calculating for Maximum Reinforcement** $0.85 f_c' b a = A_s f_y$ Where a = 0.85 c	

and for a tension controlled system

c/d ≤ 0.375

c = 0.375 x6.25 = 2.344

Therefore

a = 0.85 x 2.344 = 1.992 "

$$A_s = \frac{0.85 f_c' b a}{f_y}$$

$A_{s,max}$ = (0.85 x 4000 x 12 x 1.992) / 60000 = 1.355 in^2

$A_{s,max}$ 1.355 in^2

Calculating for Minimum Reinforcement

10.5.1

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d$$

$A_{s,min}$ = (3 x √4000 / 60000) x 12 x 6.25 = 0.237 in^2

$A_{s,min}$ 0.237 in^2

Calculating for Maximum Reinforcement Spacing

10.6.4 Control of cracking by distribution of reinforcement

The spacing "s" of steel reinforcement closest to the tension face shall not exceed

$$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c$$

Where c_c = concrete cover = 1.5 in

f_s = $2f_y / 3$ = 40000 psi

Therefore

Maximum spacing of reinforcement "s"

s = [15 x (40000 / 40000)] - (2.5 x 1.5)

s = 11.25 in

Max Spacing 11.25 in

Spreadsheet Created by:
Jahleel Castagne BEng.
jahleel_castagne@hotmail.com

Reference	Calculations	Output
ACI 318 - 08		
	Calculating for Reinforcement Requirements	
10.2.7	TheFactored Nominal flexural strength is given by	
	$\phi M_n = \phi A_s f_y [d-(a/2)]$	
	Where ϕ= 0.9	
	Where a is given by the following equation	
	$0.85 f_c' b a = A_s f_y$	
	Therefore	
	$a = A_s f_y / 0.85 f_c' b =$ 1.4706 A_s	
	$a/2 =$ 0.735 A_s	
	$\varphi M_n = \varphi A_s f_y [d-(0.735 A_s)]$	
	Which gives the following for the bottom of the slab	
	1E+05 = A_s x 0.9 x 60000 [6.25 - (0.735 A_s)]	
	1E+05 = 337500 A_s - 39690 A_s^2	
	39690 A_s^2 - 337500 A_s + 1E+05 = 0	
	$A_{s,req} =$ 0.42 in^2/ft width	
	Checking Reinforcement	
	Assume reinforcement with the following specifications	
	Bar Size = #6	
	Nominal Diameter = 0.75 in	
	Nominal Area = 0.44 in^2	
	Spacing = 7 in	
	Therefore A_s/ft width is given by	
	A_s = 0.44 x 12 / 7 = 0.754 in^2/ft width	Main Rebar

	.754285714285714 > 0.4 ?37 < 0.754285714285714 < 1.3 $A_s > A_{s,req'd}$ $A_{s,min} < A_s < A_{s,max}$	#4 Bars (0.5") 8" c-c
	Designing for Shear $Required\ Shear\ Strength\ (V_u) = \frac{wl}{2}$ V_u = (0.245 x 19.14) / 2 = 2.345 Kips $V_u = V_c + V_s$ Where	
11.2.1.1	$V_c = 2\sqrt{f_c'}b_w d$ = 2 x √4000 x 12 x 6.25 = 9487 lbs 9.49 Kips	

Reference	**Calculations**	**Output**
ACI 318 - 08	Because $V_u < \Phi V_c/2$, no shear reinforcement is required, but transverse	
11.4.6	reinforcement shall be used to account for shrinkage	
7.12.2	Vs = 0.0018 x 12 x [((2 x 8) + 7) / 2] Vs = 0.248 in^2/ft	
	Assume reinforcement with the following specifications Bar Size = #6 Nominal Diameter = 0.75 in Nominal Area = 0.44 in^2 Spacing = 7 in	
	Therefore A_s/ft width is given by A_s = 0.44 x 12 / 7 = 0.754 in^2/ft width	Shear Rebar #6bars (0.75") 7" c-c

Masonry Walls

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ASDIP Concrete 5.2.1.6

MASONRY WALL DESIGN

GEOMETRY

Block Thickness ..... 8 in = KH / 24

Use 1-#5 @ 32.0 in Vertical Bars , d' = 3.8 in

Vertical ρ = 0.0023 Solid Grouted .... N

Block Weight ..... 50 psf

SLENDERNESS

Unsupported Height H ..... 15.00 ft

Effective Height K-factor 1.00

Parapet Height ..... 0.00 ft

Lateral Stability ..... *Non-sway Wall*

Deflection = 0.15 in < Allow = 1.26 in OK

UNFACTORED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic		Ecc.	Spacing	
Uniform Load w	0.4	0.0	0.3	0.0	0.0	0.0	k/ft	0.0		in
Concentrated P	0.0	0.0	0.0	0.0	0.0	0.0	kip	4.0	48.0	in
Pressure on Wall					62.0	0.0	psf			
Pressure on Parapet					0.0	0.0	psf			

Seismic Load = *Max(0.1, 0.4 * I * Sds) * SW* =Max(0.1, 0.4 * 1.0 * 0.25) * 50 = 5.0 psf ASCE 12.11.1

PROPERTIES

Masonry f'm = 1.5 ksi Max. strain = 0.0025 Stress-strain = Whitney TMS 9.3.2

Young's Modulus *Em = 900 * f'm* = 900 * 1.5 = 1350 ksi TMS 4.2.2

Rupture fr = 115 psi TMS 9.1.9.2

Steel fy = 60.0 ksi Stress-strain = Elasto-plastic Es = 29000 ksi ACI 20.2.2.2

Radius of gyration rx = *Thick / √12* = 2.2 in

Inertia Inet = *(bw * (Thick - 2 fs)³ / 12 + 2 * Sp * fs * (Thick / 2 - fs / 2)² + Sp * fs³ / 12) * 12 / Sp*

= (11.2 * (7.63 - 2 * 1.3)³ / 12 + 2 * 32 * 1.3 * (7.63 / 2 - 1.3 / 2)² + 32 * 1.3³ / 12) * 12 / 32 = 354 in⁴/ft

AMPLIFIED FACTORED LOADS

Eff. reaction length Le = *Min (4 + 4 * Thick, Spacing)* = Min (4 + 4 * 7.6, 48.0) / 12 = 2.88 ft ACI 11.2.3.1

- Gravity At Roof

Total w dead = *w dead + P dead / Le + Hp * SW* = 0.4 + 0.0 / 2.88 = 0.4 k/ft

Total w live = *w live + P live / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

Total w RLive = *w RLive + P RLive / Le* = 0.3 + 0.0 / 2.88 = 0.3 k/ft

Total w snow = *w snow + P snow / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

M dead = *w dead * Ecc + P dead / Le * Ecc* = 0.4 * 0.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

M live = *w live * Ecc + P live / Le * Ecc* = 0.0 * 0.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

M RLive = *w RLive * Ecc + P RLive / Le * Ecc* = 0.3 * 0.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

M snow = *w snow * Ecc + P snow / Le * Ecc* = 0.0 * 0.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

- Controlling load combination: 1.2D+0.5L+0.5Lr+W , Span: 1

Axial load Pu = 1.2 * 0.8 + 0.5 * 0.0 + 0.5 * 0.3 + 0 * 0.0 + 1 * 0.0 + 0 * 0.0 = 1.1 k/ft

Mu gravity mid = 1.2 * 0.0 + 0.5 * 0.0 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-in/ft

Mu lateral mid = 1 * -20.9 + 0 * -1.7 = 20.9 k-in/ft

Mu gravity top = 1.2 * 0.0 + 0.5 * 0.0 + 0.5 * 0.0 + 0 * 0.0 = 0.0 k-in/ft

Mu lateral top = 1 * 0.0 + 0 * 0.0 = 0.0 k-in/ft

M1x grav = 0.0 k-in/ft M1x lat = 0.0 k-in/ft M2x grav = 0.0 k-in/ft M2x lat = 20.9 k-in/ft

M1x = 0.0 + 0.0 = 0.0 k-in/ft M2x = 0.0 + 20.9 = 20.9 k-in/ft (M2 is always positive)

Icr = $n * (As + Pu * Thick / (fy * 2 * d)) * (d - c)^2 + b * c^3 / 3$ = TMS Eq. 9-30

21 * (0.12 + 1.1 * 7.63 / (60.0 * 2 * 3.8)) * (3.8 - 0.7)² + 12 * 0.7³ / 3 = 29.2 in⁴/ft

Mcr = $(frm / 1000 + Pu * Spacing / b / An) * Sx$ = TMS 9.3.5.4.4

(115 / 1000 + 1.1 * 32 / 12 / 137.3) * 92.8 = 12.6 k-in/ft

Ieff = 0.75 * Inet = 0.75 * 353.8 = 29.2 in⁴/ft since Mu < Mcr TMS 9.3.5.4.3

Pe = $n^2 * Em * Ieff / (Kx * H)^2$ = n² * 1350.0 * 29.2 / (1.0 * 15.00 * 12)² = 12.0 k/ft TMS Eq. 9-29

$$\psi = \frac{1}{1 - Min\left(1.0, \frac{Pu}{Pe}\right)} >= 1.0 = \frac{1}{1 - Min\left(1.0, \frac{1.1}{12.0}\right)} = 1.10$$ TMS Eq. 9-28

Amplified moment Mcx = $\psi * M2x$ = 1.10 * 20.9 = 23.0 k-in/ft TMS Eq. 9-27

DESIGN CODES

Masonry Design	TMS 402-16
Load Combinations	ASCE 7-10/16

WALL STRENGTH

- Check the bending capacity for Pu = 1.1 k/ft and Mu = 23.0 k-in/ft Comb (1.2D+0.5L+0.5Lr+W)

By trial and error, neutral axis depth *k = c / d =* 0.19

Strain1 = *(k * d - d1) / (k * d) * Max strain =* (0.19 * 3.8 - 3.8) / (0.19 * 3.8) * 0.0025 = -0.0416

As (in²/ft)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (k/ft)	Ms = Fs(h/2-d) (k-in/ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.31	3.8	-0.0416	60.0	0.0	-7.0	0.0	0.0
					T = -7.0	Ms = 0.0	Is = 0.0

Conc. force Fc = *C = Sum (Fci) = Sum (fci * Aci)* =7.4 k/ft

Conc. moment Mc =*Sum (Mci) = Sum (Fci * (h / 2 - di))* = 28.8 k-in/ft

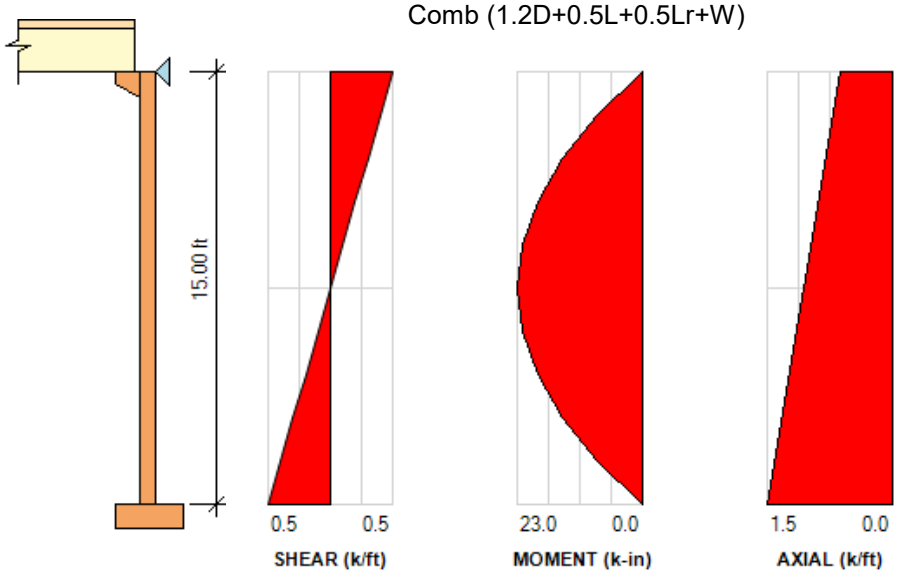
Nominal flexural strength Mn = *Ms + Mc =* 0.0 + 28.8 = 28.8 k-in/ft

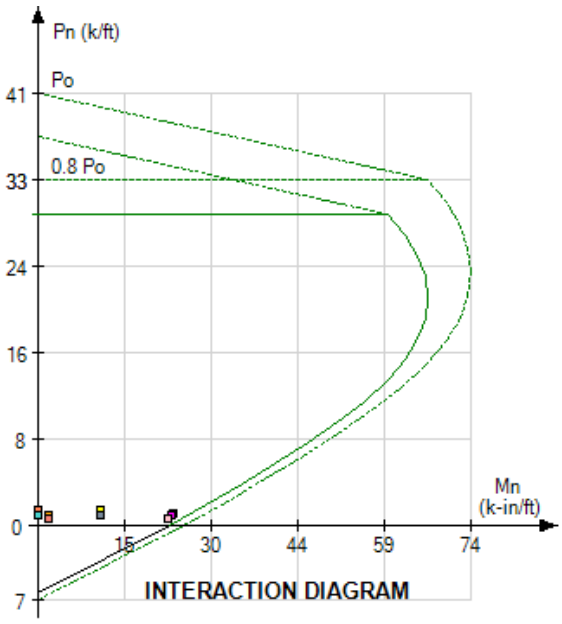
Under-strength ϕ-factor = 0.9 TMS 9.1.4.4

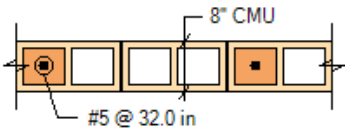
Design flexural strength ϕMn = 0.90 * 28.8 = 25.9 k-in/ft TMS 9.1.3

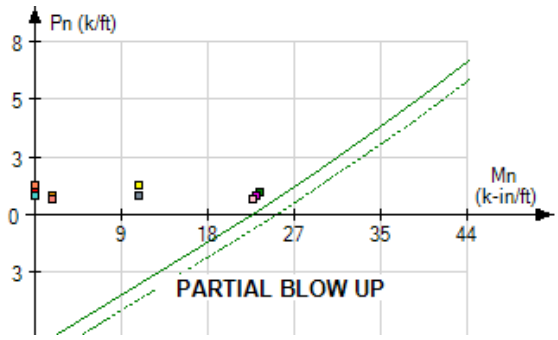
Flexural ratio = *Mu / ϕMn =* 23.0 / 25.9 = 0.89 < 1.0 OK

Comb (1.2D+0.5L+0.5Lr+W)









ASDIP Concrete 5.2.1.6

MASONRY WALL DESIGN

GEOMETRY

Block Thickness ..... 8 in = KH / 24

Use 1-#5 @ 32.0 in Vertical Bars , d' = 3.8 in

Vertical ρ = 0.0023 Solid Grouted .... N

Block Weight ..... 50 psf

SLENDERNESS

Unsupported Height H ..... 15.00 ft

Effective Height K-factor 1.00

Parapet Height ..... 0.00 ft

Lateral Stability ..... *Non-sway Wall*

Deflection = 0.21 in < Allow = 1.26 in OK

UNFACTORED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic		Ecc.	Spacing	
Uniform Load w	0.3	0.0	0.4	0.0	0.0	0.0	k/ft	6.0		in
Concentrated P	0.0	0.0	0.0	0.0	0.0	0.0	kip	4.0	48.0	in
Pressure on Wall					62.0	0.0	psf			
Pressure on Parapet					0.0	0.0	psf			

Seismic Load = *Max(0.1, 0.4 * I * Sds) * SW* =Max(0.1, 0.4 * 1.0 * 0.25) * 50 = 5.0 psf ASCE 12.11.1

PROPERTIES

Masonry f'm = 1.5 ksi Max. strain = 0.0025 Stress-strain = Whitney TMS 9.3.2

Young's Modulus *Em = 900 * f'm* = 900 * 1.5 = 1350 ksi TMS 4.2.2

Rupture fr = 115 psi TMS 9.1.9.2

Steel fy = 60.0 ksi Stress-strain = Elasto-plastic Es = 29000 ksi ACI 20.2.2.2

Radius of gyration rx = *Thick / √12* = 2.2 in

Inertia Inet = *(bw * (Thick - 2 fs)³ / 12 + 2 * Sp * fs * (Thick / 2 - fs / 2)² + Sp * fs³ / 12) * 12 / Sp*

= (11.2 * (7.63 - 2 * 1.3)³ / 12 + 2 * 32 * 1.3 * (7.63 / 2 - 1.3 / 2)² + 32 * 1.3³ / 12) * 12 / 32 = 354 in⁴/ft

AMPLIFIED FACTORED LOADS

Eff. reaction length Le = *Min (4 + 4 * Thick, Spacing)* = Min (4 + 4 * 7.6, 48.0) / 12 = 2.88 ft ACI 11.2.3.1

- Gravity At Roof

Total w dead = *w dead + P dead / Le + Hp * SW* = 0.3 + 0.0 / 2.88 = 0.3 k/ft

Total w live = *w live + P live / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

Total w Rlive = *w Rlive + P Rlive / Le* = 0.4 + 0.0 / 2.88 = 0.4 k/ft

Total w snow = *w snow + P snow / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

M dead = *w dead * Ecc + P dead / Le * Ecc* = 0.3 * 6.0 + 0.0 / 2.88 * 4.0 = 1.8 k-in/ft

M live = *w live * Ecc + P live / Le * Ecc* = 0.0 * 6.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

M Rlive = *w Rlive * Ecc + P Rlive / Le * Ecc* = 0.4 * 6.0 + 0.0 / 2.88 * 4.0 = 2.4 k-in/ft

M snow = *w snow * Ecc + P snow / Le * Ecc* = 0.0 * 6.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

- Controlling load combination: 1.2D+0.5L+0.5Lr+W , Span: 1

Axial load Pu = 1.2 * 0.7 + 0.5 * 0.0 + 0.5 * 0.4 + 0 * 0.0 + 1 * 0.0 + 0 * 0.0 = 1.0 k/ft

Mu gravity mid = 1.2 * -0.9 + 0.5 * 0.0 + 0.5 * -1.2 + 0 * 0.0 = 1.7 k-in/ft

Mu lateral mid = 1 * -20.9 + 0 * -1.7 = 20.9 k-in/ft

Mu gravity top = 1.2 * 1.8 + 0.5 * 0.0 + 0.5 * 2.4 + 0 * 0.0 = 3.4 k-in/ft

Mu lateral top = 1 * 0.0 + 0 * 0.0 = 0.0 k-in/ft

M1x grav = 3.4 k-in/ft M1x lat = 0.0 k-in/ft M2x grav = 1.7 k-in/ft M2x lat = 20.9 k-in/ft

M1x = 3.4 + 0.0 = 3.4 k-in/ft M2x = 1.7 + 20.9 = 22.6 k-in/ft (M2 is always positive)

Icr = $n * (As + Pu * Thick / (fy * 2 * d)) * (d - c)^2 + b * c^3 / 3$ = TMS Eq. 9-30

21 * (0.12 + 1.0 * 7.63 / (60.0 * 2 * 3.8)) * (3.8 - 0.7)² + 12 * 0.7³ / 3 = 29.0 in⁴/ft

Mcr = $(frm / 1000 + Pu * Spacing / b / An) * Sx$ = TMS 9.3.5.4.4

(115 / 1000 + 1.0 * 32 / 12 / 137.3) * 92.8 = 12.5 k-in/ft

Ieff = 0.75 * Inet = 0.75 * 353.8 = 29.0 in⁴/ft since Mu < Mcr TMS 9.3.5.4.3

Pe = $n^2 * Em * Ieff / (Kx * H)^2$ = n² * 1350.0 * 29.0 / (1.0 * 15.00 * 12)² = 11.9 k/ft TMS Eq. 9-29

$$\psi = \frac{1}{1 - Min\left(1.0, \frac{Pu}{Pe}\right)} >= 1.0 = \frac{1}{1 - Min\left(1.0, \frac{1.0}{11.9}\right)} = 1.09$$ TMS Eq. 9-28

Amplified moment Mcx = $\psi * M2x$ = 1.09 * 22.6 = 24.7 k-in/ft TMS Eq. 9-27

DESIGN CODES

Masonry Design	TMS 402-16
Load Combinations	ASCE 7-10/16

WALL STRENGTH

- Check the bending capacity for Pu = 1.0 k/ft and Mu = 24.7 k-in/ft Comb (1.2D+0.5L+0.5Lr+W)

By trial and error, neutral axis depth *k = c / d =* 0.18

Strain1 = *(k * d - d1) / (k * d) * Max strain =* (0.18 * 3.8 - 3.8) / (0.18 * 3.8) * 0.0025 = -0.0421

As (in²/ft)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (k/ft)	Ms = Fs(h/2-d) (k-in/ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.31	3.8	-0.0421	60.0	0.0	-7.0	0.0	0.0
					T = -7.0	Ms = 0.0	Is = 0.0

Conc. force Fc = *C = Sum (Fci) = Sum (fci * Aci)* =7.4 k/ft

Conc. moment Mc =*Sum (Mci) = Sum (Fci * (h / 2 - di))* = 28.6 k-in/ft

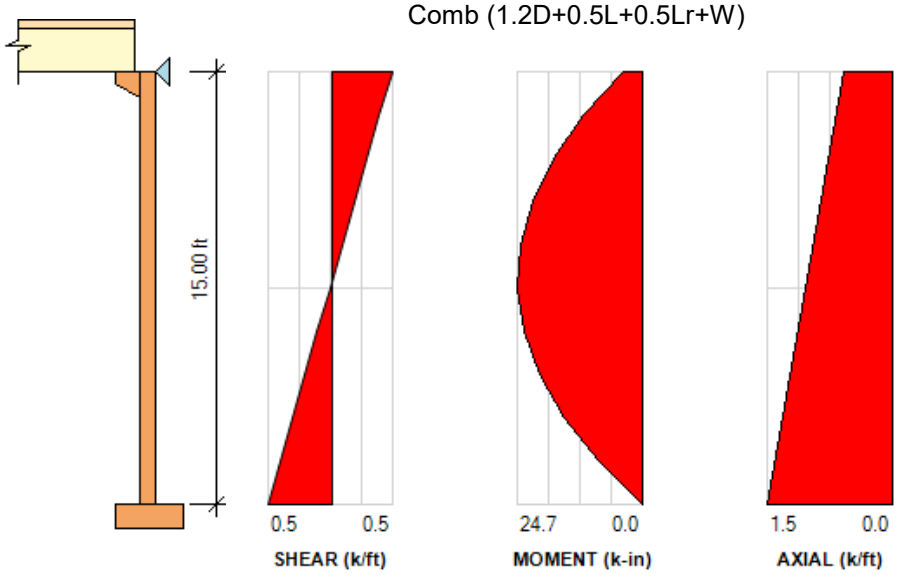
Nominal flexural strength Mn = *Ms + Mc =* 0.0 + 28.6 = 28.6 k-in/ft

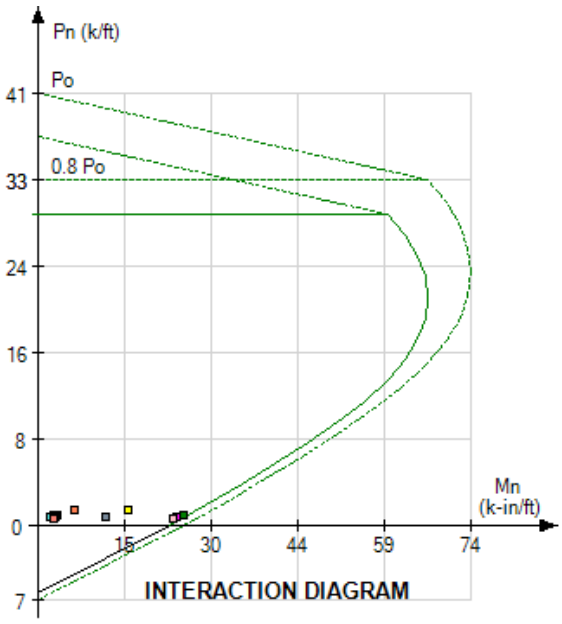
Under-strength ϕ-factor = 0.9 TMS 9.1.4.4

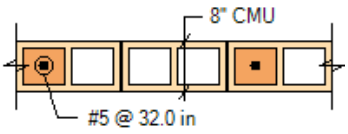
Design flexural strength ϕMn = 0.90 * 28.6 = 25.7 k-in/ft TMS 9.1.3

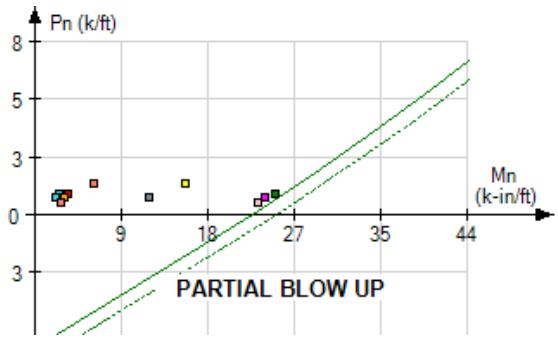
Flexural ratio = *Mu / ϕMn =* 24.7 / 25.7 = 0.96 < 1.0 OK

Comb (1.2D+0.5L+0.5Lr+W)









ASCE 7-16 Wind load Program (30.3 Part 1: Low-Rise Building Wall Components and Cladding)
Wall Components and Cladding *(Mean Roof Height h <= 60 ft)*

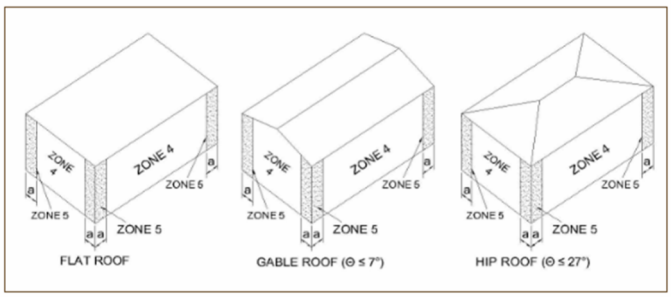
Project: LAMAS RESIDENCE -GARAGE
Prepared by: FEH of GFCE
E-mail: GFCE@GFCE.NET

Input:
ASCE 7-16 Strength Design Load & Allowable Stress Design
Wind Speed = 175 mph
Building Exposure = C (Open Terrain with Scattered Obstructions)
Internal Pressure Coefficient GCpi = 0.18 Enclosed Building
Roof angle is less than or equal to 10° or flat (External Pressure Coefficient is reduced 10%)
Kzt = 1.00
Kd = 0.85
Ke = 1.00
Mean Roof Height = 15 ft

Output:
Velocity Pressure Exposure Coefficient "Kz max." = 0.85
Per Table 26.10-1, page 268
LRFD uses wind speed Vult = 175 mph
Velocity Pressure (qh LRFD) = 56.64 psf = .00256*Kz*Kzt*Kd*Ke*V^2 lb/ft^2
Per eq.30.3-1, page 335
ASD uses wind speed Vasd[1] = 136 mph
Velocity Pressure (qh ASD) = 33.99 psf = .00256*Kz*Kzt*Kd*Ke*V^2*0.6 lb/ft^2

_	LRFD (Load Resistance Factor Design)			ASD (Allowable Stress Design)			(Per Figure 30.4-1, Page 335)		
Tributary Area Sq Feet	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	Positive Zone 4 & 5 psf	Negative Center Zone 4 psf	Negative Corner Zone 5 psf	External Pressure Coefficients		
							Positive GCp	Negative Center GCp	Negative Corner GCp
10 ft²	61.2	-66.3	-81.6	36.7	-39.8	-48.9	0.90	-0.99	-1.26
20 ft²	58.5	-63.6	-76.1	35.1	-38.1	-45.7	0.85	-0.94	-1.16
50 ft²	54.9	-60.0	-69.0	32.9	-36.0	-41.4	0.79	-0.88	-1.04
100 ft²	52.2	-57.3	-63.6	31.3	-34.4	-38.1	0.74	-0.83	-0.94
200 ft²	49.5	-54.6	-58.1	29.7	-32.7	-34.9	0.69	-0.78	-0.85
500 ft²	45.9	-51.0	-51.0	27.5	-30.6	-30.6	0.63	-0.72	-0.72
200 ft²	49.5	-54.6	-58.1	29.7	-32.7	-34.9	0.69	-0.78	-0.85

Least Width = 24 ft a = 2.4 ft (Zone 5)
10% of least width or 0.4h, whichever is smaller but not less than 4% of least width or 3 ft.



Foot notes:
[1] From Florida Building Code 7th edition 2020 in section 1609.3.1 wind speed conversion.

ASDIP Concrete 5.2.1.6

MASONRY WALL DESIGN

GEOMETRY

Block Thickness ..... 8 in = KH / 23

Use 1-#5 @ 32.0 in Vertical Bars , d' = 3.8 in

Vertical ρ = 0.0023 Solid Grouted .... N

Block Weight ..... 50 psf

SLENDERNESS

Wall Height H_1 = 14.50 H_2 = 10.00 ft

Effective Height K-factor 1.00

Parapet Height ..... 0.00 ft

Lateral Stability ..... *Non-sway Wall*

Deflection = 0.05 in < Allow = 1.22 in OK

UNFACTORED LOADS

	Dead		Live		RLive		Snow		Wind		Seismic		
	2nd	Roof	2nd	Roof	2nd	Roof	2nd	Roof	2nd	Roof	2nd	Roof	
Uniform Load w	1.5	0.8	0.5	0.0	0.0	0.5	0.0	0.0	0.0	0.0	0.0	0.0	k/ft
Concentrated P	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	kip
Pressure on Wall									62.0	62.0	0.0	0.0	psf
Pressure Parapet										0.0		0.0	psf

w Eccentricity ..... 2.0 in P Eccentricity ..... 4.0 in P Spacing ..... 48.0 in

Seismic Load = *Max(0.1, 0.4 * I * Sds) * SW* =Max(0.1, 0.4 * 1.0 * 0.25) * 50 = 5.0 psf ASCE 12.11.1

PROPERTIES

Masonry f'm = 1.5 ksi Max. strain = 0.0025 Stress-strain = Whitney TMS 9.3.2

Young's Modulus *Em = 900 * f'm* = 900 * 1.5 = 1350 ksi TMS 4.2.2

Rupture fr = 115 psi TMS 9.1.9.2

Steel fy = 60.0 ksi Stress-strain = Elasto-plastic Es = 29000 ksi ACI 20.2.2.2

Radius of gyration rx = *Thick / √12* = 2.2 in

Inertia Inet = $(bw * (Thick - 2\ fs)^3 / 12 + 2 * Sp * fs * (Thick / 2 - fs / 2)^2 + Sp * fs^3 / 12) * 12 / Sp$

= $(11.2 * (7.63 - 2 * 1.3)^3 / 12 + 2 * 32 * 1.3 * (7.63 / 2 - 1.3 / 2)^2 + 32 * 1.3^3 / 12) * 12 / 32 = 354\ in^4/ft$

AMPLIFIED FACTORED LOADS (1st Level)

Eff. reaction length Le = *Min (4 + 4 * Thick, Spacing)* = Min (4 + 4 * 7.6, 48.0) / 12 = 2.88 ft ACI 11.2.3.1

- Gravity At Roof

Total w dead = *w dead + P dead / Le + Hp * SW* = 0.8 + 0.0 / 2.88 = 0.8 k/ft

Total w live = *w live + P live / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

Total w Rlive = *w Rlive + P Rlive / Le* = 0.5 + 0.0 / 2.88 = 0.5 k/ft

Total w snow = *w snow + P snow / Le* = 0.0 + 0.0 / 2.88 = 0.0 k/ft

M dead = *w dead * Ecc + P dead / Le * Ecc* = 0.8 * 2.0 + 0.0 / 2.88 * 4.0 = 1.6 k-in/ft

M live = *w live * Ecc + P live / Le * Ecc* = 0.0 * 2.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

M Rlive = *w Rlive * Ecc + P Rlive / Le * Ecc* = 0.5 * 2.0 + 0.0 / 2.88 * 4.0 = 1.0 k-in/ft

M snow = *w snow * Ecc + P snow / Le * Ecc* = 0.0 * 2.0 + 0.0 / 2.88 * 4.0 = 0.0 k-in/ft

- Gravity At Second

Total w dead = *w dead + P dead / Le + SW =* 1.5 + 0.0 / 2.88 = 1.5 k/ft

Total w live = *w live + P live / Le =* 0.5 + 0.0 / 2.88 = 0.5 k/ft

Total w Rlive = *w Rlive + P Rlive / Le =* 0.0 + 0.0 / 2.88 = 0.0 k/ft

Total w snow = *w snow + P snow / Le =* 0.0 + 0.0 / 2.88 = 0.0 k/ft

M dead = *(w dead * Ecc + P dead / Le * Ecc) / 2 =* (1.5 * 2.0 + 0.0 / 2.88 * 4.0) / 2 = 3.0 k-in/ft

M live = *(w live * Ecc + P live / Le * Ecc) / 2 =* (0.5 * 2.0 + 0.0 / 2.88 * 4.0) / 2 = 1.0 k-in/ft

M Rlive = *(w Rlive * Ecc + P Rlive / Le * Ecc) / 2 =* (0.0 * 2.0 + 0.0 / 2.88 * 4.0) / 2 = 0.0 k-in/ft

M snow = *(w snow * Ecc + P snow / Le * Ecc) / 2 =* (0.0 * 2.0 + 0.0 / 2.88 * 4.0) / 2 = 0.0 k-in/ft

- Controlling load combination: 1.2D+0.5L+0.5Lr+W , Span: 2

Axial load Pu = 1.2 * 1.1 + 0.5 * 0.0 + 0.5 * 0.5 + 0 * 0.0 + 1 * 0.0 + 0 * 0.0 = 1.5 k/ft

Mu gravity mid = 1.2 * 0.3 + 0.5 * 0.3 + 0.5 * -0.4 + 0 * 0.0 = 0.3 k-in/ft

Mu lateral mid = 1 * -1.6 + 0 * -0.1 = 15.4 k-in/ft

Mu gravity top = 1.2 * 1.6 + 0.5 * 0.0 + 0.5 * 1.0 + 0 * 0.0 = 2.4 k-in/ft

Mu lateral top = 1 * 0.0 + 0 * 0.0 = 0.0 k-in/ft

M1x grav = 2.4 k-in/ft M1x lat = 0.0 k-in/ft M2x grav = 0.3 k-in/ft M2x lat = 15.4 k-in/ft

M1x = 2.4 + 0.0 = 2.4 k-in/ft M2x = 0.3 + 15.4 = 15.6 k-in/ft (M2 is always positive)

Icr = $n * (As + Pu * Thick / (fy * 2 * d)) * (d - c)^2 + b * c^3 / 3 =$ TMS Eq. 9-30

21 * (0.12 + 1.5 * 7.63 / (60.0 * 2 * 3.8)) * (3.8 - 0.8)² + 12 * 0.8³ / 3 = 30.1 in⁴/ft

Mcr = *(frm / 1000 + Pu * Spacing / b / An) * Sx =* TMS 9.3.5.4.4

(115 / 1000 + 1.5 * 32 / 12 / 137.3) * 92.8 = 13.4 k-in/ft

Ieff = 0.75 * Inet = 0.75 * 353.8 = 30.1 in⁴/ft since Mu < Mcr TMS 9.3.5.4.3

Pe = $n^2 * Em * Ieff / (Kx * H)^2 =$ n² * 1350.0 * 30.1 / (1.0 * 10.00 * 12)² = 27.9 k/ft TMS Eq. 9-29

$$\psi = \frac{1}{1 - Min\left(1.0, \frac{Pu}{Pe}\right)} >= 1.0 = \frac{1}{1 - Min\left(1.0, \frac{1.5}{27.9}\right)} = 1.06$$ TMS Eq. 9-28

Amplified moment Mcx = $\psi * M2x =$ 1.06 * 15.6 = 19.3 k-in/ft TMS Eq. 9-27

DESIGN CODES

Masonry Design	TMS 402-16
Load Combinations	ASCE 7-10/16

WALL STRENGTH

- Check the bending capacity for Pu = 1.5 k/ft and Mu = 19.3 k-in/ft Comb (1.2D+0.5L+0.5Lr+W)

By trial and error, neutral axis depth *k = c / d =* 0.17

Strain1 = *(k * d - d1) / (k * d) * Max strain =* (0.17 * 3.8 - 3.8) / (0.17 * 3.8) * 0.0025 = -0.0261

As (in²/ft)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (k/ft)	Ms = Fs(h/2-d) (k-in/ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.31	3.8	-0.0261	60.0	0.0	-7.0	0.0	0.0
					T = -7.0	Ms = 0.0	Is = 0.0

Conc. force Fc = *C = Sum (Fci) = Sum (fci * Aci) =*7.5 k/ft

Conc. moment Mc =*Sum (Mci) = Sum (Fci * (h / 2 - di)) =* 27.0 k-in/ft

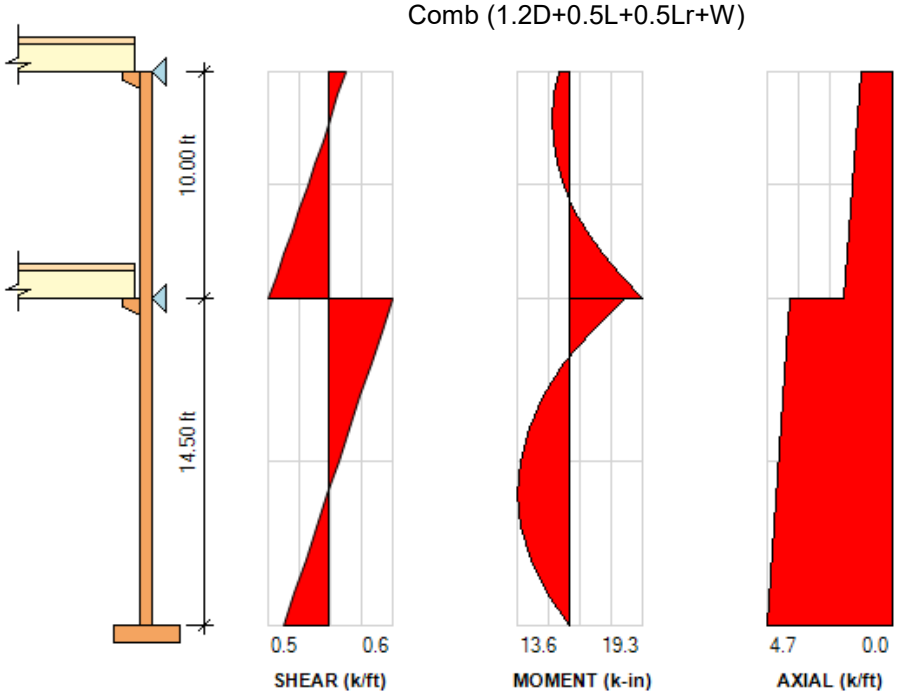
Nominal flexural strength Mn = *Ms + Mc =* 0.0 + 27.0 = 27.0 k-in/ft

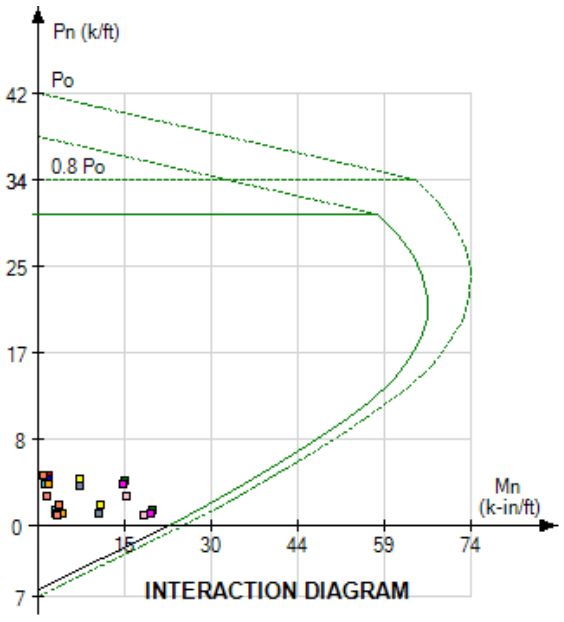
Under-strength ϕ-factor = 0.9 TMS 9.1.4.4

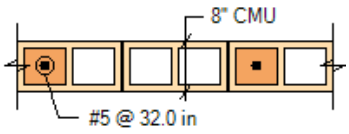
Design flexural strength ϕMn = 0.90 * 27.0 = 24.3 k-in/ft TMS 9.1.3

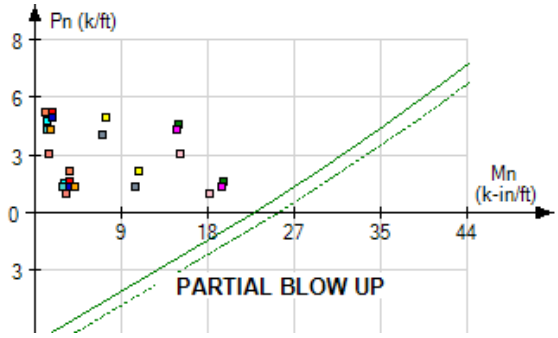
Flexural ratio = *Mu / ϕMn =* 19.3 / 24.3 = 0.80 < 1.0 OK

Comb (1.2D+0.5L+0.5Lr+W)





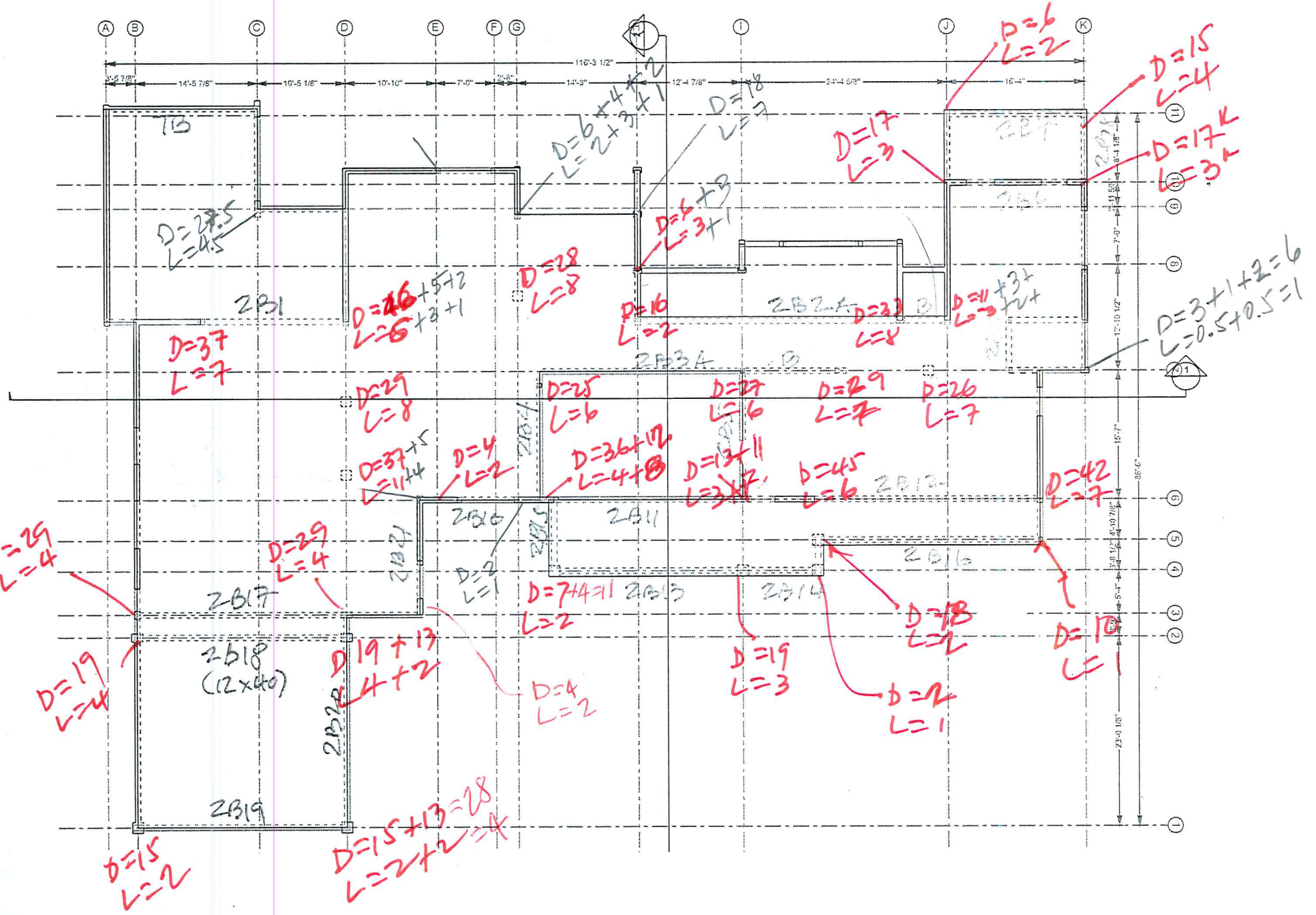




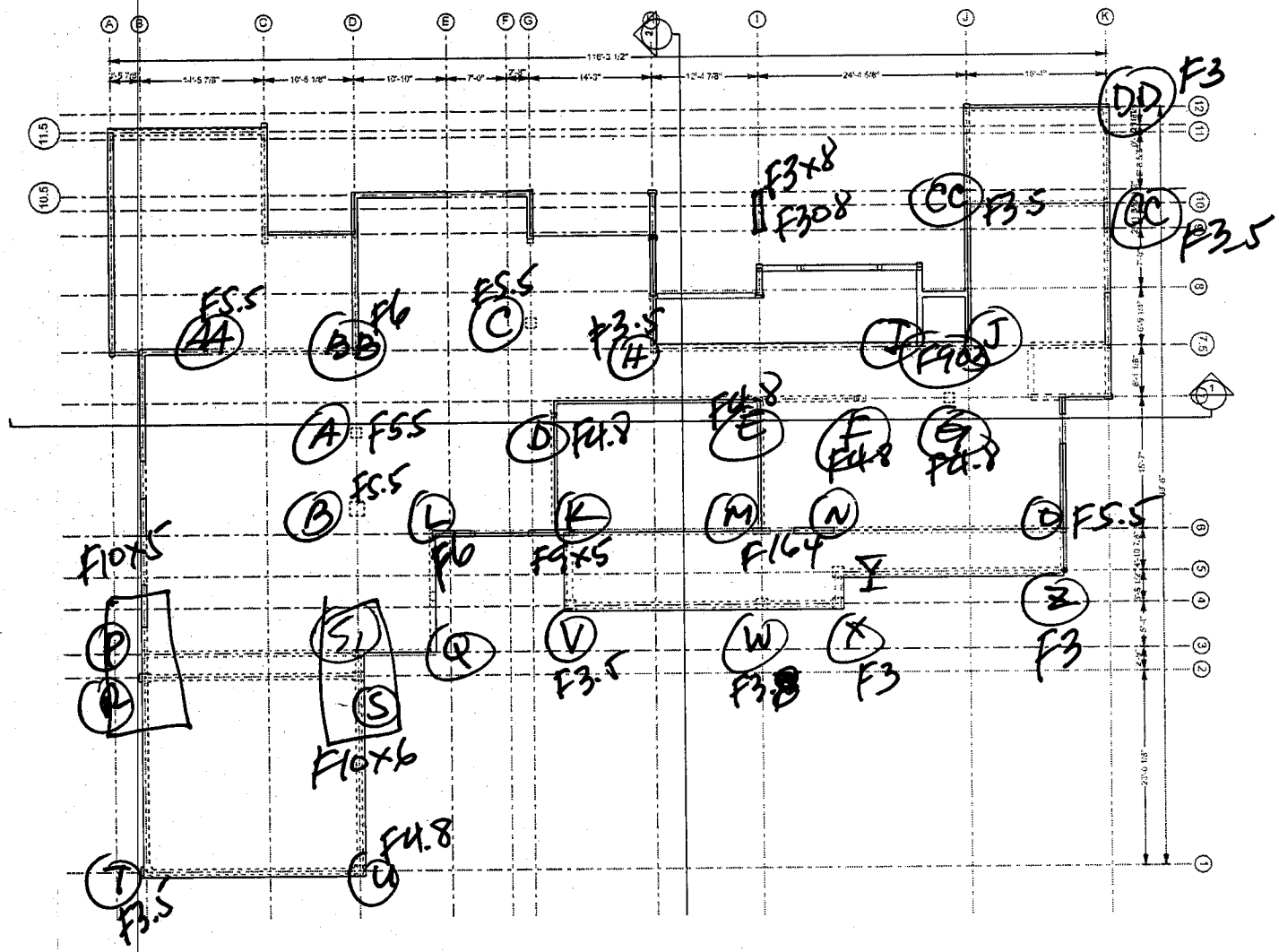


Foundations Calculations

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET


A
B
C
D
E
F
G
H
I
J
K
116'-3 1/2"
14'-5 7/8"
10'-5 1/8"
10'-10"
7'-0"
14'-3"
12'-4 7/8"
24'-4 5/8"
16'-4"
TB
2B1
2B2
2B3
2B4
2B5
2B6
2B7
2B8
2B9
2B10
2B11
2B12
2B13
2B14
2B15
2B16
2B17
2B18
(12x40)
2B19
D=6
L=2
D=15
L=4
D=17
L=3
D=17
L=3
D=18
L=7
D=6+4+2
L=2+3+1
D=6+3
L=3+1
D=28
L=8
D=16
L=2
D=33
L=8
D=11+3+2
L=2+1
D=3+1+2=6
L=0.5+0.5=1
D=37
L=7
D=46+5+2
L=6+3+1
D=29
L=8
D=25
L=6
D=27
L=6
D=29
L=7
D=26
L=7
D=37+5
L=11+4
D=4
L=2
D=36+12
L=4+8
D=13+11
L=3+7
D=45
L=6
D=42
L=7
D=2
L=1
D=7+4=11
L=2
D=19
L=3
D=18
L=2
D=10
L=1
D=2
L=1
D=29
L=4
D=29
L=4
D=19
L=4
D=19+13
L=4+2
D=4
L=2
D=15
L=2
D=15+13=28
L=2+2=4

FTG Col'N ID


A
B
C
D
E
F
G
H
I
J
K
11.5
10.5
F3
F3x8
F3x8
CC
F3.5
F3.5
F5.5
F5.5
F5.5
F3.5
F5.5
F4.8
F4.8
F4.8
F5.5
F10x5
F9x5
F16
F10x6
F4.8
F3.5
F3
F3
F3
F3.5

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PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

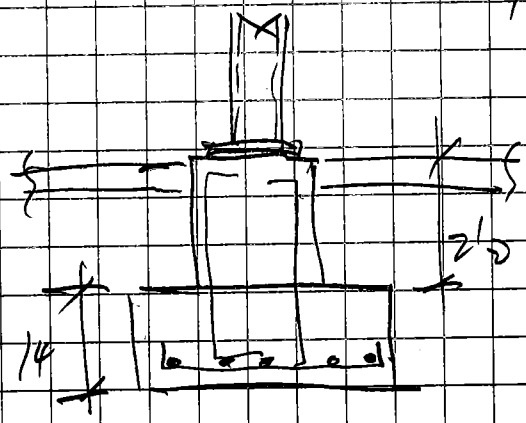
Footings

— Columns w/ similar load

(A), (B), (C)

$P_d = 38^k$

$P_L = 12^k$



$5' - 6'' \times 5' - 6'' \times 12''$

w/ 5 #6 EW

Bottom

— Columns w/ similar load

(D), (E), (F), (G)

$P_d = 29^k$

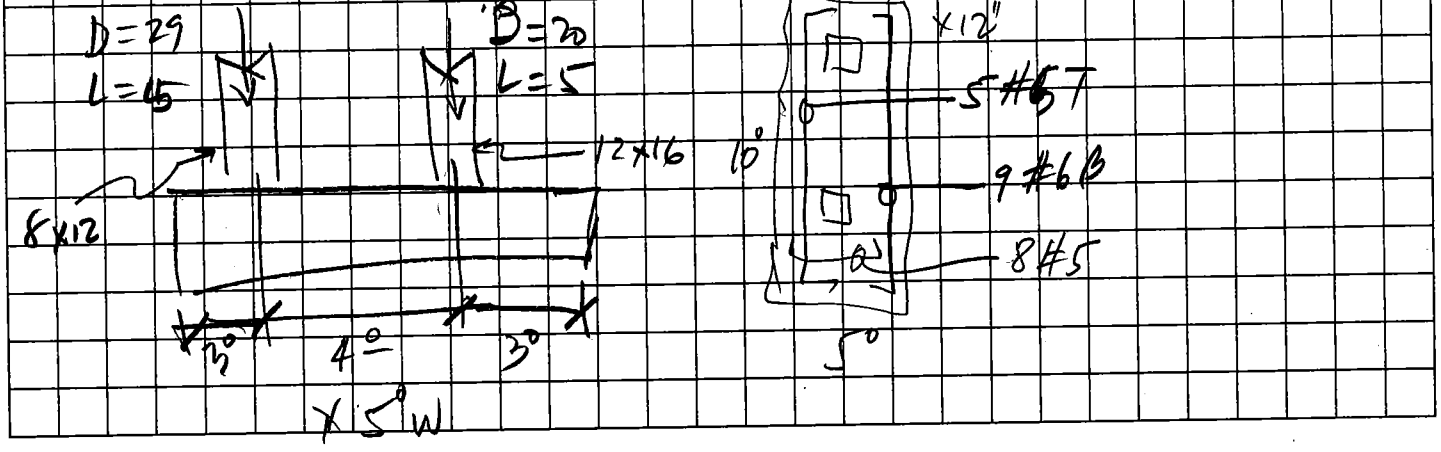
$P_L = 7^k$

$4' - 9'' \times 4' - 9'' \times 12''$

w/ 5 #5 EW

Bott.

— Column (P) & (R) Combined



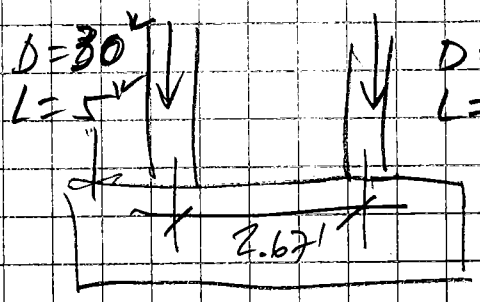
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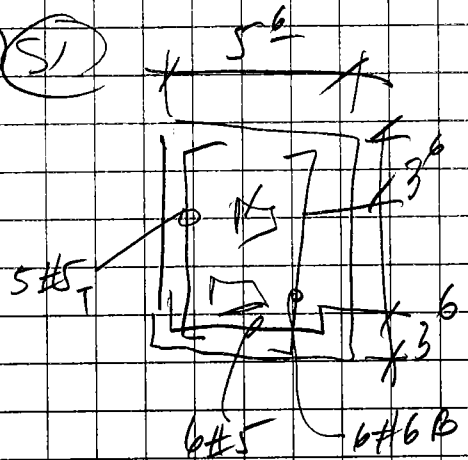
PROJECT NAME: _____

PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Column Combined (SS)





Column FTG (AA)

$P_d = 38^k$

$P_L = 7^k$ → 5'-6" x 12" w/ 5 #6 EW

Column FTG (BB)

$P_d = 50^k$

$P_L = 6^k$ 6'0" x 12" w/ 5 #6 EW

Column FTG (W)

$P_d = 28 + 4 = 32^k$

$P_L = 4^k$ 4'9" x 12" w/ 5 #6 EW

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PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

Column FTG (T)

$P_d = 17^k$ $P_L = 2^k$

$3\frac{6}{x}12''$ w/ 4#5 ew

Column FTG (V)

$P_d = 15^k + W = 17^k$

$P_L = 2^k$

$3\frac{6}{-} \times 12''$ / 4#5

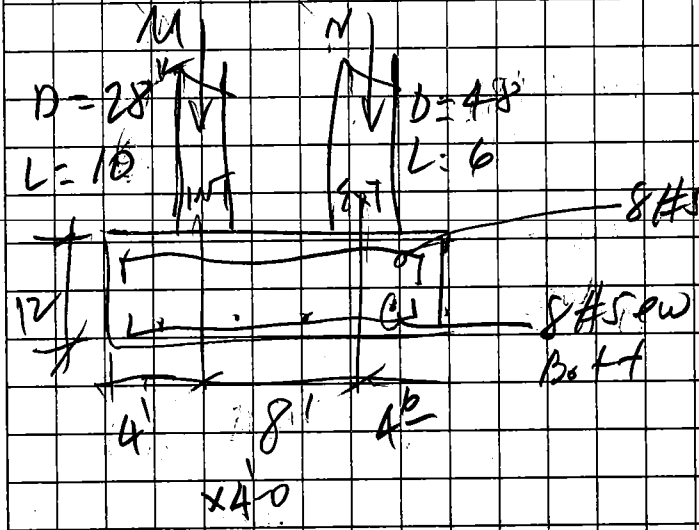
Column FTG (W)

$P_d = 21^k$

$P_L = 3^k$

$3\frac{6}{-} \times 12''$ / w 4#5

Column FTG (N)(M)



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PROJECT NAME: _____

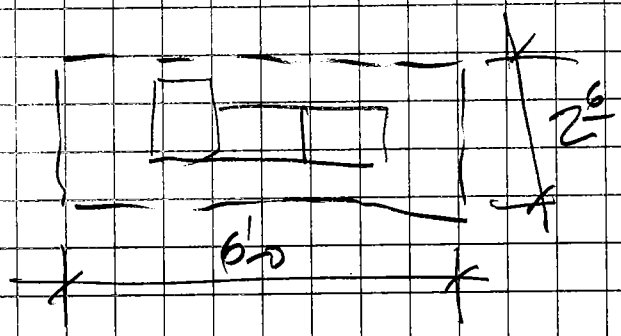
PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

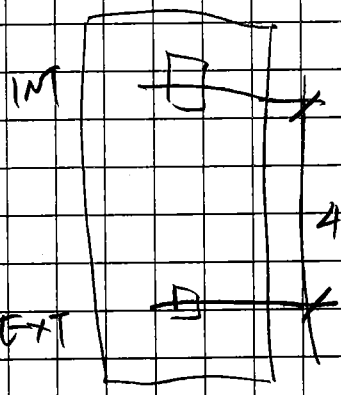
Footing Col (I)

$P_d = 22^k$

$P_L = 2^k$



Footings (O) (Z)



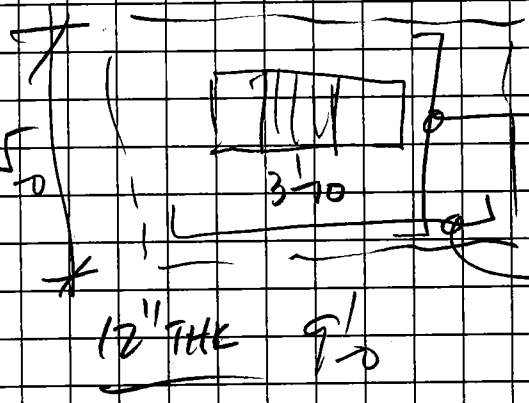
INT: $P_d = 45^k$, $P_L = 7^k$ ← 5'-6" □ x 14" w/ 5 #6

EXT: $P_d = 10^k$, $P_L = 2^k$ ← 3'-0" □ x 14" w/ 4 #4

Footing (K)

$P_d = 54^k + 7 = 61^k$

$P_L = 15^k$



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PROJECT #: _____ DATE: _____

ENG: _____ SHEET: _____ OF _____

FTG Col (C1)

$P_d = 19.3 + 1.3 = 20.6^k$

$P_L = 7.2^k$

[4" ⌀ x 12" w/ 4 #5 ew Bott

FTG Col (L)

$P_d = 48^k$

$P_L = 15^k$

[6" ⌀ x 12" 6 #5 ew

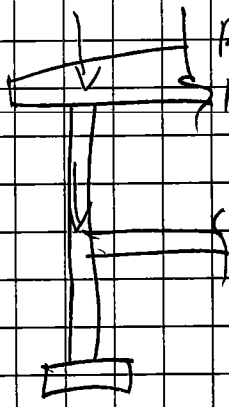
FTG Col (A)

$P_d = 18^k$

$L = 12^k$

[3'-6" x 12" 5 #4 3 #5 ew

WALL



$P_D = 15 \times 35 = .525^{k/f}$

$P_L = 15 \times 20 = .300^{k/f}$

$F_D = 0.7^{k/f}$

$F_L = 0.2^{k/f}$

$wall = 25 \times 60 = 1.5^{k/f}$

$\Sigma D = 2.73^{k/f}$

See Comp. WF24

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	5.50	ft	
Footing Width (Z-dir)	5.50	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	16.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	2.00	ft	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	0.98		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	38.0	12.0	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

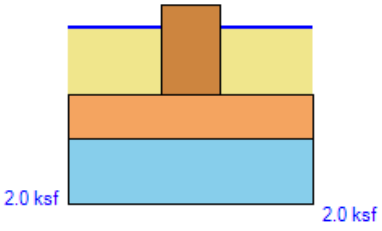
OVERTURNING

- About X-X	Force kip	Arm ft	Moment k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	3.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	3.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	2.75	2.75	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

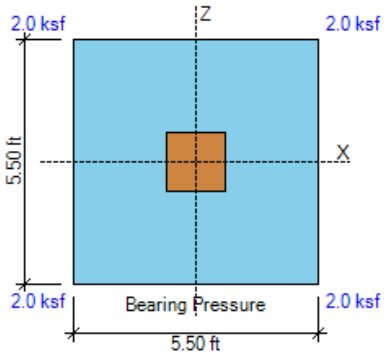
RESISTING

- About X-X	Force kip	Arm ft	Moment k-ft
Footing Weight ..	2.7	2.75	7.5
Column Weight ..	0.3	2.75	0.9
Soil Cover	3.8	2.75	10.3
Axial Force P	22.8	2.75	62.7
Water Buoyancy .	-1.1	2.75	-3.1
Rv =	28.5	RM =	78.3
- About Z-Z			
Footing Weight ..	2.7	2.75	7.5
Column Weight ..	0.3	2.75	0.9
Soil Cover	3.8	2.75	10.3
Axial Force P	22.8	2.75	62.7
Water Buoyancy .	-1.1	2.75	-3.1
Rv =	28.5	RM =	78.3



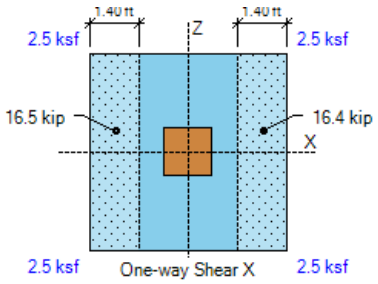
SLIDING (combDlg. 0.6D+0.6W)

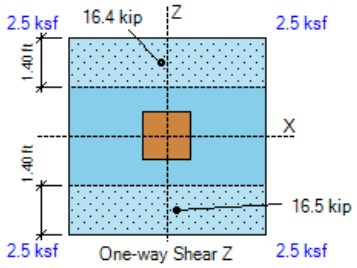
Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	12.8	kip	
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	6.5	6.5	kip
Horiz. Resisting Force	19.4	19.4	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK



UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.3	kip
Footing Selfweight	2.7	kip
Soil Cover Weight	3.8	kip
Downward Force W	6.8	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.40	
Unbalanced Moment Factor ɣvz ...	0.40	
Punching Shear due to Axial	70.0	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	70.0	psi
Effective Perimeter bo	97.0	in
Average Effective Depth d	8.3	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	1.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.43	OK

ONE-WAY SHEAR (Comb: 1.2D+1.6L+0.5Lr)

	- Side	+ Side		
One-way Shear Vux ...	16.5	16.4	kip	
One-way Shear Vuz ...	16.5	16.4	kip	
Under-strength ϕ Factor		0.75		
One-way Shear Strength ϕVcx		46.8	kip	OK
One-way Shear Strength ϕVcz		42.7	kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..		0.39		OK

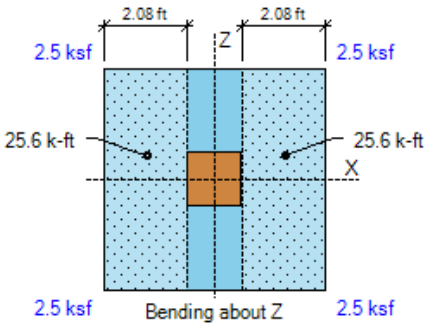
FLEXURE (Comb: 1.2D+1.6L+0.5Lr)

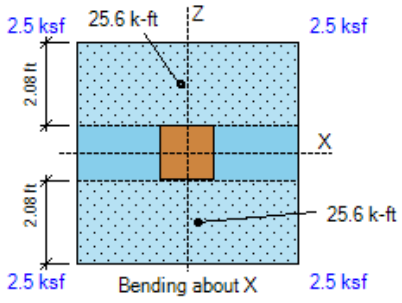
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..		N.A.		
Z-Bars Development Length Ratio ..		N.A.		
Min. Top Steel Area Ratio		N.A.		
Under-strength ϕ Factor		0.60		
Bending Strength -ϕMnx		21.7	k-ft	OK
Bending Strength -ϕMnz		21.7	k-ft	OK
Max. Bending Ratio -Mu/ϕMn		0.00		OK
Max. Bar Spacing Ratio		N.A.		

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	25.6	25.6	k-ft	
Bending Moment +Muz ...	25.6	25.6	k-ft	
Use 5 #6 Bars Parallel to X (Bot) , d = 8.6 in				
Use 5 #6 Bars Parallel to Z (Bot) , d = 7.9 in				
X-Bars Development Length Ratio ..		0.53		OK
Z-Bars Development Length Ratio ..		0.53		OK
Min. Bottom Steel Area Ratio		0.65		OK
Under-strength ϕ Factor		0.90		
Bending Strength +ϕMnx		74.1	k-ft	OK
Bending Strength +ϕMnz		81.5	k-ft	OK
Max. Bending Ratio +Mu/ϕMn		0.35		OK
Max. Bar Spacing Ratio		0.83		OK

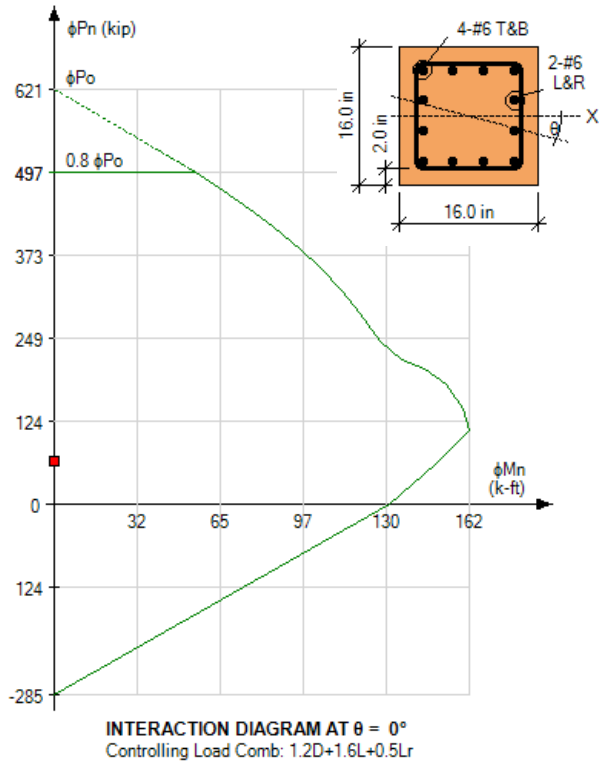
TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.3	ksi	
Column Bearing Strength ϕPn ...	2.5	ksi	OK
Footing Bearing Strength ϕPn ...	4.1	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.2D+1.6L+0.5Lr

DESIGN CODES

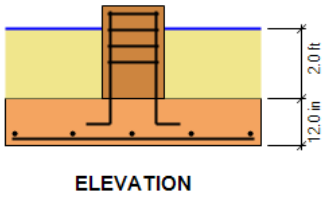
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

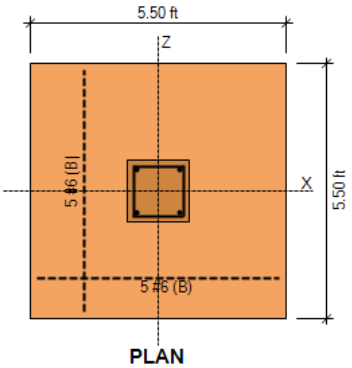
Column Concrete f'c	3.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+1.6L+0.5Lr)

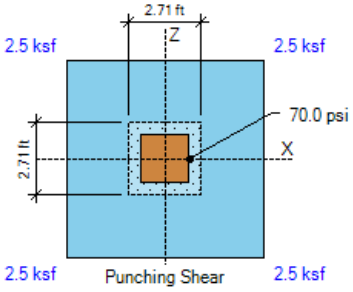
Axial Load Pu	65.4	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	495.2	kip
Flexural Strength ϕMn	150.8	k-ft
Biaxial Strength Ratio	0.13	OK



ELEVATION



PLAN



Punching Shear

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	8.00	ft	
Footing Thickness	14.0	in	OK
Soil Cover	1.00	ft	
Column Length (X-dir)	8.0	in	
Column Width (Z-dir)	53.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	0.2	ksf	
Soil Pressure at Corner 3	0.2	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.96		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.13		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	17.0	2.7	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	22.5	4.5	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: D+L)

- Overturning about X-X
- Moment Mx = 1.0 * 22.5 + 1.0 * 4.5 = 27.0 k-ft
- Shear Force Vz = 1.0 * 0.0 + 1.0 * 0.0 = 0.0 kip

  Arm = 0.00 + 14.0 / 12 = 1.17 ft Moment = 0.0 * 1.17 = 0.0 k-ft
- Passive Force = 0.0 kip Arm = 0.47 ft Moment = 0.0 k-ft
- Overturning moment X-X = 27.0 + 0.0 = 27.0 k-ft

- Resisting about X-X
- Footing weight = *1.0 * W * L * Thick * Density* = 1.0 * 8.00 * 3.00 * 14.0 / 12 * 0.15 = 4.2 kip

  Arm = *W / 2* = 8.00 / 2 = 4.00 ft Moment = 4.2 * 4.00 = 16.8 k-ft
- Pedestal weight = *1.0 * W * L * H * Density* = 1.0 * 53.0 / 12 * 8.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *W / 2 - Offset* = 8.00 / 2 - 0.0 / 12 = 4.00 ft Moment = 0.0 * 4.00 = 0.0 k-ft
- Soil cover = *1.0 * W * L * SC * Density* 1=0 * (8.00 * 3.00 - 53.0 / 12 * 8.0 / 12) * 1.0 * 110 = 2.3 kip

  Arm = *W / 2* = 8.00 / 2 = 4.00 ft Moment = 2.3 * 4.00 = 9.3 k-ft
- Buoyancy = *1.0 * W * L * γ * (SC + Thick - WT)* = 1.0 * 8.00 * 3.00 * 62 * (0.17) = -0.2 kip

  Arm = *W / 2* = 8.00 / 2 = 4.00 ft Moment = 0.2 * 4.00 = -1.0 k-ft
- Axial force P = 1.0 * 17.0 + 1.0 * 2.7 = 19.7 kip

  Arm = *W / 2 - Offset* = 8.00 / 2 - 0.0 / 12 = 4.00 ft Moment = 19.7 * 4.00 = 78.8 k-ft
- Resisting moment X-X = 16.8 + 0.0 + 9.3 + 78.8 + -1.0 = 103.9 k-ft
- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{103.9}{27.0} = 3.85 > 1.50$ OK

- Overturning about Z-Z

- Moment Mz = 1.0 * 0.0 + 1.0 * 0.0 = 0.0 k-ft

- Shear Force Vx = 1.0 * 0.0 + 1.0 * 0.0 = 0.0 kip

Arm = 0.00 + 14.0 / 12 = 1.17 ft Moment = 0.0 * 1.17 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.47 ft Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *1.0 * W * L * Thick * Density =* 1.0 * 8.00 * 3.00 * 14.0 / 12 * 0.15 = 4.2 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 4.2 * 1.50 = 6.3 k-ft

- Pedestal weight = *1.0 * W * L * H * Density =* 1.0 * 53.0 / 12 * 8.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *1.0 * W * L * SC * Density =* 1.0 * (8.00 * 3.00 - 53.0 / 12 * 8.0 / 12) * 1.0 * 110 = 2.3 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 2.3 * 1.50 = 3.5 k-ft

- Buoyancy = *1.0 * W * L * γ * (SC + Thick - WT) =* 1.0 * 8.00 * 3.00 * 62 * (0.17) = -0.2 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.4 k-ft

- Axial force P = 1.0 * 17.0 + 1.0 * 2.7 = 19.7 kip

Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 19.7 * 1.50 = 29.6 k-ft

- Resisting moment Z-Z = 6.3 + 0.0 + 3.5 + 29.6 + -0.4 = 38.9 k-ft

- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{38.9}{0.0}$ = 99.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 27.0 + 0.0 = 27.0 k-ft

Resisting moment X-X = 16.8 + 0.0 + 9.3 + -1.0 + 78.8 = 103.9 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 6.3 + 0.0 + 3.5 + -0.4 + 29.6 = 38.9 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 4.2 + 0.0 + 2.3 - 0.2 + 19.7 = 26.0 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{38.9 - 0.0}{26.0} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{103.9 - 27.0}{26.0} = 2.96 \text{ ft}$$

X-ecc = *Length / 2 - Xp* =3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 8.00 / 2 - 2.96 = 1.04 ft

Area = *Width * Length =* 8.00 * 3.00 = 24.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 8.00² / 6 = 32.0 ft³

Sz = *Width * Length² / 6 =* 8.00 * 3.00² / 6 = 12.0 ft³

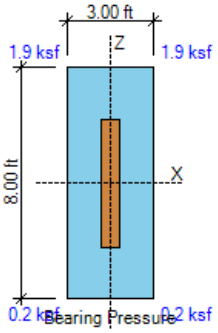
- Footing is in full bearing. Soil pressures are as follows:

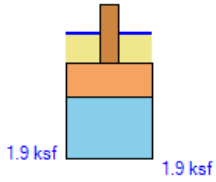
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 26.0 * (1 / 24.0 + 1.04 / 32.0 + 0.00 / 12.0) = 1.93 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 26.0 * (1 / 24.0 - 1.04 / 32.0 + 0.00 / 12.0) = 0.24 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 26.0 * (1 / 24.0 - 1.04 / 32.0 - 0.00 / 12.0) = 0.24 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 26.0 * (1 / 24.0 + 1.04 / 32.0 - 0.00 / 12.0) = 1.93 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33 (per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2)* = 4.33 * 110 * (1.00 + 14.0 / 12 / 2) = 0.75 ksf

X-Passive force = *Pressure * Thick * Width* = 0.75 * 14.0 / 12 * 8.00 = 7.0 kip

Z-Passive force = *Pressure * Thick * Length* = 0.75 * 14.0 / 12 * 3.00 = 2.6 kip

Friction force = *Resisting force * Friction coeff.* = Max (0, 14.0 * 0.45) = 6.3 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\frac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \frac{1.00 * 7.0 + 1.00 * 6.3}{0.0}$ = 99.99 > 1.50 OK

- Sliding safety factor Z-Z = $\frac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \frac{1.00 * 2.6 + 1.00 * 6.3}{0.0}$ = 89.18 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\frac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \frac{0.0 + 2.5 + 1.4 - 0.1}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

Concrete f'c = 3.0 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

d Top X-dir = *Thick - Cover - X-diameter / 2* = 14.0 - 2.0 - 0.8 / 2 = 11.6 in

d Top Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2* = 14.0 - 2.0 - 0.8 - 0.8 / 2 = 10.9 in

d Bot X-dir = *Thick - Cover - X-diameter / 2* = 14.0 - 3.0 - 0.6 / 2 = 10.7 in

d Bot Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2* = 14.0 - 3.0 - 0.6 - 0.6 / 2 = 10.1 in

*ϕVcx = 2 * ϕ * √(f'c) * Width * d / 1000* = 2 * 0.75 * √(3000) * 8.0 * 12 * 10.7 / 1000 = 84.3 kip ACI Eq. (22.5.5.1)

*ϕVcz = 2 * ϕ * √(f'c) * Length * d / 1000* = 2 * 0.75 * √(3000) * 3.0 * 12 * 10.1 / 1000 = 29.8 kip

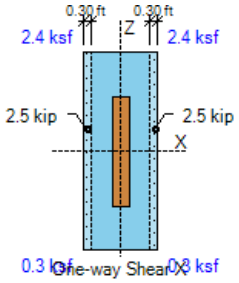
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

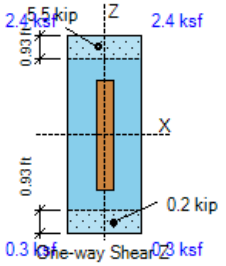
One-way shear Vux (- Side) = 2.5 kip < 84.3 kip OK

One-way shear Vux (+ Side) = 2.5 kip < 84.3 kip OK

One-way shear Vuz (- Side) = 0.2 kip < 29.8 kip OK

One-way shear Vuz (+ Side) = 5.5 kip < 29.8 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(3000) * 3.00 * 14.0² / 6 / 1000 = 4.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(3000) * 8.00 * 14.0² / 6 / 1000 = 11.4 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 16.1 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 16.1 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 42.9 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 42.9 k-ft OK

- Bottom Bars

Use 4 #5 Z-Bars *ρ = As / b d* = 1.2 / (3.00 * 12 * 10.1) = 0.0034 — q = 0.0034 * 60 / 3.0 = 0.068

Use 8 #5 X-Bars *ρ = As / b d* = 2.5 / (8.00 * 12 * 10.7) = 0.0024 — q = 0.0024 * 60 / 3.0 = 0.048

β = W / L = 8.00 / 3.00 = 2.67 *γs = 2 * β / (β + 1)* = 2 * 2.67 / (2.67 + 1) = 1.45 — ACI 13.3.3.3

Bending strength *ϕMn = ϕ * b * d² * f'c * q * (1 - 0.59 * q)* — ACI 22.2.2

ϕMnx = 0.90 * 3.00 * 12 * 10.1² * 3.0 * 0.068 * (1 - 0.59 * 0.068) = 53.9 k-ft

ϕMnz = 0.90 * 8.00 * 12 * 10.7² * 3.0 * 0.048 / 1.45 * (1 - 0.59 * 0.048 / 1.45) = 80.4 k-ft

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Mux (- Side) = 0.6 k-ft < 53.9 k-ft OK ratio = 0.01

Bottom moment Mux (+ Side) = 9.3 k-ft < 53.9 k-ft OK ratio = 0.17

Bottom moment Muz (- Side) = 5.6 k-ft < 80.4 k-ft OK ratio = 0.07

Bottom moment Muz (+ Side) = 5.6 k-ft < 80.4 k-ft OK ratio = 0.07

X-As min = *0.0018 * Width * Thick* = 0.0018 * 8.00 * 12 * 14.0 = 2.4 in² < 2.5 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 14.0 = 0.9 in² < 1.2 in² OK — ACI 8.6.1.1

X-As max for 0.005 tension strain = 13.49 in² > 2.48 in² OK — ACI 21.2.2

Z-As max for 0.005 tension strain = 5.06 in² > 1.24 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.63 / 2, 12.9 / 2) / 0.63) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.3a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (3000)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.63 * 0.07) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* = — ACI 25.4.3

X-Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (3000)½ * 1.0 * 0.7 * 0.0 * 0.63 * 0.07) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover* = 3.00 * 12 / 2 + 0.0 - 8.0 / 2 - 2.5 = 11.5 in > 6.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover* = 3.00 * 12 / 2 - 0.0 - 8.0 / 2 - 2.5 = 11.5 in > 6.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.63 / 2, 10.0 / 2) / 0.63) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (3000)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.63 * 0.07) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* = ACI 25.4.3

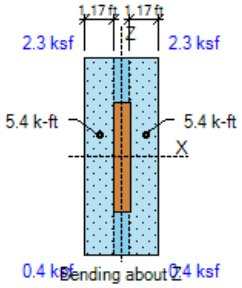
Z-Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (3000)½ * 1.0 * 0.7 * 0.0 * 0.63 * 0.17) = 6.0 in

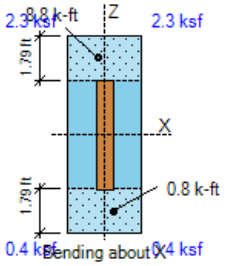
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover* = 3.00 * 12 / 2 + 0.0 - 8.0 / 2 - 2.5 = 19.0 in > 6.0 in OK

+Z Ld provided = *(Width - Col) / 2 - Offset - Cover* = 3.00 * 12 / 2 - 0.0 - 8.0 / 2 - 2.5 = 19.0 in > 6.0 in OK

X-bar spacing = 12.9 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

Area *A1 = col L * col W* = 8.0 * 53.0 = 424.0 in²

Sx = *col W * col L² / 6* = 53.0 * 8.0² / 6 = 565.3 in³

Sz = *col L * col W² / 6* = 8.0 * 53.0² / 6 = 3745.3 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 24.7 / 424.0 + 0.0 * 12 / 565.3 + 34.2 * 12 / 3745.3 = 0.2 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.00 * 12 / 2 - 0.0 - 8.0 / 2, 8.00 * 12 / 2 - 0.0 - 53.0 / 2 = 14.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.00 * 12 * 8.0 * 12, (8.0 + 2 * 14.0) * (53.0 + 2 * 14.0)] = 2916.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 3.0 * Min [2, √(2916.0 / 424.0)] = 3.3 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.65 * 5.28 * 60.0 / 424.0 = 0.5 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 3.3 + 0.5 = 3.8 ksi > 0.2 psi OK

Column *ϕPnc = ϕ * 0.85 * f'c* = 0.65 * 0.85 * 4.0 = 2.2 ksi

Column *ϕPns = ϕ * As * Fy / A1* = 0.65 * 5.28 * 60.0 / 424.0 = 0.5 ksi ACI 22.8.3.2

Column bearing *ϕPn = ϕPnc + ϕPns* = 2.2 + 0.5 = 2.7 ksi > 0.2 psi OK

Shear friction *ϕVn = ϕ * As * Fy * μ* = 0.75 * 5.28 * 60.0 * 0.6 = 142.6 kip ACI Eq. (22.9.4.2)

Shear *Vu = √(Vux² + Vuz²)* = √(0.0² + 0.0²) = 0.0 kip < 142.6 kip OK

Straight *Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Size / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (4000)½ * 0.8 / 0.7 * 0.75 * 0.06) = 12.0 in

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (3000)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 in

Ld provided = *Dowel length* = 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover* = 14.00 - 3.0 = 11.0 in > 6.0 in OK

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2* = 3.00 * 12 / 2 - 0.0 - 8.0 / 2 = 14.0 in αsx = 10

Z-Edge = *d /2* = 10.4 / 2 = 5.2 in αsz = 20

αs = *αsx + αsz* = 10 + 20 = 30 Col type = Edge β = *W / L* = 53.0 / 8.0 = 6.63 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 20 / 10 * (8.0 + 10.4 / 2 + 14.0) + 20 / 10 * (53.0 + 10.4 / 2 + 5.2) = 117.8 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge)* = (8.0 + 10.4 / 2 + 14.0) * (53.0 + 10.4 / 2 + 5.2) = 1723.0 in²

ϕVc = *ϕ * Min (2 + 4 / β, αs * d / bo + 2, 4) * √(f'c)* ACI 22.6.5.2

ϕVc = 0.75 * Min (2 + 4 / 6.63, 30 * 10.4 / 117.8 + 2, 4) * √(3000) = 107.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 24.7 + 0.31 * 1723.0 / 144 - 10.9 = 17.6 kip

b1 = *L + d / 2 + X-Edge* = 8.0 + 10.4 / 2 + 14.0 = 27.2 in b2 = *W + d / 2 + Z-Edge* = 53.0 + 10.4 / 2 + 5.2 = 63.4 in

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b2 / b1)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(63.4 / 27.2)}} = 0.50$$ ACI Eq. (8.4.4.2.2)

$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b1 / b2)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(27.2 / 63.4)}} = 0.30$$ ACI Eq. (8.4.2.3.2)

X2z = $b1^2 / (2 * b1 + b2)$ = 27.2² / (2 * 27.2 + 63.4) = 6.3 in X2x = *b2 / 2* = 63.4 / 2 = 31.7 in

Jcz = $b1 * d^3 / 6 + b1^3 * d / 6 + 2 * b1 * d * (b1 / 2 - X2z)^2 + b2 * d * X2z^2$ ACI R8.4.4.2.3

Jcz = 27.2 * 10.4³ / 6 + 27.2³ * 10.4 / 6 + 2 * 27.2 * 10.4 * (27.2 / 2 * 6.3)² + 63.4 * 10.4 * 6.3² = 95917 in⁴

Jcx = $b2 * d^3 / 12 + b2^3 * d / 12 + 2 * b1 * d * (b2 / 2)^2$ ACI R8.4.4.2.3

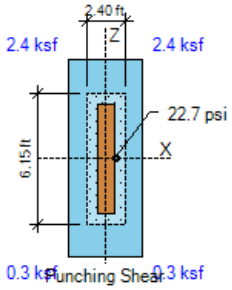
Jcx = 63.4 * 10.4³ / 12 + 2 * 27.2 * 10.4 * (63.4 / 2)² = 792420 in⁴

Stress due to P = *F / (bo * d) * 1000* = 17.6 / (117.8 * 10.4) * 1000 = 14.4 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx* = 0.50 * 34.2 * 12 * 31.7 / 792420 * 1000 = 8.3 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz* = 0.50 * 0.0 * 12 * 6.3 / 95917 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress* = 14.4 + 8.3 + 0.0 = 22.7 psi < 107.0 psi OK



COLUMN CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

Concrete f'c = 4.0 ksi Max. strain = 0.0030

Young's Modulus $Ec = 57 * (f'c * 1000)^{1/2}$ $= 57 * (4000)^{1/2}$ = 3605 ksi ACI 19.2.2.1

Rupture $fr = 7.5 * (f'c * 1000)^{1/2} / 1000 = 7.5 * (4000)^{1/2} / 1000$ = 0.47 ksi ACI Eq. (19.2.3.1)

Steel fy = 60.0 ksi Es = 29000 ksi ACI 20.2.2.2

Use 12-#6 Longitudinal Bars, As = 5.28 in², ρ = 0.012 *Use Ties #4* d' = 2.9 in

As min = $0.01 * L * W$ = 0.01 * 8.0 * 53.0 = 4.24 in² < 5.28 in² OK ACI 16.3.4.1

Axial Pu = 24.7 kip

Shear Vux = 0.0 k-ft

Shear Vuz = 0.0 k-ft

Moment Mux = 34.2 k-ft

Moment Muz = 0.0 k-ft

Moment Mu = $\sqrt{(Mux^2 + Muz^2)}$ = $\sqrt{(34.2^2 + 0.0^2)}$ = 34.2 k-ft

Rotation angle $\theta = Arc\ Tan\ (Muz / Mux)$ = Arc Tan (0.0 / 34.2) * 180 / π = 0° from X-axis

By trial and error, neutral axis depth $k = c / d$ = 0.12

Strain1 = $(k * d - d1) / (k * d) * Max\ strain$ = (0.12 * 50.1 - 2.9) / (0.12 * 50.1) * 0.0030 = 0.0019

As (in²)	d (in)	Strain	fs (ksi)	fc (ksi)	Fs = As(fs-fc) (kip)	Ms = Fs(h/2-d) (k-ft)	Is = As(h/2-d)² (in⁴)
As1 = 0.44	2.9	0.0019	54.4	3.4	22.4	44.2	245.6
As2 = 0.44	2.9	0.0019	54.4	3.4	22.4	44.2	245.6
As3 = 0.44	2.9	0.0019	54.4	3.4	22.4	44.2	245.6
As4 = 0.44	2.9	0.0019	54.4	3.4	22.4	44.2	245.6
As5 = 0.44	50.1	-0.0248	60.0	3.4	-26.4	52.0	245.6
As6 = 0.44	50.1	-0.0248	60.0	3.4	-26.4	52.0	245.6
As7 = 0.44	50.1	-0.0248	60.0	3.4	-26.4	52.0	245.6
As8 = 0.44	50.1	-0.0248	60.0	3.4	-26.4	52.0	245.6
As9 = 0.44	34.4	-0.0159	60.0	3.4	-26.4	17.3	27.3
As10 = 0.44	18.6	-0.0070	60.0	3.4	-26.4	-17.3	27.3
As11 = 0.44	34.4	-0.0159	60.0	3.4	-26.4	17.3	27.3
As12 = 0.44	18.6	-0.0070	60.0	0.0	-26.4	-17.3	27.3
					T = -121.5	Ms = 384.5	Is = 2073.8

Conc. force Fc = $C = Sum\ (Fci) = Sum\ (fci * Aci)$ = 144.2 kip

Conc. moment Mc = $Sum\ (Mci) = Sum\ (Fci * (h / 2 - di))$ = 286.5 k-ft

Nominal flexural strength Mn = $Ms + Mc$ = 384.5 + 286.5 = 671.0 k-ft

ϕ-factor slope = $(0.65 - 0.90) / (0.65 * Pn\ (balanced) - 0.9 * Pn\ (Es = 0.005)$

ϕ-factor slope = (0.65 - 0.90) / (0.65 * 783.4 - 0.9 * 454.5) = -0.0025 ACI 21.2.2

ϕ-factor = $Min\ (0.9, Max\ (0.65, slope * (Pu - 0.9 * Pn\ (Es = 0.005)) + 0.9))$

ϕ-factor = Min (0.9, Max (0.65, -0.0025 * (24.7 - 0.9 * 454.5) + 0.9)) = 0.90

Axial strength $Pn = 0.8 * 0.85 * f'c * (A1 - As) + fy * As$ = 0.8 * 0.85 * 4.0 * (424 - 5.3) + 60 * 5.3 = 1384.0 kip

Design flexural strength ϕMn = 0.90 * 671.0 = 603.9 k-ft ACI 21.2.2

Strength ratio = $Max\ (Pu / \phi Pn,\ Mu / \phi Mn)$ = Max (24.7 / (0.65 * 1384.0), 34.2 / 603.9) = 0.06 ≤ 1.0 OK

Conc. force Fc = *C = Sum (Fci) = Sum (fci * Aci)* =144.2 kip

Conc. moment Mc = *Sum (Mci) = Sum (Fci * (h / 2 - di))* = 286.5 k-ft

Nominal flexural strength Mn = *Ms + Mc =* 384.5 + 286.5 = 671.0 k-ft

ϕ-factor slope = *(0.65 - 0.90) / (0.65 * Pn (balanced) - 0.9 * Pn (Es = 0.005)*

ϕ-factor slope = (0.65 - 0.90) / (0.65 * 783.4 - 0.9 * 454.5) = -0.0025 ACI 21.2.2

ϕ-factor = *Min (0.9, Max (0.65, slope * (Pu - 0.9 * Pn (Es = 0.005)) + 0.9))*

ϕ-factor = Min (0.9, Max (0.65, -0.0025 * (24.7 - 0.9 * 454.5) + 0.9)) = 0.90

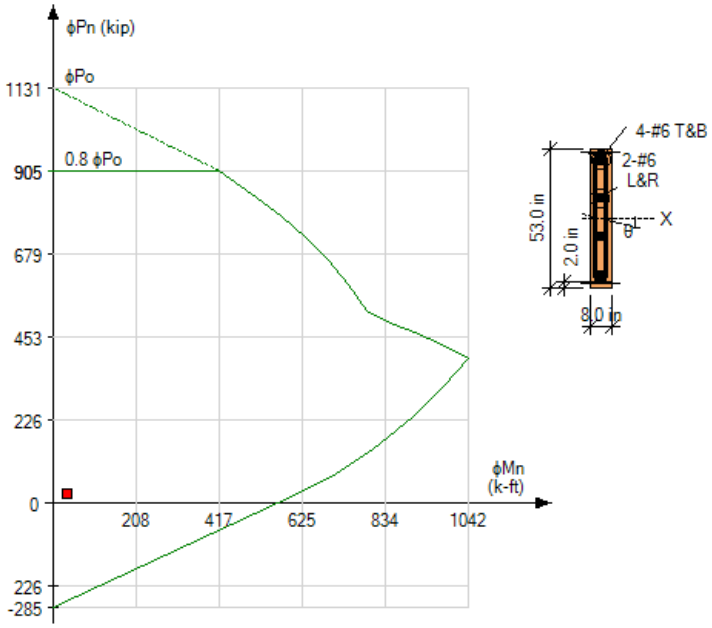
Axial strength *Pn = 0.8 * 0.85 * f'c * (A1 - As) + fy * As* = 0.8 * 0.85 * 4.0 * (424 - 5.3) + 60 * 5.3 = 1384.0 kip

Design flexural strength ϕMn = 0.90 * 671.0 = 603.9 k-ft ACI 21.2.2

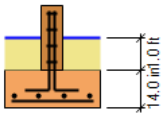
Strength ratio = *Max (Pu / ϕPn, Mu / ϕMn)* = Max (24.7 / (0.65 * 1384.0), 34.2 / 603.9) = 0.06 ≤ 1.0 OK

DESIGN CODES

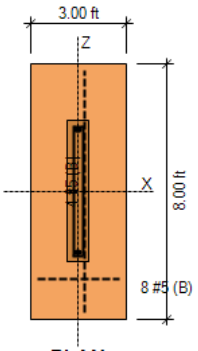
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.2D+1.6L+0.5Lr



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	6.05	ft	
Footing Width (Z-dir)	6.00	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	8.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+0.75L+0.75Lr)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	1.00		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	54.0	6.0	4.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

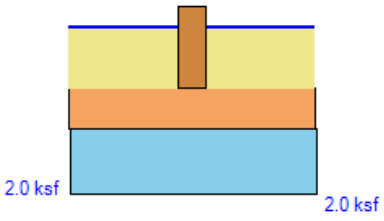
OVERTURNING

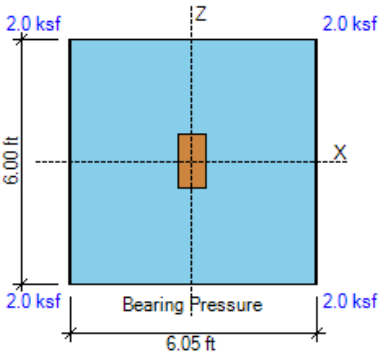
- About X-X	Force kip	Arm ft	Moment k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	3.00	3.03	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

- About X-X	Force kip	Arm ft	Moment k-ft
Footing Weight ..	3.3	3.00	9.8
Column Weight ..	0.0	3.00	0.0
Soil Cover	4.7	3.00	14.0
Axial Force P	32.4	3.00	97.2
Water Buoyancy .	-1.4	3.00	-4.1
Rv =	39.0	RM =	116.9
- About Z-Z			
Footing Weight ..	3.3	3.03	9.9
Column Weight ..	0.0	3.03	0.0
Soil Cover	4.7	3.03	14.1
Axial Force P	32.4	3.03	98.0
Water Buoyancy .	-1.4	3.00	-4.1
Rv =	39.0	RM =	117.9



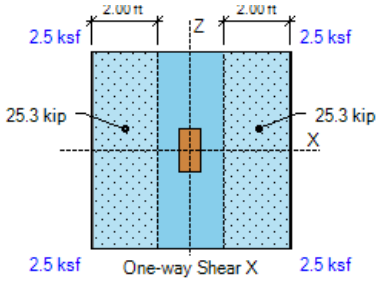


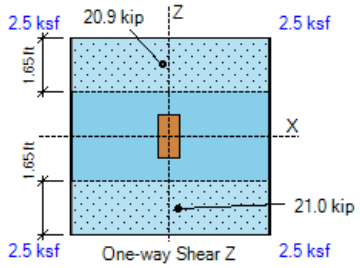
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	17.5		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	7.1	7.2	kip
Horiz. Resisting Force	24.7	24.7	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestal Selfweight	0.0	kip	
Footing Selfweight	3.3	kip	
Soil Cover Weight	4.7	kip	
Downward Force W	7.9	kip	
Uplift Safety Factor	99.99 > 1.0	OK	





PUNCHING SHEAR (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.45	
Unbalanced Moment Factor ɣvz ...	0.35	
Punching Shear due to Axial	105.7	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	105.7	psi
Effective Perimeter bo	81.0	in
Average Effective Depth d	8.3	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	2.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.64	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.2D+1.6L+0.5Lr)

	- Side	+ Side		
One-way Shear Vux ...	25.3	25.3	kip	
One-way Shear Vuz ...	21.0	20.9	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	51.0		kip	OK
One-way Shear Strength ϕVcz	47.0		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.50			OK

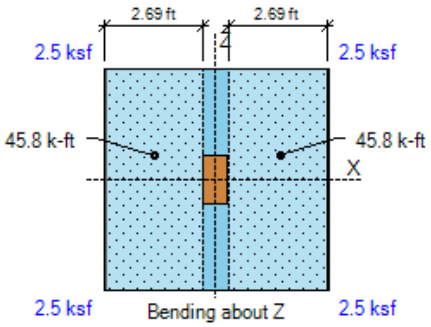
FLEXURE (Comb: 1.2D+1.6L+0.5Lr)

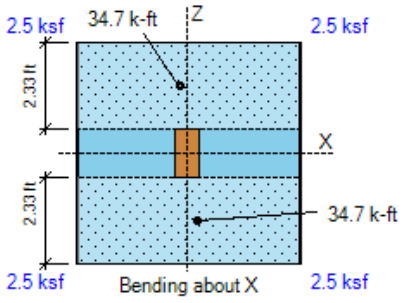
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..	N.A.			
Z-Bars Development Length Ratio ..	N.A.			
Min. Top Steel Area Ratio	N.A.			
Under-strength ϕ Factor	0.60			
Bending Strength -ϕMnx	23.9		k-ft	OK
Bending Strength -ϕMnz	23.7		k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00			OK
Max. Bar Spacing Ratio	N.A.			

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	34.7	34.7	k-ft	
Bending Moment +Muz ...	45.8	45.8	k-ft	
Use 5 #6 Bars Parallel to X (Bot) , d = 8.6 in				
Use 5 #6 Bars Parallel to Z (Bot) , d = 7.9 in				
X-Bars Development Length Ratio ..	0.40			OK
Z-Bars Development Length Ratio ..	0.47			OK
Min. Bottom Steel Area Ratio	0.71			OK
Under-strength ϕ Factor	0.90			
Bending Strength +ϕMnx	74.1		k-ft	OK
Bending Strength +ϕMnz	81.8		k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.56			OK
Max. Bar Spacing Ratio	0.93			OK

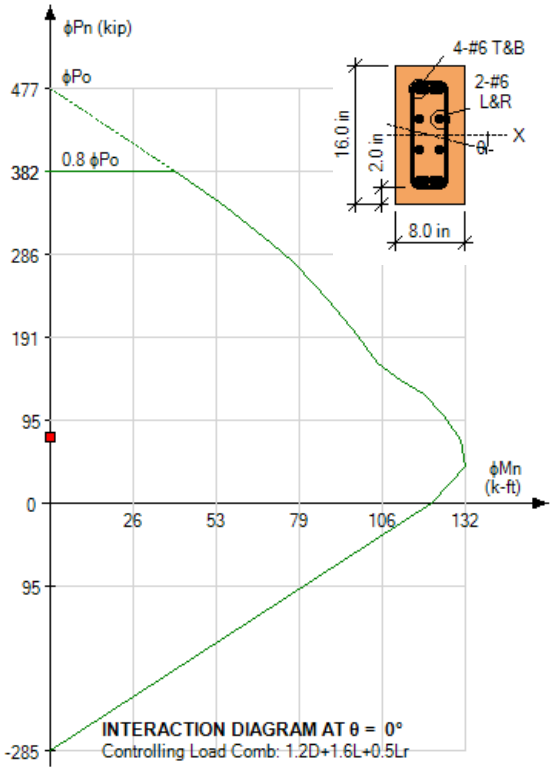
TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.6	ksi	
Column Bearing Strength ϕPn ...	3.8	ksi	OK
Footing Bearing Strength ϕPn ...	4.9	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



DESIGN CODES

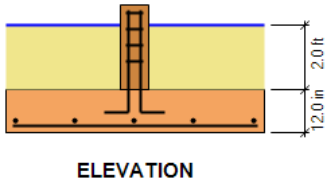
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

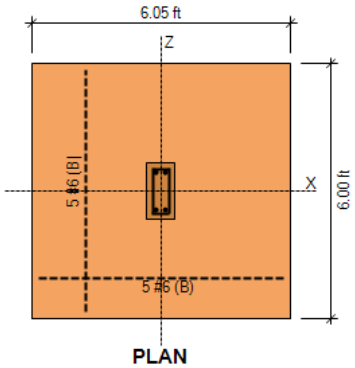
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+1.6L+0.5Lr)

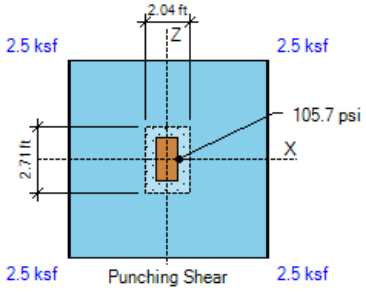
Axial Load Pu	76.4	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	381.6	kip
Flexural Strength ϕMn	131.1	k-ft
Biaxial Strength Ratio	0.20	OK



ELEVATION



PLAN



GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL GC1

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	4.05	ft	
Footing Width (Z-dir)	4.00	ft	
Footing Thickness	12.0	in	OK
Soil Cover	1.50	ft	
Column Length (X-dir)	16.0	in	
Column Width (Z-dir)	8.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+Lr)

Gross Allow. Soil Pressure ..	2.0	ksf
Soil Pressure at Corner 1	2.0	ksf
Soil Pressure at Corner 2	2.0	ksf
Soil Pressure at Corner 3	2.0	ksf
Soil Pressure at Corner 4	2.0	ksf
Bearing Pressure Ratio	1.00	OK
Ftg. Area in Contact with Soil ..	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	20.6	0.0	7.2	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

OVERTURNING

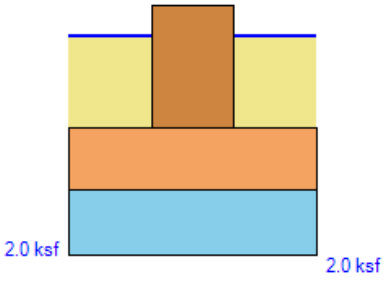
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

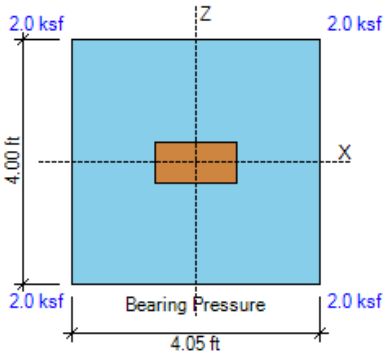
RESISTING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	1.5	2.00	2.9
Column Weight ..	0.0	2.00	0.0
Soil Cover	1.5	2.00	3.0
Axial Force P	12.4	2.00	24.7
Water Buoyancy .	-0.3	2.00	-0.6
Rv =	15.0	RM =	30.1
- About Z-Z			
Footing Weight ..	1.5	2.03	3.0
Column Weight ..	0.0	2.03	0.0
Soil Cover	1.5	2.03	3.1
Axial Force P	12.4	2.03	25.0
Water Buoyancy .	-0.3	2.00	-0.6
Rv =	15.0	RM =	30.4

	X-X	Z-Z	
Arm of Vertical Resultant =	2.00	2.03	ft
Overturning Safety Factor =	99.99	99.99	> 1.5

OK



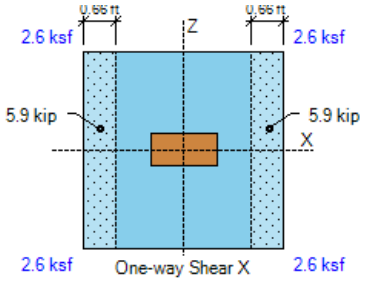


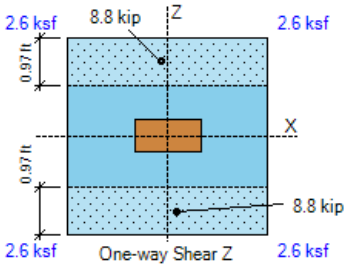
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	6.8		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	3.8	3.9	kip
Horiz. Resisting Force	10.6	10.6	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	1.5	kip
Soil Cover Weight	1.5	kip
Downward Force W	3.0	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+0.5L+1.6Lr)

Unbalanced Moment Factor ɣvx ...	0.35	
Unbalanced Moment Factor ɣvz ...	0.45	
Punching Shear due to Axial	44.0	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	44.0	psi
Effective Perimeter bo	81.5	in
Average Effective Depth d	8.4	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	2.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.27	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.2D+0.5L+1.6Lr)

	- Side	+ Side		
One-way Shear Vux ...	5.9	5.9	kip	
One-way Shear Vuz ...	8.8	8.8	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	34.3		kip	OK
One-way Shear Strength ϕVcz	32.2		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.27			OK

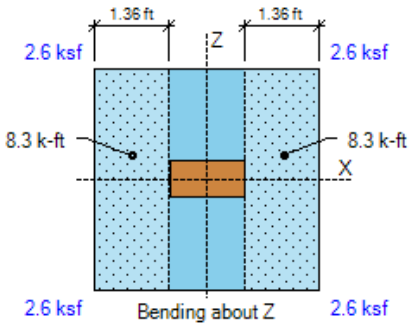
FLEXURE (Comb: 1.2D+0.5L+1.6Lr)

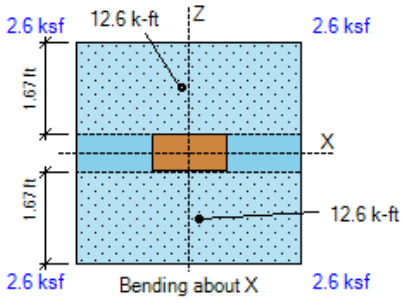
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..	N.A.			
Z-Bars Development Length Ratio ..	N.A.			
Min. Top Steel Area Ratio	N.A.			
Under-strength ϕ Factor	0.60			
Bending Strength -ϕMnx	16.0		k-ft	OK
Bending Strength -ϕMnz	15.8		k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00			OK
Max. Bar Spacing Ratio	N.A.			

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	12.6	12.6	k-ft	
Bending Moment +Muz ...	8.3	8.3	k-ft	
Use 4 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..	0.87			OK
Z-Bars Development Length Ratio ..	0.34			OK
Min. Bottom Steel Area Ratio	0.85			OK
Under-strength ϕ Factor	0.90			
Bending Strength +ϕMnx	43.1		k-ft	OK
Bending Strength +ϕMnz	46.8		k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.29			OK
Max. Bar Spacing Ratio	0.79			OK

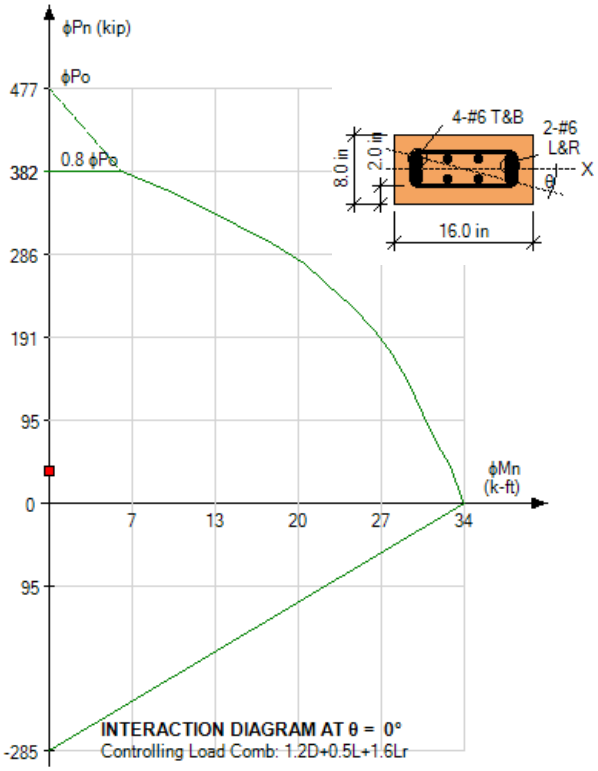
TRANSFER (Comb: 1.2D+0.5L+1.6Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.3	ksi	
Column Bearing Strength ϕPn ...	3.8	ksi	OK
Footing Bearing Strength ϕPn ...	4.9	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



DESIGN CODES

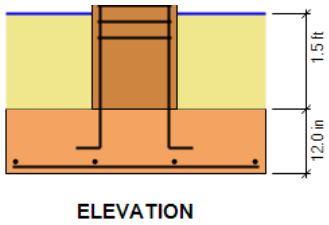
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

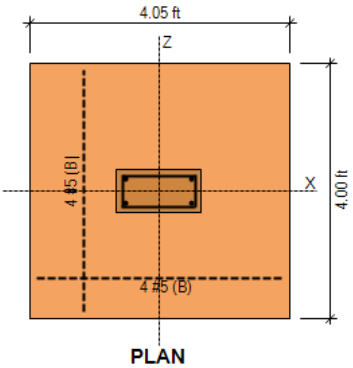
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+0.5L+1.6Lr)

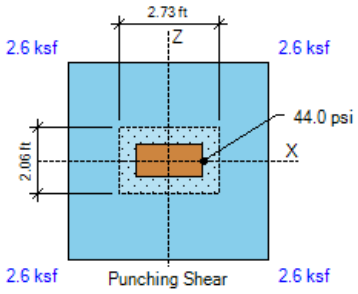
Axial Load Pu	36.2	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	381.7	kip
Flexural Strength ϕMn	32.8	k-ft
Biaxial Strength Ratio	0.09	OK



ELEVATION



PLAN



GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL K

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	**9.00**	ft	
Footing Width (Z-dir)	**5.00**	ft	
Footing Thickness	**12.0**	in	OK
Soil Cover	**2.00**	ft	
Column Length (X-dir)	**46.0**	in	
Column Width (Z-dir)	**12.0**	in	
Offset (X-dir)	**0.00**	in	OK
Offset (Z-dir)	**0.00**	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+0.75L+0.75Lr)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.95		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	**61.0**	**10.0**	**5.0**	**0.0**	**0.0**	**0.0**	kip
Moment about X Mx ..	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	k-ft
Moment about Z Mz ..	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	k-ft
Shear Force Vx	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	kip
Shear Force Vz	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

OVERTURNING

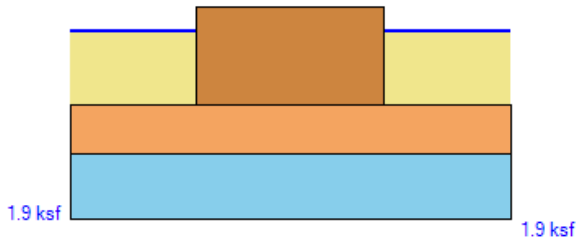
- About X-X	Force kip	Arm ft	Moment k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

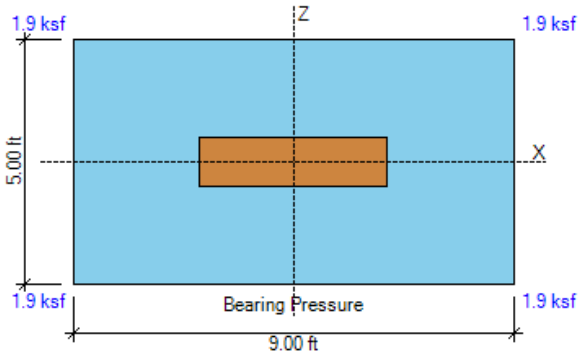
	X-X	Z-Z	
Arm of Vertical Resultant =	2.50	4.50	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

- About X-X	Force kip	Arm ft	Moment k-ft
Footing Weight ..	4.1	2.50	10.1
Column Weight ..	0.0	2.50	0.0
Soil Cover	5.4	2.50	13.6
Axial Force P	36.6	2.50	91.5
Water Buoyancy .	-1.7	2.50	-4.2
Rv =	44.4	RM =	111.0
- About Z-Z			
Footing Weight ..	4.1	4.50	18.2
Column Weight ..	0.0	4.50	0.0
Soil Cover	5.4	4.50	24.5
Axial Force P	36.6	4.50	164.7
Water Buoyancy .	-1.7	2.50	-4.2
Rv =	44.4	RM =	199.8

SPREAD FOOTING DESIGN



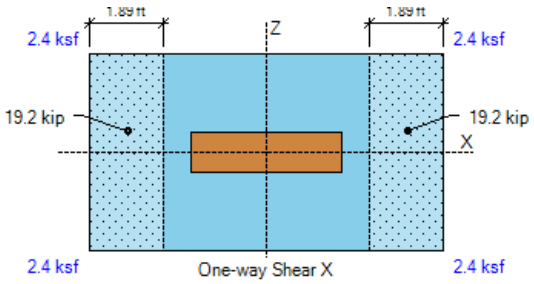


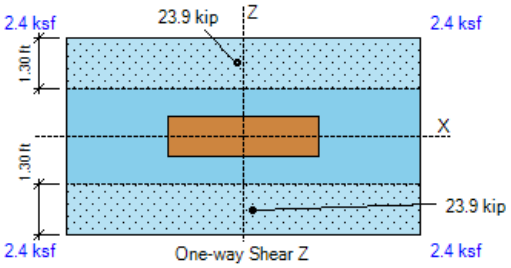
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	20.0		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	5.9	10.7	kip
Horiz. Resisting Force	25.9	30.7	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestal Selfweight	0.0	kip	
Footing Selfweight	4.1	kip	
Soil Cover Weight	5.4	kip	
Downward Force W	9.5	kip	
Uplift Safety Factor	99.99 > 1.0	OK	





PUNCHING SHEAR (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.29	
Unbalanced Moment Factor ɣvz ...	0.52	
Punching Shear due to Axial	60.7	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	60.7	psi
Effective Perimeter bo	149.5	in
Average Effective Depth d	8.4	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	3.83	
Punching Shear Strength ϕVc ..	125.0	psi
Punching Shear Ratio Vu/ϕVc	0.49	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.2D+1.6L+0.5Lr)

	- Side	+ Side		
One-way Shear Vux ...	19.2	19.2	kip	
One-way Shear Vuz ...	23.9	23.9	kip	
Under-strength ϕ Factor		0.75		
One-way Shear Strength ϕVcx		42.8	kip	OK
One-way Shear Strength ϕVcz		71.5	kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..		0.45		OK

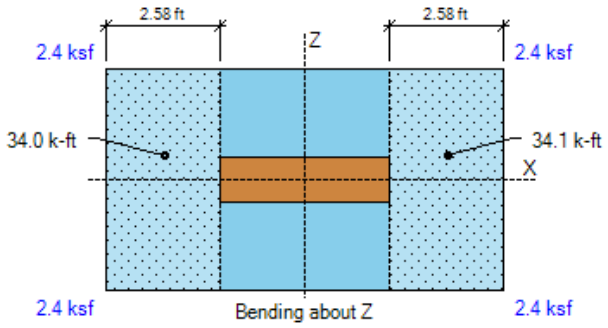
FLEXURE (Comb: 1.2D+1.6L+0.5Lr)

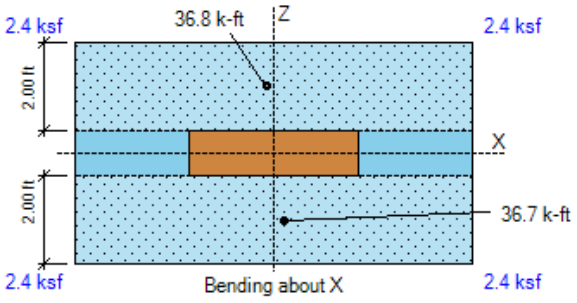
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..		N.A.		
Z-Bars Development Length Ratio ..		N.A.		
Min. Top Steel Area Ratio		N.A.		
Under-strength ϕ Factor		0.60		
Bending Strength -ϕMnx		35.5	k-ft	OK
Bending Strength -ϕMnz		19.7	k-ft	OK
Max. Bending Ratio -Mu/ϕMn		0.00		OK
Max. Bar Spacing Ratio		N.A.		

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	36.7	36.8	k-ft	
Bending Moment +Muz ...	34.0	34.1	k-ft	
Use 6 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 9 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..		0.42		OK
Z-Bars Development Length Ratio ..		0.28		OK
Min. Bottom Steel Area Ratio		0.84		OK
Under-strength ϕ Factor		0.90		
Bending Strength +ϕMnx		76.4	k-ft	OK
Bending Strength +ϕMnz		69.7	k-ft	OK
Max. Bending Ratio +Mu/ϕMn		0.49		OK
Max. Bar Spacing Ratio		0.71		OK

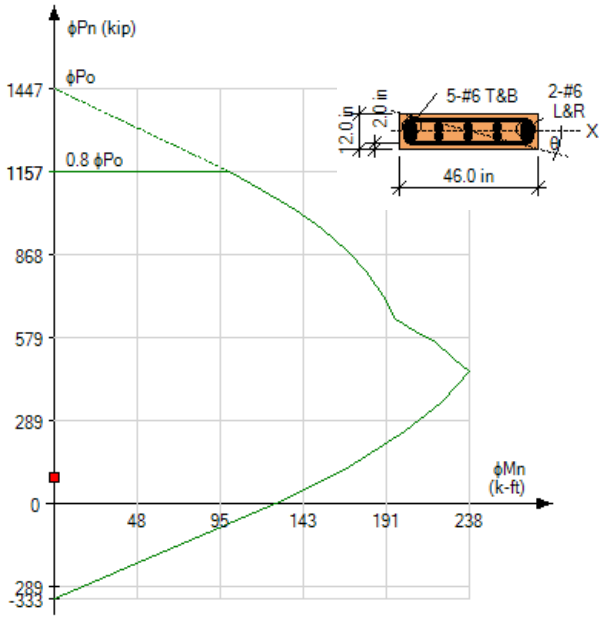
TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 14 #6 x 1.83 ft hooked dowels , As = 6.16 in²			
Bearing Stress Pbu	0.2	ksi	
Column Bearing Strength ϕPn ...	2.6	ksi	OK
Footing Bearing Strength ϕPn ...	3.8	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	166.3	kip	OK
Min. Steel Area As = 0.005 A1 ...	5.52	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.2D+1.6L+0.5Lr

DESIGN CODES

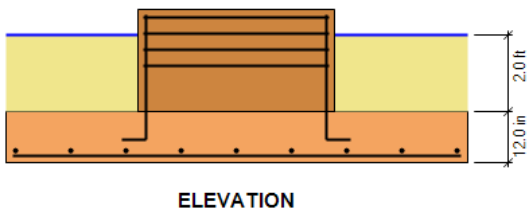
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

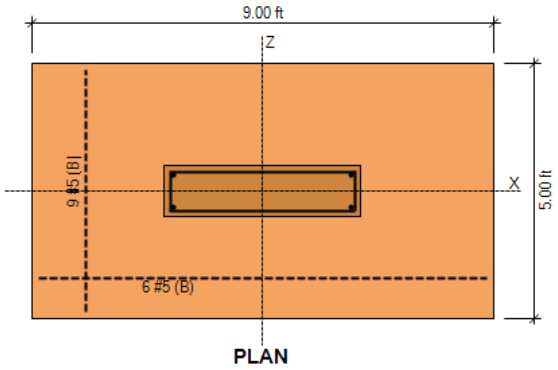
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+1.6L+0.5Lr)

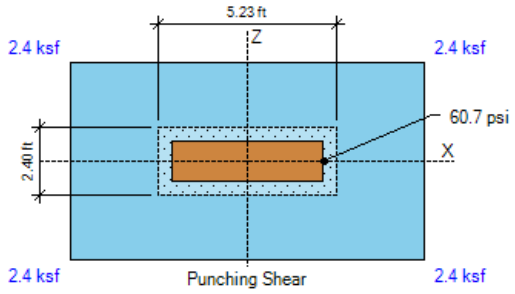
Axial Load Pu	91.7	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	1156.1	kip
Flexural Strength ϕMn	154.6	k-ft
Biaxial Strength Ratio	0.08	OK



ELEVATION



PLAN



Punching Shear

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	6.00	ft	
Footing Width (Z-dir)	6.00	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	36.0	in	
Column Width (Z-dir)	8.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+0.75L+0.75Lr)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.97		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	48.0	10.0	5.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

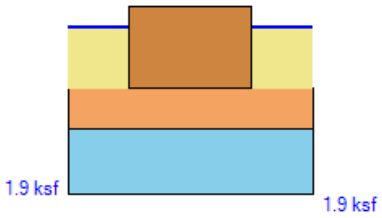
OVERTURNING

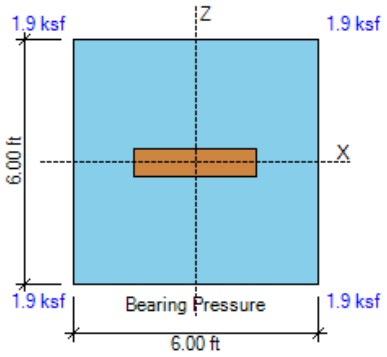
- About X-X	Force kip	Arm ft	Moment k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	3.00	3.00	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

- About X-X	Force kip	Arm ft	Moment k-ft
Footing Weight ..	3.2	3.00	9.7
Column Weight ..	0.0	3.00	0.0
Soil Cover	4.5	3.00	13.5
Axial Force P	28.8	3.00	86.4
Water Buoyancy .	-1.3	3.00	-4.0
Rv =	35.2	RM =	105.5
- About Z-Z			
Footing Weight ..	3.2	3.00	9.7
Column Weight ..	0.0	3.00	0.0
Soil Cover	4.5	3.00	13.5
Axial Force P	28.8	3.00	86.4
Water Buoyancy .	-1.3	3.00	-4.0
Rv =	35.2	RM =	105.5



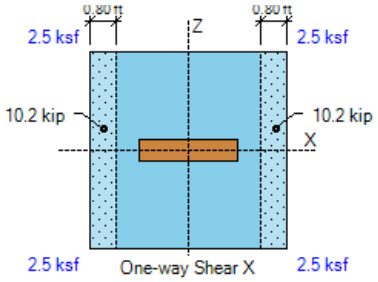


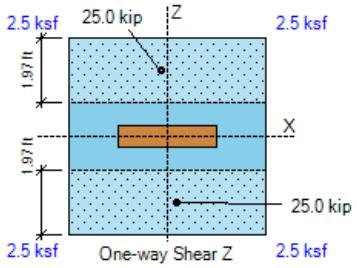
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	15.8		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	7.1	7.1	kip
Horiz. Resisting Force	23.0	23.0	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	3.2	kip
Soil Cover Weight	4.5	kip
Downward Force W	7.7	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.29	
Unbalanced Moment Factor ɣvz ...	0.52	
Punching Shear due to Axial	64.3	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	64.3	psi
Effective Perimeter bo	121.5	in
Average Effective Depth d	8.4	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	4.50	
Punching Shear Strength ϕVc ..	118.7	psi
Punching Shear Ratio Vu/ϕVc	0.54	OK

ONE-WAY SHEAR (Comb: 1.2D+1.6L+0.5Lr)

	- Side	+ Side		
One-way Shear Vux ...	10.2	10.2	kip	
One-way Shear Vuz ...	25.0	25.0	kip	
Under-strength ϕ Factor		0.75		
One-way Shear Strength ϕVcx		51.4	kip	OK
One-way Shear Strength ϕVcz		47.7	kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..		0.52		OK

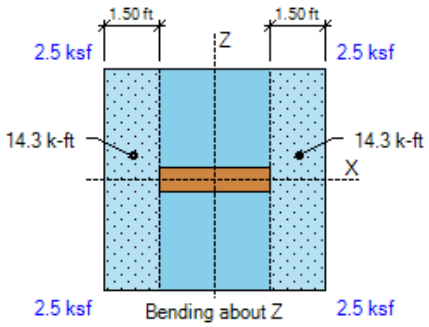
FLEXURE (Comb: 1.2D+1.6L+0.5Lr)

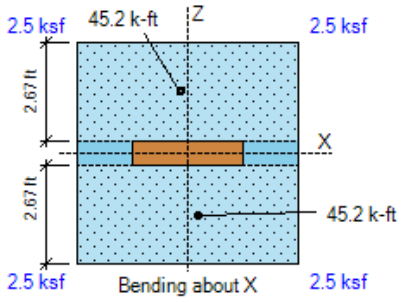
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..		N.A.		
Z-Bars Development Length Ratio ..		N.A.		
Min. Top Steel Area Ratio		N.A.		
Under-strength ϕ Factor		0.60		
Bending Strength -ϕMnx		23.7	k-ft	OK
Bending Strength -ϕMnz		23.7	k-ft	OK
Max. Bending Ratio -Mu/ϕMn		0.00		OK
Max. Bar Spacing Ratio		N.A.		

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	45.2	45.2	k-ft	
Bending Moment +Muz ...	14.3	14.3	k-ft	
Use 6 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 6 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..		0.77		OK
Z-Bars Development Length Ratio ..		0.20		OK
Min. Bottom Steel Area Ratio		0.84		OK
Under-strength ϕ Factor		0.90		
Bending Strength +ϕMnx		64.9	k-ft	OK
Bending Strength +ϕMnz		70.2	k-ft	OK
Max. Bending Ratio +Mu/ϕMn		0.70		OK
Max. Bar Spacing Ratio		0.73		OK

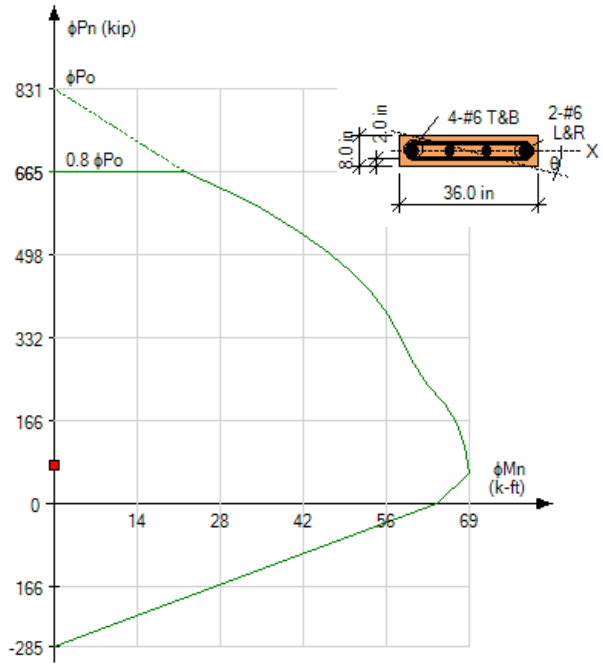
TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.3	ksi	
Column Bearing Strength ϕPn ...	2.9	ksi	OK
Footing Bearing Strength ϕPn ...	4.0	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	2.88	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.2D+1.6L+0.5Lr

DESIGN CODES

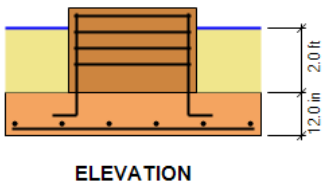
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

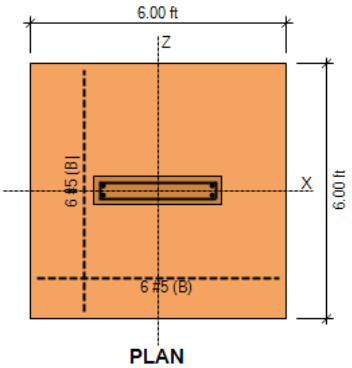
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+1.6L+0.5Lr)

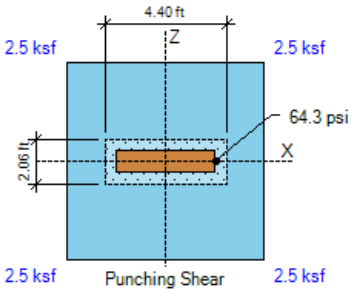
Axial Load Pu	76.1	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	664.6	kip
Flexural Strength ϕMn	70.3	k-ft
Biaxial Strength Ratio	0.11	OK



ELEVATION



PLAN



GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL T

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	3.50	ft	
Footing Width (Z-dir)	3.50	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	8.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+Lr)

Gross Allow. Soil Pressure ..	2.0	ksf
Soil Pressure at Corner 1	1.7	ksf
Soil Pressure at Corner 2	1.7	ksf
Soil Pressure at Corner 3	1.7	ksf
Soil Pressure at Corner 4	1.7	ksf
Bearing Pressure Ratio	0.84	OK
Ftg. Area in Contact with Soil ..	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	15.0	0.0	2.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

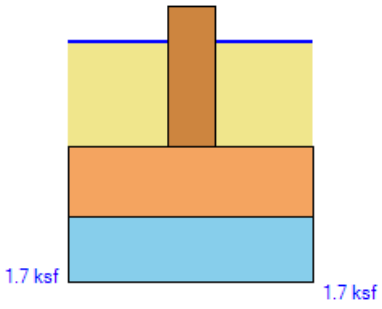
OVERTURNING

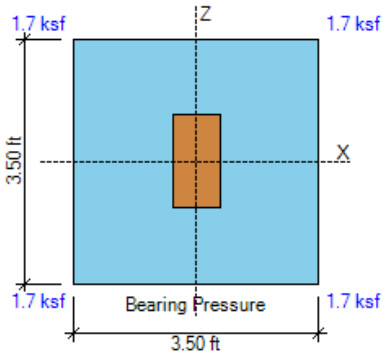
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	1.75	1.75	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	1.1	1.75	1.9
Column Weight ..	0.0	1.75	0.0
Soil Cover	1.5	1.75	2.6
Axial Force P	9.0	1.75	15.8
Water Buoyancy .	-0.5	1.75	-0.8
Rv =	11.1	RM =	19.5
- About Z-Z			
Footing Weight ..	1.1	1.75	1.9
Column Weight ..	0.0	1.75	0.0
Soil Cover	1.5	1.75	2.6
Axial Force P	9.0	1.75	15.8
Water Buoyancy .	-0.5	1.75	-0.8
Rv =	11.1	RM =	19.5



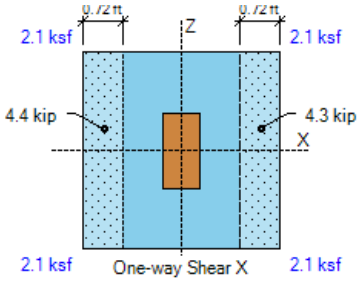


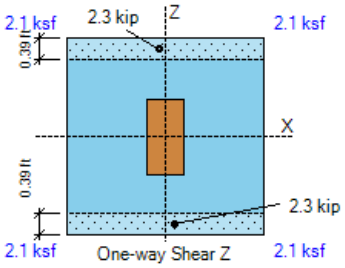
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	5.0		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	4.2	4.2	kip
Horiz. Resisting Force	9.2	9.2	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	91.78	91.78 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	1.1	kip
Soil Cover Weight	1.5	kip
Downward Force W	2.6	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+0.5L+1.6Lr)

Unbalanced Moment Factor ɣvx ...	0.49	
Unbalanced Moment Factor ɣvz ...	0.32	
Punching Shear due to Axial	24.2	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	24.2	psi
Effective Perimeter bo	82.8	in
Average Effective Depth d	8.4	in
Column Location Factor αs	30 (Edge)	
Column Aspect Ratio β	2.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.15	OK

ONE-WAY SHEAR (Comb: 1.2D+0.5L+1.6Lr)

	- Side	+ Side		
One-way Shear Vux ...	4.4	4.3	kip	
One-way Shear Vuz ...	2.3	2.3	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	30.0		kip	OK
One-way Shear Strength ϕVcz	27.8		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.15			OK

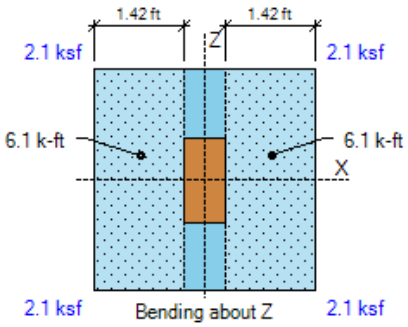
FLEXURE (Comb: 1.2D+0.5L+1.6Lr)

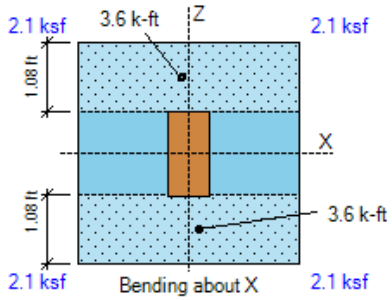
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..	N.A.			
Z-Bars Development Length Ratio ..	N.A.			
Min. Top Steel Area Ratio	N.A.			
Under-strength ϕ Factor	0.60			
Bending Strength -ϕMnx	13.8		k-ft	OK
Bending Strength -ϕMnz	13.8		k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00			OK
Max. Bar Spacing Ratio	N.A.			

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	3.6	3.6	k-ft	
Bending Moment +Muz ...	6.1	6.1	k-ft	
Use 4 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..	0.41			OK
Z-Bars Development Length Ratio ..	0.57			OK
Min. Bottom Steel Area Ratio	0.73			OK
Under-strength ϕ Factor	0.90			
Bending Strength +ϕMnx	43.0		k-ft	OK
Bending Strength +ϕMnz	46.5		k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.13			OK
Max. Bar Spacing Ratio	0.67			OK

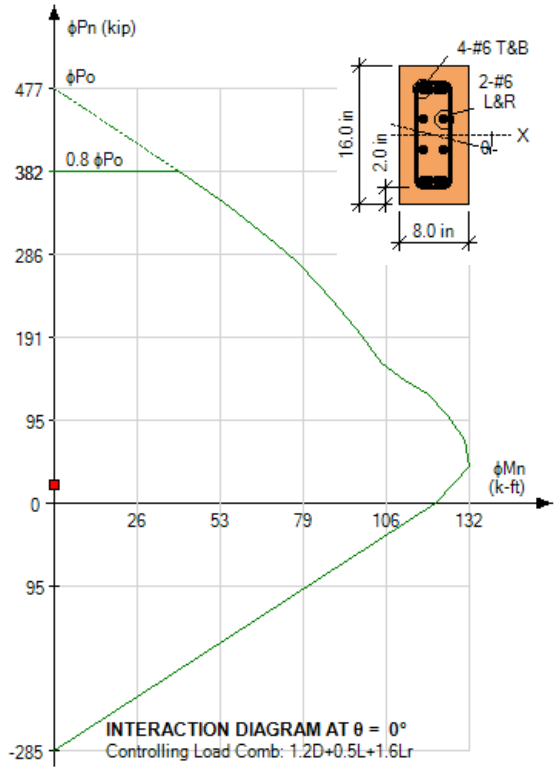
TRANSFER (Comb: 1.2D+0.5L+1.6Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.2	ksi	
Column Bearing Strength ϕPn ...	3.8	ksi	OK
Footing Bearing Strength ϕPn ...	4.9	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



DESIGN CODES

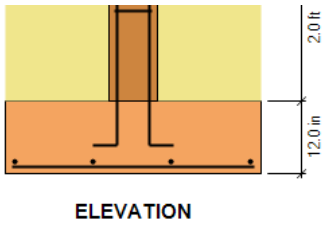
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

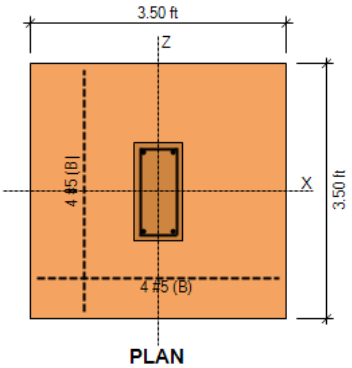
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+0.5L+1.6Lr)

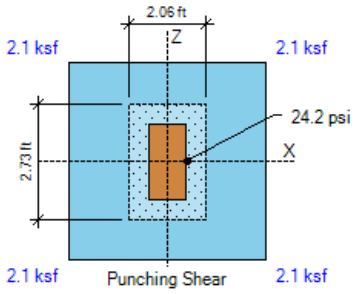
Axial Load Pu	21.2	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	381.6	kip
Flexural Strength ϕMn	127.1	k-ft
Biaxial Strength Ratio	0.06	OK



ELEVATION



PLAN



GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL W

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	3.75	ft	
Footing Width (Z-dir)	3.75	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	8.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+Lr)

Gross Allow. Soil Pressure ..	2.0	ksf
Soil Pressure at Corner 1	2.0	ksf
Soil Pressure at Corner 2	2.0	ksf
Soil Pressure at Corner 3	2.0	ksf
Soil Pressure at Corner 4	2.0	ksf
Bearing Pressure Ratio	0.98	OK
Ftg. Area in Contact with Soil ..	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	20.5	0.0	3.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

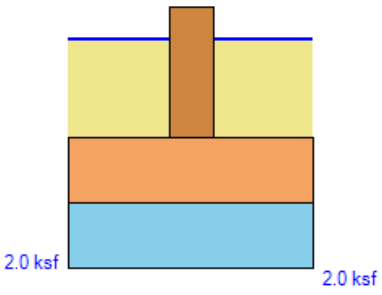
OVERTURNING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

RESISTING

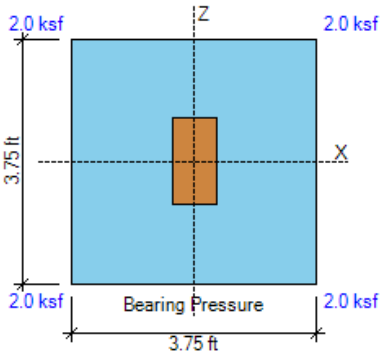
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	1.3	1.88	2.4
Column Weight ..	0.0	1.88	0.0
Soil Cover	1.7	1.88	3.3
Axial Force P	12.3	1.88	23.1
Water Buoyancy .	-0.5	1.88	-1.0
Rv =	14.8	RM =	27.7
- About Z-Z			
Footing Weight ..	1.3	1.88	2.4
Column Weight ..	0.0	1.88	0.0
Soil Cover	1.7	1.88	3.3
Axial Force P	12.3	1.88	23.1
Water Buoyancy .	-0.5	1.88	-1.0
Rv =	14.8	RM =	27.7

	X-X	Z-Z	
Arm of Vertical Resultant =	1.88	1.88	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK



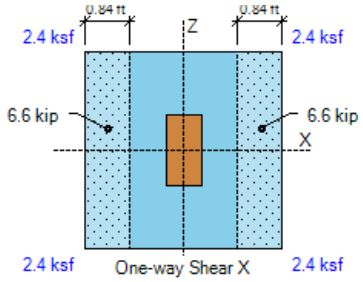
SLIDING (combDlg. 0.6D+0.6W)

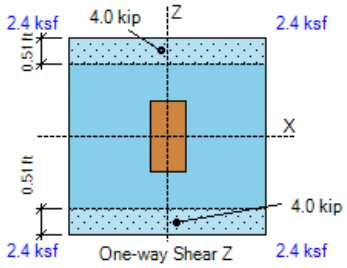
Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	6.7	kip	
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	4.5	4.5	kip
Horiz. Resisting Force	11.1	11.1	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK



UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	1.3	kip
Soil Cover Weight	1.7	kip
Downward Force W	3.0	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+0.5L+1.6Lr)

Unbalanced Moment Factor ɣvx ...	0.49	
Unbalanced Moment Factor ɣvz ...	0.31	
Punching Shear due to Axial	33.4	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	33.4	psi
Effective Perimeter bo	85.8	in
Average Effective Depth d	8.4	in
Column Location Factor αs	30 (Edge)	
Column Aspect Ratio β	2.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.20	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.2D+0.5L+1.6Lr)

	- Side	+ Side		
One-way Shear Vux ...	6.6	6.6	kip	
One-way Shear Vuz ...	4.0	4.0	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	32.1		kip	OK
One-way Shear Strength ϕVcz	29.8		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.21			OK

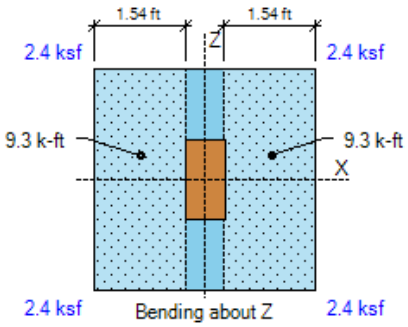
FLEXURE (Comb: 1.2D+0.5L+1.6Lr)

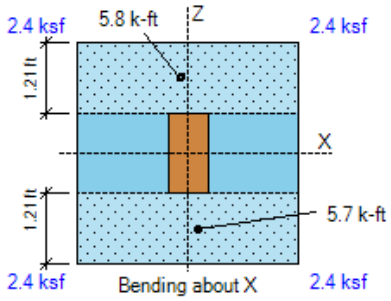
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..	N.A.			
Z-Bars Development Length Ratio ..	N.A.			
Min. Top Steel Area Ratio	N.A.			
Under-strength ϕ Factor	0.60			
Bending Strength -ϕMnx	14.8		k-ft	OK
Bending Strength -ϕMnz	14.8		k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00			OK
Max. Bar Spacing Ratio	N.A.			

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	5.7	5.8	k-ft	
Bending Moment +Muz ...	9.3	9.3	k-ft	
Use 4 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..	0.38			OK
Z-Bars Development Length Ratio ..	0.50			OK
Min. Bottom Steel Area Ratio	0.78			OK
Under-strength ϕ Factor	0.90			
Bending Strength +ϕMnx	43.2		k-ft	OK
Bending Strength +ϕMnz	46.7		k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.20			OK
Max. Bar Spacing Ratio	0.72			OK

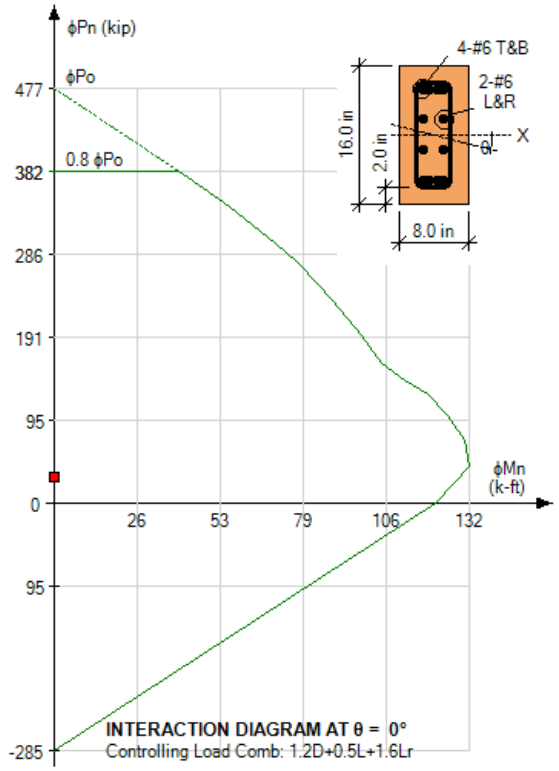
TRANSFER (Comb: 1.2D+0.5L+1.6Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.2	ksi	
Column Bearing Strength ϕPn ...	3.8	ksi	OK
Footing Bearing Strength ϕPn ...	4.9	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



DESIGN CODES

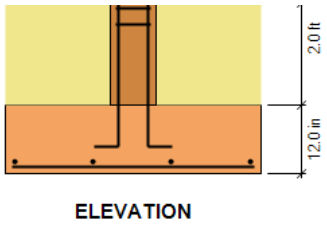
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

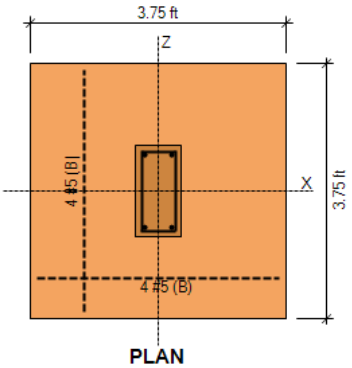
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+0.5L+1.6Lr)

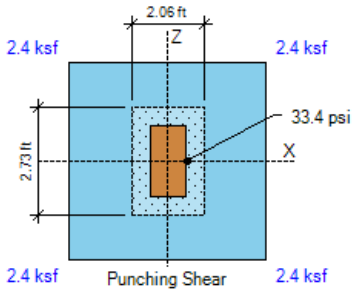
Axial Load Pu	29.4	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	381.6	kip
Flexural Strength ϕMn	128.3	k-ft
Biaxial Strength Ratio	0.08	OK



ELEVATION



PLAN



GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL Y

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	6.00	ft	
Footing Width (Z-dir)	2.50	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	36.0	in	
Column Width (Z-dir)	8.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+Lr)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.94		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	22.0	0.0	2.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

OVERTURNING

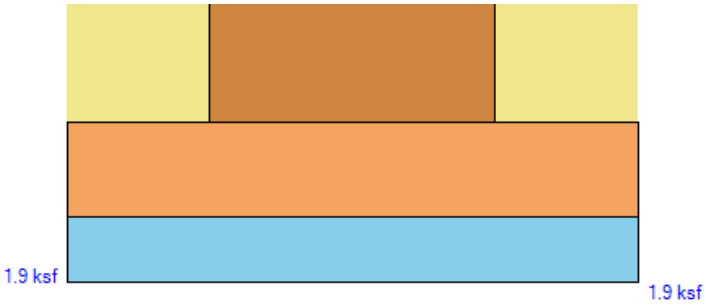
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

RESISTING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	1.4	1.25	1.7
Column Weight ..	0.0	1.25	0.0
Soil Cover	1.7	1.25	2.1
Axial Force P	13.2	1.25	16.5
Water Buoyancy .	-0.6	1.25	-0.7
Rv =	15.7	RM =	19.6
- About Z-Z			
Footing Weight ..	1.4	3.00	4.1
Column Weight ..	0.0	3.00	0.0
Soil Cover	1.7	3.00	5.1
Axial Force P	13.2	3.00	39.6
Water Buoyancy .	-0.6	1.25	-0.7
Rv =	15.7	RM =	47.1

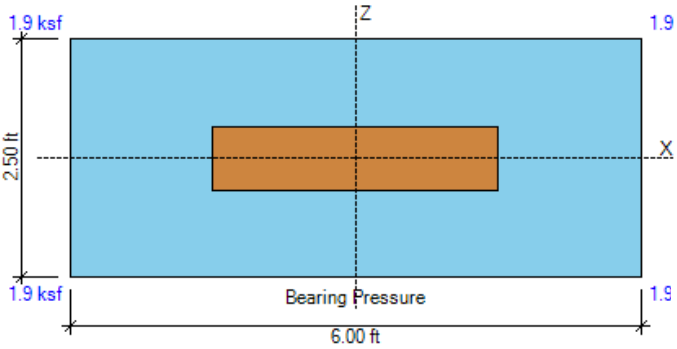
	X-X	Z-Z	
Arm of Vertical Resultant =	1.25	3.00	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

SPREAD FOOTING DESIGN



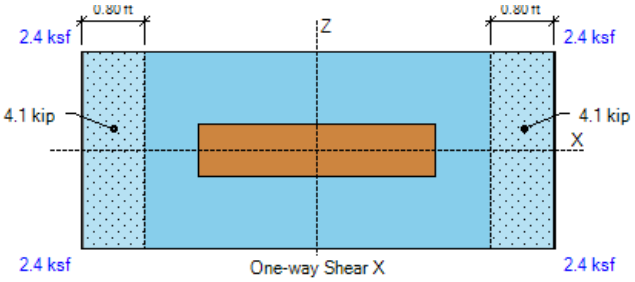
SLIDING (combDlg. 0.6D+0.6W)

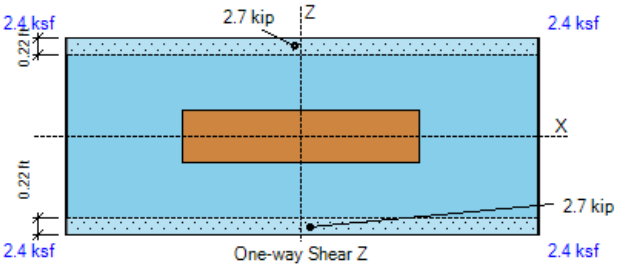
Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	7.1	kip	
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	3.0	7.1	kip
Horiz. Resisting Force	10.0	14.2	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK



UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	1.4	kip
Soil Cover Weight	1.7	kip
Downward Force W	3.1	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.4D)

Unbalanced Moment Factor ɣvx ...	0.33	
Unbalanced Moment Factor ɣvz ...	0.48	
Punching Shear due to Axial	28.0	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	28.0	psi
Effective Perimeter bo	90.8	in
Average Effective Depth d	8.4	in
Column Location Factor αs	30 (Edge)	
Column Aspect Ratio β	4.50	
Punching Shear Strength ϕVc ..	118.7	psi
Punching Shear Ratio Vu/ϕVc	0.24	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.4D)

	- Side	+ Side		
One-way Shear Vux ...	4.1	4.1	kip	
One-way Shear Vuz ...	2.7	2.7	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	21.4		kip	OK
One-way Shear Strength ϕVcz	47.7		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.19			OK

FLEXURE (Comb: 1.4D)

- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

X-Bars Development Length Ratio ..	N.A.		
Z-Bars Development Length Ratio ..	N.A.		
Min. Top Steel Area Ratio	N.A.		
Under-strength ϕ Factor	0.60		
Bending Strength -ϕMnx	23.7	k-ft	OK
Bending Strength -ϕMnz	9.9	k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00		OK
Max. Bar Spacing Ratio	N.A.		

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	5.2	5.2	k-ft	
Bending Moment +Muz ...	5.8	5.8	k-ft	

Use 5 #5 Bars Parallel to X (Bot) , d = 8.7 in

Use 6 #5 Bars Parallel to Z (Bot) , d = 8.1 in

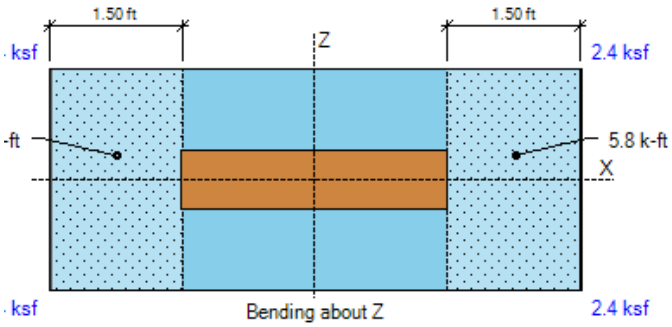
X-Bars Development Length Ratio ..	0.77		OK
Z-Bars Development Length Ratio ..	0.71		OK
Min. Bottom Steel Area Ratio	0.84		OK
Under-strength ϕ Factor	0.90		
Bending Strength +ϕMnx	46.5	k-ft	OK
Bending Strength +ϕMnz	56.3	k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.11		OK
Max. Bar Spacing Ratio	0.73		OK

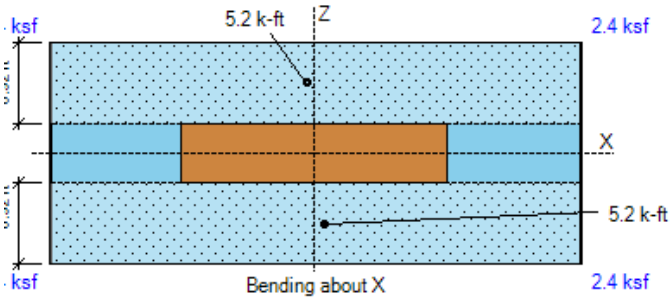
TRANSFER (Comb: 1.4D)

Bearing Under-strength ϕ Factor	0.65		

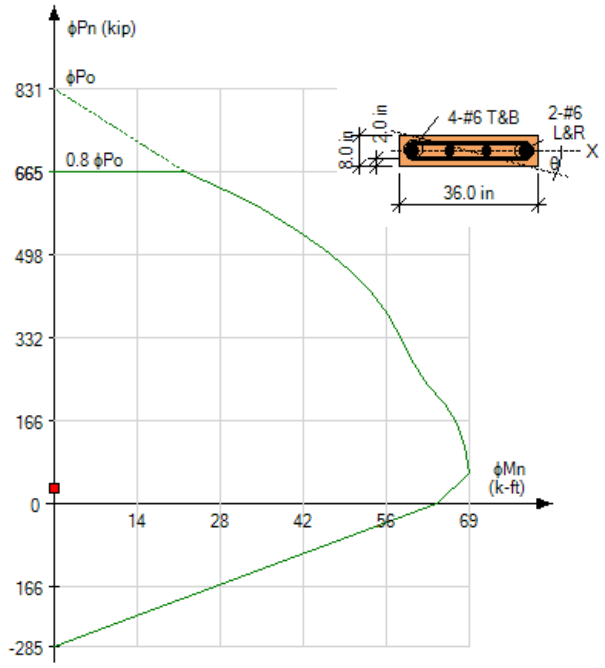
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²

Bearing Stress Pbu	0.1	ksi	
Column Bearing Strength ϕPn ...	2.9	ksi	OK
Footing Bearing Strength ϕPn ...	4.0	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	2.88	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.4D

DESIGN CODES

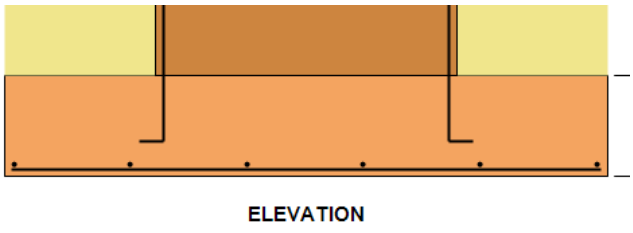
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

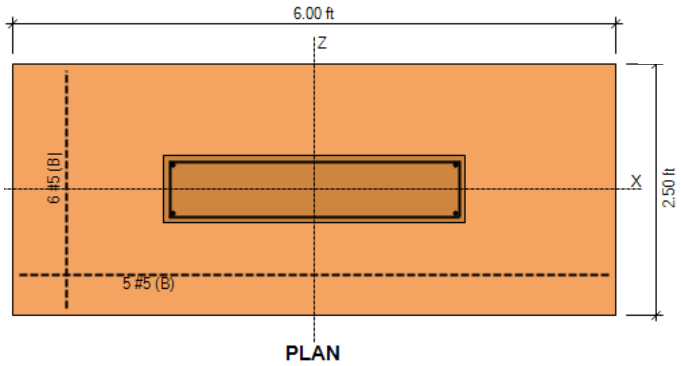
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.4D)

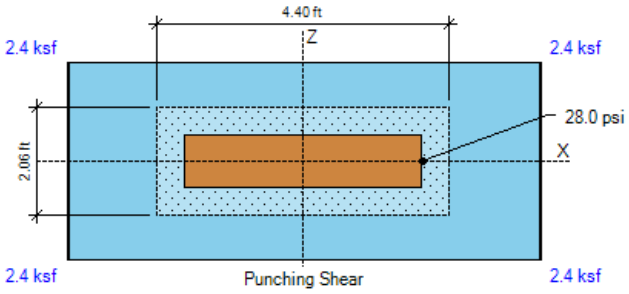
Axial Load Pu	30.8	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	664.6	kip
Flexural Strength ϕMn	67.3	k-ft
Biaxial Strength Ratio	0.05	OK



ELEVATION



PLAN



Punching Shear

GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL O

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	5.50	ft	
Footing Width (Z-dir)	5.50	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	36.0	in	
Column Width (Z-dir)	8.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+0.75L+0.75Lr)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	0.98		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	45.0	5.0	2.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

OVERTURNING

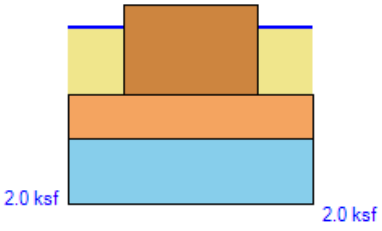
- About X-X	Force kip	Arm ft	Moment k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

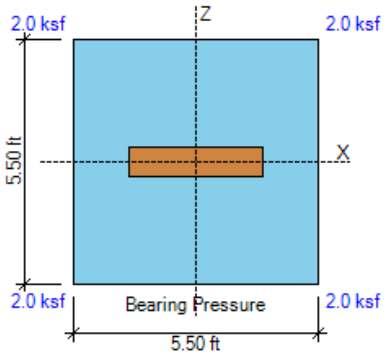
	X-X	Z-Z	
Arm of Vertical Resultant =	2.75	2.75	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

- About X-X	Force kip	Arm ft	Moment k-ft
Footing Weight ..	2.7	2.75	7.5
Column Weight ..	0.0	2.75	0.0
Soil Cover	3.7	2.75	10.3
Axial Force P	27.0	2.75	74.3
Water Buoyancy .	-1.1	2.75	-3.1
Rv =	32.3	RM =	88.9
- About Z-Z			
Footing Weight ..	2.7	2.75	7.5
Column Weight ..	0.0	2.75	0.0
Soil Cover	3.7	2.75	10.3
Axial Force P	27.0	2.75	74.3
Water Buoyancy .	-1.1	2.75	-3.1
Rv =	32.3	RM =	88.9

SPREAD FOOTING DESIGN



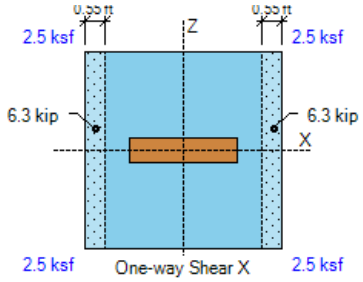


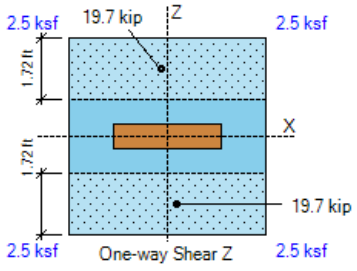
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.		0.45	
Friction Force at Base		14.5	kip
Passive Pressure Coeff. Kp		4.33	
	X-X	Z-Z	
Passive Force @ Ftg ...	6.5	6.5	kip
Horiz. Resisting Force	21.1	21.1	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.0	kip
Footing Selfweight	2.7	kip
Soil Cover Weight	3.7	kip
Downward Force W	6.5	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.4D)

Unbalanced Moment Factor γvx ...	0.27	
Unbalanced Moment Factor γvz ...	0.55	
Punching Shear due to Axial	49.9	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	49.9	psi
Effective Perimeter bo	126.8	in
Average Effective Depth d	8.4	in
Column Location Factor αs	30 (Edge)	
Column Aspect Ratio β	4.50	
Punching Shear Strength ϕVc ..	118.7	psi
Punching Shear Ratio Vu/ϕVc	0.42	OK

ONE-WAY SHEAR (Comb: 1.4D)

	- Side	+ Side		
One-way Shear Vux ...	6.3	6.3	kip	
One-way Shear Vuz ...	19.7	19.7	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	47.1		kip	OK
One-way Shear Strength ϕVcz	43.7		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.45			OK

FLEXURE (Comb: 1.4D)

- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

X-Bars Development Length Ratio ..	N.A.		
Z-Bars Development Length Ratio ..	N.A.		
Min. Top Steel Area Ratio	N.A.		
Under-strength ϕ Factor	0.60		
Bending Strength -ϕMnx	21.7	k-ft	OK
Bending Strength -ϕMnz	21.7	k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00		OK
Max. Bar Spacing Ratio	N.A.		

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	33.5	33.5	k-ft	
Bending Moment +Muz ...	9.0	9.0	k-ft	

Use 5 #5 Bars Parallel to X (Bot) , d = 8.7 in

Use 6 #5 Bars Parallel to Z (Bot) , d = 8.1 in

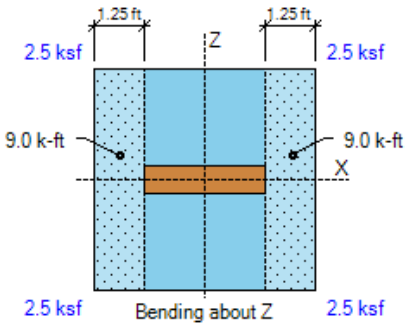
X-Bars Development Length Ratio ..	0.96		OK
Z-Bars Development Length Ratio ..	0.23		OK
Min. Bottom Steel Area Ratio	0.92		OK
Under-strength ϕ Factor	0.90		
Bending Strength +ϕMnx	64.7	k-ft	OK
Bending Strength +ϕMnz	58.7	k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.52		OK
Max. Bar Spacing Ratio	0.83		OK

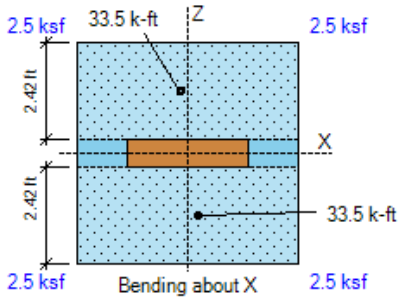
TRANSFER (Comb: 1.4D)

Bearing Under-strength ϕ Factor	0.65		

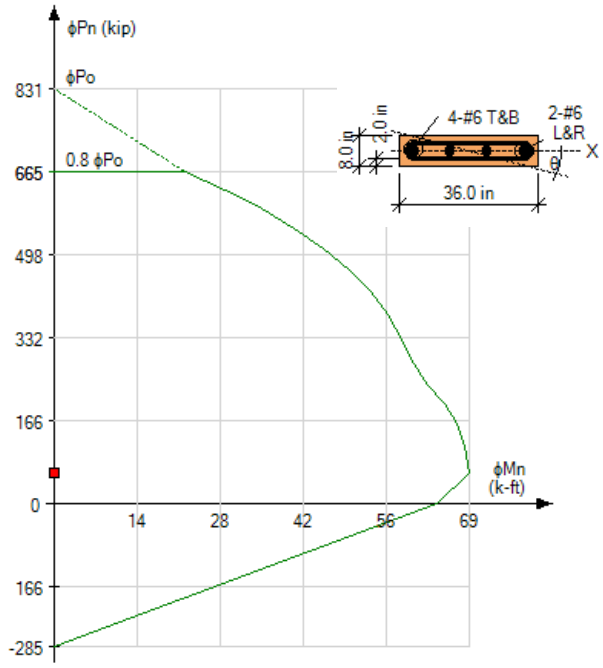
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²

Bearing Stress Pbu	0.2	ksi	
Column Bearing Strength ϕPn ...	2.9	ksi	OK
Footing Bearing Strength ϕPn ...	4.0	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	2.88	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.4D

DESIGN CODES

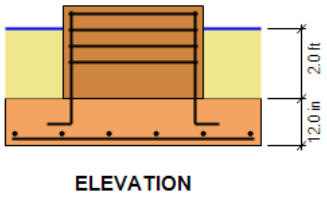
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

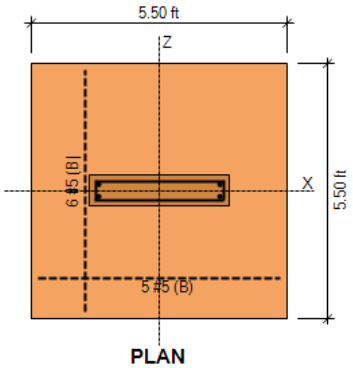
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.4D)

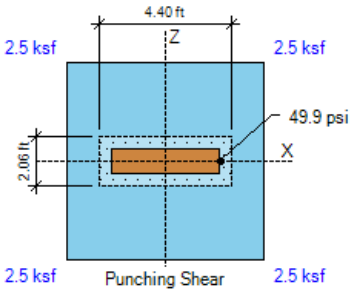
Axial Load Pu	63.0	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	664.6	kip
Flexural Strength ϕMn	70.3	k-ft
Biaxial Strength Ratio	0.09	OK



ELEVATION



PLAN



Punching Shear

GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -COL U

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	4.75	ft	
Footing Width (Z-dir)	4.75	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	8.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	N. A.		

SOIL PRESSURES (D+Lr)

Gross Allow. Soil Pressure ..	2.0	ksf
Soil Pressure at Corner 1	1.9	ksf
Soil Pressure at Corner 2	1.9	ksf
Soil Pressure at Corner 3	1.9	ksf
Soil Pressure at Corner 4	1.9	ksf
Bearing Pressure Ratio	0.95	OK
Ftg. Area in Contact with Soil ..	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	32.0	0.0	4.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

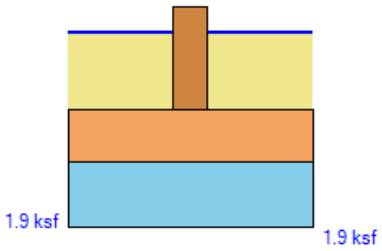
OVERTURNING

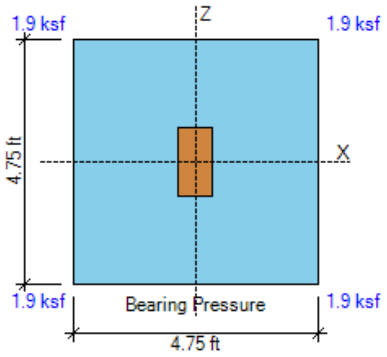
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	1.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	2.38	2.38	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	2.0	2.38	4.8
Column Weight ..	0.0	2.38	0.0
Soil Cover	2.9	2.38	6.8
Axial Force P	19.2	2.38	45.6
Water Buoyancy .	-0.8	2.38	-2.0
Rv =	23.2	RM =	55.2
- About Z-Z			
Footing Weight ..	2.0	2.38	4.8
Column Weight ..	0.0	2.38	0.0
Soil Cover	2.9	2.38	6.8
Axial Force P	19.2	2.38	45.6
Water Buoyancy .	-0.8	2.38	-2.0
Rv =	23.2	RM =	55.2



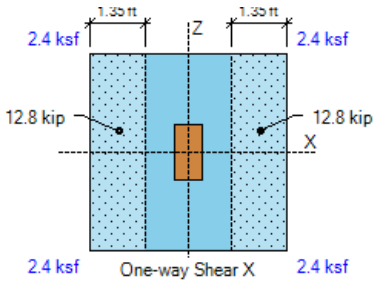


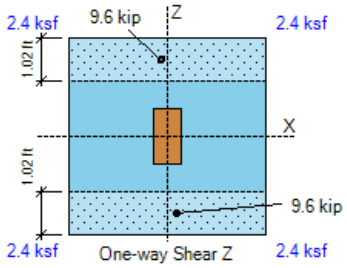
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Friction Force at Base	10.5		kip
Passive Pressure Coeff. Kp	4.33		
	X-X	Z-Z	
Passive Force @ Ftg ...	5.6	5.6	kip
Horiz. Resisting Force	16.1	16.1	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestal Selfweight	0.0	kip	
Footing Selfweight	2.0	kip	
Soil Cover Weight	2.9	kip	
Downward Force W	4.9	kip	
Uplift Safety Factor	99.99 > 1.0	OK	





PUNCHING SHEAR (Comb: 1.4D)

Unbalanced Moment Factor ɣvx ...	0.45	
Unbalanced Moment Factor ɣvz ...	0.35	
Punching Shear due to Axial	58.9	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	58.9	psi
Effective Perimeter bo	81.0	in
Average Effective Depth d	8.3	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	2.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.36	OK

SPREAD FOOTING DESIGN

ONE-WAY SHEAR (Comb: 1.4D)

	- Side	+ Side		
One-way Shear Vux ...	12.8	12.8	kip	
One-way Shear Vuz ...	9.6	9.6	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	40.4		kip	OK
One-way Shear Strength ϕVcz	36.9		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.32			OK

FLEXURE (Comb: 1.4D)

- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

X-Bars Development Length Ratio ..	N.A.		
Z-Bars Development Length Ratio ..	N.A.		
Min. Top Steel Area Ratio	N.A.		
Under-strength ϕ Factor	0.60		
Bending Strength -ϕMnx	18.7	k-ft	OK
Bending Strength -ϕMnz	18.7	k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00		OK
Max. Bar Spacing Ratio	N.A.		

- Bottom Bars -	- Side	+ Side	
Bending Moment +Mux ...	13.8	13.8	k-ft
Bending Moment +Muz ...	19.7	19.7	k-ft

Use 5 #6 Bars Parallel to X (Bot) , d = 8.6 in

Use 5 #6 Bars Parallel to Z (Bot) , d = 7.9 in

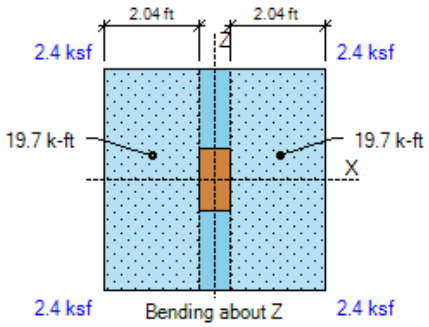
X-Bars Development Length Ratio ..	0.55		OK
Z-Bars Development Length Ratio ..	0.67		OK
Min. Bottom Steel Area Ratio	0.56		OK
Under-strength ϕ Factor	0.90		
Bending Strength +ϕMnx	73.5	k-ft	OK
Bending Strength +ϕMnz	80.9	k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.24		OK
Max. Bar Spacing Ratio	0.71		OK

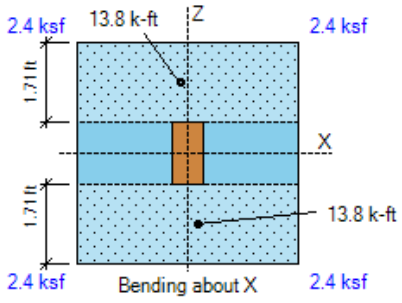
TRANSFER (Comb: 1.4D)

Bearing Under-strength ϕ Factor	0.65		

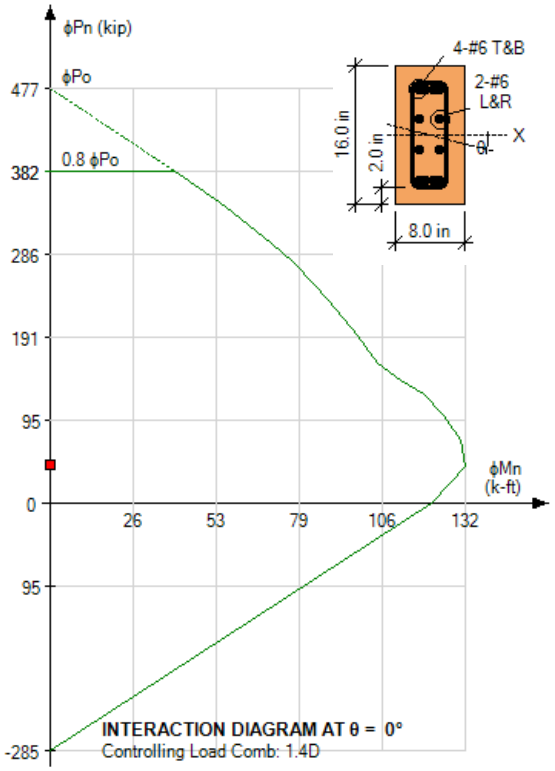
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²

Bearing Stress Pbu	0.4	ksi	
Column Bearing Strength ϕPn ...	3.8	ksi	OK
Footing Bearing Strength ϕPn ...	4.9	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



DESIGN CODES

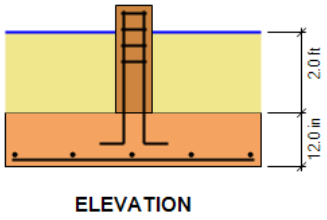
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

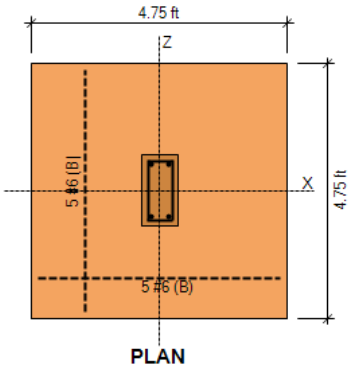
Column Concrete f'c	4.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.4D)

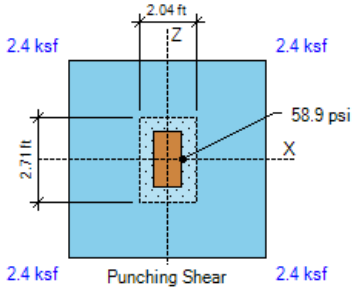
Axial Load Pu	44.8	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	381.6	kip
Flexural Strength ϕMn	133.2	k-ft
Biaxial Strength Ratio	0.12	OK



ELEVATION



PLAN



COMBINED FOOTING DESIGN

GEOMETRY

Column to Column Distance	4.00		ft	
	Exterior	Interior		
Column Length (X)	12.0	12.0	in	
Column Width (Z)	8.0	16.0	in	
Edge Distance (X)	3.00	3.00	ft	OK
Footing Width (Z)	5.00	5.00	ft	
Footing Thickness	12.0		in	OK
Soil Cover	2.00		ft	
Pedestal Height	N. A.			

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.00		ksf	
Overburden Pressure	0.32		ksf	
Reaction Force	85.9		kip	
Reaction Eccentricity	0.23		ft	
Ftg. Area in Contact with Soil	100.0		%	
	Exterior	Interior		
Bearing Pressure	1.96	1.48	ksf	
Bearing Ratio	0.98	0.74		OK

APPLIED LOADS

- Exterior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	35.0	5.0	0.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
- Interior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	25.0	5.0	0.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip

MATERIALS

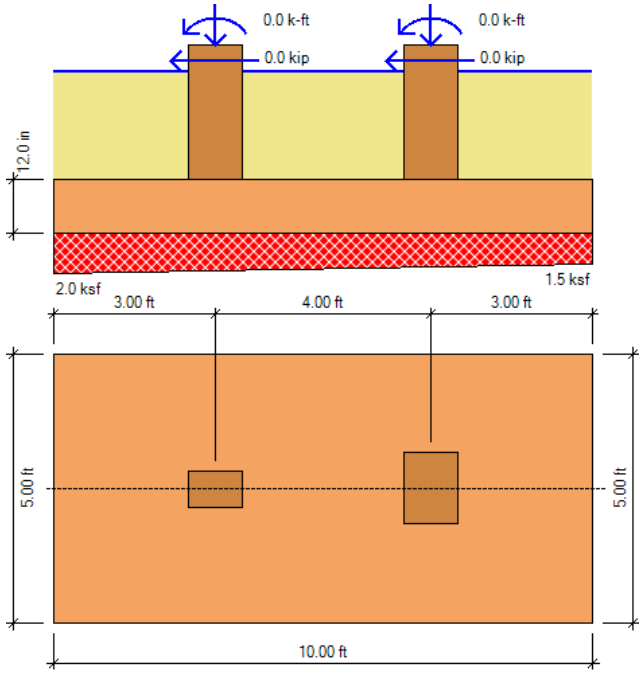
	Columns	Footing	
Concrete f'c	4.0	3.0	ksi
Reinf. Steel fy	60.0	60.0	ksi
Soil Cover Density		120.0	pcf

SLIDING (Comb. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Passive Pressure Coeff. Kp	4.33		
Friction Force at Base	20.5	kip	
Passive Force @ Ftg	6.5	kip	
Horiz. Resisting Force	27.0	kip	
Horiz. Sliding Force	0.0	kip	
Sliding Safety Factor	99.99 > 1.5		OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

	OVERTURNING				RESISTING			
	Force	Arm	Moment		Force	Arm	Moment
- About Z-Z	kip	ft	k-ft	- About Z-Z	kip	ft	k-ft
Ext. Moment Mz ...	0.0	0.0	0.0	Soil Cover	6.9	5.00	34.6
Ext. Shear Vx	0.0	1.00	0.0	Footing Weight ...	4.5	5.00	22.5
Int. Moment Mz	0.0	0.0	0.0	Ext. Column	0.0	7.00	0.0
Int. Shear Vx	0.0	1.00	0.0	Int. Column	0.0	3.00	0.0
Passive Pressure	0.0	0.40	0.0	Ext. Axial P	21.0	7.00	147.0
Rh =	0.0	OTM =	0.0	Int. Axial P	15.0	3.00	45.0
Arm of Vertical Resultant	4.74	ft		Water Buoyancy .	-1.9	5.00	-9.4
Overturning Safety Factor ..	99.99	> 1.5	OK	Rv =	45.5	RM =	215.7

FOOTING AT EXTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	18.0	9.6	kip
Shear Strength ϕVcx ..	41.3	41.3	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.44 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ... 0.43

Punching Shear due to Axial	73.5	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	73.5	psi
Effective Perimeter bo	73.5	in
Average Effective Depth d	8.4	in
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.45	OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.34 ft

No Top Reinf. Use Plain Concrete Strength

Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in

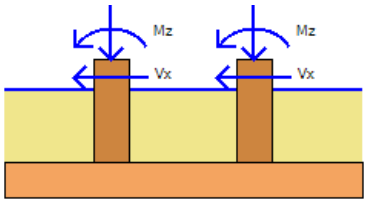
Bottom Steel Min. Area Ratio ..... 0.49 OK

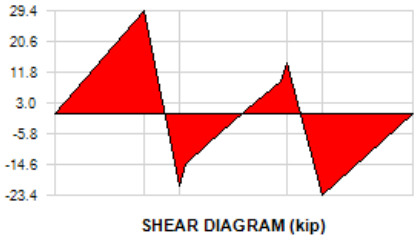
	Top	Bot	
Bending Moment Mux ..	0.0	10.5	k-ft
Bending Strength ϕMnx	9.2	42.1	k-ft

Max. Bending Ratio Mux/ϕMnx ..... 0.25 OK

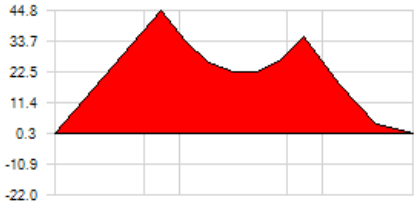
Top Z-Bars Develop. Length Ratio . N.A. OK

Bot Z-Bars Develop. Length Ratio .. 0.50 OK





SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

COMBINED FOOTING DESIGN

FOOTING AT INTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	4.6	13.7	kip
Shear Strength ϕVcx ..	41.3	41.3	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.33 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor γvx ...	0.38	
Punching Shear due to Axial	43.9	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	43.9	psi
Effective Perimeter bo	89.5	in
Average Effective Depth d	8.4	in
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.27	OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.34 ft

No Top Reinf. Use Plain Concrete Strength

Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in

Bottom Steel Min. Area Ratio ..... 0.49 OK

	Top	Bot	
Bending Moment Mux ..	0.0	6.4	k-ft
Bending Strength ϕMnx	9.2	42.1	k-ft

Max. Bending Ratio Mux/ϕMnx ..... 0.15 OK

Top Z-Bars Develop. Length Ratio . N.A. OK

Bot Z-Bars Develop. Length Ratio .. 0.60 OK

LONGITUDINAL REINF.

- Long. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Use 6 #5 Bars Parallel to X (Top) , d = 9.7 in

Use 6 #5 Bars Parallel to X (Bot) , d = 8.7 in

	Top	Bot	
Min. Steel Area Ratio ...	0.37	0.84	OK
Bending Moment Muz ..	22.0	44.8	k-ft
Bending Strength ϕMnz	78.0	69.7	k-ft

Max. Bending Ratio Muz/ϕMnz ..... 0.64 OK

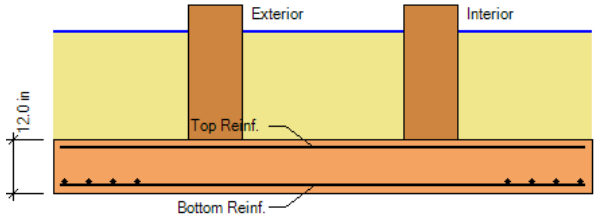
Ext. Bot X-Bars Dev. Length Ratio . 0.15 OK

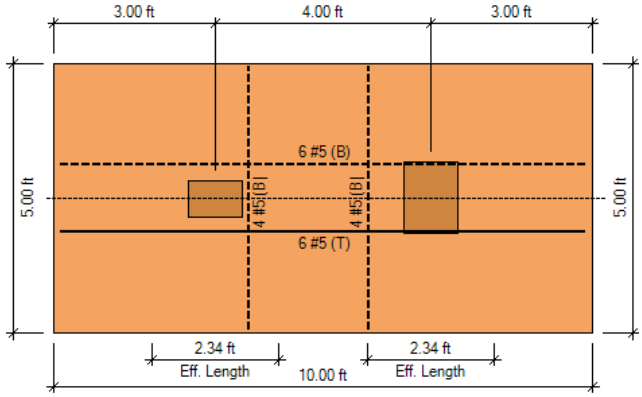
Int. Bot X-Bars Dev. Length Ratio ... 0.15 OK

LOAD TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor 0.65

	Ext.	Int.		
Compressive Stress Pu	0.5	0.2	ksi	
Column Bearing ϕPn	4.4	3.3	ksi	OK
Footing Bearing ϕPn	5.5	4.4	ksi	OK
Shear Force Vu	**0.0**	**0.0**	kip	
Shear Friction ϕVn	142.6	142.6	kip	OK
Dowel Steel Area As	**5.28**	**5.28**	in²	
Min. Steel Area As min ..	0.96	1.92	in²	OK
Dev. Length Ratio at Col.	0.50	0.50		OK
Dev. Length Ratio at Ftg.	0.67	0.67		OK





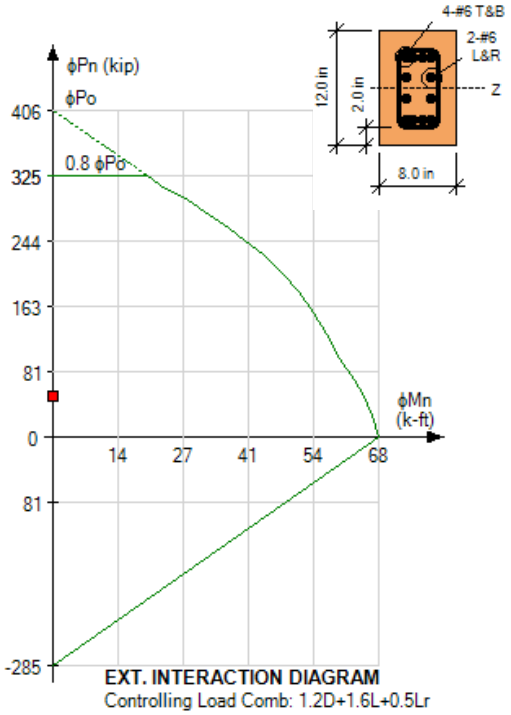
UPLIFT CALCS (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestals Selfweight	0.0	kip	
Footing Selfweight	4.5	kip	
Soil Cover Weight	6.9	kip	
Downward Force W	11.4	kip	
Uplift Safety Factor	99.99	> 1.0	OK

EXT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	50.0	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	323.3	kip
Flexural Strength ϕMn	65.2	k-ft
Biaxial Strength Ratio	0.15	OK

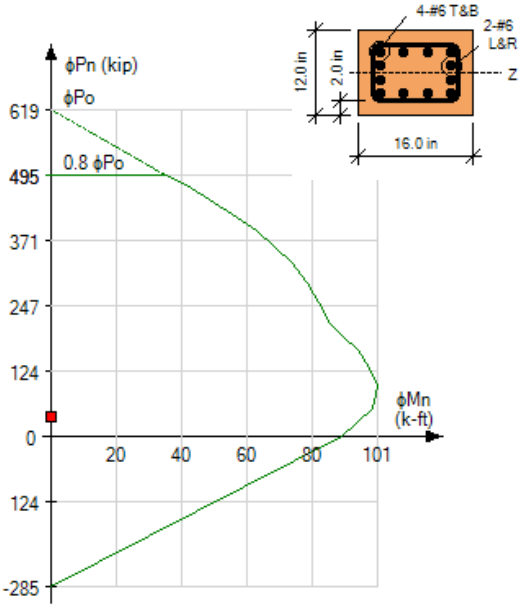


EXT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

INT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	38.0	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	494.2	kip
Flexural Strength ϕMn	95.8	k-ft
Biaxial Strength Ratio	0.08	OK



INT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG COLNS S & S1

ASDIP Foundation 4.8.1.0

COMBINED FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Column to Column Distance	2.67		ft	
	Exterior	Interior		
Column Length (X)	12.0	12.0	in	
Column Width (Z)	8.0	16.0	in	
Edge Distance (X)	3.50	3.50	ft	OK
Footing Width (Z)	5.50	5.50	ft	
Footing Thickness	14.0		in	OK
Soil Cover	2.00		ft	
Pedestal Height	N. A.			

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.00		ksf	
Overburden Pressure	0.33		ksf	
Reaction Force	99.7		kip	
Reaction Eccentricity	-0.03		ft	
Ftg. Area in Contact with Soil	100.0		%	
	Exterior	Interior		
Bearing Pressure	1.84	1.91	ksf	
Bearing Ratio	0.92	0.95		OK

APPLIED LOADS

- Exterior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	35.0	5.0	0.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
- Interior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	35.0	7.0	0.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip

MATERIALS

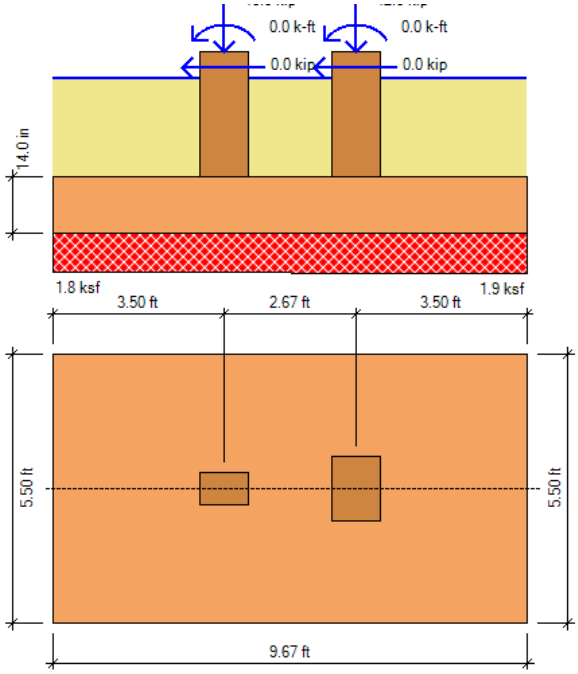
	Columns	Footing	
Concrete f'c	4.0	3.0	ksi
Reinf. Steel fy	60.0	60.0	ksi
Soil Cover Density		120.0	pcf

SLIDING (Comb. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Passive Pressure Coeff. Kp	4.33		
Friction Force at Base	23.7	kip	
Passive Force @ Ftg	8.6	kip	
Horiz. Resisting Force	32.3	kip	
Horiz. Sliding Force	0.0	kip	
Sliding Safety Factor	99.99 > 1.5		OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

	OVERTURNING				RESISTING			
	Force	Arm	Moment		Force	Arm	Moment
- About Z-Z	kip	ft	k-ft	- About Z-Z	kip	ft	k-ft
Ext. Moment Mz ...	0.0	0.0	0.0	Soil Cover	7.4	4.84	35.6
Ext. Shear Vx	0.0	1.17	0.0	Footing Weight ...	5.6	4.84	27.0
Int. Moment Mz	0.0	0.0	0.0	Ext. Column	0.0	6.17	0.0
Int. Shear Vx	0.0	1.17	0.0	Int. Column	0.0	3.50	0.0
Passive Pressure	0.0	0.47	0.0	Ext. Axial P	21.0	6.17	129.6
Rh =	0.0	OTM =	0.0	Int. Axial P	21.0	3.50	73.5
Arm of Vertical Resultant		4.84 ft		Water Buoyancy .	-2.3	4.84	-11.2
Overturning Safety Factor ..		99.99 > 1.5	OK	Rv =	52.6	RM =	254.5

FOOTING AT EXTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	22.3	1.3	kip
Shear Strength ϕVcx ..	56.3	56.3	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.40 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ... 0.42

Punching Shear due to Axial	52.7	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	52.7	psi
Effective Perimeter bo	81.5	in
Average Effective Depth d	10.4	in
Punching Shear Strength ϕVc ..	164.3	psi

Punching Shear Ratio Vu/ϕVc ..... 0.32 OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.68 ft

No Top Reinf. Use Plain Concrete Strength

Use 4 #5 Bars Parallel to Z (Bot) , d = 10.1 in

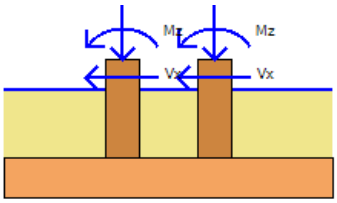
Bottom Steel Min. Area Ratio ..... 0.65 OK

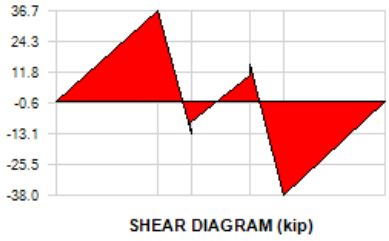
	Top	Bot	
Bending Moment Mux ..	0.0	15.0	k-ft
Bending Strength ϕMnx	14.4	53.6	k-ft

Max. Bending Ratio Mux/ϕMnx ..... 0.28 OK

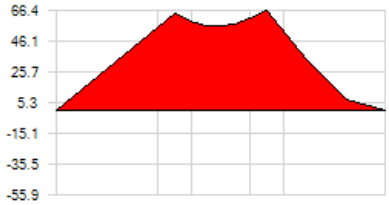
Top Z-Bars Develop. Length Ratio . N.A. OK

Bot Z-Bars Develop. Length Ratio .. 0.44 OK





SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

COMBINED FOOTING DESIGN

FOOTING AT INTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	0.6	23.2	kip
Shear Strength ϕVcx ..	56.3	56.3	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.41 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor γvx ...	0.38	
Punching Shear due to Axial	44.5	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	44.5	psi
Effective Perimeter bo	97.5	in
Average Effective Depth d	10.4	in
Punching Shear Strength ϕVc ..	164.3	psi

Punching Shear Ratio Vu/ϕVc ..... 0.27 OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.68 ft

No Top Reinf. Use Plain Concrete Strength

Use 4 #5 Bars Parallel to Z (Bot) , d = 10.1 in

Bottom Steel Min. Area Ratio ..... 0.65 OK

	Top	Bot	
Bending Moment Mux ..	0.0	11.4	k-ft
Bending Strength ϕMnx	14.4	53.6	k-ft

Max. Bending Ratio Mux/ϕMnx ..... 0.21 OK

Top Z-Bars Develop. Length Ratio . N.A. OK

Bot Z-Bars Develop. Length Ratio .. 0.52 OK

LONGITUDINAL REINF.

- Long. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Use 5 #5 Bars Parallel to X (Top) , d = 11.7 in

Use 7 #5 Bars Parallel to X (Bot) , d = 10.7 in

	Top	Bot	
Min. Steel Area Ratio ...	0.93	0.87	OK
Bending Moment Muz ..	55.9	66.4	k-ft
Bending Strength ϕMnz	79.6	100.6	k-ft

Max. Bending Ratio Muz/ϕMnz ..... 0.70 OK

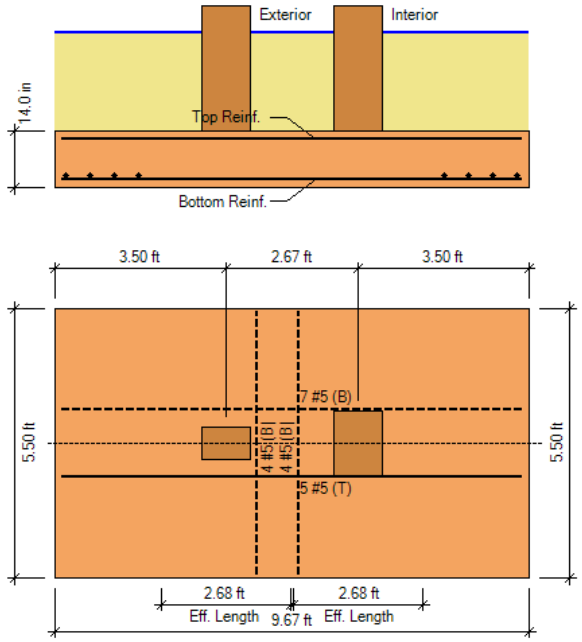
Ext. Bot X-Bars Dev. Length Ratio . 0.26 OK

Int. Bot X-Bars Dev. Length Ratio ... 0.26 OK

LOAD TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor 0.65

	Ext.	Int.		
Compressive Stress Pu	0.5	0.3	ksi	
Column Bearing ϕPn	4.4	3.3	ksi	OK
Footing Bearing ϕPn	5.5	4.4	ksi	OK
Shear Force Vu	**0.0**	**0.0**	kip	
Shear Friction ϕVn	142.6	142.6	kip	OK
Dowel Steel Area As	**5.28**	**5.28**	in²	
Min. Steel Area As min ..	0.96	1.92	in²	OK
Dev. Length Ratio at Col.	0.50	0.50		OK
Dev. Length Ratio at Ftg.	0.55	0.55		OK



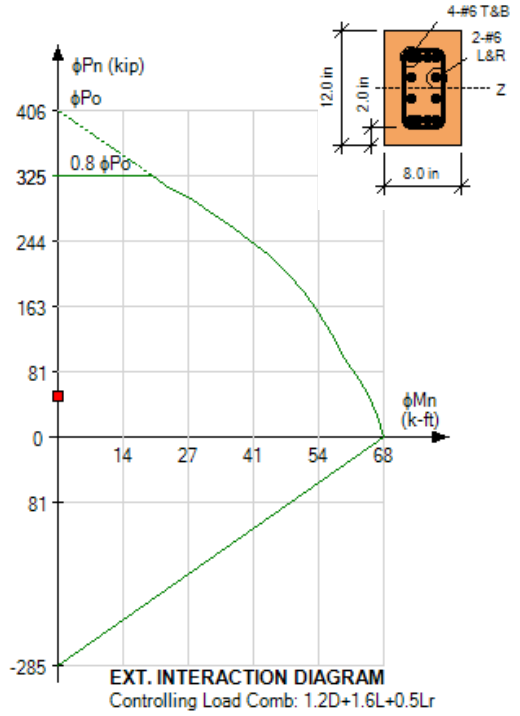
UPLIFT CALCS (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestals Selfweight	0.0	kip	
Footing Selfweight	5.6	kip	
Soil Cover Weight	7.4	kip	
Downward Force W	13.0	kip	
Uplift Safety Factor	99.99	> 1.0	OK

EXT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	50.0	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	323.3	kip
Flexural Strength ϕMn	65.2	k-ft
Biaxial Strength Ratio	0.15	OK

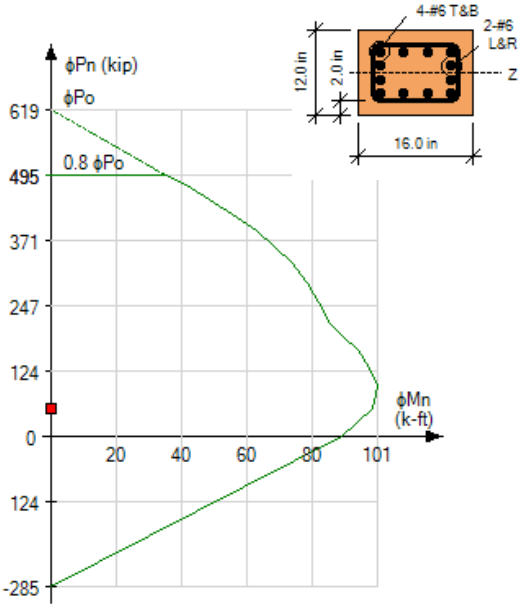


EXT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

INT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	53.2	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	494.2	kip
Flexural Strength ϕMn	98.2	k-ft
Biaxial Strength Ratio	0.11	OK



INT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG -STEEL COLUMN D-E-F-G

ASDIP Foundation 4.8.1.0

SPREAD FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	4.75	ft	
Footing Width (Z-dir)	4.75	ft	
Footing Thickness	12.0	in	OK
Soil Cover	2.00	ft	
Column Length (X-dir)	16.0	in	
Column Width (Z-dir)	16.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Pedestal Height	2.00	ft	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure ..	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.95		OK
Ftg. Area in Contact with Soil ..	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	29.0	7.0	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

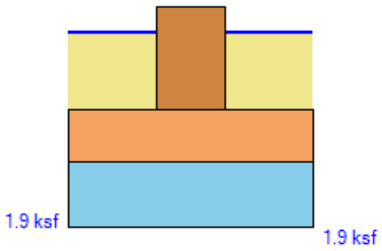
OVERTURNING

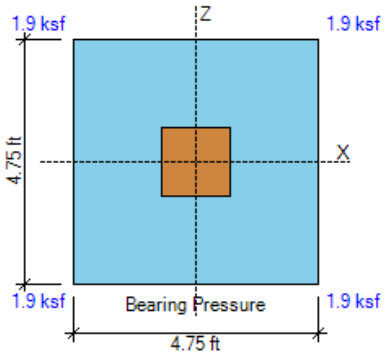
	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Moment Mx	---	---	0.0
Shear Force Vz	0.0	3.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0
- About Z-Z			
Moment Mz	---	---	0.0
Shear Force Vx	0.0	3.00	0.0
Passive Pressure	0.0	0.40	0.0
Rh =	0.0	OTM =	0.0

	X-X	Z-Z	
Arm of Vertical Resultant =	2.38	2.38	ft
Overturning Safety Factor =	99.99	99.99	> 1.5
			OK

RESISTING

	Force	Arm	Moment
- About X-X	kip	ft	k-ft
Footing Weight ..	2.0	2.38	4.8
Column Weight ..	0.3	2.38	0.8
Soil Cover	2.7	2.38	6.5
Axial Force P	17.4	2.38	41.3
Water Buoyancy .	-0.8	2.38	-2.0
Rv =	21.6	RM =	51.4
- About Z-Z			
Footing Weight ..	2.0	2.38	4.8
Column Weight ..	0.3	2.38	0.8
Soil Cover	2.7	2.38	6.5
Axial Force P	17.4	2.38	41.3
Water Buoyancy .	-0.8	2.38	-2.0
Rv =	21.6	RM =	51.4



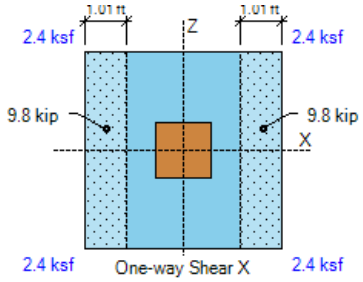


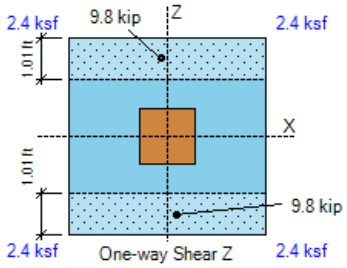
SLIDING (combDlg. 0.6D+0.6W)

Footing-Soil Friction Coeff.		0.45	
Friction Force at Base		9.7	kip
Passive Pressure Coeff. Kp		4.33	
	X-X	Z-Z	
Passive Force @ Ftg ...	5.6	5.6	kip
Horiz. Resisting Force	15.4	15.4	kip
Horiz. Sliding Force	0.0	0.0	kip
Sliding Safety Factor ...	99.99	99.99 > 1.5	OK

UPLIFT (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip
Pedestal Selfweight	0.3	kip
Footing Selfweight	2.0	kip
Soil Cover Weight	2.7	kip
Downward Force W	5.1	kip
Uplift Safety Factor	99.99 > 1.0	OK





PUNCHING SHEAR (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.40	
Unbalanced Moment Factor ɣvz ...	0.40	
Punching Shear due to Axial	46.0	psi
Punching Shear due to Mx	0.0	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	46.0	psi
Effective Perimeter bo	97.5	in
Average Effective Depth d	8.4	in
Column Location Factor αs	40 (Interior)	
Column Aspect Ratio β	1.00	
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.28	OK

ONE-WAY SHEAR (Comb: 1.2D+1.6L+0.5Lr)

	- Side	+ Side		
One-way Shear Vux ...	9.8	9.8	kip	
One-way Shear Vuz ...	9.8	9.8	kip	
Under-strength ϕ Factor	0.75			
One-way Shear Strength ϕVcx	40.7		kip	OK
One-way Shear Strength ϕVcz	37.8		kip	OK
Max. One-way Shear Ratio Vu/ϕVc ..	0.26			OK

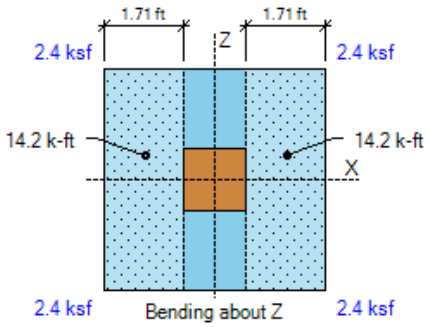
FLEXURE (Comb: 1.2D+1.6L+0.5Lr)

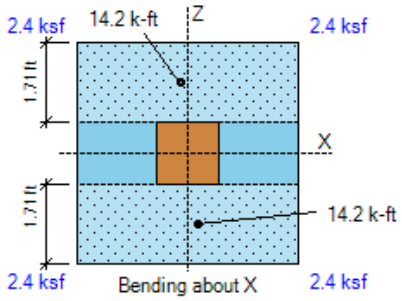
- Top Bars -	- Side	+ Side		
Bending Moment -Mux ...	0.0	0.0	k-ft	
Bending Moment -Muz ...	0.0	0.0	k-ft	
No Top Reinforcement Provided at the Footing				
Use Plain Concrete Flexural Strength at Top				
X-Bars Development Length Ratio ..	N.A.			
Z-Bars Development Length Ratio ..	N.A.			
Min. Top Steel Area Ratio	N.A.			
Under-strength ϕ Factor	0.60			
Bending Strength -ϕMnx	18.7		k-ft	OK
Bending Strength -ϕMnz	18.7		k-ft	OK
Max. Bending Ratio -Mu/ϕMn	0.00			OK
Max. Bar Spacing Ratio	N.A.			

- Bottom Bars -	- Side	+ Side		
Bending Moment +Mux ...	14.2	14.2	k-ft	
Bending Moment +Muz ...	14.2	14.2	k-ft	
Use 5 #5 Bars Parallel to X (Bot) , d = 8.7 in				
Use 5 #5 Bars Parallel to Z (Bot) , d = 8.1 in				
X-Bars Development Length Ratio ..	0.67			OK
Z-Bars Development Length Ratio ..	0.67			OK
Min. Bottom Steel Area Ratio	0.79			OK
Under-strength ϕ Factor	0.90			
Bending Strength +ϕMnx	54.0		k-ft	OK
Bending Strength +ϕMnz	58.4		k-ft	OK
Max. Bending Ratio +Mu/ϕMn	0.26			OK
Max. Bar Spacing Ratio	0.71			OK

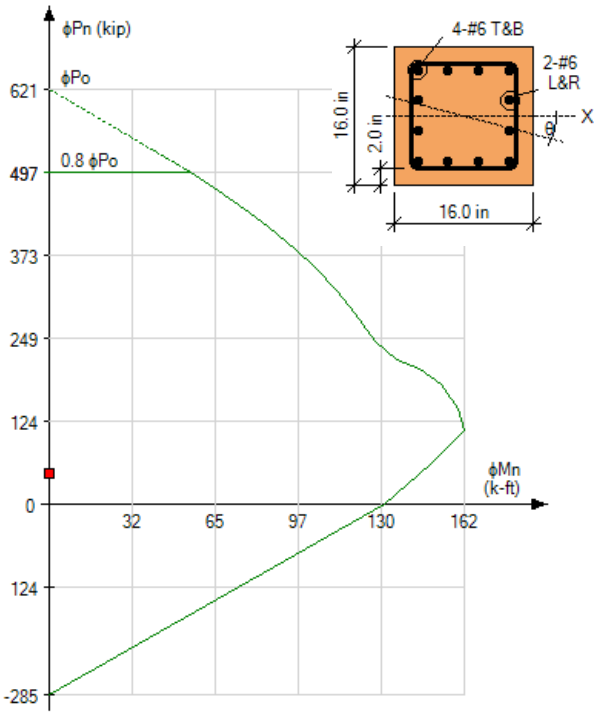
TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor	0.65		
Use 12 #6 x 1.83 ft hooked dowels , As = 5.28 in²			
Bearing Stress Pbu	0.2	ksi	
Column Bearing Strength ϕPn ...	2.5	ksi	OK
Footing Bearing Strength ϕPn ...	4.1	ksi	OK
Shear Force Transfer Vu	0.0	kip	
Shear Friction Strength ϕVn	142.6	kip	OK
Min. Steel Area As = 0.005 A1 ...	1.28	in²	OK
Develop. Length Ratio at Column	0.55		OK
Develop. Length Ratio at Footing	0.67		OK





SPREAD FOOTING DESIGN



INTERACTION DIAGRAM AT θ = 0°
Controlling Load Comb: 1.2D+1.6L+0.5Lr

DESIGN CODES

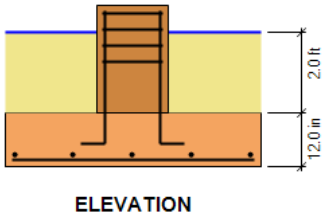
Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

MATERIALS

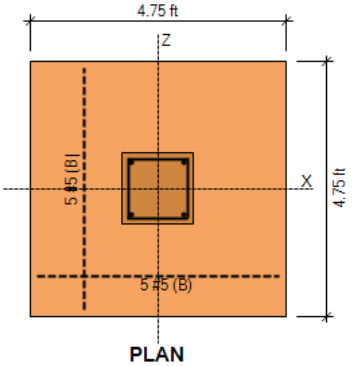
Column Concrete f'c	3.0	ksi
Column Dowels fy	60.0	ksi
Column Ties fy	60.0	ksi
Footing Concrete f'c	3.0	ksi
Footing Rebars fy	60.0	ksi
Soil Cover Density	110.0	pcf

COLUMN (Comb: 1.2D+1.6L+0.5Lr)

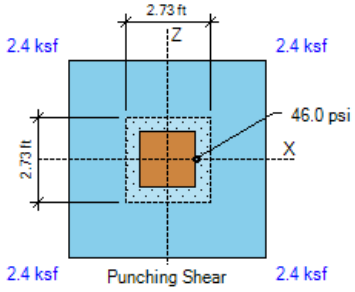
Axial Load Pu	46.6	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	495.2	kip
Flexural Strength ϕMn	145.6	k-ft
Biaxial Strength Ratio	0.09	OK



ELEVATION



PLAN



Punching Shear

GF CONSULTING ENGINEERS INC
13170 SW 128 St. #204
MIAMI, FL

Project:
Engineer: FH
Descrip: FTG COLNS O & Z

ASDIP Foundation 4.8.1.0

COMBINED FOOTING DESIGN

www.asdipsoft.com

GEOMETRY

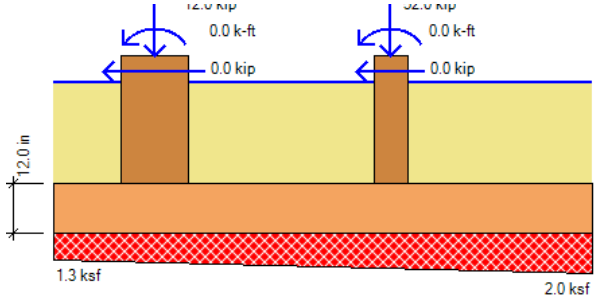
Column to Column Distance	4.67		ft	
	Exterior	Interior		
Column Length (X)	16.0	8.0	in	
Column Width (Z)	16.0	24.0	in	
Edge Distance (X)	2.00	4.00	ft	OK
Footing Width (Z)	4.50	4.50	ft	
Footing Thickness	12.0		in	OK
Soil Cover	2.00		ft	
Pedestal Height	N. A.			

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.00		ksf	
Overburden Pressure	0.31		ksf	
Reaction Force	79.0		kip	
Reaction Eccentricity	-0.37		ft	
Ftg. Area in Contact with Soil	100.0		%	
	Exterior	Interior		
Bearing Pressure	1.30	1.99	ksf	
Bearing Ratio	0.65	0.99		OK

APPLIED LOADS

- Exterior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	10.0	2.0	1.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
- Interior Col.	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	45.0	7.0	2.0	0.0	0.0	0.0	kip
Moment about Z Mz	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip

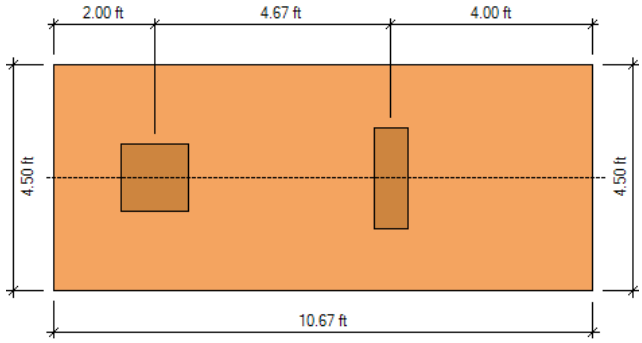


MATERIALS

	Columns	Footing	
Concrete f'c	4.0	3.0	ksi
Reinf. Steel fy	60.0	60.0	ksi
Soil Cover Density		120.0	pcf

SLIDING (Comb. 0.6D+0.6W)

Footing-Soil Friction Coeff.	0.45		
Passive Pressure Coeff. Kp	4.33		
Friction Force at Base	18.9	kip	
Passive Force @ Ftg	5.8	kip	
Horiz. Resisting Force	24.7	kip	
Horiz. Sliding Force	0.0	kip	
Sliding Safety Factor	99.99 > 1.5		OK



DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

	OVERTURNING			RESISTING			
	Force	Arm	Moment		Force	Arm	Moment
- About Z-Z	kip	ft	k-ft	- About Z-Z	kip	ft	k-ft
Ext. Moment Mz ...	0.0	0.0	0.0	Soil Cover	6.5	5.34	34.5
Ext. Shear Vx	0.0	1.00	0.0	Footing Weight ...	4.3	5.34	23.1
Int. Moment Mz	0.0	0.0	0.0	Ext. Column	0.0	8.67	0.0
Int. Shear Vx	0.0	1.00	0.0	Int. Column	0.0	4.00	0.0
Passive Pressure	0.0	0.40	0.0	Ext. Axial P	6.0	8.67	52.0
Rh =	0.0	OTM =	0.0	Int. Axial P	27.0	4.00	108.0
Arm of Vertical Resultant	5.72	ft		Water Buoyancy .	-1.8	5.34	-9.6
Overturning Safety Factor ..	99.99	> 1.5	OK	Rv =	42.0	RM =	240.1

FOOTING AT EXTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	3.7	5.8	kip
Shear Strength ϕVcx ..	37.2	37.2	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.15 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor ɣvx ...	0.45	
Punching Shear due to Axial	9.7	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	9.7	psi
Effective Perimeter bo	96.8	in
Average Effective Depth d	8.4	in
Punching Shear Strength ϕVc ..	164.3	psi
Punching Shear Ratio Vu/ϕVc	0.06	OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.68 ft

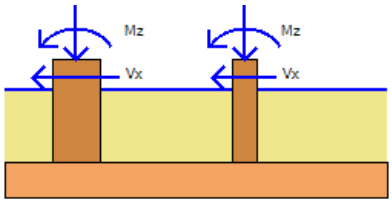
Use 0 #5 Bars Parallel to Z (Top) , d = 9.1 in

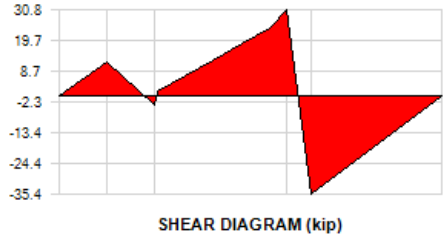
Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in

Bottom Steel Min. Area Ratio ..... 0.56 OK

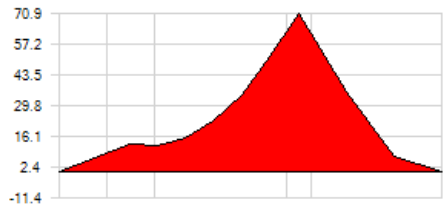
	Top	Bot	
Bending Moment Mux ..	0.0	4.9	k-ft
Bending Strength ϕMnx	10.6	42.4	k-ft

Max. Bending Ratio Mux/ϕMnx	0.12	OK
Top Z-Bars Develop. Length Ratio .	0.35	OK
Bot Z-Bars Develop. Length Ratio ..	0.35	OK





SHEAR DIAGRAM (kip)



MOMENT DIAGRAM (k-ft)

COMBINED FOOTING DESIGN

FOOTING AT INTERIOR COL.

- One-way Shear (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.75

	- Side	+ Side	
One-way Shear Vux ...	22.5	26.9	kip
Shear Strength ϕVcx ..	37.2	37.2	kip

Max. One-way Shear Ratio Vu/ϕVc .. 0.72 OK

- Punching Shear (Comb: 1.2D+1.6L+0.5Lr)

Unbalanced Moment Factor γvx ...	0.29	
Punching Shear due to Axial	64.8	psi
Punching Shear due to Mz	0.0	psi
Punching Shear Stress Vu	64.8	psi
Effective Perimeter bo	102.8	in
Average Effective Depth d	8.4	in
Punching Shear Strength ϕVc ..	136.9	psi

Punching Shear Ratio Vu/ϕVc ..... 0.47 OK

- Transv. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Under-strength ϕ Factor 0.90

Footing Effective Length ..... 2.01 ft

Use 0 #5 Bars Parallel to Z (Top) , d = 9.1 in

Use 4 #5 Bars Parallel to Z (Bot) , d = 8.1 in

Bottom Steel Min. Area Ratio ..... 0.42 OK

	Top	Bot	
Bending Moment Mux ..	0.0	2.9	k-ft
Bending Strength ϕMnx	7.9	41.6	k-ft

Max. Bending Ratio Mux/ϕMnx ..... 0.07 OK

Top Z-Bars Develop. Length Ratio . 0.46 OK

Bot Z-Bars Develop. Length Ratio .. 0.46 OK

LONGITUDINAL REINF.

- Long. Flexure (Comb: 1.2D+1.6L+0.5Lr)

Use 8 #5 Bars Parallel to X (Top) , d = 9.7 in

Use 8 #5 Bars Parallel to X (Bot) , d = 8.7 in

	Top	Bot	
Min. Steel Area Ratio ...	0.14	0.70	OK
Bending Moment Muz ..	11.4	70.9	k-ft
Bending Strength ϕMnz	102.1	90.9	k-ft

Max. Bending Ratio Muz/ϕMnz ..... 0.78 OK

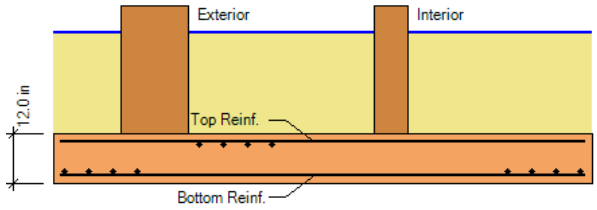
Ext. Bot X-Bars Dev. Length Ratio . 0.20 OK

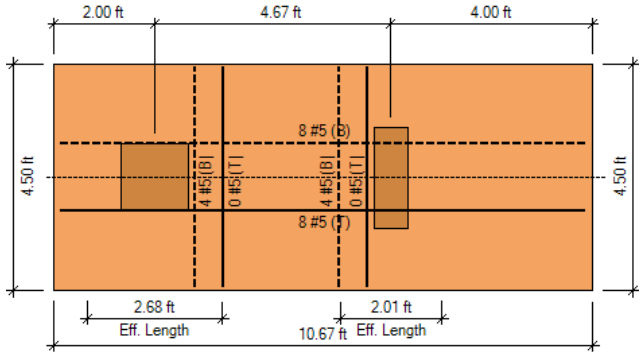
Int. Bot X-Bars Dev. Length Ratio ... 0.12 OK

LOAD TRANSFER (Comb: 1.2D+1.6L+0.5Lr)

Bearing Under-strength ϕ Factor 0.65

	Ext.	Int.		
Compressive Stress Pu	0.1	0.3	ksi	
Column Bearing ϕPn	3.0	3.3	ksi	OK
Footing Bearing ϕPn	4.1	4.4	ksi	OK
Shear Force Vu	**0.0**	**0.0**	kip	
Shear Friction ϕVn	142.6	142.6	kip	OK
Dowel Steel Area As	**5.28**	**5.28**	in²	
Min. Steel Area As min ..	2.56	1.92	in²	OK
Dev. Length Ratio at Col.	0.50	0.50		OK
Dev. Length Ratio at Ftg.	0.67	0.67		OK





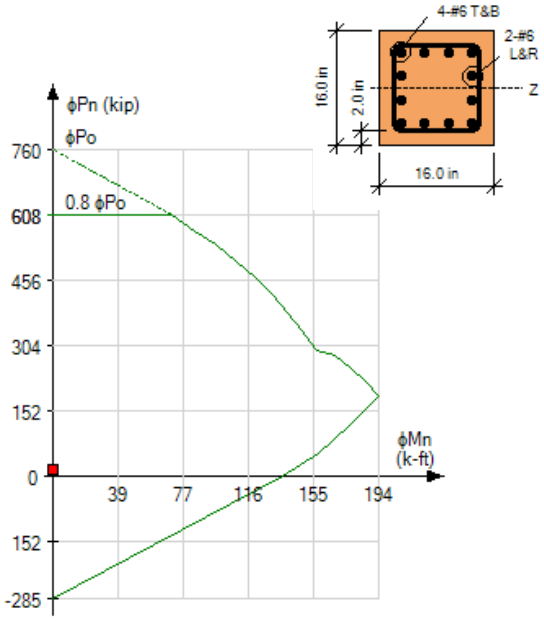
UPLIFT CALCS (Comb: 0.6D+0.6W)

Uplift Force P	0.0	kip	
Pedestals Selfweight	0.0	kip	
Footing Selfweight	4.3	kip	
Soil Cover Weight	6.5	kip	
Downward Force W	10.8	kip	
Uplift Safety Factor	99.99	> 1.0	OK

EXT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	15.7	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	603.8	kip
Flexural Strength ϕMn	144.0	k-ft
Biaxial Strength Ratio	0.03	OK

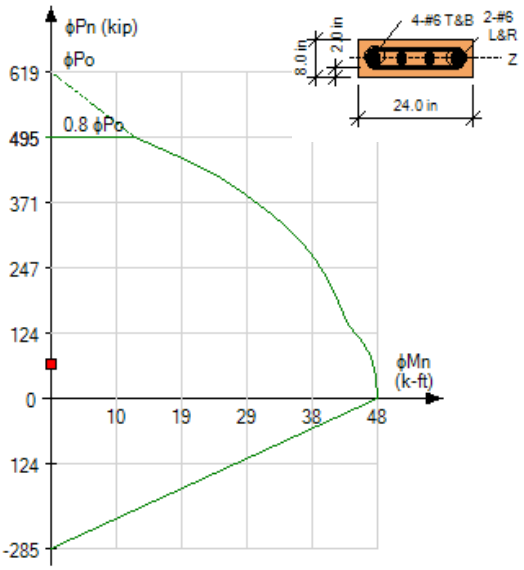


EXT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

INT. COLUMN (Comb: 1.2D+1.6L+0.5Lr)

Use 12-#6 Longitudinal Bars, Use Ties #4

Axial Load Pu	66.2	kip
Bending Moment Mu	0.0	k-ft
Axial Strength ϕPn	494.9	kip
Flexural Strength ϕMn	47.3	k-ft
Biaxial Strength Ratio	0.13	OK



INT. INTERACTION DIAGRAM
Controlling Load Comb: 1.2D+1.6L+0.5Lr

WALL FOOTING DESIGN

GEOMETRY

Footing Width	2.00	ft	
Footing Thickness	12.0	in	
Soil Cover	2.00	ft	OK
Water Table	3.00	ft	
Wall Thickness	8.0	in	
Wall Offset	0.0	in	OK

SOIL PRESSURES (D+0.75L+0.75Lr)

Gross Allow. Soil Pressure	2.0	ksf	
Max. Bearing Pressure	1.8	ksf	
Min. Bearing Pressure	1.8	ksf	
Bearing Pressure Ratio	0.92		OK
Ftg. Area in Contact w/ Soil	100.0	%	
Eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	2.7	0.2	0.3	0.0	0.0	0.0	k/ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft/ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	k/ft

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

  Arm = 0.00 + 12.0 / 12 = 1.00 ft Moment = 0.0 * 1.0 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.40 ft Moment = 0.0 * 0.40 = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 1.00 * 2.00 * 12.0 / 12 * 0.15 = 0.2 kip

  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft Moment = 0.2 * 1.00 = 0.2 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 12.0 / 12 * 8.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *L / 2 - Offset =* 2.00 / 2 - 0.0 / 12 = 1.00 ft Moment = 0.0 * 1.00 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (1.00 * 2.00 - 12.0 / 12 * 8.0 / 12) * 2.0 * 110 = 0.2 kip

  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft Moment = 0.2 * 1.00 = 0.2 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 1.00 * 2.00 * 62 * (2.00 + 12.0 / 12 - 3.00) = 0.0 kip

  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft Moment = 0.0 * 1.00 = 0.0 k-ft

- Axial force P = 0.6 * 2.7 + 0.6 * 0.0 = 1.6 k-ft

  Arm = *L / 2 - Offset =* 2.00 / 2 - 0.0 / 12 = 1.00 ft Moment = 1.6 * 1.00 = 1.6 k-ft

- Resisting moment Z-Z = 0.2 + 0.0 + 0.2 + 1.6 + 0.0 = 2.0 k-ft

- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{2.0}{0.0}$ = 99.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.75L+0.75Lr)

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft/ft

Resisting moment Z-Z = 0.3 + 0.3 + 0.0 + 3.1 = 3.7 k-ft/ft

Resisting force = *Footing + Soil - Buoyancy + P* = 0.3 + 0.3 - 0.0 + 3.1 = 3.7 k/ft

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{3.7 - 0.0}{3.7} = 1.00 \text{ ft}$$

X-ecc = *Width / 2 - Xp* = 2.00 / 2 - 1.00 = 0.00 ft

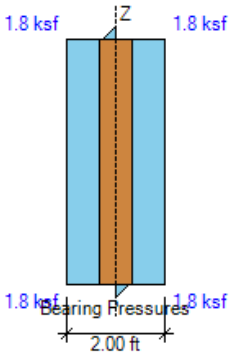
Area = *Width * Length* = 1.00 * 2.00 = 2.0 ft²

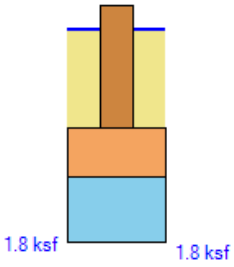
Sz = *Length * Width² / 6* = 1.00 * 2.00² / 6 = 0.7 ft³

Bearing length = *Min (Width , 3 * Xp)* = Min (2.00, 3 * 1.00) = 2.00 ft = Ftg. Width = 2.00 ft

Max. bearing = *Rv * (1/A + X-ecc / Sz)* = 3.7 * (1 / 2.0 + 0.00 / 0.7) = 1.8 ksf < 2.0 ksf OK

Min. bearing = *Rv * (1/A - X-ecc / Sz)* = 3.7 * (1 / 2.0 - 0.00 / 0.7) = 1.8 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28 deg (assumed) Active coefficient ka = 0.00

Passive coefficient *kp = 1 / ka* = 1 / 0.00 = 4.33

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2)* = 4.33 * 110 / 1000 * (2.00 + 12.0 / 12 / 2) = 1.19 ksf

X-Passive force = *Pressure * Thick * Length* = 1.19 * 12.0 / 12 * 1.00 = 1.2 k/ft

Friction force = *Resisting force * Friction coeff.* = Max (0, 2.0 * 0.45) = 0.9 k/ft

Use 100% of Passive + 100% of Friction for sliding resistance

$$\text{- Sliding safety factor X-X} = \frac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \frac{1.00 * 1.2 + 1.00 * 0.9}{0.0} = 99.99 > 1.50 \text{ OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\frac{Footing + Cover - Buoyancy}{Uplift\ load} = \frac{0.2 + 0.2 - 0.0}{0.0}$ = 99.99 > 1.00 OK

SHEAR CALCULATIONS (Comb: 1.4D)

Concrete f'c = 3.0 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

d Top X-dir = *Thick - Cover - X-diameter / 2* = 12.0 - 2.0 - 0.8 / 2 = 9.6 in

d Top Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2* = 12.0 - 2.0 - 0.8 - 0.8 / 2 = 8.9 in

d Bot X-dir = *Thick - Cover - X-diameter / 2* = 12.0 - 3.0 - 0.6 / 2 = 8.7 in

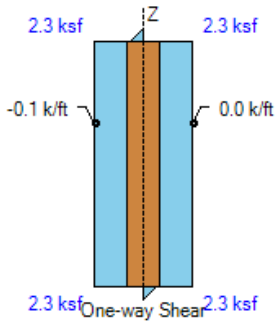
d Bot Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2* = 12.0 - 3.0 - 0.6 - 0.6 / 2 = 8.1 in

$\phi Vcx = 2 * \phi * \sqrt{f'c} * Width * d / 1000$ = 2 * 0.75 * √(3000) * 2.0 * 12 * 8.7 / 1000 = 8.6 kip ACI Eq. (22.5.5.1)

- Shear forces calculated as the volume of the bearing pressures under the effective areas:

One-way shear Vux (- Side) = -0.1 k/ft < 8.6 k/ft OK

One-way shear Vux (+ Side) = 0.0 k/ft < 8.6 k/ft OK



BENDING CALCULATIONS (Comb: 1.2D+0.5L+1.6Lr)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.55 * √(3000) * 1.00 * 12.0² / 6 / 1000 = 3.6 k-ft/ft ACI Eq. (14.5.2.1a)

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Muz (- Side) = 0.0 k-ft/ft < 3.6 k-ft/ft OK

Top moment -Muz (+ Side) = 0.0 k-ft/ft < 3.6 k-ft/ft OK

- Bottom Bars

Use 3 #5 Z-Bars *ρ = As / b d* = 0.9 / (2.00 * 12 * 8.1) = 0.0048 q = 0.0048 * 60 / 3.0 = 0.096

Use #5 @ 12.0 in X-Bars *ρ = As / b d* = 0.3 / (1.00 * 12 * 8.7) = 0.0030 q = 0.0030 * 60 / 3.0 = 0.059

Bending strength *ϕMn = ϕ * b * d² * f'c * q * (1 - 0.59 * q)* ACI 22.2.2

ϕMnz = 0.90 * 1.00 * 12 * 8.7² * 3.0 * 0.059 * (1 - 0.59 * 0.059) = 11.7 k-ft/ft

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Muz (- Side) = 0.4 k-ft/ft < 11.7 k-ft/ft OK ratio = 0.04

Bottom moment Muz (+ Side) = 0.4 k-ft/ft < 11.7 k-ft/ft OK ratio = 0.04

X-As min = *0.0018 * Length * Thick* = 0.0018 * 1.00 * 12 * 12.0 = 0.3 in²/ft < 0.3 in²/ft OK

Z-As min = *0.0018 * Width * Thick* = 0.0018 * 2.00 * 12 * 12.0 = 0.5 in² < 0.9 in² OK

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.63 / 2, 12.0 / 2) / 0.63) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Location / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

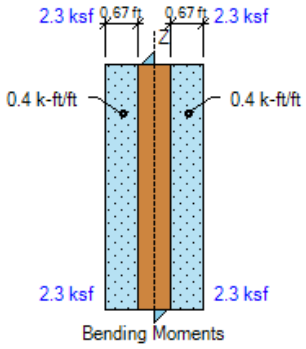
X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (3000)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.63 * 0.04) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio) =* ACI 25.4.3

X-Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (3000)½ * 1.0 * 0.7 * 0.0 * 0.63 * 0.04) = 6.0 in

-X Ld provided = *(Width - Wall) / 2 + Offset - Cover* = 2.00 * 12 / 2 + 0.0 - 8.0 / 2 - 2.0 = 6.0 in > 6.0 in OK

+X Ld provided = *(Width - Wall) / 2 - Offset - Cover* = 2.00 * 12 / 2 - 0.0 - 8.0 / 2 - 2.0 = 6.0 in > 6.0 in OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+0.5L+1.6Lr)

Area *A1 = wall L * t =* 8.0 * 12.0 = 96.0 in²

Sx = *wall L * t² / 6 =* 12.0 * 8.0² / 6 = 128.0 in³

Bearing *Pbu = P / A1 + Mz / Sx =* 3.8 / 96.0 + 0.0 * 12 / 128.0 = 0.0 ksi

Min edge = *W / 2 - X-offset - wall t / 2* = 2.00 * 12 / 2 - 0.0 - 8.0 / 2 = 8.0 in

Area *A2 = Min [L * W, (wall L + 2 * Min edge) * (wall t + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.00 * 12 * 1.0 * 12, (8.0 + 2 * 8.0) * (12.0 + 2 * 8.0)] = 288.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* =0.65 * 0.85 * 3.0 * Min [2, √(288.0 / 96.0)] = 2.9 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.65 * 0.12 * 60.0 / 96.0 = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.9 + 0.0 = 2.9 ksi > 0.0 ksi OK

Wall *ϕPnc = ϕ * 0.85 * f'c =* 0.65 * 0.85 * 3.0 = 1.7 ksi

Wall *ϕPns = ϕ * As * Fy / A1 =* 0.65 * 0.12 * 60.0 / 96.0 = 0.0 ksi ACI 22.8.3.2

Wall bearing *ϕPn = ϕPnc + ϕPns =* 1.7 + 0.0 = 1.7 ksi > 0.0 ksi OK

Shear friction *ϕVn = ϕ * As * Fy * μ =* 0.75 * 0.12 * 60.0 * 0.6 = 3.1 k/ft ACI Eq. (22.9.4.2)

Shear *Vu = Max (Vux, Vuz) =* Max (0.0, 0.0) = 0.0 k/ft < 3.1 k/ft OK

Straight *Ld req = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

Ld req = Max (12.0, 3 / 40 * 60.0 * 1000 / (3000)½ * 1.0 * 0.8 / 2.5 * 0.63 * 0.00) = 12.0 in

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (3000)½ * 1.0 * 0.7 * 0.0 * 0.63 * 1.00) = 6.0 in

Dowel Ld provided = 36.0 in > 12.0 in OK

Dowel Ldh provided =*Footing thickness - Cover =* 12.00 - 3.0 = 9.0 in > 6.0 in OK

Wall f'c = 3.0 ksi Max. strain = 0.0030 Dowel fy = 60.0 ksi

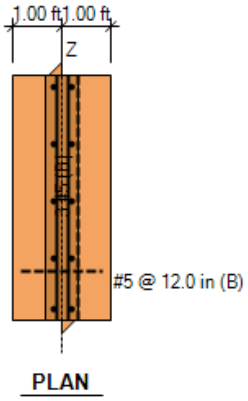
Use #5 dowels @ 32.0 in , As = 0.12 in²

Wall As min = *0.0012 * L * t =* 0.0012 * 12.0 * 8.0 = 0.12 in² < 0.12 in² OK ACI 11.6.1

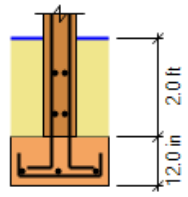
DESIGN CODES

Concrete Design ..... ACI 318-14

Load Combinations ..... ASCE 7-10/16



PLAN



SECTION

NOA's

13170 SW 128TH Street, #204 Miami, FL 33186 • Phone #: 305-971-GFCE (4323) • FAX: 305-971-4332
www.gfce.net E-MAIL: GFCE@GFCE.NET

MiTek®
Residential North America

Product Evaluation Report
MiTek® Inc.
UGTS & USC Uplift Girder Ties

Florida Product Approval # 2033.1-2 R6
Florida Building Code 2020
Per Rule 61G20-3.005(1)(d)
Method 1-d

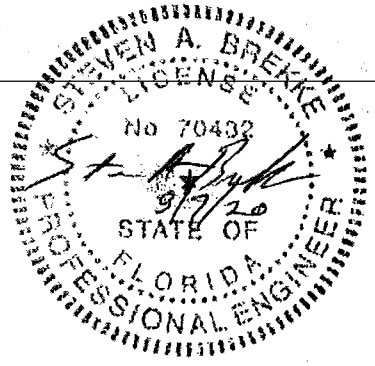
Category: Structural Components
Subcategory: Wood Connector
Compliance Method: 61G20-3.005
HVHZ

Product Manufacturer:
MiTek Inc.
16023 Swingley Ridge Rd
Chesterfield, MO 63017

Engineer Evaluator:
Steven Brekke P.E.
FL P.E. #70432

Validator:
Johnathan Green, P.E.
FL P.E. #88223

Contents:
Evaluation Report Pages 1-4

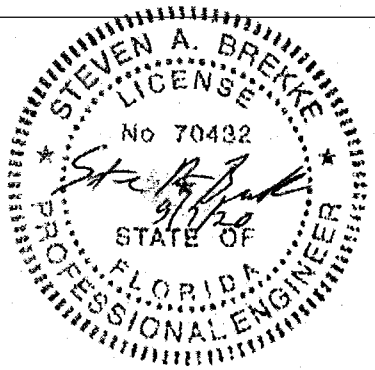


FL NOA 2033 R7

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MiTek®

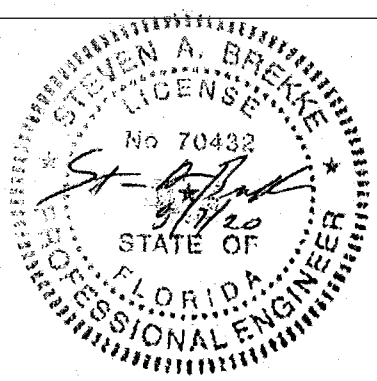
Compliance Statement:	The product as described in this report has demonstrated compliance with the Florida Building Code 2020, Sections 2304.10.3 and 2314.
Product Description:	The UGTS & USC is an uplift girder truss tie and consists of a steel bucket welded to two straps to connect a wood member to a masonry wall. The connector is used to transfer uplift load between wood members and masonry walls.
Product Material:	Strap: 10 Ga. steel ASTM A1011-40 Seat: 10 Ga. steel ASTM A1011-40
Materials:	**Wood** Wood members must be Southern Pine with a minimum specific gravity (G) of 0.55, or an approved member with an equivalent specific gravity (G) of 0.55. Wood members shall have a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). Wood members must be used under continuously dry conditions and where sustained temperatures are limited to 100° F. **Preservative and Fire-Retardant-treated Wood** Connectors or fasteners used in contact with preservative-treated and fire-retardant-treated wood shall be in accordance with Section 2304.10.5. **Nail** Nails shall meet the requirements in the Table and the requirements specified in ASTM F 1667. **Fasteners** Required fastener types and sizes are specified in the Table. **Masonry** Material and quality of masonry construction must comply with the applicable sections of FBC 2020.



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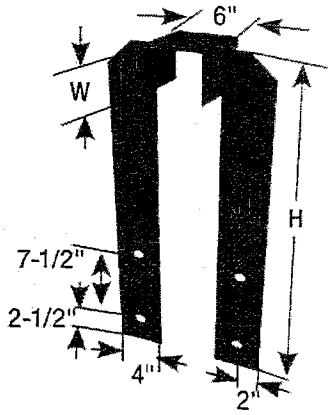
Design Procedure:	The allowable loads shown in the Table are based on allowable stress design and include a 60% increase for wind and seismic load conditions per Section 11.3.2 of the NDS. No further increases are permitted. The allowable loads are based on wood members with a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). If moisture content exceeds this value, a wet service factor, C_m, must be applied per Section 11.3.3 of the NDS. The allowable loads are based on wood members under continuously dry conditions and where sustained temperatures are limited to 100° F. Wood members that will experience sustained exposure to temperatures exceeding 100°F shall be adjusted by the temperature factor, C_t, per Section 11.3.4 of the NDS. Wood members to which the anchor is attached must be designed by a registered design professional per FBC 2020. Fasteners shall be installed to fully grouted and reinforced masonry units (CMU) type S or better mortar, or reinforced concrete ($f'c$ = 2,500 psi at 28 days) designed by a registered design professional per FBC 2020.
Installation:	Install the anchor in accordance with this evaluation report and the manufacturer's installation instructions.
Testing and Specification:	1. ASTM D1761-88(00) Standard Test Methods for Mechanical Fasteners in Wood. Test Report No.3018 02-50883.3 dated 11/19/2002 by Twin City Testing Corporation Test Report No.031205 dated 11/30/2001 and revised 04/18/2002 by Twin City Testing Corporation 2. ASTM D 1761 to ASTM D7147-11 comparison letter, sealed by Steven Brekke, P.E. 3. ANSI/AWC National Design Specifications for Wood Construction (NDS) 2018
Quality Assurance Entity:	The manufacturer has established compliance of roof panel products in accordance with Florida Building Code and Rule 61G20-3.005 (3) for the manufacturing assurance program audited by an approved quality assurance entity.



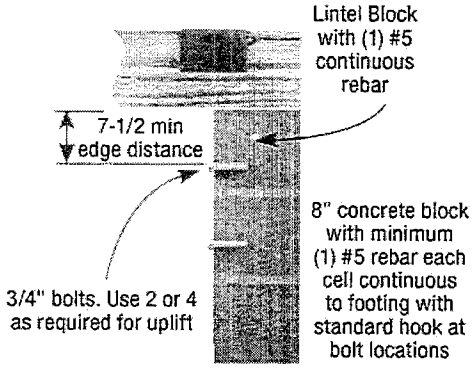
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Description	USP Stock No.	Steel Gauge	Dimensions		Fastener Schedule[2,3,4]				Allowable Loads (Lbs.)
					Rafter/Truss[5]		Concrete / Masonry Wall		DF-L / SP
			W	H	Qty	Type	Qty	Bolt Dia.	Uplift[1] 160%
3 Ply Flat	UGTS3F	10	4-3/4	23	8	16d	2	3/4	7813
	USC3F	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
4 Ply Flat	UGTS4F	10	6-1/2	23	8	16d	2	3/4	7813
	USC4F	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
4/12 pitch	UGTS43	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS44	10	6-1/2	23	8	16d	2	3/4	7813
	USC43	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC44	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
5/12 pitch	UGTS53	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS54	10	6-1/2	23	8	16d	2	3/4	7813
	USC53	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC54	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
6/12 pitch	UGTS63	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS64	10	6-1/2	23	8	16d	2	3/4	7813
	USC63	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC64	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
7/12 pitch	UGTS73	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS74	10	6-1/2	23	8	16d	2	3/4	7813
	USC73	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC74	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
8/12 pitch	UGTS83	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS84	10	6-1/2	23	8	16d	2	3/4	7813
	USC83	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC84	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133

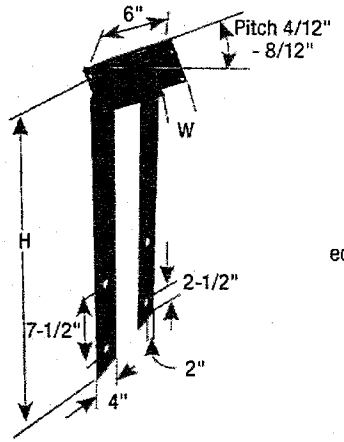
1) Uplift loads have been increased 60% for wind or seismic loads; no further increase shall be permitted.
2) Use Powers Fasteners 3/4" - 6" Wedge Bolts; or equal, installed in accordance with manufacturer's specifications.
3) Fasteners shall be installed to fully grouted and reinforced concrete masonry or reinforced concrete (f'c = 2500 psi at 28 days).
4) Bolts shall conform to ASTM A 307 or better.
5) Minimum nail embedment shall be 1-5/8" for 16d nails.



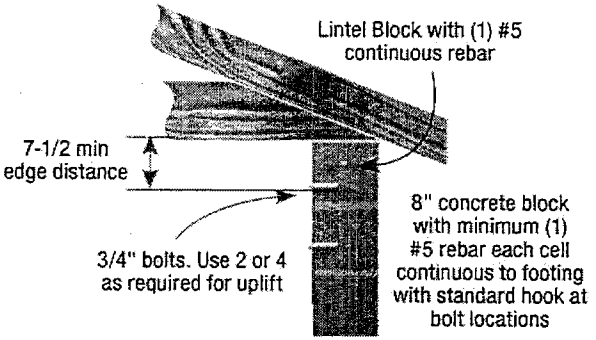
USC F Uplift Girder Tie (4-bolt design)



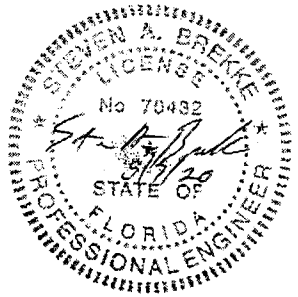
Typical USC F installation (UGTS F similar)



USC Uplift Girder Tie (4-bolt design)



Typical USC installation (UGTS similar)



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mitek-us.com

Name: UGTS & USC Uplift Girder Ties

Sheet 1	Steven Brekke PE
Date:	PROFESSIONAL ENGINEER
Aug 7, 2020	License #: 70432

MiTek®
Residential North America

Product Evaluation Report
MiTek® Inc.
UGTS & USC Uplift Girder Ties

Florida Product Approval # 2033.1-2 R6
Florida Building Code 2020
Per Rule 61G20-3.005(1)(d)
Method 1-d

Category: Structural Components
Subcategory: Wood Connector
Compliance Method: 61G20-3.005
HVHZ

Product Manufacturer:
MiTek Inc.
16023 Swingley Ridge Rd
Chesterfield, MO 63017

Engineer Evaluator:
Steven Brekke P.E.
FL P.E. #70432

Validator:
Johnathan Green, P.E.
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Contents:
Evaluation Report Pages 1-4



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Compliance Statement:	The product as described in this report has demonstrated compliance with the Florida Building Code 2020, Sections 2304.10.3 and 2314.
Product Description:	The UGTS & USC is an uplift girder truss tie and consists of a steel bucket welded to two straps to connect a wood member to a masonry wall. The connector is used to transfer uplift load between wood members and masonry walls.
Product Material:	Strap: 10 Ga. steel ASTM A1011-40 Seat: 10 Ga. steel ASTM A1011-40
Materials:	**Wood** Wood members must be Southern Pine with a minimum specific gravity (G) of 0.55, or an approved member with an equivalent specific gravity (G) of 0.55. Wood members shall have a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). Wood members must be used under continuously dry conditions and where sustained temperatures are limited to 100° F. **Preservative and Fire-Retardant-treated Wood** Connectors or fasteners used in contact with preservative-treated and fire-retardant-treated wood shall be in accordance with Section 2304.10.5. **Nail** Nails shall meet the requirements in the Table and the requirements specified in ASTM F 1667. **Fasteners** Required fastener types and sizes are specified in the Table. **Masonry** Material and quality of masonry construction must comply with the applicable sections of FBC 2020.



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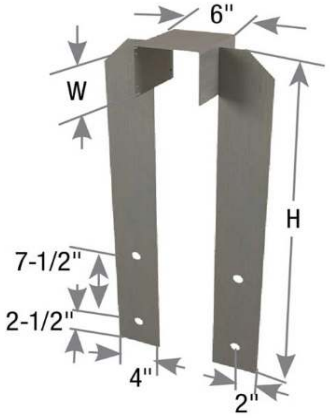
Design Procedure:	The allowable loads shown in the Table are based on allowable stress design and include a 60% increase for wind and seismic load conditions per Section 11.3.2 of the NDS. No further increases are permitted. The allowable loads are based on wood members with a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). If moisture content exceeds this value, a wet service factor, C_m, must be applied per Section 11.3.3 of the NDS. The allowable loads are based on wood members under continuously dry conditions and where sustained temperatures are limited to 100° F. Wood members that will experience sustained exposure to temperatures exceeding 100°F shall be adjusted by the temperature factor, C_t, per Section 11.3.4 of the NDS. Wood members to which the anchor is attached must be designed by a registered design professional per FBC 2020. Fasteners shall be installed to fully grouted and reinforced masonry units (CMU) type S or better mortar, or reinforced concrete (f'c = 2,500 psi at 28 days) designed by a registered design professional per FBC 2020.
Installation:	Install the anchor in accordance with this evaluation report and the manufacturer's installation instructions.
Testing and Specification:	1. ASTM D1761-88(00) Standard Test Methods for Mechanical Fasteners in Wood. Test Report No.3018 02-50883.3 dated 11/19/2002 by Twin City Testing Corporation Test Report No.031205 dated 11/30/2001 and revised 04/18/2002 by Twin City Testing Corporation 2. ASTM D 1761 to ASTM D7147-11 comparison letter, sealed by Steven Brekke, P.E. 3. ANSI/AWC National Design Specifications for Wood Construction (NDS) 2018
Quality Assurance Entity:	The manufacturer has established compliance of roof panel products in accordance with Florida Building Code and Rule 61G20-3.005 (3) for the manufacturing assurance program audited by an approved quality assurance entity.



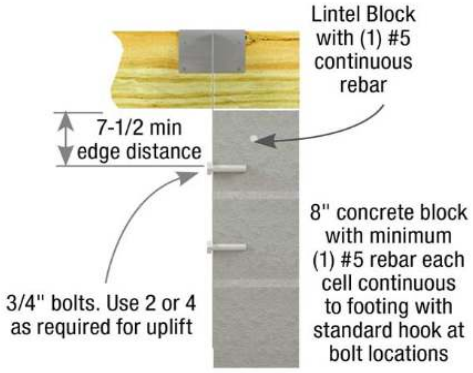
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Description	USP Stock No.	Steel Gauge	Dimensions		Fastener Schedule[2,3,4]				Allowable Loads (Lbs.)
					Rafter/ Truss[5]		Concrete / Masonry Wall		DF-L / SP
									Uplift[1]
			W	H	Qty	Type	Qty	Bolt Dia.	160%
3 Ply Flat	UGTS3F	10	4-3/4	23	8	16d	2	3/4	7813
	USC3F	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
4 Ply Flat	UGTS4F	10	6-1/2	23	8	16d	2	3/4	7813
	USC4F	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
4/12 pitch	UGTS43	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS44	10	6-1/2	23	8	16d	2	3/4	7813
	USC43	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC44	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
5/12 pitch	UGTS53	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS54	10	6-1/2	23	8	16d	2	3/4	7813
	USC53	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC54	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
6/12 pitch	UGTS63	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS64	10	6-1/2	23	8	16d	2	3/4	7813
	USC63	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC64	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
7/12 pitch	UGTS73	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS74	10	6-1/2	23	8	16d	2	3/4	7813
	USC73	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC74	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
8/12 pitch	UGTS83	10	4-3/4	23	8	16d	2	3/4	7813
	UGTS84	10	6-1/2	23	8	16d	2	3/4	7813
	USC83	10	4-3/4	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133
	USC84	10	6-1/2	30-1/2	8	16d	2	3/4	7813
					8	16d	4	3/4	10133

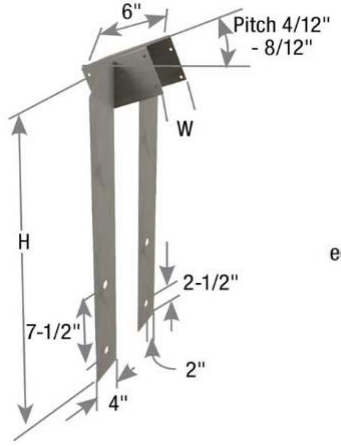
1) Uplift loads have been increased 60% for wind or seismic loads; no further increase shall be permitted.
2) Use Powers Fasteners 3/4″ - 6″ Wedge Bolts; or equal, installed in accordance with manufacturer's specifications.
3) Fasteners shall be installed to fully grouted and reinforced concrete masonry or reinforced concrete (f'c = 2500 psi at 28 days).
4) Bolts shall conform to ASTM A 307 or better.
5) Minimum nail embedment shall be 1-5/8″ for 16d nails.



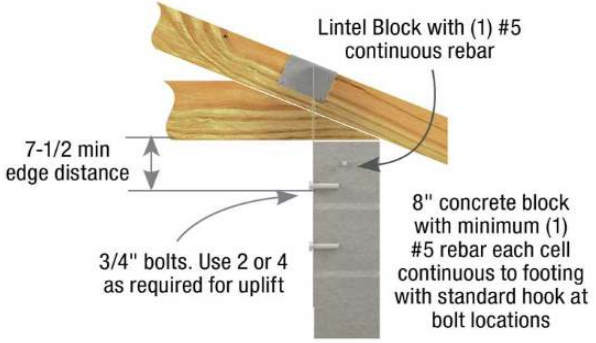
USC F Uplift Girder Tie (4-bolt design)



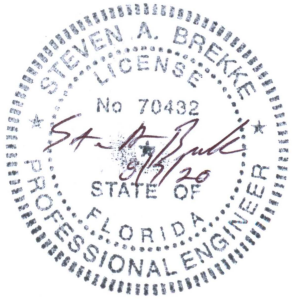
Typical USC F installation (UGTS F similar)



USC Uplift Girder Tie (4-bolt design)



Typical USC installation (UGTS similar)



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Name: UGTS & USC Uplift Girder Ties

Sheet 1	Steven Brekke PE
Date:	PROFESSIONAL ENGINEER
Aug 7, 2020	License #: 70432

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ESR-2552

Reissued March 2020
Revised December 2020
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DIVISION: 06 00 00—WOOD, PLASTICS AND COMPOSITES
Section: 06 05 23—Wood, Plastic, and Composite Fastenings

REPORT HOLDER:

SIMPSON STRONG-TIE COMPANY INC.

EVALUATION SUBJECT:

SIMPSON STRONG-TIE® FACE-MOUNT HANGERS FOR SAWN LUMBER, STRUCTURAL COMPOSITE LUMBER (SCL), PREFABRICATED WOOD I-JOISTS AND GLULAM BEAMS (ENGINEERED WOOD PRODUCTS)

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021, 2018, 2015, 2012 and 2009 *International Building Code®* (IBC)
- 2021, 2018, 2015, 2012 and 2009 *International Residential Code®* (IRC)

For evaluation for compliance with codes adopted by the Los Angeles Department of Building and Safety (LADBS), see ESR-2552 LABC and LARC Supplement.

Property evaluated:

Structural

2.0 USES

The Simpson Strong-Tie® face-mount hangers described in this report are used as wood framing connectors in wood construction in accordance with Section 2304.10.4 of the 2021 IBC, 2304.10.3 of the 2018 and 2015 IBC and Section 2304.9.3 of the 2012 and 2009 IBC. The face-mount hangers may also be used in structures regulated under the IRC when an engineered design is submitted in accordance with Section R301.1.3 of the IRC.

3.0 DESCRIPTION

3.1 General:

With the exception of the CBH Hangers Series, the Simpson Strong-Tie face-mount hangers described in this report are U-shaped hangers that have prepunched holes for the installation of nails or Simpson Strong-Drive SDS screws, depending on the hanger model, into the face of the supporting wood header or beam or ledger. The CBH Hanger is a die-formed steel plate with a welded stud and prepunched holes for the installation of Simpson Strong-Drive SDS screws.

3.1.1: LU Series Hangers: The LU series hangers are formed from No. 20 gage galvanized steel. See Table 1 for hanger dimensions, required fasteners, and allowable loads; and Figure 1 for a drawing of a typical LU series hanger.

3.1.2 LUCZ Series Hangers: The LUCZ hangers have concealed flanges to allow for installation near the end of a supporting member such as a ledger or header. The hangers are formed from No. 18 gage galvanized steel coated with a G185 zinc coating. See Table 2 and Figure 2 for hanger dimensions, required fastener schedule, allowable loads and a typical installation detail.

3.1.3 Hangers: The U series hangers are formed from No. 16 gage galvanized steel. The hangers are face-nailed to the supporting wood header, and nailed to the supported solid sawn lumber, structural composite lumber or prefabricated wood I-joists with web stiffeners to accept the required size and number of joist nails shown in Table 3. See Table 3 for the U series hanger model numbers, hanger dimensions, required fasteners, and allowable loads. See Figure 3 for a drawing of a typical U series joist hanger.

3.1.4 HU and HUC Series Hangers: The HU and HUC series hangers are formed from No. 14 gage galvanized steel, and are face-nailed to the supporting wood header/beam and nailed to the solid sawn lumber, supported structural composite lumber or prefabricated wood I-joists with web stiffeners. HU hangers having a seat width (W) equal to or greater than $2^{9}/_{16}$ inches (65 mm) are available with concealed flanges and are specified with the model designation HUC. See Table 4 for hanger model numbers, hanger dimensions, required fasteners (minimum and maximum) and allowable loads based on the minimum and maximum nailing. The HU and HUC hangers have triangular and round holes in both the U-shaped portion supporting the wood joist and in the flanges attached to the supporting wood header/beam. The minimum allowable loads are achieved by filling only the round holes with the type and size of nails shown in Table 4, and the maximum allowable loads are achieved by filling all holes (both the round and triangular) with size of nails shown in Table 4. See Figures 4A and 4B for drawings of typical HU and HUC hangers, and Figure 4C shows a typical installation of an HU hanger supporting a prefabricated wood I-joist with approved web stiffeners.

3.1.5 HUCQ Series Hangers: The HUCQ series hangers are formed from No. 14 gage galvanized steel and are used to support glulam beams or other approved wood members. Other approved wood members must be laterally supported at the ends in accordance with Section 2308.4.6 of the IBC and Section R502.7 of the IRC. The hangers have

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prepunched holes for the installation of Simpson Strong-Drive SDS series wood screws (SDS). See Table 5 for the HUCQ series model numbers, hanger dimensions, required fasteners and allowable loads. See Figure 5 for a drawing of the HUCQ series hanger.

3.1.6 LUS Series Hangers: The LUS series hangers are formed from No. 18 gage galvanized steel. The hangers have prepunched holes for the installation of nails that are driven at a 45-degree angle through the joist and into the header, which is described as double shear nailing in the installation instructions. See Table 6 for the hanger dimensions, required fasteners, and allowable loads; and Figure 6 for a drawing of a typical LUS series hanger.

3.1.7: MUS Joist Hanger: The MUS series hangers are formed from No. 18 gage galvanized steel. The U-shaped portion of the hangers has prepunched holes for the installation of joist nails that are driven at an angle through the joist and into the header, which is described as double shear nailing in the installation instructions. See Table 7 for the hanger dimensions, required fasteners, and allowable loads; Figure 7 for a drawing of a typical MUS series hanger.

3.1.8 HUS and HUSC Series Hangers: The HUS and HUSC series hangers having a seat width, *W*, of $3^{9}/_{16}$ inches (90 mm) are formed from No. 14 gage galvanized steel. The HUS1.81/10 hanger with a seat width, *W*, of $1^{13}/_{16}$ inches (46 mm) is formed from No. 16 gage galvanized steel. The HUS models having a seat width, *W*, of $3^{9}/_{16}$ inches (90 mm) are available with concealed flanges, which are specified with the model designation HUSC. The hangers have prepunched holes for the installation of nails that are driven at a 45-degree angle through the joist and into the header, which is described as double shear nailing in the installation instructions. See Table 8 for HUS series hanger models, hanger dimensions, required fasteners, and allowable loads. See Figure 8 for a drawing of a typical HUS hanger.

3.1.9 HHUS Series Hangers: The HHUS series hangers are formed from No. 14 gage galvanized steel. The hangers have pre-punched holes for the installation of nails that are driven at a 45 degree angle through the joist and into the header, which is described as double shear nailing in the installation instructions. See Table 9 for the hanger model numbers, hanger dimensions, required fasteners, and allowable loads. See Figure 9 for a drawing of a typical HHUS hanger.

3.1.10 HGUS Series Hangers: The HGUS series hangers are formed from No. 12 gage galvanized steel and are used to support glulam beams or other approved wood members. Other approved wood members must be laterally supported at the ends in accordance with Section 2308.4.6 of the IBC and Section R502.7 of the IRC. The hangers have prepunched holes for the installation of nails that are driven at a 45-degree angle through the joist and into the header, which is described as double shear nailing in the installation instructions. See Table 10 for the HGUS series hanger model numbers, hanger dimensions, required fasteners and allowable loads. See Figure 10 for a drawing of a typical HGUS hanger.

3.1.11 HTU Series Hangers: The HTU hangers are designed to support trusses installed with full or partial heel heights and gaps between the truss and the supporting girders of up to, but not exceeding, $^{1}/_{2}$ inch (12.7 mm), as shown in Tables 11A and 11C, and $^{1}/_{8}$ inch (3.2 mm) as shown in Table 11B. Minimum and maximum nailing options are given in Tables 11A, 11B, and 11C to address varying heel heights and support conditions. The HTU hangers are formed from No. 16 gage galvanized steel. See Table 11A and Figures 11A and 11B for hanger dimensions, required fastener schedule, allowable loads and an installation detail for installations in which the gap between the truss and the supporting girders is less than or equal to $^{1}/_{2}$ inch (12.7 mm). See Table 11B and Figures 11A and 11B for hanger dimensions, required fastener schedule, allowable loads and an installation detail for installations in which the gap between the truss and the supporting girders is less than or equal to $^{1}/_{8}$ inch (3.2 mm). See Table 11C and Figures 11A and 11C for hanger dimensions, required fastener schedule, allowable loads and an installation detail for installations in which the minimum allowable number of nails is driven into the supporting girder, and the gap between the truss and supporting girder is less than or equal to $^{1}/_{2}$ inch (12.7 mm).

3.1.12 LGU, MGU, HGU and HHGU Series Hangers: The LGU and MGU series hangers are formed from No. 10 gage galvanized steel; HGU series hangers are formed from No. 7 gage galvanized steel; and HHGU series hangers are formed from No. 3 gage steel. The LGU, MGU, HGU and HHGU series hangers are used to support glulam beams or other approved wood members. Other approved wood members must be laterally supported at the ends in accordance with Section 2308.4.6 of the IBC and Section R502.7 of the IRC. The hangers have pre-punched holes for the installation of Simpson Strong-Drive SDS series wood screws (SDS). See Table 12 for the LGU, MGU, HGU and HHGU series model numbers, hanger dimensions, required fasteners and allowable loads. See Figure 12 for a drawing of the HHGU series hanger and a typical installation detail.

3.1.13 SUR/L Series Hangers: The SUR/L series hangers are formed from No. 16 gage galvanized steel. The SUR and SUL hangers are identical except they are skewed at 45 degrees to the right (SUR) and the left (SUL), respectively. See Table 13 for hanger models, hanger dimensions, required fasteners, and allowable loads. See Figure 13A for a drawing of a typical SUL hanger and Figure 13B for a typical SUR hanger installation.

3.1.14 HSUR/L Series Hangers: The HSUR/L series hangers are formed from No. 14 gage galvanized steel. The HSUR and HSUL hangers are identical except they are skewed at 45 degrees to the right (HSUR) and the left (HSUL), respectively, and are designed to support prefabricated wood I-joists having approved web stiffeners, and structural composite lumber (SCL). See Table 14 for the hanger model numbers, hanger dimensions, required fasteners, and allowable loads. See Figure 14A for a drawing of a typical HSUR hanger, and Figure 14B for a typical HSUR hanger installation.

3.1.15 IUS Series Hangers: The IUS series hangers are formed from No. 18 gage galvanized steel and are used exclusively to support prefabricated wood I-joists to a supporting wood member. See Table 15A for the IUS series hanger model numbers, hanger dimensions, and required fasteners; and Table 15B for allowable loads. See Figures 15A, 15B, and 15C for drawings of a typical IUS hanger and a typical IUS hanger installation.

3.1.16 MIU Series Joist Hangers: The MIU series hangers are formed from No. 16 gage galvanized steel and are used to support prefabricated wood I-joists or other approved wood members. Other approved wood members must be laterally supported at the ends in accordance with Section 2308.8.2 of the IBC and Section R502.7 of the IRC. See Table 16 for the MIU series hanger model numbers, hanger dimensions, required fasteners, and allowable loads. See Figure 16A for a drawing of a typical MIU hanger and Figure 16B for a drawing of a typical MIU hanger installation.

3.1.17 DU, DHU and DHUTF Series Hangers: The DU, DHU and DHUTF series hangers are face-mount and top-flange hangers that are formed from either No. 14 gage or No. 12 gage galvanized steel. They are designed to transfer joist floor loads to a wood stud wall through two layers of $^5/_8$-inch-thick (15.9 mm) gypsum board complying with ASTM C1396, and are installed using a combination of nails and Simpson Strong-Drive SDS series wood screws (SDS), which are recognized under ESR-2236. These hangers are installed after the two layers of $^5/_8$" Type X gypsum board is installed and fastened using, at a minimum, the fastener schedule in Item 14.1-5 in IBC Table 721.1(2). See Table 17A for hanger dimensions and allowable loads. See Table 17B for fastener schedule. See Figure 17 for a drawing of the DU, DHU and DHUTF hangers and a typical installation.

3.1.18 CBH Series Hangers: The CBH series hangers are face-mounted and installed in inverted pairs. They are formed from No. 3 gage steel. They are used to support glulam beams or other approved wood members and are installed using Simpson Strong-Drive SDS series wood screws (SDS), which are recognized under ESR-2236. See Table 18A & 18B for hanger dimensions, fastener schedules and allowable loads. See Figure 18 for a drawing of the CBH hangers and a typical installation.

3.2 Materials:

3.2.1 Steel: All hangers described in this report, with the exception of the CBH, HTU, HGUS and HHGU series hangers, are manufactured from galvanized steel complying with ASTM A653, SS designation, Grade 33, with a minimum yield strength, F_y, of 33,000 psi (227 MPa) and a minimum tensile strength, F_u, of 45,000 psi (310 MPa). The HTU and HGUS series hangers are manufactured from galvanized steel complying with ASTM A653, SS designation, Grade 40, with a minimum yield strength, F_y, of 40,000 psi (276 MPa) and a minimum tensile strength, F_u, of 55,000 psi (379 MPa). The HHGU and CBH series hangers are manufactured from nongalvanized steel complying with ASTM A1011 SS, Grade 33, with a minimum yield strength, F_y, of 33,000 psi (227 MPa) and a minimum tensile strength, F_u, of 52,000 psi (358 MPa). Base-metal thicknesses for the hangers in this report are as follows:

NOMINAL THICKNESS (gage)	MINIMUM BASE-METAL THICKNESS (inch)
No. 3	0.2285
No. 7	0.1715
No. 10	0.1275
No. 12	0.0975
No. 14	0.0685
No. 16	0.0555
No. 18	0.0444
No. 20	0.0335

For **SI:** 1 inch = 25.4 mm.

The galvanized zinc coating conforms to ASTM A924 and ASTM A653 with a G90 designation. Some models (designated with a model number ending with Z) are available with a G185 zinc coating specification in accordance with ASTM A653. Some models (designated with a model number ending with HDG) are available with a hot-dip galvanization, also known as "batch" galvanization, in accordance with ASTM A123, with a minimum specified coating weight of 2.0 ounces of zinc per square foot of surface area (600 g/m^2), total for both sides. With the exception of the LUCZ, model numbers in this report do not include the Z or HDG ending, but the information shown applies. The lumber treater or holder of this report (Simpson Strong-Tie Company) should be contacted for recommendations on minimum corrosion resistance of steel connectors in contact with the specific proprietary preservative treated or fire retardant treated lumber.

3.2.2 Wood: Wood headers/beams to which the connectors are face-nailed must be either sawn lumber, structural glued-laminated timber, or engineered lumber having a minimum specific gravity of 0.50 (minimum equivalent specific gravity of 0.50 for engineered lumber), and having a maximum moisture content of 19 percent (16 percent for engineered lumber) except as noted in Section 4.1. The thickness of the supporting wood member (header) must be equal to or greater than the length of the fasteners specified in the tables in this report, or as required by wood member design, whichever is greater.

Supported wood members that are prefabricated wood I-joists or SCL must be recognized in a current evaluation report, which will specify the allowable shear capacity and allowable reactions at supports for the proprietary engineered wood lumber. When required, web stiffeners for prefabricated wood I-joists must comply with specifications noted in the applicable evaluation report for the I-joists. The hangers described in this evaluation report may support I-joists having a current ICC-ES evaluation report. The evaluation report for the prefabricated wood I-joist must specify a minimum bearing length that is equal to or less than the hanger bearing length. When the hangers are used to support prefabricated wood I-joists, joist end reactions must not exceed the allowable joist end reaction per the I-joist manufacturer's code report. For installation with engineered wood members, minimum allowable fastener spacing and end and edge distances, as specified in the applicable evaluation report for the engineered wood product, must be met.

3.2.3 Fasteners: Nails used for hangers described in this report must comply with ASTM F1667 and have the following minimum fastener dimensions and bending yield strengths (F_{yb}):

FASTENERS	SHANK DIAMETER (inches)	NAIL LENGTH (inches)	F_{yb} (psi)
10d × $1^1/_2$	0.148	$1^1/_2$	90,000
10d	0.148	3	90,000
16d × $2^1/_2$	0.162	$2^1/_2$	90,000
16d	0.162	$3^1/_2$	90,000

For **SI:** 1 inch = 25.4 mm, 1 psi = 6.895 kPa.

Some of the joist hangers described in this report are attached to the wood members using Simpson Strong-Drive SDS series wood screws as described in ESR-2236.

Fasteners used in contact with preservative treated or fire retardant treated lumber must comply with Section 2304.10.6 of the 2021 IBC, 2304.10.5 of the 2018 and 2015 IBC, Section 2304.9.5 of the 2012 and 2009 IBC, 2015, and applicable sections of the IRC. The chemical treatment manufacturer or this report holder (Simpson Strong-Tie Company) should be contacted for recommendations on minimum corrosion resistance of fasteners and connection capacities of fasteners used with the specific proprietary preservative treated or fire retardant treated lumber. Unless otherwise indicated, fasteners listed in this report as 10d and 16d are 10d common and 16d common nails, respectively.

4.0 DESIGN AND INSTALLATION

4.1 Design:

The tabulated allowable loads shown in this report are based on allowable stress design (ASD) and include the load duration factor, C_D, corresponding with the applicable loads in accordance with the NDS.

Tabulated allowable loads apply to products connected to wood used under dry conditions and where sustained temperatures are 100ºF (37.8ºC) or less. When products are installed to wood having a moisture content greater than 19 percent (16 percent for engineered wood products), or where wet service is expected, the allowable loads must be adjusted by the wet service factor, C_M, specified in the NDS. When connectors are installed in wood that will experience sustained exposure to temperatures exceeding 100ºF (37.8ºC), the allowable loads in this report must be adjusted by the temperature factor, C_t, specified in the NDS.

Connected wood members must be analyzed for load-carrying capacity at the connection in accordance with the NDS and the applicable evaluation report for engineered wood products.

4.2 Installation:

Installation of the connectors must be in accordance with this evaluation report and the manufacturer's published installation instructions. In the event of a conflict between this report and the manufacturer's published installation instructions, this report governs.

5.0 CONDITIONS OF USE

The Simpson Strong-Tie face-mount hangers described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 The connectors must be manufactured, identified and installed in accordance with this report and the manufacturer's published installation instructions. A copy of the instructions must be available at the jobsite at all times during installation.

5.2 Calculations showing compliance with this report must be submitted to the code official. The calculations must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

5.3 Adjustment factors noted in Section 4.1 and the applicable codes must be considered, where applicable.

5.4 Connected wood members and fasteners must comply, respectively, with Sections 3.2.2 and 3.2.3 of this report.

5.5 Use of connectors with preservative or fire retardant treated lumber must be in accordance with Section 3.2.1 of this report. Use of fasteners with preservative treated or fire retardant treated lumber must be in accordance with Section 3.2.3 of this report.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Joist Hangers and Similar Devices (AC13), dated March 2018 (editorially revised December 2020).

Tests and calculations in accordance with the ICC-ES Acceptance Criteria for Dowel-Type Threaded Fasteners used in Wood (AC233), dated February 2020.

7.0 IDENTIFICATION

7.1 The products described in this report are identified with a die-stamped or an adhesive label indicating the name of the manufacturer (Simpson Strong-Tie) the model number, and the number of an index evaluation report (ESR-2523) that is used as an identifier for the products recognized in this report.

7.2 The report holder's contact information is the following:

SIMPSON STRONG TIE COMPANY INC.
5956 WEST LAS POSITAS BOULEVARD
PLEASANTON, CALIFORNIA 94588
(800) 999-5099
www.strongtie.com

TABLE 1—ALLOWABLE LOADS FOR THE LU SERIES JOIST HANGERS

MODEL No.	DIMENSIONS[1] (inches)			FASTENERS[2] (Quantity-Type)		ALLOWABLE LOADS[3,4,5] (lbf)						
						Uplift[6]	Download					
	W	H	B	Header[5]	Joist	C_D = 1.6	C_D = 1.0		C_D = 1.15		C_D = 1.25	
							10d	16d	10d	16d	10d	16d
LU24	$1^{9}/_{16}$	$3^{1}/_{8}$	$1^{1}/_{2}$	4	2-10d x $1^{1}/_{2}$	240	465	555	530	630	570	655
LU26	$1^{9}/_{16}$	$4^{3}/_{4}$	$1^{1}/_{2}$	6	4-10d x $1^{1}/_{2}$	540	695	835	800	950	860	1,030
LU28	$1^{9}/_{16}$	$6^{3}/_{8}$	$1^{1}/_{2}$	8	6-10d x $1^{1}/_{2}$	850	930	1,110	1,065	1,180	1,145	1,180
LU210	$1^{9}/_{16}$	$7^{13}/_{16}$	$1^{1}/_{2}$	10	6-10d x $1^{1}/_{2}$	850	1,160	1,390	1,330	1,580	1,430	1,615

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 1 for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]LU Series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]The quantity of 10d or 16d common nails specified in the "Header" column under "Fasteners" is required to achieve the tabulated allowable loads shown in the Allowable Download "10d" or "16d" columns.
[6]Allowable uplift loads are for hangers installed with either 10d or 16d common nails into the supporting header/beam, and have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.

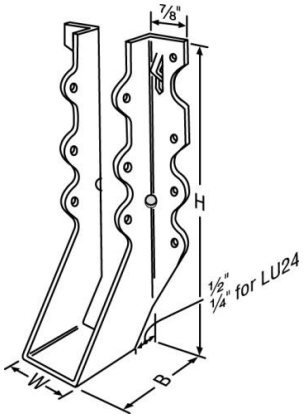


FIGURE 1—LU SERIES HANGER
(See Table 1)

TABLE 2—DIMENSIONS, NAILING SCHEDULES AND DESIGN VALUES FOR LUCZ SERIES HANGERS

MODEL No.	DIMENSIONS (inches)		FASTENERS [1] (Quantity-Type)		ALLOWABLE LOADS [2, 3, 4, 5] (lbf)					
	W	H	Into Supporting Member	Into Supported Member	Uplift [6]	Download				
					C_D=1.60	C_D =0.9	C_D =1.0	C_D =1.15	C_D =1.25	C_D =1.60
LUC26Z	$1^9/_{16}$	$4^3/_4$	6 - 10dx$1^1/_2$	4 - 10d x $1^1/_2$	730	640	710	810	875	1,100
			6 - 10d		730	640	710	810	875	1,100
			6 - 16d		730	760	845	965	1,040	1,315
LUC210Z	$1^9/_{16}$	$7^3/_4$	10 - 10d x $1^1/_2$	6 - 10d x $1^1/_2$	985	1,065	1,185	1,345	1,455	1,830
			10 - 10d		985	1,065	1,185	1,345	1,455	1,830
			10 - 16d		985	1,270	1,410	1,605	1,735	2,180

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]Allowable loads correspond to installations where all pre-punched nail holes in the hanger are filled with nails. Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[2]Tabulated allowable loads are for installations in wood members complying with Section 3.2.2 of this report.
[3]Tabulated loads must be selected based on the applicable load duration factor, C_D, as permitted by the applicable building code. See Sections 4.1 and 4.2 for design and installation requirements.
[4]The maximum allowable gap between the joist end and the supporting member is $^1/_8$ inch (3.2 mm).
[5]LUCZ series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm), for nominal 2x6 joists supported by the LUC26Z and nominal 2x10 joists supported by the LUC210Z.
[6]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The tabulated allowable uplift loads must be reduced proportionally when other load durations govern.

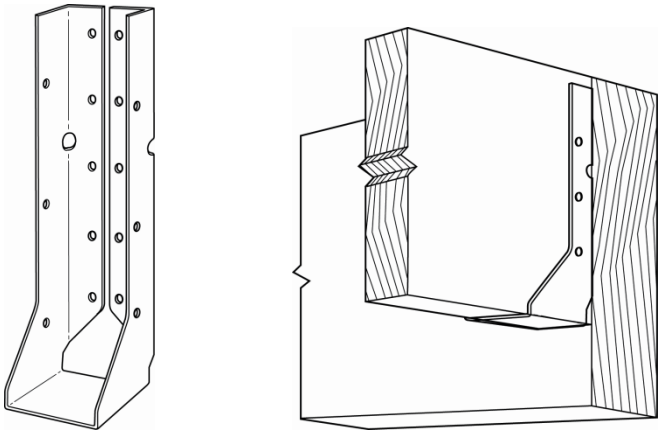


FIGURE 2—LUCZ SERIES HANGER AND INSTALLATION DETAIL

TABLE 3—ALLOWABLE LOADS FOR THE U SERIES JOIST HANGERS

MODEL No.	DIMENSIONS[1] (inches)			FASTENERS[2] (Quantity-Type)		ALLOWABLE LOADS[4,5,6] (lbf)						
						Uplift [6]	Download					
	W	H	B	Header[5]	Joist	C_D = 1.6	C_D = 1.0		C_D = 1.15		C_D = 1.25	
							10d	16d	10d	16d	10d	16d
U24	1 9/16	3 1/8	2	4	2-10d x 1 1/2	240	490	575	550	650	590	705
U26	1 9/16	4 3/4	2	6	4-10d x 1 1/2	535	730	865	830	980	890	1,055
U210	1 9/16	7 13/16	2	10	6-10d x 1 1/2	990	1,220	1,440	1,380	1,565	1,480	1,565
U214	1 9/16	10	2	12	8-10d x 1 1/2	990	1,465	1,730	1,655	1,955	1775	2,110
U14	1 13/16	10 1/4	2	14	6-10d x 1 1/2	970	-	2,015	-	2,285	-	2,465
U24R	2 1/16	3 5/8	2	4	2-10d x 1 1/2	240	490	575	550	650	590	705
U26R	2 1/16	5 5/8	2	8	4-10d x 1 1/2	535	975	1,150	1,105	1,305	1,185	1,410
U210R	2 1/16	9 1/8	2	14	6-10d x 1 1/2	990	1,710	2,015	1,930	2,280	2,070	2,465
U3510/14	2 5/16	9	2	14	6-10d x 1 1/2	970	-	2,015	-	2,285	-	2,465
U3510/20	2 5/16	10 5/16	2	16	6-10d x 1 1/2	970	-	2,305	-	2,615	-	2,820
U34	2 9/16	3 3/8	2	4	2-10d x 1 1/2	240	490	575	550	650	590	705
U36	2 9/16	5 3/8	2	8	4-10d x 1 1/2	535	975	1,150	1,105	1,305	1,185	1,410
U310	2 9/16	8 7/8	2	14	6-10d x 1 1/2	990	1,710	2,015	1,930	2,280	2,070	2,465
U314	2 9/16	10 1/2	2	16	6-10d x 1 1/2	990	1,950	2,305	2,210	2,610	2,370	2,815
U24-2	3 1/8	3	2	4	2-10d	240	490	575	550	650	590	705
U26-2	3 1/8	5	2	8	4-10d	535	975	1,150	1,105	1,305	1,185	1,410
U210-2	3 1/8	8 1/2	2	14	6-10d	990	1,750	2,015	1,930	2,280	2,070	2,465
U44	3 9/16	2 7/8	2	4	2-10d	240	490	575	550	650	590	705
U46	3 9/16	4 7/8	2	8	4-10d	535	975	1,150	1,105	1,305	1,185	1,410
U410	3 9/16	8 3/8	2	14	6-10d	990	1,710	2,015	1,930	2,280	2,070	2,465
U414	3 9/16	10	2	16	6-10d	990	1,950	2,305	2,210	2,610	2,370	2,815
U44R	4 1/16	2 5/8	2	4	2-16d	240	490	575	550	650	590	705
U46R	4 1/16	4 5/8	2	8	4-16d	535	975	1,150	1,105	1,305	1,185	1,410
U410R	4 1/16	8 1/8	2	14	6-16d	990	1,710	2,015	1,930	2,280	2,070	2,465
U26-3	4 5/8	4 1/4	2	8	4-10d	535	975	1,150	1,105	1,305	1,185	1,410
U210-3	4 5/8	7 3/4	2	14	6-10d	990	1,710	2,015	1,930	2,280	2,070	2,465
U3510-2	4 3/4	8 3/4	2	14	6-10d x 1 1/2	970	-	2,015	-	2,285	-	2,465
U3512-2	4 3/4	11 1/4	2	16	6-10d x 1 1/2	970	-	2,305	-	2,615	-	2,820
U66	5 1/2	5	2	8	4-10d	535	975	1,150	1,105	1,305	1,185	1,410
U610	5 1/2	8 1/2	2	14	6-10d	990	1,710	2,015	1,930	2,280	2,070	2,465
U610	5 1/2	8 1/2	2	14	6-10d x 1 1/2	970	1,705	2,015	1,930	2,285	2,075	2,465
U66R	6	5	2	8	4-16d	535	975	1,150	1,105	1,305	1,185	1,410
U610R	6	8 1/2	2	14	6-16d	990	1,710	2,015	1,930	2,280	2,070	2,465

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 3 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]U Series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]The quantity of 10d or 16d common nails specified in the "Header" column under "Fasteners" is required to achieve the tabulated allowable loads shown in the Allowable Download "10d" or "16d" columns.
[6]Allowable uplift loads are for hangers installed with either 10d or 16d common nails into the supporting header/beam, and have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.

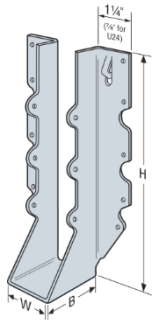


FIGURE 3—U SERIES JOIST HANGER

TABLE 4 - ALLOWABLE LOADS FOR THE HU HANGER SERIES

MODEL NO.	HANGER DIMENSIONS[1] (inches)			FASTENERS[2] (Quantity-Type)		ALLOWABLE LOADS (lbf)[2,3,4]			
	W	H	B	Header	Joist	Uplift[5]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
HU26	1 9/16	3 1/16	2 1/4	4-16d	2-10d x 1 1/2	305	595	670	720
HU28		5 1/4		6-16d	4-10d x 11/2	605	895	1,010	1,080
HU210		7 1/8		8-16d	4-10d x 11/2	605	1,190	1,345	1,440
HU212		9		10-16d	6-10d x 11/2	1,135	1,490	1,680	1,800
HU214		10 1/8		12-16d	6-10d x 11/2	1,135	1,790	2,015	2,160
HU216		12 15/16		18-16d	8-10d x 11/2	1,510	2,680	3,025	3,240
HU1.81/5	1 13/16	5 3/8	2 1/2	16-16d	6-10d x 11/2	915	2,380	2,685	2,890
HU7		6 11/16		16-16d	8-10d x 11/2	1,515	2,380	2,685	2,890
HU9		9 5/16		24-16d	10-10d x 11/2	1,795	3,570	4,030	4,335
HU11		11 1/16		30-16d	10-10d x 11/2	1,795	4,465	4,705	4,810
HU14		13 11/16		36-16d	14-10d x 11/2	1,795	5,055	5,275	5,420
HU2.1/9	2 1/8	9 3/16	2 1/2	14-16d	6-10d x 11/2	915	2,085	2,350	2,530
HU2.1/11		11		16-16d	6-10d x 11/2	915	2,380	2,685	2,890
HU359	2 3/8	8 15/16	2 1/2	18-16d	10-10d x 11/2	1,795	2,680	3,020	3,250
HU3511		11 1/16		22-16d	10-10d x 11/2	1,795	3,275	3,695	3,970
HU3514		13 1/2		24-16d	12-10d x 11/2	1,795	3,570	4,030	4,335
HU3516/22		14 1/4		20-16d	8-10d x 11/2	1,515	2,975	3,360	3,610
HU3524/30		18		24-16d	14-10d x 11/2	1,795	3,570	4,030	4,335
HU34	2 9/16	3 3/8	2 1/2	4-16d	2-10d x 11/2	380	595	670	720
HU36		5 3/16		8-16d	4-10d x 11/2	605	1,190	1,345	1,440
HU38		6 9/16		10-16d	4-10d x 11/2	605	1,490	1,680	1,800
HU310		8 7/8		14-16d	6-10d x 11/2	915	2,085	2,350	2,530
HU312		10 5/8		16-16d	6-10d x 11/2	915	2,380	2,685	2,890
HU314		12 3/8		18-16d	8-10d x 11/2	1,515	2,680	3,020	3,250
HU316		14 1/8		20-16d	8-10d x 11/2	1,515	2,975	3,360	3,610
HU2.75/10	2 3/4	9	2 1/2	18-16d	10-10d x 11/2	1,795	2,680	3,020	3,250
HU2.75/12		10 3/4		22-16d	10-10d x 11/2	1,795	3,275	3,695	3,970
HU2.75/14		13		24-16d	14-10d x 11/2	1,795	3,570	4,030	4,335
HU2.75/16		14 1/16		26-16d	14-10d x 11/2	1,795	3,870	4,365	4,695
HU24-2	3 1/8	3 1/16	2 1/2	4-16d	2-10d	380	595	670	720
HU26-2		4 15/16		8-16d	4-10d	755	1,190	1,345	1,440
HU28-2		6 5/16		10-16d	4-10d	755	1,490	1,680	1,800
HU210-2		8 9/16		18-16d	10-10d	1,795	2,680	3,020	3,250
HU212-2		10 9/16		22-16d	10-10d	1,795	3,275	3,695	3,970
HU214-2		12 1/8		18-16d	8-10d	1,510	2,680	3,025	3,240
HU216-2		13 7/8		26-16d	12-10d	1,795	3,870	4,365	4,695
HU3.25/10.5	3 1/4	9	2 1/2	22-16d	10-10d	1,795	3,275	3,695	3,970
HU3.25/12		11		24-16d	12-10d	1,795	3,570	4,030	4,335

Please refer to next page for continuation of Table 4.

TABLE 4 (CONTINUED) - ALLOWABLE LOADS FOR THE HU HANGER SERIES

MODEL NO.	HANGER DIMENSIONS[1] (inches)			FASTENERS[2] (Quantity-Type)		ALLOWABLE LOADS (lbf)[3,4,5]			
	W	H	B	Header	Joist	Uplift[6]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
HU44	3 9/16	2 7/8	2 1/2	4-16d	2-10d	380	595	670	720
HU46		4 11/16		8-16d	4-10d	755	1,190	1,345	1,440
HU48		6 3/32		10-16d	4-10d	755	1,490	1,680	1,800
HU410		8 3/8		14-16d	6-10d	1,135	2,085	2,350	2,520
HU412		10 5/16		16-16d	6-10d	1,135	2,385	2,690	2,880
HU414		11 7/8		18-16d	8-10d	1,510	2,680	3,025	3,240
HU416		13 5/8		20-16d	8-10d	1,510	2,980	3,360	3,600
HU4.12/9	4 1/8	8 5/8	2 1/2	18-16d	10-10d	1,795	2,680	3,020	3,250
HU4.12/11		10 5/16		22-16d	10-10d	1,795	3,275	3,695	3,970
HU4.28/9	4 9/32	9	2 1/2	18-16d	8-10d	1,515	2,680	3,020	3,250
HU4.28/11		11		22-16d	8-10d	1,515	3,275	3,695	3,970
HU26-3	4 11/16	4 9/16	2 1/2	8-16d	4-10d	755	1,190	1,345	1,440
HU210-3		8		14-16d	6-10d	1,135	2,085	2,350	2,520
HU212-3		9 3/4		16-16d	6-10d	1,135	2,385	2,690	2,880
HU214-3		12 1/16		18-16d	8-10d	1,510	2,680	3,025	3,240
HU216-3		13 7/8		20-16d	8-10d	1,510	2,980	3,360	3,600
HU4.75/9	4 3/4	9	2 1/2	18-16d	8-10d	1,515	2,680	3,020	3,250
HU4.75/11		11		22-16d	8-10d	1,515	3,275	3,695	3,970
HU3514-2		13 1/4		18-16d	8-10d	1,515	2,680	3,020	3,250
HU3516-2		15 1/4		26-16d	12-10d	1,795	3,870	4,365	4,695
HU3520-2		19 1/4		26-16d	12-10d	1,795	3,870	4,365	4,695
HU310-2	5 1/8	7 13/16	2 1/2	14-16d	6-10d	1,135	2,085	2,350	2,530
HU312-2		10 5/8		16-16d	6-10d	1,135	2,380	2,685	2,890
HU314-2		12 5/8		18-16d	8-10d	1,510	2,680	3,020	3,250
HU5.125/12	5 1/4	10 1/4	2 1/2	22-16d	8-16d	1,515	3,275	3,695	3,970
HU5.125/13.5		13 1/4		26-16d	12-16d	2,695	3,870	4,365	4,695
HU5.125/16		13 7/8		26-16d	12-16d	2,695	3,870	4,365	4,695
HU66	5 1/2	4 3/16	2 1/2	8-16d	4-16d	895	1,190	1,345	1,440
HU68		5 13/16		10-16d	4-16d	895	1,490	1,680	1,800
HU610		7 5/8		14-16d	6-16d	1,345	2,085	2,350	2,520
HU612		9 3/8		16-16d	6-16d	1,345	2,385	2,690	2,880
HU614		11 5/8		18-16d	8-16d	1,780	2,680	3,025	3,240
HU616		13 3/8		20-16d	8-16d	1,780	2,980	3,360	3,600
HU210-4	6 1/8	8 3/8	2 1/2	14-16d	6-16d	1,345	2,085	2,350	2,520
HU410-2	7 1/8	8 9/16	2 1/2	18-16d	8-16d	1795	2680	3020	3250
HU412-2		10 13/16		22-16d	8-16d	1795	3275	3695	3970
HU414-2		13 7/8		26-16d	12-16d	2695	3870	4365	4695
HU88	7 1/2	6 5/8	2 1/2	10-16d	4-16d	895	1,490	1,680	1,800
HU810		8 3/8		14-16d	6-16d	1,345	2,085	2,350	2,520
HU812		10 1/8		16-16d	6-16d	1,345	2,385	2,690	2,880
HU814		11 7/8		18-16d	8-16d	1,780	2,680	3,025	3,240
HU816		13 5/8		20-16d	8-16d	1,780	2,680	3,360	3,600

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figures 4a and 4b for definitions of hanger nomenclature (W, H, B).
[2]The Fastener column with "minimum quantity" refers to hangers installed with the designated type of nails into only round pre-punched holes of the hanger, and the Fastener column with "maximum quantity" refers to HU series and HUC series hangers installed with the designated type of nails into both round and triangle pre-punched holes of the hanger. The hanger size and fastener quantity are used to determine allowable loads listed in the table.
[3]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[4]HU series hangers with widths (W) equal to or greater than 2 $^{9}/_{16}$ inches (65 mm) are available with header flanges turned in (concealed) and are identified with the model designation HUC#. See Figure 4b.
[5]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.

[6]HU Series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[7]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.

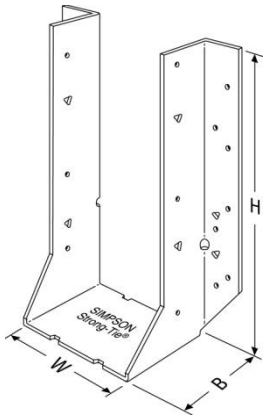


FIGURE 4A—HU SERIES HANGER
(See Table 4)

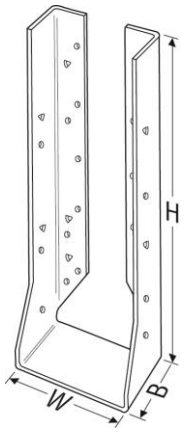


FIGURE 4B—HUC SERIES HANGER
(See Table 4, Footnote 4)

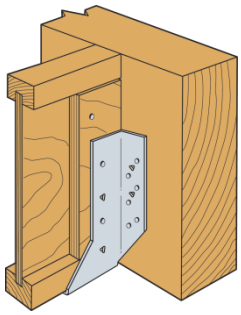


FIGURE 4C—TYPICAL HU HANGER

TABLE 5—ALLOWABLE LOADS FOR THE HUCQ SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			FASTENERS (Quantity-Type)		ALLOWABLE LOADS[2,3,4] (lbs)			
						Uplift[4]	Download		
	W	H	B	Header	Joist	C_D=1.6	C_D=1.0	C_D=1.15	C_D=1.25
HUCQ1.81/9-SDS	1 13/16	9	3	8-SDS1/4" x 1 3/4"	4-SDS1/4" x 1 3/4"	1,310	2,000	2,300	2,500
HUCQ1.81/11-SDS	1 13/16	11	3	10-SDS1/4" x 1 3/4"	4-SDS1/4" x 1 3/4"	1,310	2,500	2,875	3,125
HUCQ310-SDS	2 9/16	9	3	8-SDS1/4" x 2 1/2"	4-SDS1/4" x 2 1/2"	1,220	3,120	3,590	3,685
HUCQ210-2-SDS	3 1/4	9	3	12-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	1,220	4,680	4,865	4,865
HUCQ410-SDS	3 9/16	9	3	12-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	4,500	4,500	4,500
HUCQ412-SDS	3 9/16	11	3	14-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	5,045	5,045	5,045
HUCQ210-3-SDS	4 5/8	9	3	12-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,215	4,680	4,865	4,865
HUCQ5.25/9-SDS	5 1/4	9	3	12-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	4,500	4,500	4,500
HUCQ5.25/11-SDS	5 1/4	11	3	14-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	5,045	5,045	5,045
HUCQ610-SDS	5 9/16	9	3	12-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	4,680	5,210	5,210
HUCQ612-SDS	5 9/16	11	3	14-SDS1/4" x 2 1/2"	6-SDS1/4" x 2 1/2"	2,365	5,210	5,210	5,210

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 5 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[4]For supported members other than glulam beams, lateral support must be provided at the ends in accordance with Section 2308.8.2 of the IBC or Section R502.7 of the IRC.

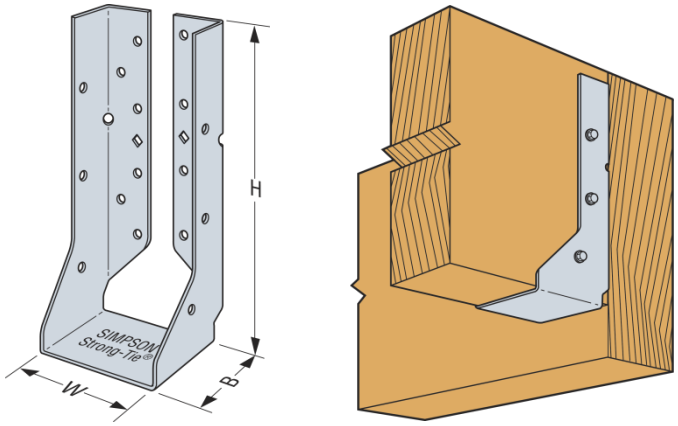


FIGURE 5—DIMENSIONS AND TYPICAL INSTALLATION OF THE HUCQ SERIES HANGERS

TABLE 6 ALLOWABLE LOADS FOR THE LUS SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			COMMON NAILS[2] (Quantity-Type)		ALLOWABLE LOADS[3,4] (lbf)			
	W	H	B	Header	Joist[5]	Uplift[6]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
LUS24	1 9/16	3 1/8	1 3/4	4-10d	2-10d	435	670	765	820
LUS26	1 9/16	4 3/4	1 3/4	4-10d	4-10d	1,165	865	990	1,060
LUS28	1 9/16	6 5/8	1 3/4	6-10d	4-10d	1,165	1,100	1,260	1,350
LUS210	1 9/16	7 13/16	1 3/4	8-10d	4-10d	1,165	1,335	1,530	1,640
LUS36	2 9/16	5 1/4	2	4-16d	4-16d	1,060	1,030	1,170	1,265
LUS310	2 9/16	7 9/32	2	6-16d	4-16d	1,070	1,315	1,490	1,610
LUS24-2	3 1/8	3 1/8	2	4-16d	2-16d	410	800	905	980
LUS26-2	3 1/8	4 15/16	2	4-16d	4-16d	1,060	1,030	1,170	1,265
LUS28-2	3 1/8	7	2	6-16d	4-16d	1,060	1,315	1,490	1,610
LUS210-2	3 1/8	8 15/16	2	8-16d	6-16d	1,445	1,830	2,075	2,245
LUS214-2	3 1/8	10 15/16	2	10-16d	6-16d	1,445	2,110	2,395	2,590
LUS44	3 9/16	3	2	4-16d	2-16d	410	800	905	980
LUS46	3 9/16	4 3/4	2	4-16d	4-16d	1,060	1,030	1,170	1,265
LUS48	3 9/16	6 3/4	2	6-16d	4-16d	1,060	1,315	1,490	1,610
LUS410	3 9/16	8 3/4	2	8-16d	6-16d	1,445	1,830	2,075	2,245
LUS414	3 9/16	10 3/4	2	10-16d	6-16d	1,445	2,110	2,395	2,590
LUS26-3	4 5/8	4 1/8	2	4-16d	4-16d	1,060	1,030	1,170	1,265
LUS28-3	4 5/8	6 1/4	2	6-16d	4-16d	1,060	1,315	1,490	1,610
LUS210-3	4 5/8	8 13/16	2	8-16d	6-16d	1,445	1,830	2,075	2,245

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 6 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]LUS Series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]Joist nails must be driven at a 45 degree angle through the joist into the header/beam (double shear nailing) to achieve the tabulated loads.
[6]Allowable uplift loads have been increased for wind or earthquake loading with no further increase is allowed. The allowable uplift loads must be reduced when other load durations govern.

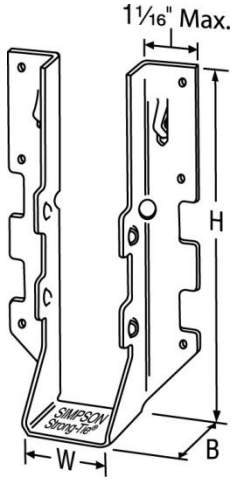


FIGURE 6—LUS SERIES HANGER
(See Table 6)

TABLE 7—ALLOWABLE LOADS FOR THE MUS SERIES HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			COMMON NAILS[2] (Quantity-Type)		ALLOWABLE LOADS[3,4] (lbf)			
	W	H	B	Header	Joist[5]	Uplift[6]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
MUS26	$1^{9}/_{16}$	$5^{3}/_{16}$	2	6–10d	6–10d	930	1,295	1,480	1,560
MUS28	$1^{9}/_{16}$	$6^{3}/_{4}$	2	8–10d	8–10d	1,320	1,730	1,975	2,125

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 7 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]MUS series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]Joist nails must be driven at a 45 degree angle through the joist into the header/beam (double shear nailing) to achieve the tabulated loads.
[6]Allowable uplift loads have been increased for wind or earthquake loading with no further increase is allowed. The allowable uplift loads must be reduced when other load durations govern.

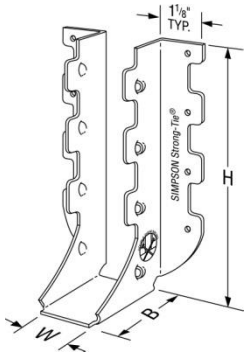


FIGURE 7—MUS HANGER (See Table 7)

TABLE 8—ALLOWABLE LOADS FOR THE HUS AND HUSC SERIES HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			COMMON NAILS[2] (Quantity-Type)		ALLOWABLE LOADS[3,4,7] (lbf)			
	W	H	B	Header	Joist[5]	Uplift[6]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
HUS26	$1^5/_8$	$5^3/_8$	3	14-16d	6-16d	1,320	2,735	3095	3230
HUS28	$1^5/_8$	$7^1/_{16}$	3	22-16d	8-16d	1,760	4095	4095	4095
HUS210	$1^5/_8$	$9^1/_{16}$	3	30-16d	10-16d	2,635	5,450	5,795	5,830
HUS1.81/10	$1^{13}/_{16}$	$8^7/_8$	3	30-16d	10-16d	2,675	5,510	5,830	5,830
HUS46	$3^9/_{16}$	$4^5/_{16}$	2	4-16d	4-16d	1,165	1,090	1,225	1,320
HUS48	$3^9/_{16}$	$6^{15}/_{16}$	2	6-16d	6-16d	1,320	1,630	1,840	1,980
HUS410	$3^9/_{16}$	$8^{15}/_{16}$	2	8-16d	8-16d	3,265	2,175	2,455	2,640
HUS412	$3^9/_{16}$	$10^3/_4$	2	10-16d	10-16d	3,435	2,720	3,070	3,300
HUS26-2	$3^1/_8$	$5^3/_{16}$	2	4-16d	4-16d	1,165	1,090	1,225	1,320
HUS28-2	$3^1/_8$	$7^3/_{16}$	2	6-16d	6-16d	1,320	1,630	1,840	1,980
HUS210-2	$3^1/_8$	$9^3/_{16}$	2	8-16d	8-16d	3,285	2,175	2,455	2,640
HUS212-2	$3^1/_8$	11	2	10-16d	10-16d	3,435	2,720	3,070	3,300

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]Refer to Figure 8 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]HUS series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]Joist nails must be driven at a 45 degree angle through the joist into the header/beam (double shear nailing) to achieve the tabulated loads.
[6]Allowable uplift loads have been increased for wind or earthquake loading with no further increase is allowed. The allowable uplift loads must be reduced when other load durations govern.
[7]HUS series hangers with widths (W) equal to or greater than $3^9/_{16}$ inches (90 mm) are available with header flanges turned in (concealed) and are identified with the model designation HUSC#.

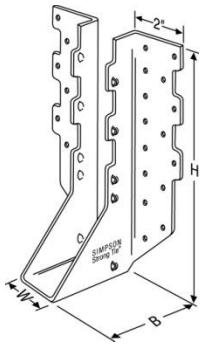


FIGURE 8—HUS SERIES HANGER (See Table 8)

TABLE 9 - ALLOWABLE LOADS FOR THE HHUS SERIES HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			COMMON NAILS[2] (Quantity-Type)		ALLOWABLE LOADS[3,4] (lbf)			
	W	H	B	Header	Joist[5]	Uplift[6]	Download		
						C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
HHUS26-2	3 5/16	5 7/16	3	14-16d	6-16d	1,320	2,820	3,185	3,425
HHUS28-2	3 5/16	7 1/2	3	22-16d	8-16d	1,760	4,255	4,800	5,165
HHUS210-2	3 5/16	9 1/8	3	30-16d	10-16d	3,550	5,690	6,420	6,485
HHUS46	3 5/8	5 1/4	3	14-16d	6-16d	1,320	2,820	3,185	3,425
HHUS48	3 5/8	7 1/8	3	22-16d	8-16d	1,760	4,255	4,800	5,165
HHUS410	3 5/8	9	3	30-16d	10-16d	3,550	5,690	6,420	6,485
HHUS5.50/10	5 1/2	9	3	30-16d	10-16d	3,550	5,690	6,420	6,485
HHUS7.25/10	7 1/4	9	3 5/16	30-16d	10-16d	3,550	5,690	6,420	6,485

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 9 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]HUS series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125 inch (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[5]Joist nails must be driven at a 45 degree angle through the joist into the header/beam to achieve the tabulated loads.
[6]Allowable uplift loads have been increased for wind or earthquake loading with no further increase is allowed. The allowable uplift loads must be reduced when other load durations govern.

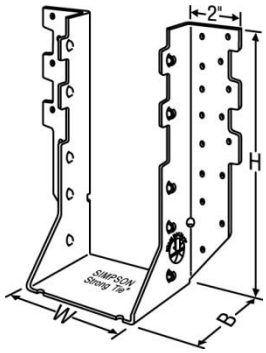


FIGURE 9—HHUS SERIES HANGER (See Table 9)

TABLE 10—ALLOWABLE LOADS FOR THE HGUS SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (Inches)			FASTENERS[2] (Quantity-Type)		ALLOWABLE LOADS[3,5,6] (lbs)			
						Uplift[4]	Download		
	W	H	B	Header	Joist[3]	C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
HGUS26	1 5/8	5 3/8	5	20-16d	8-16d	875	4,340	4,850	5,170
HGUS28	1 5/8	7 1/8	5	36-16d	12-16d	1,650	7,275	7,275	7,275
HGUS210	1 5/8	9 1/8	5	46-16d	16-16d	2,090	9,100	9,100	9,100
HGUS2.75/10	2 3/4	8 7/8	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS2.75/12	2 3/4	10 7/8	4	56-16d	20-16d	5,040	9,400	9,400	9400
HGUS2.75/14	2 3/4	12 7/8	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS3.25/10	3 1/4	8 5/8	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS3.25/12	3 1/4	10 5/8	4	56-16d	20-16d	5,040	9,400	9,400	9,400
HGUS26-2	3 7/16	5 7/16	4	20-16d	8-16d	2,155	4,340	4,850	5,170
HGUS28-2	3 7/16	7 3/16	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS210-2	3 7/16	9 3/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS46	3 5/8	5 5/16	4	20-16d	8-16d	2,155	4,360	4,885	5,230
HGUS48	3 5/8	7 1/16	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS410	3 5/8	9 1/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS412	3 5/8	10 7/16	4	56-16d	20-16d	5,040	9,400	9,400	9,400
HGUS414	3 5/8	12 7/16	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS26-3	4 15/16	5 7/16	4	20-16d	8-16d	2,155	4,340	4,850	5,170
HGUS28-3	4 15/16	7 3/16	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS210-3	4 15/16	9 3/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS212-3	4 15/16	10 3/4	4	56-16d	20-16d	5,205	11,835	13,215	14,085
HGUS214-3	4 15/16	12 3/4	4	66-16d	22-16d	5,360	13,735	14,360	14,360
HGUS5.25/10	5 1/4	9 1/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS5.25/12	5 1/4	10 5/8	4	56-16d	20-16d	5,040	9,400	9,400	9,400
HGUS5.5/8	5 1/2	6 15/16	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS5.5/10	5 1/2	8 15/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS5.5/12	5 1/2	10 1/2	4	56-16d	20-16d	5,040	9,400	9,400	9,400
HGUS5.5/14	5 1/2	12 1/2	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS5.62/10	5 5/8	8 7/8	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS5.62/12	5 5/8	10 7/16	4	56-16d	20-16d	5,040	9,400	9,400	9,400
HGUS5.62/14	5 5/8	12 7/16	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS26-4	6 9/16	5 7/16	4	20-16d	8-16d	2,155	4,340	4,850	5,170
HGUS28-4	6 9/16	7 3/16	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS210-4	6 9/16	9 3/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS212-4	6 9/16	10 9/16	4	56-16d	20-16d	5,205	11,835	13,215	14,085
HGUS214-4	6 9/16	12 9/16	4	66-16d	22-16d	5,360	13,735	14,360	14,360
HGUS6.88/10	6 7/8	8 13/16	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS6.88/12	6 7/8	10 13/16	4	56-16d	20-16d	5,045	9,400	9,400	9,400
HGUS6.88/14	6 7/8	12 13/16	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS7.25/8	7 1/4	7 1/4	4	36-16d	12-16d	3,235	7,460	7,460	7,460
HGUS7.25/10	7 1/4	8 5/8	4	46-16d	16-16d	4,095	9,100	9,100	9,100
HGUS7.25/12	7 1/4	10 5/8	4	56-16d	20-16d	5,040	9,400	9,400	9,499
HGUS7.25/14	7 1/4	12 5/8	4	66-16d	22-16d	5,515	9,695	9,695	9,695
HGUS7.37/10	7 3/8	8 9/16	4	46-16d	16-16d	4,065	9,095	9,095	9,095
HGUS7.37/12	7 3/8	10 9/16	4	56-16d	20-16d	5,205	11,835	13,215	14,085
HGUS7.37/14	7 3/8	12 9/16	4	66-16d	22-16d	5,515	13,735	14,360	14,360

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 10 (this page) for definitions of hanger nomenclature (W, H, B).
[2]Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.
[3]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[4]Joist nails must be driven at a 45 degree angle through the joist into the header/beam (double shear nailing) to achieve tabulated loads.
[5]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[6]HGUS series hangers provide torsional resistance, which is defined as the moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to the vertical position of the joist is 0.125" (3.2 mm). The height, H, of the joist hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.

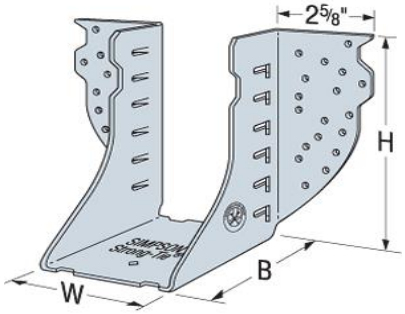


FIGURE 10—HGUS SERIES JOIST HANGER

TABLE 11A—DIMENSIONS, NAILING SCHEDULES AND DESIGN VALUES FOR HTU SERIES HANGERS ($^1/_2$ Inch Maximum Gap between Supporting Member and Supported Member – Maximum Number of Nails into Supporting Member)

MODEL No.	DIMENSIONS [3] (inches)			FASTENERS [4] (Quantity-Type)		ALLOWABLE LOADS [5, 6, 7] (lbf)					
	W	H	B	Into Supporting Member	Into Supported Member	Uplift [8]	Download				
						C_D=1.6	C_D=0.9	C_D=1.0	C_D=1.15	C_D=1.25	C_D=1.6
Single 2X Sizes											
HTU26 ($^1/_2$" Gap – Min Nail)[1]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	11-10dx$1^1/_2$	635	2,395	2,395	2,395	2,395	2,395
	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	14-10dx$1^1/_2$	1,175	2,640	2,940	3,100	3,100	3,100
HTU26 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	20-10dx$1^1/_2$	1,215	2,640	2,940	3,320	3,580	3,630
HTU28 ($^1/_2$" Gap – Min Nail)[1]	$1^5/_8$	$7^1/_{16}$	$3^1/_2$	26-16d	14-10dx$1^1/_2$	1,110	3,430	3,770	3,770	3,770	3,770
HTU28 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$7^1/_{16}$	$3^1/_2$	26-16d	26-10dx$1^1/_2$	1,920	3,430	3,820	4,315	4,655	5,015
HTU210 ($^1/_2$" Gap – Min Nail)[1]	$1^5/_8$	$9^1/_{16}$	$3^1/_2$	32-16d	14-10dx$1^1/_2$	1250	3,600	3,600	3,600	3,600	3,600
HTU210 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$9^1/_{16}$	$3^1/_2$	32-16d	32-10dx$1^1/_2$	3255	4,225	4,705	5,020	5,020	5,020
Double 2X Sizes											
HTU26-2 ($^1/_2$" Gap – Min Nail)[1]	$3^5/_{16}$	$5^7/_{16}$	$3^1/_2$	20-16d	14-10d	1,515	2,640	2,940	3,320	3,500	3,500
HTU26-2 ($^1/_2$" Gap – Max Nail)[2]	$3^5/_{16}$	$5^7/_{16}$	$3^1/_2$	20-16d	20-10d	1,910	2,640	2,940	3,320	3,500	3,500
HTU28-2 ($^1/_2$" Gap – Min Nail)[1]	$3^5/_{16}$	$7^1/_{16}$	$3^1/_2$	26-16d	14-10d	1,490	3,430	3,820	3,980	3,980	3,980
HTU28-2 ($^1/_2$" Gap – Max Nail)[2]	$3^5/_{16}$	$7^1/_{16}$	$3^1/_2$	26-16d	26-10d	3,035	3,430	3,820	4,315	4,655	5,520
HTU210-2 ($^1/_2$" Gap – Min Nail)[1]	$3^5/_{16}$	$9^1/_{16}$	$3^1/_2$	32-16d	14-10d	1,755	4,225	4,255	4,255	4,255	4,255
HTU210-2 ($^1/_2$" Gap – Max Nail)[2]	$3^5/_{16}$	$9^1/_{16}$	$3^1/_2$	32-16d	32-10d	3,855	4,225	4,705	5,310	5,730	6,470

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]The suffix '($^1/_2$" Gap – Min Nail)' corresponds to installed conditions where the gap between the supporting member and supported member is more than $^1/_8$ inch (3.2 mm) and less than or equal to $^1/_2$ inch (12.7 mm), and, at a minimum, the number of nails specified in the table above are installed into the supported wood truss. Refer to Figure 11B on page 19 for a typical installation detail.

[2]The suffix '($^1/_2$" Gap – Max Nail)' corresponds to installed conditions where the gap between the supporting member and supported member is more than $^1/_8$ inch (3.2 mm) and less than or equal to $^1/_2$ inch (12.7 mm), and all of the pre-punched nail holes in the U-shaped portion of the hanger supporting the truss (joist) are filled with nails. This is designated in the table as "Max Nail" and is shown in Figure 11B.

[3]Refer to Figure 11A for definitions of hanger nomenclature (W, H, B).

[4]Allowable loads correspond to installations where the maximum possible number of nails is driven into the supporting member. Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.

[5]Tabulated allowable loads are for instal1ations in wood members complying with Section 3.2.2 of this report.

[6]Tabulated loads must be selected based on the applicable load duration factor, C_D, as permitted by the applicable building code. See Sections 4.1 and 4.2 for design and installation requirements.

[7]HTU series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm), for joists having a height no greater than the height (H) of the hanger.

[8]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The tabulated allowable uplift loads must be reduced proportionally when other load durations govern.

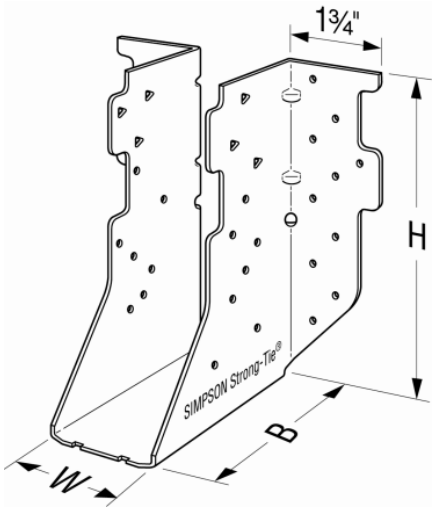


FIGURE 11A—HTU SERIES HANGER

TABLE 11B—DIMENSIONS, NAILING SCHEDULES AND DESIGN VALUES FOR HTU SERIES HANGERS ($^1/_8$ Inch Maximum Gap between Supporting Member and Supported Member – Maximum Number of Nails into Supporting Member)

MODEL No.	DIMENSIONS[3] (inches)			FASTENERS[4] (Quantity-Type)		ALLOWABLE LOADS[5, 6, 7] (lbf)					
	W	H	B	Into Supporting Member	Into Supported Member	Uplift[8]	Download				
						C_D=1.6	C_D=0.9	C_D=1.0	C_D=1.15	C_D=1.25	C_D=1.6
Single 2X Sizes											
HTU26 ($^1/_8$" Gap – Min Nail)[1]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	11-10dx$1^1/_2$	640	2,640	2,670	2,670	2,670	2,670
	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	14-10dx$1^1/_2$	1,250	2,640	2,940	3,200	3,200	3,200
HTU26 ($^1/_8$" Gap – Max Nail)[2]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	20-16d	20-10dx$1^1/_2$	1,555	2,640	2,940	3,320	3,580	4,010
HTU28 ($^1/_8$" Gap – Min Nail)[1]	$1^5/_8$	$7^1/_{16}$	$3^1/_2$	26-16d	14-10dx$1^1/_2$	1,235	3,430	3,820	3,895	3,895	3,895
HTU28 ($^1/_8$" Gap – Max Nail)[2]	$1^5/_8$	$7^1/_{16}$	$3^1/_2$	26-16d	26-10dx$1^1/_2$	2,020	3,430	3,820	4,315	4,655	5,435
HTU210 ($^1/_8$" Gap – Min Nail)[1]	$1^5/_8$	$9^1/_{16}$	$3^1/_2$	32-16d	14-10dx$1^1/_2$	1,330	4,225	4,300	4,300	4,300	4,300
HTU210 ($^1/_8$" Gap – Max Nail)[2]	$1^5/_8$	$9^1/_{16}$	$3^1/_2$	32-16d	32-10dx$1^1/_2$	3,315	4,225	4,705	5,310	5,730	5,995
Double 2X Sizes											
HTU26-2 ($^1/_8$" Gap – Min Heel)[1]	$3^5/_{16}$	$5^7/_{16}$	$3^1/_2$	20-16d	14-10d	1,515	2,640	2,940	3,320	3,580	3,910
HTU26-2 ($^1/_8$" Gap – Max Nail)[2]	$3^5/_{16}$	$5^7/_{16}$	$3^1/_2$	20-16d	20-10d	2,175	2,640	2,940	3,320	3,580	4,480
HTU28-2 ($^1/_8$" Gap – Min Nail)[1]	$3^5/_{16}$	$7^1/_{16}$	$3^1/_2$	26-16d	14-10d	1,530	3,430	3,820	4,310	4,310	4,310
HTU28-2 ($^1/_8$" Gap – Max Nail)[2]	$3^5/_{16}$	$7^1/_{16}$	$3^1/_2$	26-16d	26-10d	3,485	3,430	3,820	4,315	4,655	5,825
HTU210-2 ($^1/_8$" Gap – Min Nail)[1]	$3^5/_{16}$	$9^1/_{16}$	$3^1/_2$	32-16d	14-10d	1,755	4,225	4,705	4,815	4,815	4,815
HTU210-2 ($^1/_8$" Gap – Max Nail)[2]	$3^5/_{16}$	$9^1/_{16}$	$3^1/_2$	32-16d	32-10d	4,110	4,225	4,705	5,310	5,730	6,515

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]The suffix '($^1/_8$" Gap – Min Nail)' corresponds to installed conditions where the gap between the supporting member and supported wood truss is $^1/_8$ inch (3.2 mm) or less, and at a minimum, the number of nails specified in the table above are installed into the supported wood truss. Refer to Figure 10B for a typical installation detail.

[2]The suffix '($^1/_8$" Gap – Max Nail)' corresponds to installed conditions where the gap between the supporting member and supported wood truss is $^1/_8$ inch (3.2 mm) or less, and the all of the pre-punched nail holes in the U-shaped portion of the hanger supporting the truss (joist) are filled with nails. This is designated in the table as "Max Nail" and is shown in Figure 11B on this page.

[3]Refer to Figure 11A for definitions of hanger nomenclature (W, H, B).

[4]Allowable loads correspond to installations where the maximum possible number of nails is installed into the supporting member. Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.

[5]Tabulated allowable loads are for installations in wood members complying with Section 3.2.2 of this report.

[6]Tabulated loads must be selected based on the applicable load duration factor, C_D, as permitted by the applicable building code. See Sections 4.1 and 4.2 for design and installation requirements.

[7]HTU series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm), for joists having a height no greater than the height (H) of the hanger.

[8]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The tabulated allowable uplift loads must be reduced proportionally when other load durations govern.

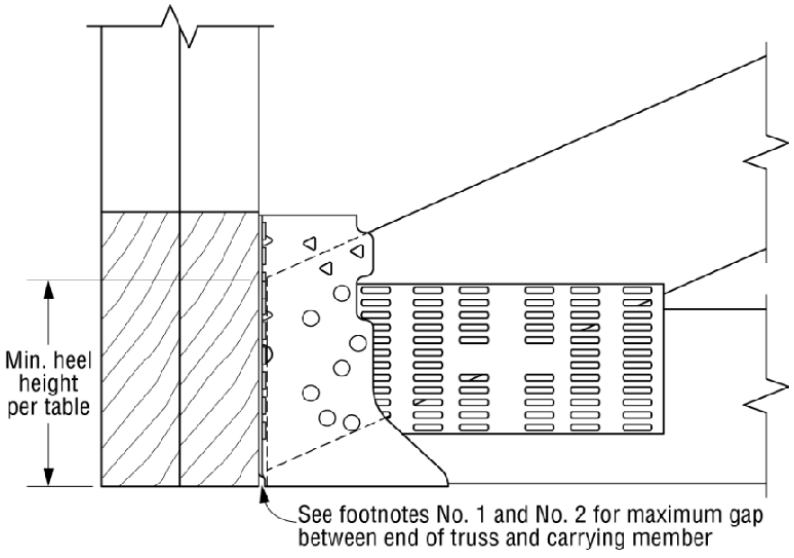


FIGURE 11B—TYPICAL HTU INSTALLATION

TABLE 11C—DIMENSIONS, NAILING SCHEDULES AND DESIGN VALUES FOR HTU SERIES HANGERS – ALTERNATE INSTALLATION ($^1/_2$ Inch Maximum Gap between Supporting Member and Supported Member - Minimum Number of Nails into Supporting Member)

MODEL No. [1, 2]	DIMENSIONS [3] (inches)			FASTENERS [4] (Quantity-Type)		ALLOWABLE LOADS [5, 6, 7] (lbf)					
	W	H	B	Into Supporting Member	Into Supported Member	Uplift [8]	Download				
						C_D=1.6	C_D=0.9	C_D=1.0	C_D=1.15	C_D=1.25	C_D=1.6
HTU26 ($^1/_2$" Gap – Min Nail)[1]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	10-16d	14-10dx$1^1/_2$	845	1,320	1,470	1,660	1,790	1,875
HTU26 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$5^7/_{16}$	$3^1/_2$	10-16d	20-10dx$1^1/_2$	1,240	1,320	1,470	1,660	1,790	2,220
HTU28 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$7^1/_{16}$	$3^1/_2$	20-16d	26-10dx$1^1/_2$	1,920	2,640	2,940	3,320	3,580	3,905
HTU210 ($^1/_2$" Gap – Max Nail)[2]	$1^5/_8$	$9^1/_{16}$	$3^1/_2$	20-16d	32-10dx$1^1/_2$	2,880	2,640	2,940	3,320	3,580	3,905

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]The suffix '($^1/_2$" Gap – Min Nail)' corresponds to installed conditions where the gap between the supporting member and supported member is more than $^1/_8$ inch (3.2 mm) and less than or equal to $^1/_2$ inch (12.7 mm), and at a minimum, the number of nails specified in the table above are installed into the supported wood truss. Refer to Figure 11C for an alternate installation detail.

[2]The suffix '($^1/_2$" Gap – Max Nail)' corresponds to installed conditions where the gap between the supporting member and supported member is more than $^1/_8$ inch (3.2 mm) and less than or equal to $^1/_2$ inch (12.7 mm), and the all of the pre-punched nail holes in the U-shaped portion of the hanger supporting the truss (joist) are filled with nails. This is designated in the table as "Max Nail" and is shown in Figure 10C.

[3]Refer to Figure 11A for definitions of hanger nomenclature (W, H, B).

[4]Allowable loads correspond to installations where the minimum allowable number of nails is installed into the supporting member. Refer to Section 3.2.3 of this report for nail sizes and required minimum physical properties.

[5]Tabulated allowable loads are for installations in wood members complying with Section 3.2.2 of this report.

[6]Tabulated loads must be selected based on the applicable load duration factor, C_D, as permitted by the applicable building code. See Sections 4.1 and 4.2 for design and installation requirements.

[7]HTU series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm), for joists having a height no greater than the height (H) of the hanger.

[8]Allowable uplift loads have been increased for wind or earthquake loading with no further increase allowed. The tabulated allowable uplift loads must be reduced proportionally when other load durations govern.

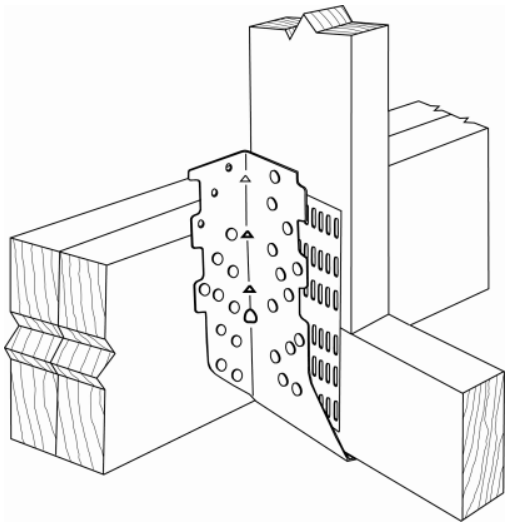


FIGURE 11C—ALTERNATE HTU INSTALLATION

TABLE 12—ALLOWABLE LOADS FOR THE LGU, MGU, HGU AND HHGU SERIES HANGERS

MODEL No.	DIMENSIONS[1] (inches)					FASTENERS (Quantity-Type)		ALLOWABLE LOADS[2,6,7] (lbs)			
								Uplift[5]	Download		
	W	H[4] (min)	B	h_a[3]	a	Header	Joist	C_D=1.6	C_D=1.0	C_D=1.15	C_D=1.25
LGU3.25-SDS	3 1/4	8	4 1/2	7 3/8	3 1/4	16 SDS1/4"x 2 1/2"	12 SDS1/4"x 2 1/2"	5,555	6,720	7,310	7,310
LGU3.63-SDS	3 5/8										
LGU5.25-SDS	5 1/4										
MGU3.63-SDS	3 5/8	9 1/4	4 1/2	8 5/8	4	24 SDS1/4"x 2 1/2"	16 SDS1/4"x 2 1/2"	7,260	9,450	9,450	9,450
MGU5.25-SDS	5 1/4										
MGU5.50-SDS	5 1/2										
MGU5.62-SDS	5 5/8										
MGU7.00-SDS	7										
HGU3.63-SDS	3 5/8	11	5 1/4	10 3/8	4 3/4	36 SDS1/4"x 2 1/2"	24 SDS1/4"x 2 1/2"	9.460	13,160	13,160	13,160
HGU5.25-SDS	5 1/4										
HGU5.50-SDS	5 1/2										
HGU5.62-SDS	5 5/8										
HGU7.00-SDS	7										
HGU7.25-SDS	7 1/4										
HGU9.00-SDS	9										
HHGU5.25-SDS	5 1/4	13	5 1/4	12 3/8	4 3/4	44 SDS1/4"x 2 1/2"	28 SDS1/4"x 2 1/2"	10,050	18,480	20,455	20,455
HHGU5.50-SDS	5 1/2										
HHGU5.62-SDS	5 5/8										
HHGU7.00-SDS	7							14,550	18,480	20,455	20,445
HHGU7.25-SDS	7 1/4										
HHGU9.00-SDS	9										

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 12 (this page) for definitions of hanger nomenclature (W, H, B, h_a and a).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]The supporting member (header) height must be at least at high as the flange height, h_a.
[4]The H dimension must be specified. The maximum H dimension is 30 inches.
[5]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[6]The allowable loads are based on the use of an engineered wood joist member with an allowable $F_{c\perp}$ of 650 psi. When use is to support members having an $F_{c\perp}$ of less than 650 psi, it must be verified that the combination of bearing capacity and joist nail capacity is adequate.
[7]For supported members other than glulam beams, lateral support must be provided at the ends in accordance with Section 2308.8.2 of the IBC or Section R502.7 of the IRC.

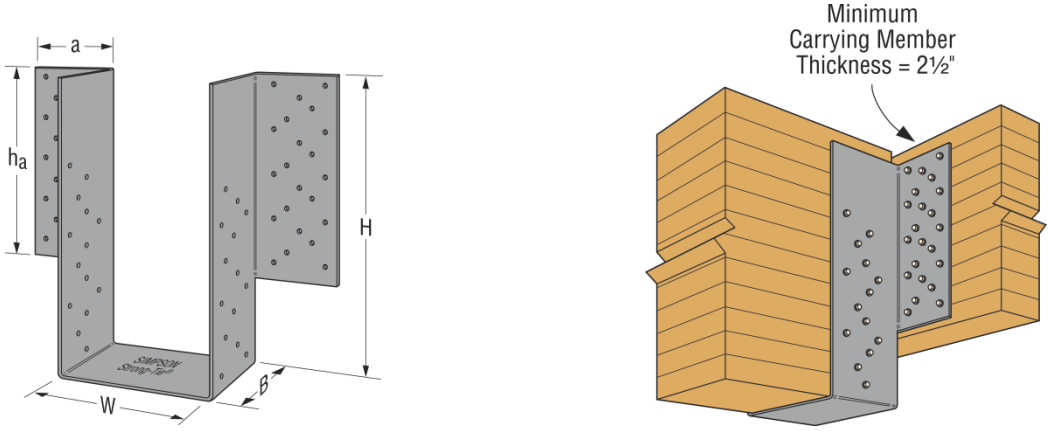


FIGURE 12—DIMENSIONS AND TYPICAL INSTALLATION OF THE LGU, MGU, HGU AND HHGU SERIES HANGERS

TABLE 13—ALLOWABLE LOADS FOR THE SUR/SUL SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (inches)					FASTENERS (Quantity-Type)		ALLOWABLE LOADS[2,6,7] (lbs)			
								Uplift[5]	Download		
	W	H	B	A_1	A_2	Header	Joist	C_D=1.6	C_D=1.0	C_D=1.15	C_D=1.25
SUR/L24	1 9/16	3 1/16	2	1 1/8	1 1/4	4-16d	4-10d x 1 1/2	395	575	650	705
SUR/L26	1 9/16	5	2	1 1/8	1 5/16	6-16d	6-10d x 1 1/2	675	865	980	1,055
SUR/L210	1 9/16	8	2	1 1/8	1 5/16	10-16d	10-10d x 1 1/2	1,445	1,440	1,635	1,760
SUR/L214	1 9/16	10	2	1 1/8	1 5/16	12-16d	12-10d x 1 1/2	1,890	1,730	1,960	2,115
SUR/L1.81/9	1 13/16	9	3	1 5/8	2 5/16	12-16d	2-10d x 1 1/2	210[3,4]	1,730	1,955	1,955
SUR/L1.81/11	1 13/16	11	3	1 5/8	2 5/16	16-16d	2-10d x 1 1/2	210[3,4]	2,300	2,560	2,560
SUR/L1.81/14	1 13/16	13 3/4	3	1 5/8	2 5/16	20-16d	2-10d x 1 1/2	210[3,4]	2,560	2,560	2,560
SUR/L2.06/9	2 1/16	9 1/16	3 3/16	1 5/8	2 1/8	14-16d	2-10d x 1 1/2	210[3]	2,015	2,285	2,465
SUR/L2.06/11	2 1/16	11 1/4	3 3/16	1 5/8	2 1/8	16-16d	2-10d x 1 1/2	210[3]	2,300	2,610	2,665
SUR/L2.06/14	2 1/16	13 5/8	3 3/16	1 5/8	2 1/8	18-16d	2-10d x 1 1/2	210[3]	2,590	2,665	2,665
SUR/L2.1/9	2 1/8	9 1/16	3 3/16	1 9/16	2 1/8	14-16d	2-10d x 1 1/2	210[3]	2,015	2,285	2,465
SUR/L2.1/11	2 1/8	11 3/16	3 3/16	1 9/16	2 1/8	16-16d	2-10d x 1 1/2	210[3]	2,300	2,610	2,665
SUR/L2.1/14	2 1/8	13 9/16	3 3/16	1 9/16	2 1/8	18-16d	2-10d x 1 1/2	210[3]	2,590	2,665	2,665
SUR/L2.37/9	2 3/8	8 15/16	3 3/16	1 5/16	2 1/8	14-16d	2-10d x 1 1/2	210[3]	2,015	2,285	2,465
SUR/L2.37/11	2 3/8	11 3/16	3 3/16	1 5/16	2 1/8	16-16d	2-10d x 1 1/2	210[3]	2,300	2,610	2,665
SUR/L2.37/14	2 3/8	13 7/16	3 3/16	1 5/16	2 1/8	18-16d	2-10d x 1 1/2	210[3]	2,590	2,665	2,665
SUR/L2.56/9	2 9/16	8 13/16	3 3/16	1 1/8	2 1/8	14-16d	2-10d x 1 1/2	210[3]	2,015	2,285	2,465
SUR/L2.56/11	2 9/16	11 3/16	3 3/16	1 1/8	2 1/8	16-16d	2-10d x 1 1/2	210[3]	2,300	2,615	2,665
SUR/L2.56/14	2 9/16	13 5/16	3 3/16	1 1/8	2 1/8	18-16d	2-10d x 1 1/2	210[3]	2,590	2,665	2,665
SUR/L26-2	3 1/8	4 15/16	2 5/8	1 1/2	2 3/8	8-16d	4-16d x 2 1/2	725	1.150	1,305	1,325
SUR/L210-2	3 1/8	8 11/16	2 5/8	1 7/16	2 3/8	14-16d	6-16d x 2 1/2	1,160	2,015	2,285	2,345
SUR/L214-2	3 1/8	12 11/16	2 7/16	1 7/16	2 3/8	18-16d	8-16d x 2 1/2	1,505	2,345	2,345	2,345
SUR/L46	3 9/16	4 3/4	2 5/8	1	2 3/8	8-16d	4-16d	730	1,150	1,305	1,325
SUR/L410	3 9/16	8 1/2	2 5/8	1	2 3/8	14-16d	6-16d	1,160	2,015	2,285	2,345
SUR/L414	3 9/16	12 1/2	2 5/8	1	2 3/8	18-16d	8-16d	1,490	2,345	2,345	2,345

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 13a (this page) for definitions of hanger nomenclature (W, H, B, A1, A2).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]Optional triangle holes may be filled (requires web stiffeners for I-joist) for additional uplift resistance. When a total of six 10dx1 1/2" nails are installed into the joist for 9- and 11-inch models, an uplift value of 825 lbs may be used. When a total of eight 10dx1 1/2" nails are installed into the joist for 14-inch models, an uplift value of 1,190 lbs may be used.
[4]Uplift value based on use of solid sawn lumber or LVL joist member. When using an I-joist as the joist member, allowable uplift must be taken as 140 lbs.
[5]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[6]SUR/L series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm). The height (H) of the hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[7]The 2-2x ans 4x SUR/L models are available with the A2 flanges concealed and are specified with the model designation SUR/LC

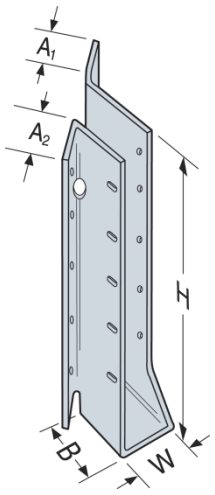


FIGURE 13A—SUL SERIES JOIST HANGER

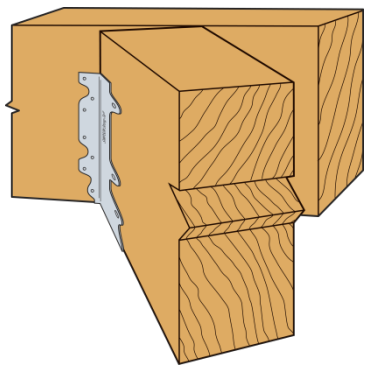


FIGURE 13B—TYPICAL SUR HANGER INSTALLATION

TABLE 14—ALLOWABLE LOADS FOR THE HSUR/HSUL SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (inches)					FASTENERS (Quantity-Type)		ALLOWABLE LOADS[2,3,6,7] (lbs)			
	W	H	B	A_1	A_2	Header	Joist	Uplift[4]	Download		
								$C_D = 1.6$	$C_D = 1.0$	$C_D = 1.15$	$C_D = 1.25$
HSUR/L26-2	3 1/8	4 15/16	2 7/16	1 3/4	2 3/8	12-16d	4-16d x 2 1/2	725	1,785	1,795	1,795
HSUR/L210-2	3 1/8	8 11/16	2 7/16	1 3/4	2 3/8	20-16d	6-16d x 2 1/2	1,160	2,975	3,355	3,410
HSUR/L214-2	3 1/8	12 11/16	2 7/16	1 3/4	2 3/8	26-16d	8-16d x 2 1/2	1,490	3,865	4,360	4,685
HSUR/L46	3 9/16	4 3/4	2 7/16	1	2 3/16	12-16d	4-16d	725	1,785	1,795	1,795
HSUR/L410	3 9/16	8 1/2	2 7/16	1	2 3/16	20-16d	6-16d	1,150	2,975	3,355	3,410
HSUR/L414	3 9/16	12 1/2	2 7/16	1	2 3/16	26-16d	8-16d	1,490	3,865	4,360	4.685
HSUR/L4.12/9	4 1/8	9	3	1 7/16	2 3/8	12-16d	2-10d x 1 1/2	145[4]	1,725	1,725	1,725
HSUR/L4.12/11	4 1/8	11 1/8	3	1 7/16	2 3/8	16-16d	2-10d x 1 1/2	145[4]	2,380	2,685	2,885
HSUR/L4.12/14	4 1/8	13 3/4	3	1 7/16	2 3/8	20-16d	2-10d x 1 1/2	145[4]	2,975	3,330	3,330
HSUR/L4.12/16	4 1/8	15 3/4	3	1 7/16	2 3/8	24-16d	2-10d x 1 1/2	145[4]	3,330	3,330	3,330
HSUR/L4.28/9	4 5/16	9	3	1 7/16	2 3/8	12-16d	2-10d x 1 1/2	145[4]	1,7255	1,725	1,725
HSUR/L4.28/11	4 5/16	11 1/8	3	1 7/16	2 3/8	16-16d	2-10d x 1 1/2	145[4]	2,380	2,685	2,885
HSUR/L4.75/9	4 3/4	8 15/16	2 3/4	1 7/16	2 3/8	12-16d	2-10d x 1 1/2	145[4]	1,725	1,725	1,725
HSUR/L4.75/11	4 3/4	10 15/16	2 3/4	1 7/16	2 3/8	16-16d	2-10d x 1 1/2	145[4]	2,380	2,685	2,885
HSUR/L4.75/14	4 3/4	13 3/4	2 3/4	1 7/16	2 3/8	20-16d	2-10d x 1 1/2	145[4]	2,975	3,330	3,330
HSUR/L4.75/16	4 3/4	15 3/4	2 3/4	1 7/16	2 3/8	24-16d	2-10d x 1 1/2	145[4]	3,330	3,330	3,330
HSUR/L5.12/9	5 1/8	9	2 13/16	1 7/16	2 3/8	12-16d	2-10d x 1 1/2	145[4]	1,725	1,725	1,725
HSUR/L5.12/11	5 1/8	11	2 13/16	1 7/16	2 3/8	16-16d	2-10d x 1 1/2	145[4]	2,380	2,685	2,885
HSUR/L5.12/14	5 1/8	13 3/4	2 13/16	1 7/16	2 3/8	20-16d	2-10d x 1 1/2	145[4]	2,975	3,330	3,330
HSUR/L5.12/16	5 1/8	15 3/4	2 13/16	1 7/16	2 3/8	24-16d	2-10d x 1 1/2	145[4]	3,330	3,330	3,330

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 14a (this page) for definitions of hanger nomenclature (W, H, B, A1, A2).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]When I-joists are the supported member as shown in Figure 7b, each I-joist must have web stiffeners installed in accordance with the I-joist manufacturer's evaluation report, and the minimum required quantity and type of nails, as specified in the column entitled FASTENERS must be nailed directly into the web stiffeners and/or I-joist.
[4]Optional triangular holes may be filled (requires web stiffeners) for additional uplift. When a total of six 10d x 1 1/2" nails are installed into the joist, an uplift value of 505 lbs may be used.
[5]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[6]HSUR/L series hangers provide torsional resistance, which is defined as a moment of not less than 75 pounds (334 N) times the depth of the joist at which the lateral movement of the top or bottom of the joist with respect to its vertical position is 0.125 inch (3.2 mm). The height (H) of the hanger must be at least 60 percent of the height of the joist unless additional lateral restraint is provided, as designed by others.
[7]The 2-2x ans 4x HSUR/L models are available with the A2 flanges concealed and are specified with the model designation HSUR/LC

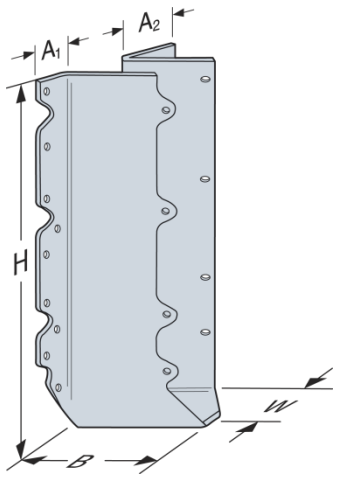


FIGURE 14A—HSUR JOIST HANGER

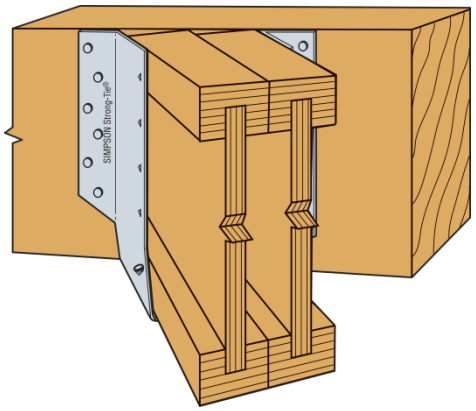


FIGURE 14B—TYPICAL HSUR JOIST HANGER INSTALLATION (Web Stiffeners not shown for Clarity–See Footnote 3 to Table 7)

TABLE 15A—APPLICABLE MODEL NUMBERS FOR THE IUS SERIES I-JOIST HANGERS[1]

MODEL NO.	DIMENSIONS[2] (Inches)			FASTENERS (Quantity-Type)		MODEL NO.	DIMENSIONS[2] (Inches)			FASTENERS (Quantity-Type)	
	W	H	B	Header	Joist		W	H	B	Header	Joist
						IUS2.37/16(min)	$2^{7}/_{16}$	16	2	14-10d	—
						IUS2.37/16(max)				16-10d	—
IUS1.81/9.5	$1^{7}/_{8}$	$9^{1}/_{2}$	2	8-10d	—	IUS2.56/9.25	$2^{5}/_{8}$	$9^{1}/_{4}$	2	8-10d	—
IUS1.81/11.88		$11^{7}/_{8}$		10-10d	—	IUS2.56/9.5		$9^{1}/_{2}$		8-10d	—
IUS1.81/14(min)		14		12-10d	—	IUS2.56/11.88		$11^{7}/_{8}$		10-10d	—
IUS1.81/14(max)				14-10d	—	IUS2.56/14(min)		14		12-10d	—
IUS1.81/16(min)		16		14-10d	—	IUS2.56/14(max)				14-10d	—
IUS1.81/16(max)				16-10d	—	IUS2.56/16(min)		16		14-10d	—
IUS2.06/9.5	$2^{1}/_{8}$	$9^{1}/_{2}$	2	8-10d	—	IUS2.56/16(max)				16-10d	—
IUS2.06/11.88		$11^{7}/_{8}$		10-10d	—	IUS3.56/9.5	$3^{5}/_{8}$	$9^{1}/_{2}$	2	10-10d	—
IUS2.06/14(min)		14		12-10d	—	IUS3.56/11.88		$11^{7}/_{8}$		12-10d	—
IUS2.06/14(max)				14-10d	—	IUS3.56/14(min)		14		12-10d	—
IUS2.06/16(min)		16		14-10d	—	IUS3.56/14(max)				14-10d	—
IUS2.06/16(max)				16-10d	—	IUS3.56/16(min)		16		14-10d	—
IUS2.37/9.5	$2^{7}/_{16}$	$9^{1}/_{2}$	2	8-10d	—	IUS3.56/16(max)				16-10d	—
IUS2.37/11.88		$11^{7}/_{8}$		10-10d	—	(This section of the table is intentionally blank)					
IUS2.37/14(min)		14		12-10d	—						
IUS2.37/14(max)				14-10d	—						

For **SI:** 1 inch = 25.4 mm.

[1]Model numbers ending with (min) refer to nails installed into only round holes of the hanger, and the model numbers ending with (max) refer to nails installed into both round and triangle holes of the hanger.
[2]Refer to Figures 1a and 1b (next page) for definitions of hanger nomenclature (W, H, B).

TABLE 15B—ALLOWABLE LOADS FOR THE IUS SERIES I-JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (inches)			COMMON NAILS (Quantity-Size)		ALLOWABLE LOADS[2,5] (lbs)			
						Uplift [3,4]	Download		
	W	H	B	Header	Joist	C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
IUS	$1^{7}/_{8}$ to $2^{5}/_{8}$	$9^{1}/_{4}$ to 16	2	8-10d	—	70	950	1,080	1,165
				10-10d	—	70	1,185	1,345	1,455
				12-10d	—	70	1,420	1,615	1,745
				14-10d	—	70	1,660	1,805	1,805
				16-10d	—	70	1,805	1,805	1,805
IUS	$2^{11}/_{16}$ to $3^{5}/_{8}$	$9^{1}/_{2}$ to 16	2	10-10d	—	70	1,185	1,345	1,455
				12-10d	—	70	1,420	1,615	1,745
				14-10d	—	70	1,660	1,805	1,805
				16-10d	—	70	1,805	1,805	1,805

For **SI:** 1 inch = 25.4 mm, 1 pound = 4.45 N.

[1]Refer to Figures 1a and 1b (next page) for definitions of hanger nomenclature (W, H, B).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads need not be reduced when other load durations govern.
[4]Additional uplift capacity is available when installing 2-10dx$1^{1}/_{2}$-inch-long nails through the triangular holes that are pre-punched in the U-shaped portion of the hanger and into the stiffened web of the prefabricated wood I-joist. When these additional nails are used, the maximum allowable uplift load is 345 lbs (C_D = 1.6).
[5]The allowable loads are based on the use of prefabricated wood I-joists having flanges with an allowable compression perpendicular-to-grain, $F_{c\perp}$, of 750 psi. When use is to support wood I-joists with flanges having an $F_{c\perp}$ of less than 750 psi, it must be verified that the combination of bearing capacity and joist nail capacity is adequate.

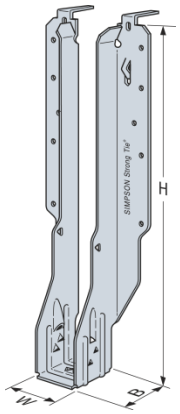


FIGURE 15A
TYPICAL IUS HANGER
(SEAT STYLE A)

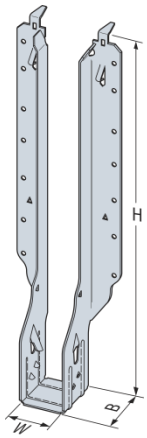


FIGURE 15B
TYPICAL IUS HANGER
(SEAT STYLE B)

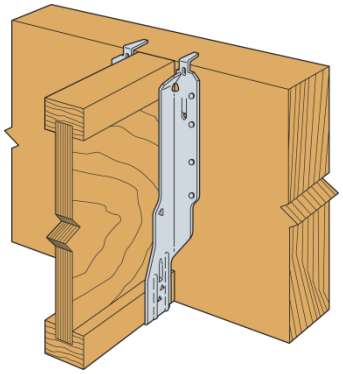


FIGURE 15C—TYPICAL IUS HANGER INSTALLATION
(Supported Wood I-joist without Web Stiffeners)

TABLE 16—ALLOWABLE LOADS FOR THE MIU SERIES JOIST HANGERS

MODEL NO.	DIMENSIONS[1] (in.)			FASTENERS[3,4]		ALLOWABLE LOADS (lbs) [2,5,6,7]			
						Uplift[3,4,5]	Download		
	W	H	B	Header	Joist	C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
MIU1.56/7	1 9/16	6 15/16	2 1/2	10-16d	4-10d x 1 1/2	555	1,440	1,635	1,760
MIU1.56/9		8 15/16		16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU1.56/11		11 1/16		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU1.56/14		13 7/16		22-16d	6-10d x 1 1/2	965	3,170	3,550	3,610
MIU1.81/7	1 13/16	6 15/16	2 1/2	10-16d	4-10d x 1 1/2	555	1,440	1,635	1,760
MIU1.81/9		8 13/16		16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU1.81/11		11 1/16		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU1.81/14		13 5/16		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU1.81/16		15 5/16		24-16d	6-10d x 1 1/2	965	3,455	3,902	4,045
MIU1.81/18		17 5/16		26-16d	6-10d x 1 1/2	965	3,745	4,020	4,045
MIU2.1/11	2 1/8	11 1/16	2 1/2	20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU2.37/7	2 3/8	6 15/16	2 1/2	10-16d	4-10d x 1 1/2	555	1,440	1,635	1,760
MIU2.37/9		9		16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU2.37/11		11 1/16		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU2.37/14		13 1/2		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU2.37/16		15 1/2		24-16d	6-10d x 1 1/2	965	3,455	3,920	4,045
MIU2.37/18		17 1/2		26-16d	6-10d x 1 1/2	965	3,745	4,045	4,045
MIU2.37/20		19 1/2		28-16d	6-10d x 1 1/2	965	4,030	4,060	4,060
MIU2.56/9	2 9/16	8 15/16	2 1/2	16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU2.56/11		11 1/16		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU2.56/13		12 1/4		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU2.56/14		13 7/16		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU2.56/16		15 7/16		24-16d	6-10d x 1 1/2	965	3,455	3,920	4,045
MIU2.56/18		17 7/16		26-16d	6-10d x 1 1/2	965	3,745	4,045	4,045
MIU2.56/20		19 7/16		28-16d	6-10d x 1 1/2	965	4,030	4,060	4,060
MIU3.12/9	3 1/8	9 1/16	2 1/2	16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU3.12/11		11 1/8		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU3.56/9	3 9/16	8 13/16	2 1/2	16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU3.56/11		11 1/8		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU3.56/14		13 5/16		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU3.56/16		15 5/16		24-16d	6-10d x 1 1/2	965	3,455	3,920	4,045
MIU3.56/18		17 5/16		26-16d	6-10d x 1 1/2	965	3,745	4,045	4,045
MIU3.56/20		19 5/16		28-16d	6-10d x 1 1/2	965	4,030	4,060	4,060
MIU4.12/9	4 1/8	9 1/16	2 1/2	16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU4.12/11		11 1/8		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU4.12/14		13 9/16		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU4.12/16		15 9/16		24-16d	6-10d x 1 1/2	965	3,455	3,920	4,045
MIU4.28/9	4 5/16	9	2 1/2	16-16d	6-10d x 1 1/2	965	2,305	2,615	2,820
MIU4.28/11		11 1/8		20-16d	6-10d x 1 1/2	965	2,880	3,135	3,135
MIU4.28/14		13 1/2		22-16d	6-10d x 1 1/2	965	3,170	3,595	3,875
MIU4.28/16		15 1/2		24-16d	6-10d x 1 1/2	965	3,455	3,920	4,045

Please refer to next page for continuation of Table 16.

TABLE 16—ALLOWABLE LOADS FOR THE MIU SERIES JOIST HANGERS (Continued)

MODEL NO.	DIMENSIONS[1] (in.)			FASTENERS[3,4]		ALLOWABLE LOADS (lbs)[2,5,6,7]			
						Uplift[3,4,5]	Download		
	W	H	B	Header	Joist	C_D = 1.6	C_D = 1.0	C_D = 1.15	C_D = 1.25
MIU4.75/9	$4^3/_4$	9	$2^1/_2$	16-16d	6-10d x $1^1/_2$	965	2,305	2,615	2,820
MIU4.75/11		$11^1/_{16}$		20-16d	6-10d x $1^1/_2$	965	2,880	3,135	3,135
MIU4.75/14		$13^1/_2$		22-16d	6-10d x $1^1/_2$	965	3,170	3,595	3,875
MIU4.75/16		$15^1/_2$		24-16d	6-10d x $1^1/_2$	965	3,455	3,920	4,045
MIU4.75/18		$17^1/_2$		26-16d	6-10d x $1^1/_2$	965	3,745	4,045	4,045
MIU4.75/20		$19^1/_2$		28-16d	6-10d x $1^1/_2$	965	4,030	4,060	4,060
MIU5.12/7	$5^1/_8$	$6^{15}/_{16}$	$2^1/_2$	10-16d	4-10d x $1^1/_2$	555	1,440	1,635	1,760
MIU5.12/9		$8^{13}/_{16}$		16-16d	6-10d x $1^1/_2$	965	2,305	2,615	2,820
MIU5.12/11		$11^1/_8$		20-16d	6-10d x $1^1/_2$	965	2,880	3,135	3,135
MIU5.12/14		$13^5/_{16}$		22-16d	6-10d x $1^1/_2$	965	3,170	3,595	3,875
MIU5.12/16		$15^5/_{16}$		24-16d	6-10d x $1^1/_2$	965	3,455	3,920	4,045
MIU5.12/18		$17^5/_{16}$		26-16d	6-10d x $1^1/_2$	965	3,745	4,045	4,045
MIU5.12/20		$19^5/_{16}$		28-16d	6-10d x $1^1/_2$	965	4,030	4,060	4,060

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Refer to Figure 8a (this page) for definitions of hanger nomenclature (W, H and B).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.
[3]The uplift loads have been increased for wind or earthquake loading with no further increase allowed. The allowable uplift loads must be reduced when other load durations govern.
[4]Total number of joist nails specified in the table must be installed to resist tabulated uplift loads. The bottom two 10dx$1^1/_2$ nails must be installed at a 45-degree angle into the bottom flange of the prefabricated wood I-joist, as shown in the detail in Figure 8b (on this page). These nails are designated as PAN nails.
[5]Web stiffeners must be installed on wood I-joists, with the following exception: An alternate reduced uplift capacity of 230 lbs can be obtained without using web stiffeners and all of the specified joist nails indicated in the table by installing two angled PAN nails at the hanger seat into the bottom flange of the wood I-joist as shown in the detail in Figure 8b (on this page).
[6]For joist members other than prefabricated wood I-joists, lateral support must be provided at the ends in accordance with Section 2308.8.2 of the IBC or Section R502.7 of the IRC.
[7]The tabulated allowable loads are based on the use of prefabricated wood I-joists having flanges with an allowable compression perpendicular-to-grain, $F_{c\perp}$, of 750 psi. When use is to support members having an $F_{c\perp}$ of less than 750 psi, it must be verified that the combination of bearing capacity and joist nail capacity is adequate.

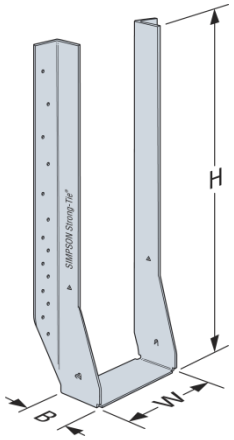


FIGURE 16A—MIU JOIST HANGER

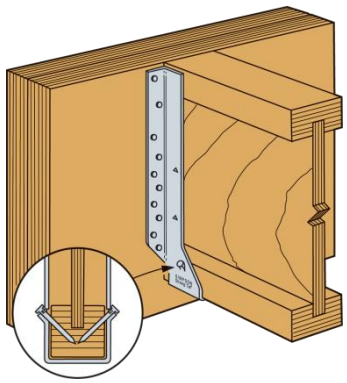


FIGURE 16B—ALTERNATE MIU JOIST HANGER INSTALLATION (See Footnotes 3 and 4 to Table 16)

TABLE 17A—ALLOWABLE LOADS FOR DU, DHU AND DHUTF SERIES JOIST HANGERS

MODEL SERIES	PRODUCT DIMENSIONS (inches)				INSTALLATION CONDITION [1,6,7]	ALLOWABLE LOADS (lbs)	
	W	H	Ga	B		Uplift C_D=1.60[3,4]	Download C_D=1.00[2]
DU	$1^{9}/_{16}$ to $2^{3}/_{8}$	$9^{1}/_{8}$ to $15^{15}/_{16}$	14	2.5	Over (2) layers $^{5}/_{8}$" gypsum wallboard	95	1,110
DHU DHUTF	$1^{9}/_{16}$ to $3^{9}/_{16}$	$9^{1}/_{8}$ to $23^{15}/_{16}$	12	3	Over (2) layer $^{5}/_{8}$" gypsum wallboard	95	1,410[5]

For **SI**: 1 inch = 25.4 mm, 1 lbf = 4.45N.

[1]Loads assume two layers of $^{5}/_{8}$-inch-thick Type X gypsum wallboard fastened using, at a minimum, the fastener schedule in Items 14.1-5 in IBC Table 721.1(2). Wall assembly must consist of minimum two 2x4 top plate members with studs spaced not more than 16 inches on center. Top plate member splices must occur at a stud location.

[2]Hangers spaced closer than 16 inches on center shall reduce capacity proportionately.

[3]Triangle nail holes may be filled with (4) additional 10d x $1^{1}/_{2}$ inch long nails to achieve an allowable uplift load of 750 lbs.

[4]DF carried members with minimum 1½ inches x $2^{1}/_{2}$ inches solid sawn dimensions increase the allowable uplift load to 170 lbs.

[5]Allowable download may be increased to 1,610 lbf for DF/SP using the DHU3.56/24 or DHUTF3.56/24 model hangers.

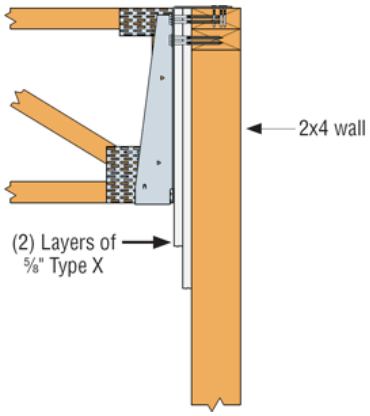
[6]DU and DHU are mounted with the top of the hanger flush with the top of the wall and tight to the gypsum wallboard.

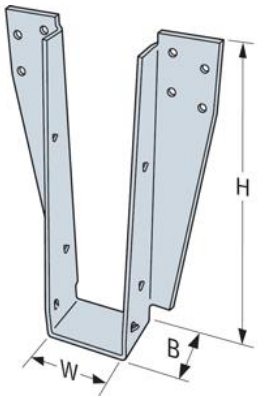
[7]Wall top plates must be restrained to prevent rotation. Use an SSP stud plate tie to connect the wall top plate to each wall stud on the opposite face of the wall or provide equivalent restraint by another method as determined by the designer. The Simpson Strong-tie Company SSP is a cold-formed, galvanized steel, strap connector that has a bent end that when installed bears on the top of the wall top plate. The SSP is to be attached to the side of the wall top plates with three 10d common nails, and attached to the wall stud with four 10d common nails.

TABLE 17B—FASTENER SCHEDULE FOR DHU, DHU AND DHUTF SERIES HANGERS

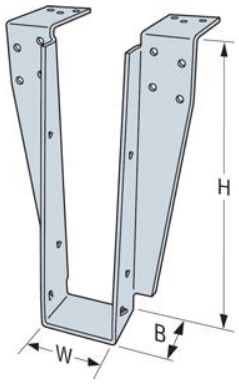
SERIES	FASTENERS		
	Joist[1]	Face	Top
DU	(2) 10d x $1^{1}/_{2}$"	(4) SDS $^{1}/_{4}$" x 3.5"	-
DHU	(2) 10d x $1^{1}/_{2}$"	(8) SDS $^{1}/_{4}$" x 3.5"	-
DHUTF	(2) 10d x $1^{1}/_{2}$"	(8) SDS $^{1}/_{4}$" x 3.5"	(6) 10d x $1^{1}/_{2}$"

[1]The two nails must be installed in the positive angle nailing (PAN) holes close to the hanger seat.

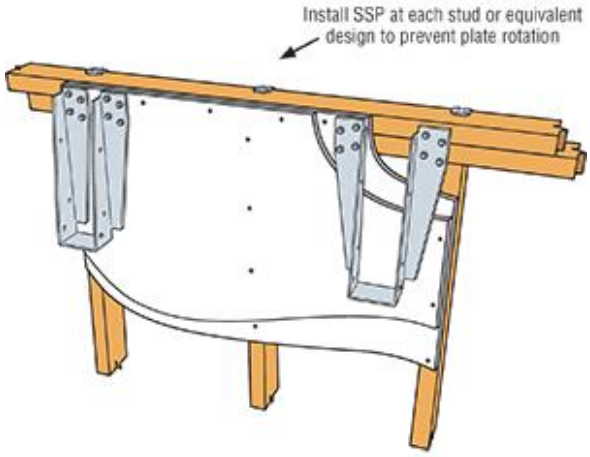




DHU (DU Similiar)



DHUTF



Typical DHU Installation

FIGURE 17—DU, DHU AND DHUTF HANGERS

TABLE 18A - CBH JOIST TO BEAM ALLOWABLE LOADS

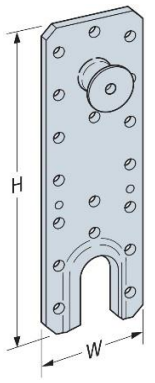
MODEL NO.	DIMENSIONS[1] (inches)		FASTENERS (Quantity-Type)		ALLOWABLE DOWNLOADS[2]
	W	H	Joist	Beam	C_D=1.00/ 1.15/1.25
CBH2.37x5.5	2 3/8	5 1/2	13-SDS1/4"x6"	13-SDS1/4"x3"	4830
CBH2.37x7.63	2 3/8	7 5/8	18-SDS1/4"x6"	18-SDS1/4"x3"	6510
CBH2.37x9.75	2 3/8	9 3/4	23-SDS1/4"x6"	23-SDS1/4"x3"	8785

TABLE 18B - CBH JOIST TO COLUMN ALLOWABLE LOADS

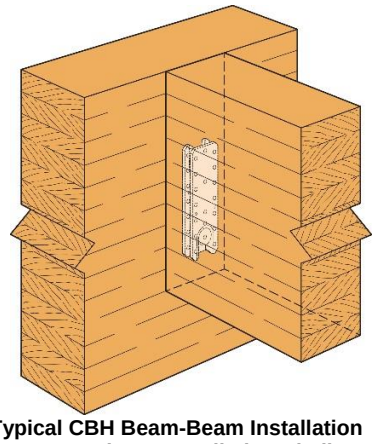
MODEL NO.	DIMENSIONS[1] (inches)		FASTENERS (Quantity-Type)		ALLOWABLE DOWNLOADS[2]
	W	H	Joist	Beam	C_D=1.00/ 1.15/1.25
CBH2.37x5.5	2 3/8	5 1/2	13-SDS1/4"x6"	13-SDS1/4"x6"	5455
CBH2.37x7.63	2 3/8	7 5/8	18-SDS1/4"x6"	18-SDS1/4"x6"	6630
CBH2.37x9.75	2 3/8	9 3/4	23-SDS1/4"x6"	23-SDS1/4"x6"	8940

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

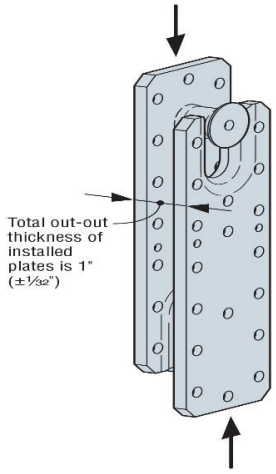
[1]Refer to Figure 18 (this page) for definitions of hanger nomenclature (W, H).
[2]Tabulated allowable loads must be selected based on duration of load as permitted by the applicable building code.



CBH Series Hanger



Typical CBH Beam-Beam Installation
Beam-Column Installation Similar



Typical CBH Assembly

FIGURE 18—CBH CONCEALED BEAM HANGERS

DIVISION: 06 00 00—WOOD, PLASTICS AND COMPOSITES
Section: 06 05 23—Wood, Plastic, and Composite Fastenings

REPORT HOLDER:

SIMPSON STRONG-TIE COMPANY INC.

EVALUATION SUBJECT:

SIMPSON STRONG-TIE® FACE-MOUNT HANGERS FOR SAWN LUMBER, STRUCTURAL COMPOSITE LUMBER (SCL), PREFABRICATED WOOD I-JOISTS AND GLULAM BEAMS (ENGINEERED WOOD PRODUCTS)

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that Simpson Strong-Tie® face-mount hangers for sawn lumber, structural composite lumber (SCL), prefabricated wood I-joists and glulam beams (engineered wood products), described in ICC-ES evaluation report ESR-2552, have also been evaluated for compliance with the codes noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

Applicable code editions:

- 2020 *City of Los Angeles Building Code* (LABC)
- 2020 *City of Los Angeles Residential Code* (LARC)

2.0 CONCLUSIONS

The Simpson Strong-Tie® face mount hangers for sawn lumber, structural composite lumber (SCL), prefabricated wood I-joists and glulam beams (engineered wood products), described in Sections 2.0 through 7.0 of the evaluation report ESR-2552, comply with the LABC Chapter 23, and the LARC, and are subjected to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The Simpson Strong-Tie® face mount hangers for sawn lumber, structural composite lumber (SCL), prefabricated wood I-joists and glulam beams (engineered wood products), described in this evaluation report supplement, must comply with all of the following conditions:

- All applicable sections in the evaluation report ESR-2552.
- The design, installation, conditions of use and identification are in accordance with the 2018 *International Building Code®* (2018 IBC) provisions noted in the evaluation report ESR-2552.
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.
- The supported end of joist or beam must be within $^1/_4$-inch from the supporting member.
- Solid blocking must be required for all joist hangers supporting roof joists having one end twisted more than one-half degree per foot of length relative to the other end, except as specifically noted in the evaluation report.
- Under the LARC, an engineered design in accordance with LARC Section R301.1.3 must be submitted.

This supplement expires concurrently with the evaluation report, reissued March 2020 and revised December 2020.

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ICC-ES Evaluation Report

ESR-2552 FBC Supplement

Issued July 2020
Revised December 2020
This report is subject to renewal March 2021

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DIVISION: 06 00 00—WOOD, PLASTICS, AND COMPOSITES
Section: 06 05 23—Wood, Plastic, and Composite Fastenings

REPORT HOLDER:

SIMPSON STRONG-TIE COMPANY INC.

EVALUATION SUBJECT:

SIMPSON STRONG-TIE® FACE-MOUNT HANGERS FOR SAWN LUMBER, STRUCTURAL COMPOSITE LUMBER (SCL), PREFABRICATED WOOD I-JOISTS AND GLULAM BEAMS (ENGINEERED WOOD PRODUCTS)

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Simpson Strong-Tie® face-mount hangers, described in ICC-ES evaluation report ESR-2552, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2020 and 2017 *Florida Building Code—Building*
- 2020 and 2017 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Simpson Strong-Tie® face-mount hangers, described in Sections 2.0 through 7.0 of ICC-ES evaluation report ESR-2552, comply with the *Florida Building Code—Building*, and the *Florida Building Code—Residential*, provided the design requirements are determined in accordance with the *Florida Building Code—Building* or the *Florida Building Code—Residential,* as applicable. The installation requirements noted in ICC-ES evaluation report ESR-2552 for the 2018 and 2015 *International Building Code®* meet the requirements of the *Florida Building Code—Building* or the *Florida Building Code—Residential,* as applicable.

Use of the Simpson Strong-Tie® face-mount hangers has also been found to be in compliance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building*, and the *Florida Building Code—Residential* with the following condition:

a. For connections subject to uplift, the connection must be designed for no less than 700 pounds (3114 N).

For products falling under Florida Rule 61G20-3, verification that the report holder's quality assurance program is audited by a quality assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official when the report holder does not possess an approval by the Commission).

This supplement expires concurrently with the evaluation report ESR-2552, reissued March 2020 and revised December 2020.

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Product Evaluation Report
MiTek® Inc.
RT16M Masonry Rafter Tie

Florida Product Approval #10739 R4
Florida Building Code 2020
Per Rule 61G20-3
Method 1-d

Category: Structural Components
Subcategory: Wood Connector
Compliance Method: 61G20-(1)(d)
HVHZ

Product Manufacturer:
MiTek Inc.
16023 Swingley Ridge Rd
Chesterfield, MO 63017

Engineer Evaluator:
Steven Brekke, P.E.
FL P.E. #70432

Validator:
Johnathan Green, P.E.
FL P.E. #88223

Contents:
Evaluation Report Pages 1-4



MiTek Inc.
16023 Swingley Ridge Road
Chesterfield, MO 63017-5746
uspcustomerservice@mii.com
1-800-328-5934

Compliance Statement:

The product as described in this report has demonstrated compliance with the Florida Building Code 2020, Sections 2304.10.3.

Product Description:

Designed as a retrofit connector for trusses installed on top plates. Can also be used as a hold-down for a roof or floor system.

Product Material:

18 Ga. Steel ASTM A653 Gr 40 G90 Galvanized

Materials:

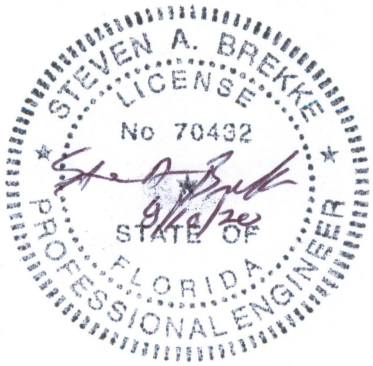
Wood
Wood members must be Douglas Fir with a minimum specific gravity (G) of 0.50, or an approved member with an equivalent specific gravity (G) of 0.50 or greater. Wood members shall have a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). Wood members must be used under continuously dry conditions and where sustained temperatures are limited to 100° F.

Preservative and Fire-Retardant-treated Wood
Connectors or fasteners used in contact with preservative-treated and fire-retardant-treated wood shall be in accordance with Section 2304.10.5.

Nail
Nails shall meet the requirements in Table and the Requirements specified in ASTM F 1667.

Fasteners
Required fastener types and sizes are specified in the Table.

Masonry
Material and quality of masonry construction must comply with the applicable sections of FBC 2020. Minimum concrete or grout compressive strength shall be 2500 psi.



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Design Procedure:

The allowable loads shown in the Table are based on allowable stress design and include a 60% increase for wind and seismic load conditions per Section 11.3.2 of the NDS. No further increases are permitted.

The allowable loads are based on wood members with a moisture content not exceeding 19 percent (16 percent for structural engineered lumber). If moisture content exceeds this value, a wet service factor, C_m, must be applied per Section 11.3.3 of the NDS. The allowable loads are based on wood members under continuously dry conditions and where sustained temperatures are limited to 100° F. Wood members that will experience sustained exposure to temperatures exceeding 100°F shall be adjusted by the temperature factor, C_t, per Section 11.3.4 of the NDS.

Wood members to which the anchor is attached must be designed by a registered design professional per FBC 2020. Masonry constructions must be a minimum of fully grouted, C90, CMU block designed by a registered design professional per FBC 2020. Concrete compressive strength shall be 2,500 psi or greater at 28 days.

Installation:

Install the rafter tie in accordance with this evaluation report and the manufacturer's installation instructions.

Testing and Specification:

1. ASTM D1761-88(2000): Job No. 08-6760 by Product Testing Inc.
2. ASTM D 1761 to ASTM D7147-11 comparison letter, sealed by Steven Brekke, P.E.
3. ANSI/AWC National Design Specifications for Wood Construction (NDS) 2018

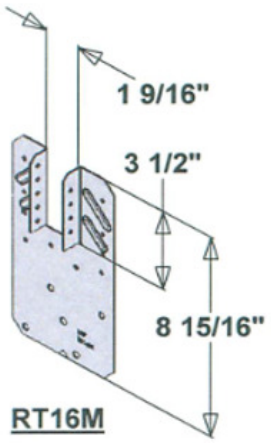
Quality Assurance Entity:

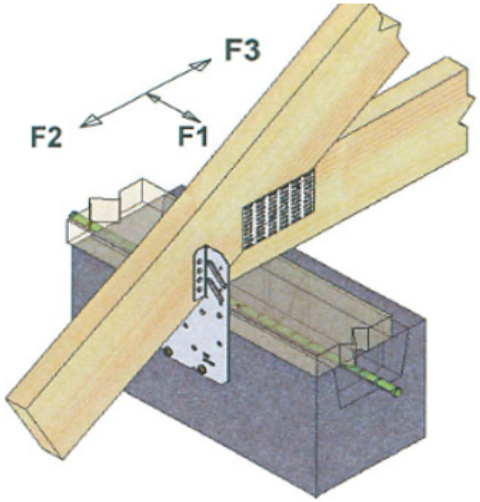
The manufacturer has established compliance of roof panel products in accordance with Florida Building Code and Rule 61G20-3.005 (3) for manufacturing under a quality assurance program audited by an approved quality assurance entity.

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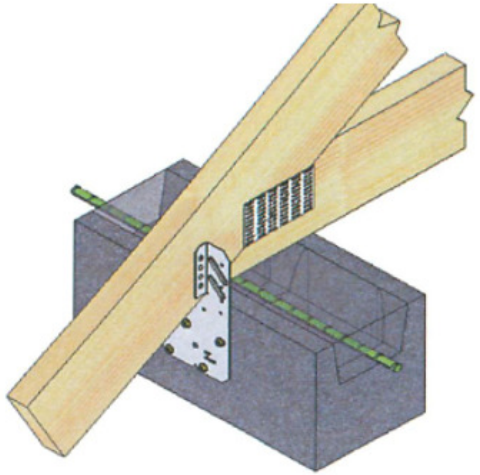


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INSTALLATION WITH TOP PLATE



INSTALLATION WITHOUT TOP PLATE

STOCK NO.	STEEL PROPERTIES [1]		FASTENER SCHEDULE [2, 3, 6]						ALLOWABLE LOADS (lbs.) [4,5]			
			Rafter/Truss		Top Plate		CMU/Concrete[7]		Uplift	Parallel to Wall (F1)	Perpendicular to Wall (F2)	Perpendicular to Wall (F3)
	Gauge	Grade	Qty	Size	Qty	Type	Qty	Type[5]	160%	160%	160%	160%
RT16M	18	40	9	10d x $1^1/_2$	--	--	4	$^1/_4$" Ø x $1^3/_4$" Tapcon®	1395	630	125	490
			9	10d x $1^1/_2$	4	16dC	2		1360	630	125	490

For SI: 1 inch = 25.4 mm, 1 psi = 6.89 kPa, 1 lbf = 4.45 N

[1]Steel conforms to ASTM A653 Gr40 standards with a minimum galvanized coating of G90.

[2]A 10d x $1^1/_2$ common nail has a diameter of 0.148 inch and a length of $1^1/_2$ inches.

[3]A 16d common nail has a diameter of 0.162 inch and a length of $3^1/_2$ inches.

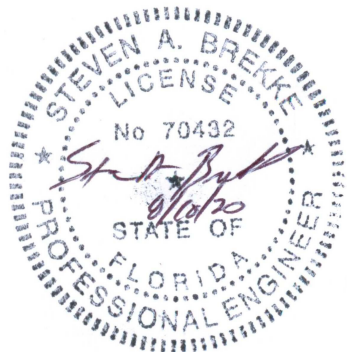
[4]Allowable loads have been increased 60% for wind and seismic loads. No further increase is permitted.

[5]Install with 1/4" x 1-3/4" Tapcon® Concrete Screws in accordance with the manufacturer's installation specifications. Concrete screws are not supplied with the RT16M connector.

[6]Install concrete screws in lower two holes for Single Top Plate or Conventional Raised Foundation or Modular Home Installations.

[7]Concrete compressive strength shall be 2,500 psi or greater at 28 days.

[8]Moisture barrier may be required.



MiTek Inc.
16023 Swingley Ridge Road
Chesterfield, MO 63017 800-328-5934
mitek-us.com

Name: RT16M Masonry Rafter Tie

Sheet 1
Date:
Aug 8, 2020

Steven Brekke PE
PROFESSIONAL ENGINEER
License #: 70432

Jax Apex Technology, Inc.
9000 Regency Square Boulevard, Suite 100
Jacksonville, FL 32211

Evaluation reports are the opinion of the engineer who prepared the report, based on the findings, and in no way constitute or imply approval by a local building authority. The engineer, in review of the data submitted, finds that, in his opinion, the product, material, system, or method of construction specifically identified in this report conforms with or is a suitable alternate to that specified in the Florida Building Code, SUBJECT TO THE LIMITATIONS IN THIS REPORT

Jeffrey P. Arneson, P.E., a licensed Florida professional engineer and employee of Jax Apex Technology, Inc. (Apex Technology) has reviewed the data submitted for compliance with the Florida Building Code. Neither Jeffrey P. Arneson, nor Apex Technology, are responsible for any errors or omissions to any documents, calculations, drawings, specifications, tests, or summaries prepared and submitted by the design professional or preparer of record who are listed in the Substantiating Data section of this report.

REPORT NO: SIM202001

CATEGORY: Structural Components

SUBCATEGORY: Wood Connectors

SUBMITTED BY:

SIMPSON STRONG-TIE COMPANY, INC.
5956 W. LAS POSITAS BOULEVARD
PLEASANTON, CA 94588

1. CERTIFICATION OF INDEPENDENCE:

Jeffrey P. Arneson, the Florida engineer who prepared this report, and Apex Technology have no financial interest in the manufacturing, sales, or distribution of the products included in this report. Jeffrey P. Arneson and Apex Technology comply with all criteria as stated in Florida Administrative Code Chapter 61G20-3.

2. PRODUCT NAMES:

Truss to Wall Connectors
MTSM16, MTSM20, HTSM16, HTSM20, HM9, HGAM10, H16, H16S, H16-2, H16-2S

Embedded Truss Anchors
META12, META16, META18, META20, META24, META40, HETA12, HETA16, HETA20, HETA24, HETA40, HHETA16, HHETA20, HHETA24, HHETA40, HETAL12, HETAL16, HETAL20, LTA2, DETAL20

Wood to Masonry Straps
MSTAM24, MSTAM36, MSTCM40, MSTCM60

Girder Tiedowns
LGT2, LGT3, LGT4, FGTR, FGTRHL, FGTRHR

3. SCOPE OF EVALUATION:

Load evaluation as a structural component using the requirements of the 7*th Edition (2020) Florida Building Code, Building* and *7th Edition (2020) Florida Building Code, Residential*.

4. DESCRIPTION:

4.1 MTSM and HTSM Twist Straps. The MSTM and HTSM twist straps are used to anchor wood trusses, rafters, or beams to masonry or concrete walls. They fasten to the wood member with 0.148"×3" or 0.148"×1½" nails, and fasten to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. MTSM twist straps are manufactured from 16 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. HTSM twist straps are manufactured from 14 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Twist strap fastener schedules, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.2 HM9 Hurricane Tie. The HM9 is used to anchor wood trusses, rafters, or joists to masonry or concrete. The HM9 fastens to the wood member with ¼"×1½" long Strong-Drive® SDS Heavy-Duty Connector screws and fastens to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The HM9 is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Hurricane tie fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.3 HGAM10 Hurricane Gusset Angle. The HGAM10 is used to anchor wood trusses, rafters, joists, or beams to masonry or concrete. The HGAM10 fastens to the wood member with ¼"×1½" long Strong-Drive® SDS Heavy-Duty Connector screws and fastens to masonry with ¼"×2¾" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The HGAM10 is manufactured from 14 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Angle fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.4 LTA2 Lateral Truss Anchor. The LTA2 is used to anchor wood trusses, rafters, or beams to masonry or concrete. The LTA2 fastens to the wood member with 0.148"×1½" nails and has legs which are embedded into the wall system. The LTA2 is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Lateral truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.5 H16, H16S, H16-2, and H16-2S Hurricane Ties. The H16, H16S, H16-2, and H16-2S are used to anchor wood trusses or rafters to masonry or concrete walls. These connectors wrap over the top of the rafter or truss. The straps are fastened to a masonry wall with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. These connectors are manufactured from No. 18 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 1. See Figures 1 and 2 for additional details.

4.6 META, HETA, HHETA, and HETAL Embedded Truss Anchors. Embedded truss anchors are used to anchor a wood member (usually a truss) to masonry or concrete. Embedded truss anchors fasten to a single-ply wood truss with 0.148"×1½" nails or

to a multi-ply truss with 0.162"×3½" nails. META, HETA, and HHETA models can be installed in a single application or doubled to achieve additional capacity. They are embedded in the masonry or concrete to a depth indicated on the side of the anchor (4" for META, HETA, and HHETA, and $5^{1}/_{16}$" for HETAL). The strap portion of the anchor is 1⅛" wide. The anchors are manufactured from steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish, with the exception of the HETAL truss seat which is manufactured from steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Steel thickness is as specified in Table 2. Embedded truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 2 for single installations and Table 3 for double installations. See Figures 3 and 4 for additional details of single and double embedded truss anchors.

4.7 DETAL20 Double Embedded Truss Anchor. The DETAL20 is a high capacity connector used to anchor single-ply wood trusses or rafters to masonry or concrete. The DETAL fastens to the wood member with 0.148"×1½" nails. It is embedded in the masonry or concrete to a depth of 4½". The strap portion of the anchor is 1⅛" wide and is manufactured from 16 gauge steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish. The truss seat is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Embedded truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 3. See Figure 4 for additional details.

4.8 MSTAM and MSTCM Strap Ties. The MSTAM and MSTCM strap ties are used to provide a tension connection between wood members and a masonry or concrete structure. The MSTAM straps are 1¼" wide for use on single 2× and thicker members. They fasten to the wood member with 0.148"×3" nails and fasten to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The MSTCM straps are 3" wide for use on double 2× or single 4× and larger members. They fasten to the wood member with 0.148"×3¼" nails and fasten to masonry with ¼"×2¼" Titen Turbo or Titen 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The MSTCM straps have countersunk nail slots for a lower nailing profile and coined edges for safer handling. MSTAM and MSTCM straps are manufactured from steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish. Steel thickness is as specified in Table 4. Masonry strap fastener schedule, dimensions, and allowable loads are shown in Table 4. See Figure 5 for additional details.

4.9 FGTR, FGTRHL, and FGTRHR Face Mount Girder Tiedowns. The FGTR is a non-pitch specific girder tie down that can be used in new construction or retrofit applications to tie down a girder truss or beam to a concrete or masonry. The FGTR can be installed in a single application or doubled to achieve additional uplift capacity. The FGTR plate fastens to the wood member with Strong-Drive® SDS Heavy-Duty Connector screws and the strap fastens to masonry or concrete with ½"×5" Titen HD® Heavy-Duty Screw Anchors. The FGTRHL and FGTRHR are designed with the strap at a 45 degree angle for corner hip applications. The FGTR straps are manufactured from 7 gauge ASTM A1011 Grade 33 steel and the plates are made from 3 gauge ASTM A1011 Grade 33 steel. Both components have a powder coat painted finish. Girder tie down fastener schedule, dimensions, and allowable loads are shown in Table 5. See Figure 6 for additional details.

4.10 LGT2 Light Truss/Girder Tiedown. The LGT2 is used to anchor a 2-ply wood truss or beam (maximum 3" wide) to wood, concrete, or masonry. The LGT2 fastens to the wood truss or beam with 0.148"×3¼" nails or #9×1½" Strong-Drive® SD Connector screws. For wood wall applications, it is fastened with 0.148"×3¼" nails

or #9×1½" Strong-Drive SD Connector screws. For masonry or concrete applications, it fastens to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The LGT2 is manufactured from 14 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

4.11 LGT3-SDS2.5 Light Truss/Girder Tiedown. The LGT3-SDS2.5 is used to anchor a 3-ply wood truss or beam (maximum 5" wide) to wood, concrete, or masonry. The LGT3-SDS2.5 fastens to the wood truss or beam with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws. For wood wall applications, it is fastened to wood studs and plates with 0.148"×3¼" nails. For masonry or concrete applications, it is fastened with ⅜"×5" Titen HD® Heavy-Duty Screw Anchors. The LGT3-SDS2.5 is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

4.12 LGT4-SDS3 Light Truss/Girder Tiedown. The LGT4-SDS3 is used to anchor a 4-ply wood truss or beam (maximum 6½" wide) to wood, concrete, or masonry. The LGT4-SDS3 fastens to the wood truss or beam with ¼"×3" long Strong-Drive® SDS Heavy-Duty Connector screws. For wood wall applications, it is fastened to wood studs and plates with 0.148"×3¼" nails. For masonry or concrete applications, it is fastened with ⅜"×5" Titen HD® Heavy-Duty Screw Anchors. The LGT4-SDS3 is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

5. MATERIALS:

5.1 Steel. Steel specifications for each product listed in this evaluation report shall be as indicated in the previous section. In addition to the standard G90 coating, some models (designated with a model number ending with Z) are available with a G185 zinc coating specification in accordance with ASTM A653. Some models (designated with a model number ending with HDG) are available with a hot-dip galvanization, also known as “batch” galvanization, in accordance with ASTM A123, with a minimum specified coating weight of 2.0 ounces of zinc per square foot of surface area, total for both sides. Some models (designated with a model number ending with SS) are available in Type 316L stainless material manufactured in accordance with ASTM A240 sheet, strip or plate and ASTM A480 (General Requirements) in the following designation: UNS designation S31603, AISI Type 316L. Model numbers in this report may not include the Z, HDG, or SS ending, but the information shown applies.

5.2 Wood. Wood members to which these connectors are fastened shall be solid sawn lumber, glued-laminated lumber, or structural composite lumber having dimensions consistent with the connector dimensions shown in Tables 1 through 6. Unless otherwise noted, lumber shall be Southern Pine (SP) or Douglas Fir-Larch (DF) having a minimum specific gravity of 0.55 or 0.50, respectively. Where indicated by SPF, lumber shall be Spruce-Pine-Fir having a minimum specific gravity of 0.42.

5.3 Concrete/Masonry. Concrete and masonry design specifications shall be the stricter of the specifications by the engineer of record, the Florida Building Code minimum standards, the following, or as noted in the report:

Material	Specification	Minimum Compressive Strength
Concrete, f'_c	-	2,500 psi
Masonry, f'_m	ASTM C1314	1,500 psi
Masonry Unit	ASTM C90	2,000 psi
Mortar	ASTM C270 Type S	1,800 psi (or by proportions)
Grout	ASTM C476	2,000 psi (or by proportions)

5.4 Nails. Unless noted otherwise, nails shall be common nails. Nails shall comply with ASTM F1667 and shall have the minimum bending yield strengths F_{yb}:

Nail Pennyweight	Nail Shank Diameter (inch)	Nail Length (inch)	F_{yb} (psi)
10d×1½	0.148	1½	90,000
10d Common	0.148	3	90,000
16d sinker	0.148	3¼	90,000
16d Common	0.162	3½	90,000

Fasteners for galvanized connectors in pressure-preservative treated wood shall be hot-dipped zinc coated galvanized steel with coating weights in accordance with ASTM A153 or steel mechanically galvanized in accordance with ASTM B695, Class 55. Fasteners for stainless steel connectors shall be stainless steel except where otherwise permitted by the treatment manufacturer.

The allowable loads of stainless-steel connectors match those of carbon-steel connectors when installed with Simpson Strong-Tie stainless-steel, SCNR ring-shank nails, unless noted otherwise.

5.5 Strong-Drive® SDS Heavy-Duty Connector Screws. Fasteners used with the connectors described in Tables 1, 5, and 6, and shown in Figures 2, 6, 8, and 10 of the report must be Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector wood screws recognized in FL9589. Model numbers shown in this report may not include the full SDS model number after the connector model number (e.g., LGT3-SDS2.5), but the information shown applies. SDS screws used in contact with preservative-treated or fire-retardant-treated lumber must, as a minimum, comply with FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.6 Strong-Drive® SD Connector Screws. Certain connectors described in this report may be installed with Simpson Strong-Tie® Strong-Drive SD Connector wood screws recognized in FL9589. Installation shall be as specified in FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.7 Titen Turbo™ and Titen® 2 Concrete and Masonry Screw Anchors. Titen Turbo™ screw anchors shown in Tables 1, 4, and 6 and Figures 2, 5, and 7 are recognized in FL16230. Alternately, Titen® 2 screw anchors of the same diameter and length may be substituted for Titen Turbo models shown with no change to the

tabulated allowable loads. Titen 2 screw anchors are recognized in FL16230. Installation for both screw anchors shall be as specified in FL16230.

5.8 Titen HD® Heavy-Duty Screw Anchors. Anchors shown in Tables 5 and 6 and Figure 6 shall be Simpson Strong-Tie® Titen HD Heavy-Duty Screw Anchors for masonry or concrete as recognized in FL15730. Installation shall be as specified in FL15730.

6. INSTALLATION:

Installation shall be in accordance with this report and the most recent edition of the Simpson Strong-Tie *Wood Construction Connectors* catalog. Information in this report supersedes any conflicting information between information provided in this report and the catalog.

7. SUBSTANTIATING DATA:

Test data submitted by Testing Engineers Inc., Product Testing, Inc., and Simpson Strong-Tie, and signed and sealed calculations performed by Bryan Wert, P.E., in accordance with the 7th *Edition (2020) Florida Building Code, Building* and *Residential* codes.

Product	Test Lab	Test Number
MTSM, HTSM	Testing Engineers, Inc.	H756, I841, O555, O564
HM9	Product Testing, Inc.	02-3793
HGAM10	Product Testing, Inc.	L007, L018
	Testing Engineers, Inc.	H046, H141
H16, H16S, H16-2, H16-2S	Testing Engineers, Inc.	H591, I376
META, HETA, HHETA, HETAL	Product Testing, Inc.	K966, K997, L008, L011, L012, L013, L014, L015, L019, L020, L021, L022, L023, P670, Q219, Q220, Q221
	Testing Engineers, Inc.	D792, D793, D844, N218
DETAL	Testing Engineers, Inc.	O795, O796, O797, O798, O799
LTA2	Testing Engineers, Inc.	P966, P967, P968, Q018, Q019, Q020, Q856
MSTCM	Testing Engineers, Inc.	N471
FGTR	Product Testing, Inc.	K930, K931, K987
FGTRHL/R	Product Testing, Inc.	K932
LGT2	Simpson Strong-Tie	23-19086, 23-19111
	Testing Engineers, Inc.	I839, L921, L922
LGT3	Testing Engineers, Inc.	L233, L234, L431, L918
LGT4	Testing Engineers, Inc.	O113, O393, O394, O587

8. FINDINGS:

The connectors listed in this evaluation report comply with the 7th *Edition (2020) Florida Building Code, Building*, and 7th *Edition (2020) Florida Building Code, Residential* when installed in accordance with this report.

9. LIMITATIONS:

1. Maximum allowable loads shall not exceed the allowable loads listed in this report. Allowable loads listed in this report are based on allowable stress design. The loads in this report are not applicable to Load and Resistance Factor Design.

2. Capacity of wood members is not covered by this report. Capacity of wood members must be checked by the building designer.

3. Connectors in this report having a galvanized coating thickness less than G185 shall not be installed in contact with preservative-treated wood products that are exposed to rainfall or ground moisture.

4. Allowable loads for more than one direction for a single connection cannot be added together. A design load that can be divided into components in the directions given must be evaluated as follows:

   (Design Uplift / Allowable Uplift) + (Design Lateral Parallel to Plate / Allowable Lateral Parallel to Plate) + (Design Lateral Perpendicular to Plate / Allowable Lateral Perpendicular to Plate) < 1.0

   As an alternate, the embedded truss anchors in Tables 2 and 3 can be evaluated using the following: The design load in each direction shall not exceed the published allowable load in that direction multiplied by 0.75.

10. ALLOWABLE LOADS:

The tables that follow reference the allowable loads for the aforementioned products.

TABLE 1: MTSM, HTSM, HM9, HGAM10, H16, and LTA2 Dimensions, Fasteners, and Allowable Loads												
Model No.	Ga.	L (in.)	Fasteners			Allowable Loads (lb.)					
			Rafter / Truss	GFCMU (Titen Turbo™)[6]	Concrete (Titen Turbo™)[6]	DF/SP (160)			SPF/HF (160)		
						Uplift	F_1	F_2	Uplift	F_1	F_2
MTSM16	16	16	(7) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	830	120[3]	90[3]	715	120[3]	90[3]
MTSM20	16	20	(7) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	830			715		
HTSM16	14	16	(8) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	1,110			955		
HTSM20	14	20	(10) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	1,110			955		
HM9[9]	18	–	(4) ¼"×1½" SDS	(5) ¼"×2¼"	(5) ¼"×1¾"	760	670	190	760	670	190
HGAM10[9]	14	–	(4) ¼"×1½" SDS	(4) ¼"×2¾"	(4) ¼"×1¾"	810	875	1,105[5]	585	630	795[5]
H16[10]	18	18¾"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370	–	–	1,180	–	–
H16S[10]	18	11 11/16"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
H16-2[10]	18	18¾"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
H16-2S[10]	18	11 11/16"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
LTA2 Perp.-to-Wall Installation	18	–	(10) 0.148"×1½"	Embed	Embed	1,180[7]	415	875	990	415	735
LTA2 Parallel-to-Wall Installation	18	–	(10) 0.148"×1½"	Embed	Embed	1,180[7]	950	220	990	800	220

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. MTSM and HTSM do not have to be wrapped over the truss/rafter to achieve tabulated loads.
3. Lateral loads apply for the MTSM and HTSM twist straps when the first seven nail holes near the bend line are filled on the truss/rafter side. Any other fasteners required can be installed in any open hole.
4. Allowable loads for the HGAM10 are for one connector. A minimum truss/rafter thickness of 2½" must be used when framing anchors are installed on each side of the joist or truss.
5. HGAM10 F_2 loads are for forces into the connector. F_2 loads away from the connector are 640 lb. (DF/SP) and 460 lb. (SPF/HF).
6. Hex-head model required. Minimum edge distance for Titen Turbo™ screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.
7. LTA2 allowable uplift on SP is 1,350 lb. for perpendicular-to-wall installation and parallel-to-wall installation.
8. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
9. The HM9KT and HGAM10KTA are kits with (20) HM9 and (10) HGAM10 connectors packaged with Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws and 2¼" and 2¾" Titen® 2 screw anchors, respectively. (1¾" Titen 2 screw anchors for concrete installations sold separately)
10. H16 models are pre-sloped at a 5:12 pitch and can be used on pitches from 3:12 to 7:12. The minimum heel height for H16 models is 4" and the maximum heel height is 13½" (Use H16 or H16-2).

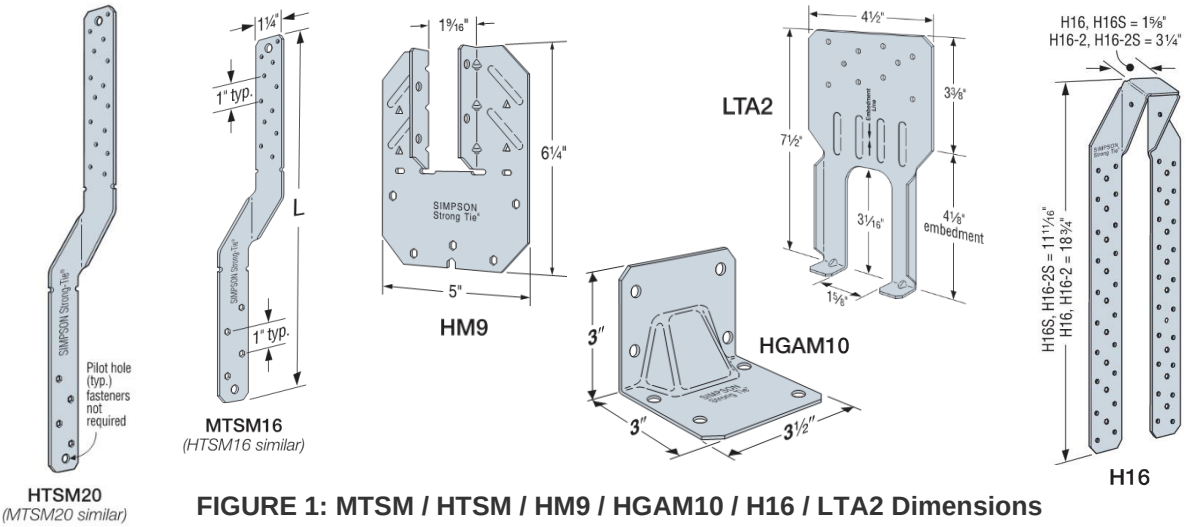


FIGURE 1: MTSM / HTSM / HM9 / HGAM10 / H16 / LTA2 Dimensions

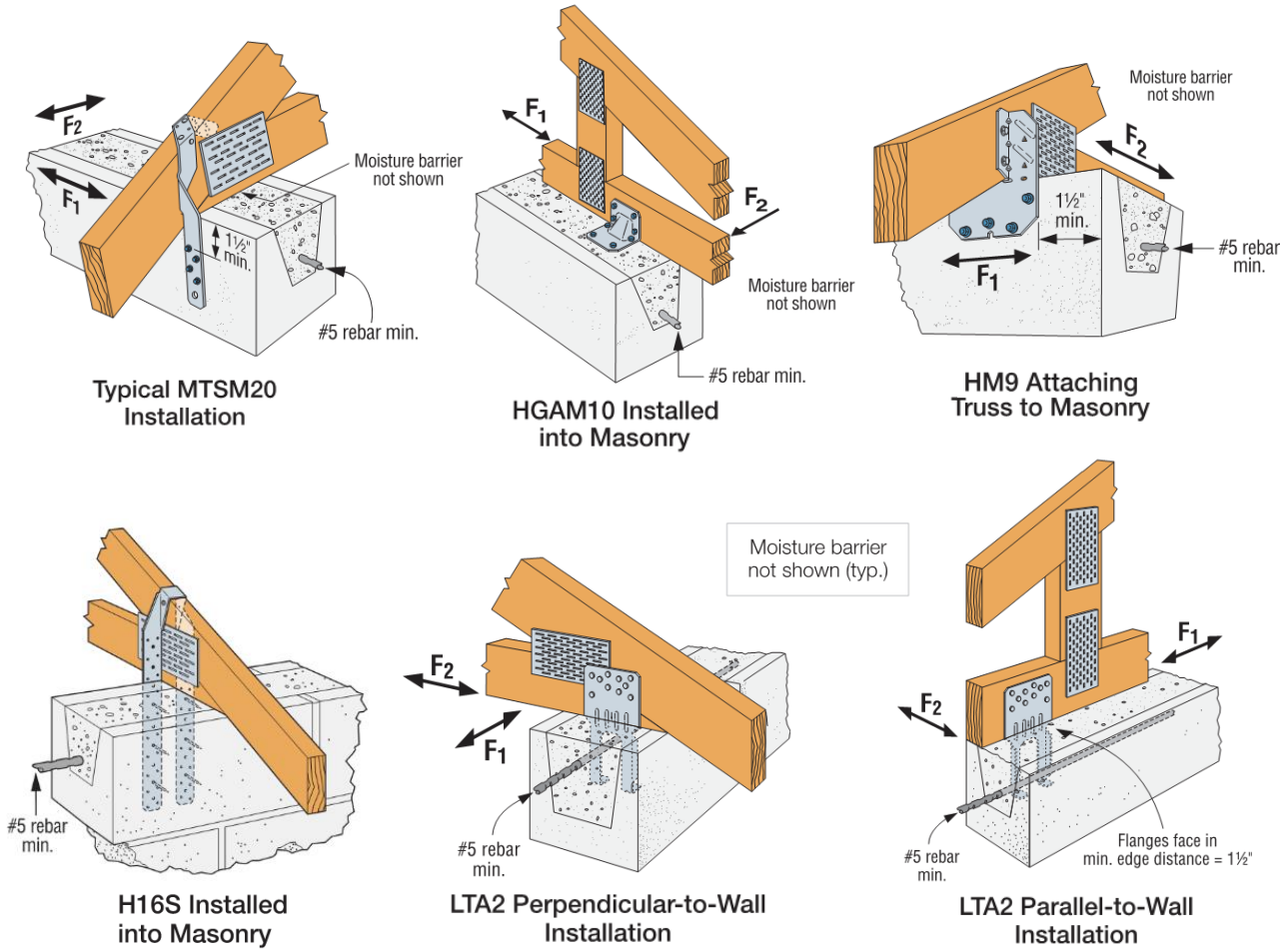


FIGURE 2: MTSM / HTSM / HM9 / HGAM10 / H16 / LTA2 Installation Details

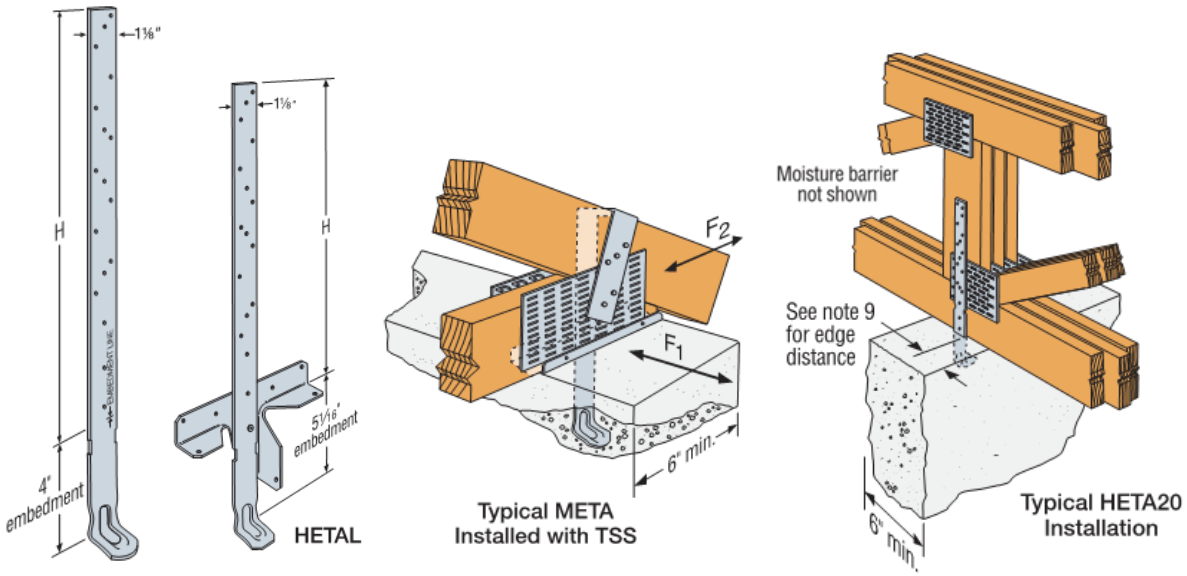


FIGURE 3: META / HETA / HHETA / HETAL Dimensions and Installation Details

TABLE 2: META, HETA, HHETA, and HETAL Dimensions, Fasteners, and Allowable Loads								
Model No.	Ga.	H (in.)	SP Allowable Loads (lb.)					
			1-ply Rafter / Truss (0.148"×1½" Nails)		2- or 3-ply Rafter / Truss (0.162"×3½" Nails)		F_1 (160)	F_2 (160)
			Qty.[2]	Uplift (160)	Qty.[2]	Uplift (160)		
META12	18	8	7	1,420	6	1,450	340	770
META16		12	8	1,450	6	1,450		
META18		14	8	1,450	6	1,450		
META20		16	7	1,420	5	1,210		
			8	1,450	6	1,450		
META24		20	8	1,450	6	1,450		
META40		36	8	1,450	6	1,450		
HETA12	16	8	7	1,455	7	1,730	340	770
HETA16		12	9	1,810	8	1,810		
HETA20		16	8	1,655	7	1,730		
			9	1,810	8	1,810		
HETA24		20	9	1,810	8	1,810		
HETA40		36	9	1,810	8	1,810		
HHETA16	14	12	10	2,120	9	2,120	340[7]	770
HHETA20		16	9	1,935	8	2,030		
			10	2,120	9	2,120		
HHETA24		20	10	2,120	9	2,120		
HHETA40		36	10	2,120	9	2,120		
HETAL12	16	7	10[3]	1,040	10[3]	1,235	390[4]	1,040
HETAL16		11	14[3]	1,810	13[3]	1,810		
HETAL20		15	14[3]	1,810	13[3]	1,810		

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. The META, HETA, and HHETA, require the lowest four nail holes be filled.
3. Five nails must be installed into the truss seat of the HETAL.
4. F_1 load towards face of HETAL is 1,870 lb.
5. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
6. It is acceptable to use a reduced number of fasteners in any model listed provided that there is a reduction in uplift load capacity. The load per nail can be approximated by dividing the allowable load by the number of nails. There should be a minimum of 4 nails installed in the strap. Lateral loads do not apply when fewer than 7 fasteners are used with the HETA and HHETA anchors or less than (6) 0.162"×3½" or (7) 0.148"×1½" fasteners are used with the META anchor.
7. The HHETA allowable F_1 load can be increased to 435 lb. if the strap is wrapped over the truss and a minimum of 12 nails are installed.
8. Straps do not have to be wrapped over the truss/rafter to achieve tabulated loads unless specifically noted otherwise.
9. Minimum edge distance for META, HETA, and HHETA is 1½" for concrete and 2" for masonry.

TABLE 3: DETAL20 and Double META, HETA, and HHETA Fasteners and Allowable Loads										
Model No.	Qty.	Application	SP Allowable Loads (lb.)							
			1-ply Rafter / Truss				2- or 3-ply Rafter / Truss			
			0.148"×1½" Nail Qty.[4]	Uplift	F_1	F_2	0.162"×3½" Nail Qty.[4]	Uplift	F_1	F_2
DETAL20	1	GFCMU	18	2,480	2,000[5]	1,370	—	—	—	—
		Concrete	18	2,480	2,000	1,465	—	—	—	—
META	2	GFCMU	10	1,875	Refer to Table 2[6]		14	1,795	1,285[7]	1,080
		Concrete	10	1,875			14	2,435	1,285[7]	1,080
HETA	2	GFCMU	10	1,920			12	2,365	1,350[7]	1,430
		Concrete	10	1,920			12	2,560	1,350[7]	1,430
HHETA	2	GFCMU	10	1,920			12	2,365	1,350[7]	1,430
		Concrete	10	1,920			14	3,180	1,350[7]	1,430

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
3. Install with spoons facing outward and spaced no more than ⅛" wider than the truss width.
4. The DETAL20 requires (6) 0.148"×1½" nails in the truss seat and (6) 0.148"×1½" nails into each strap. For all other models, install half of the required number of fasteners in each strap.
5. F_1 loads for the DETAL20 in GFCMU may cause an additional 1/32" deflection beyond the standard ⅛" limit.
6. Lateral loads for double META, HETA, and HHETA apply only to 2- or 3-ply applications with anchors spaced a minimum of 3" apart. For 1-ply applications, use lateral loads in Table 2. DETAL lateral loads apply to 1-ply application.
7. F_1 loads for double META, HETA, and HHETA may cause an additional 1/16" deflection beyond the standard ⅛" limit when the straps are installed not wrapped over the heel as shown.

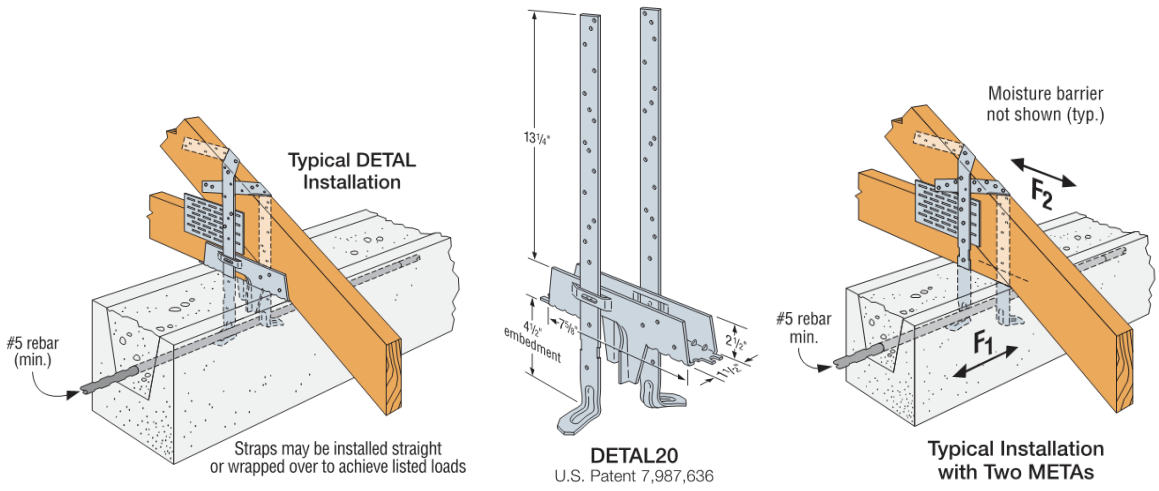


FIGURE 4: DETAL20 Dimensions and Double META / HETA / HHETA Installation Details

TABLE 4: MSTAM and MSTCM Dimensions, Fasteners, and Allowable Loads								
Model No.	Ga.	W (in.)	L (in.)	Fasteners			Allowable Tension / Uplift (lb.)	
				Rafter / Truss	GFCMU (Titen Turbo™)[3]	Concrete (Titen Turbo™)[3]	DF/SP/SPF/HF (160)	
							GFCMU	Concrete
MSTAM24	18	1¼	24	(9) 0.148"×3"	(5) ¼"×2¼"	(5) ¼"×1¾"	1,425	1,450
MSTAM36	16	1¼	36	(13) 0.148"×3"	(8) ¼"×2¼"	(8) ¼"×1¾"	1,870	1,870
MSTCM40	16	3	40¼	(26) 0.148"×3¼"	(14) ¼"×2¼"	(14) ¼"×1¾"	4,220	4,220
MSTCM60	16	3	59½	(26) 0.148"×3¼"	(14) ¼"×2¼"	(14) ¼"×1¾"	4,220	4,220

Notes:

1. Tension loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
3. Hex-head model required. Minimum edge distance for Titen Turbo™ screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.

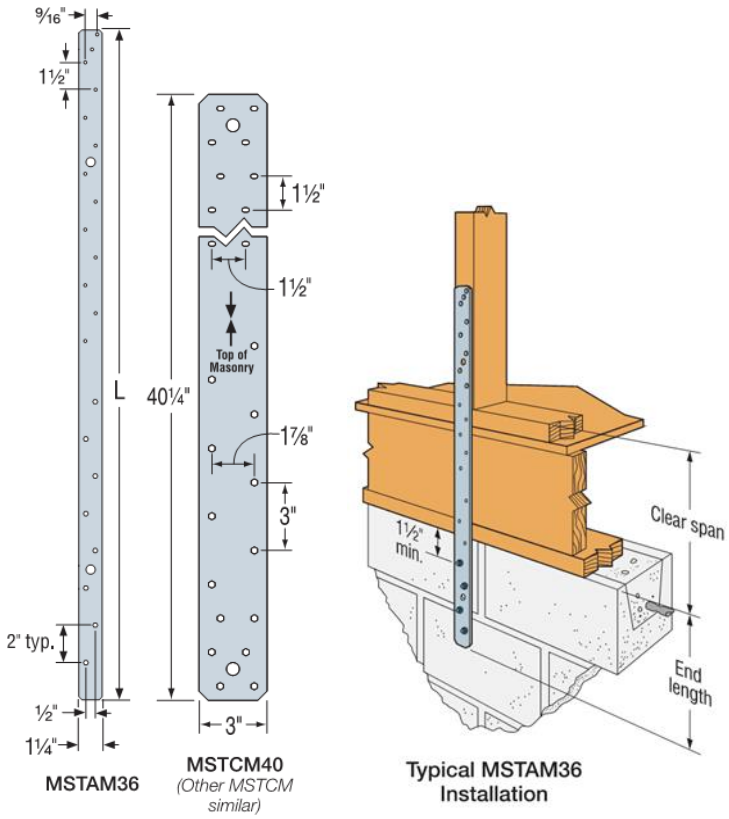


FIGURE 5: MSTAM / MSTCM Dimensions and Installation Details

TABLE 5: FGTR, FGTRHL, and FGTRHR Fasteners and Allowable Loads					
Model No.	Qty.	Fasteners		Allowable Uplift Load (lb.)	
		Rafter / Truss (Strong-Drive® SDS)	GFCMU / Concrete (Titen HD®)	DF/SP (160)	SPF/HF (160)
FGTR	1	(18) ¼"×3"	(2) ½"×5"	4,725[6]	3,400
	2	(36) ¼"×3"	(4) ½"×5"	8,885	6,395
FGTRHL/R	1	(18) ¼"×3"	(2) ½"×5"	3,635	2,615

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Products shall be attached to grouted and reinforced CMU walls or reinforced concrete walls that are designed to transfer the uplift loads to the foundation.
3. Minimum edge distance for the Titen HD anchor is 4".
4. Titen HD anchors should be spaced in every other hole on the part.
5. Attached members must be designed to resist the applied loads.
6. Uplift for FGTR corner installations shall be limited to 4,425 lb.
7. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.

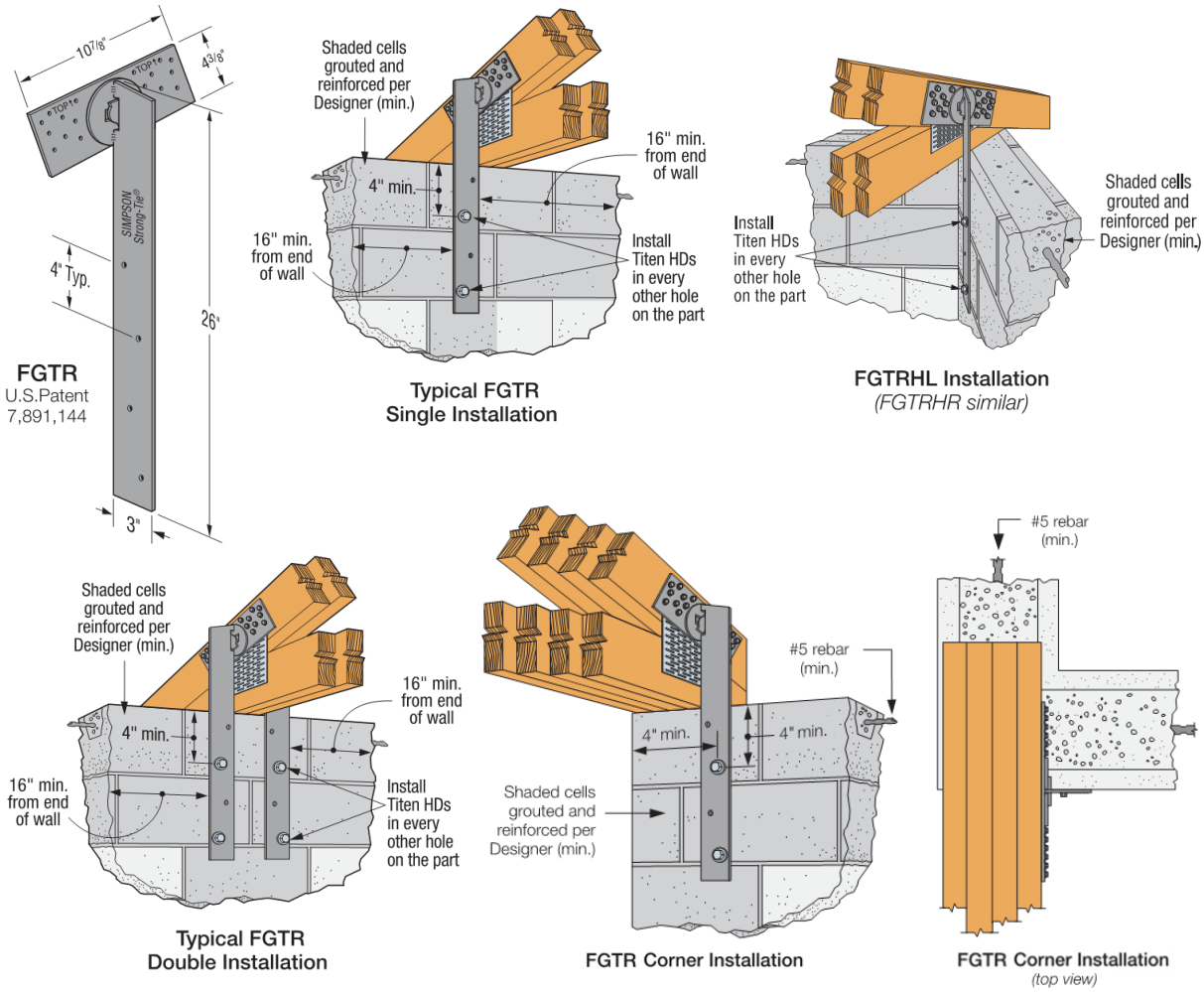


FIGURE 6: FGTR / FGTRHL / FGTRHR Dimensions and Installation Details

TABLE 6: LGT Fasteners and Allowable Loads										
Model No.	Qty.	No. of Plies	Applica-tion	Fasteners			Allowable Loads (lb.)			
							Uplift (160)		Lateral (160)	
				Rafter / Truss	Wood Studs / Framing	GFCMU / Concrete	DF/SP	SPF/HF	DF/SP/SPF/HF	
									F_1	F_2
LGT2	1	2	Wood	(16) 0.148"×3¼"	(14) 0.148"×3¼"	—	2,040	1,755	700[3]	170[3]
				(16) #9×1½" SD	(14) #9×1½" SD	—	2,465	2,125		
			GFCMU or Conc.	(16) 0.148"×3¼"	—	(7) ¼"×2¼" Titen Turbo[5]	2,030	1,750		
LGT3-SDS2.5	1	3	Wood	(12) ¼"×2½" SDS	(21) 0.148"×3¼"	—	3,480	2,505	795	385
			GFCMU or Conc.	(12) ¼"×2½" SDS	—	(4) ⅜"×5" Titen HD	3,285	2,365		
LGT4-SDS3	1	4	Wood	(16) ¼"×3" SDS	(30) 0.148"×3¼"	—	4,060	2,920	2,000[4]	675[4]
			GFCMU or Conc.	(16) ¼"×3" SDS	—	(4) ⅜"×5" Titen HD	3,285	2,365		

Notes:
1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Tabulated LGT2 uplift with SD screws is for DF lumber. LGT2 uplift using SD screws with SP lumber is 2,670 lb.
3. LGT2 lateral loads require installation of optional (4) 0.148"×3¼" in triangle fastener holes into top plates.
4. LGT4 lateral loads require installation of optional (7) 0.148"×3¼" in triangle fastener holes into top plates.
5. Hex-head model required. For concrete applications, use ¼"×1¾" Titen Turbo™. Minimum edge distance for Titen Turbo screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.
6. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.

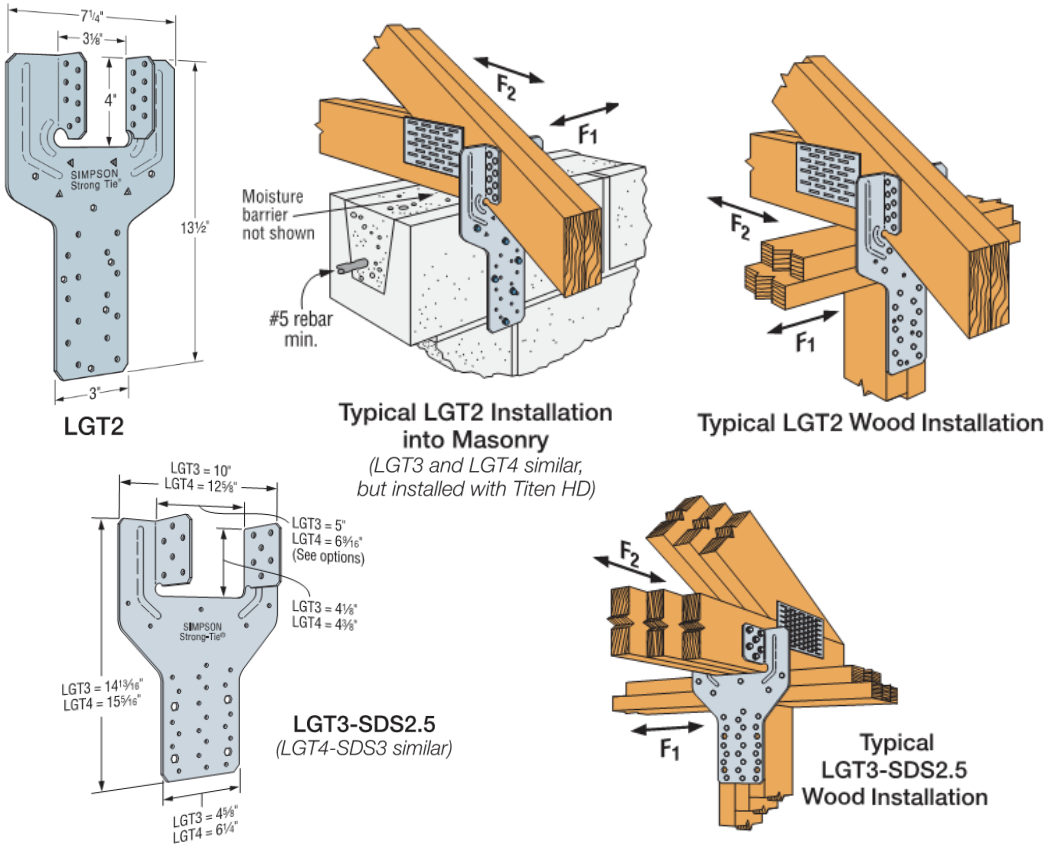


FIGURE 7: LGT2 / LGT3 / LGT4 Dimensions and Installation Details

11. REFERENCES:

Florida Building Code, Building 7th Edition (2020)

Section 104.11	Alternative materials, design, and methods of construction and equipment
Chapter 19	Concrete
Chapter 21	Masonry
Chapter 22	Steel
Chapter 23	Wood

Florida Building Code, Residential 7th Edition (2020)

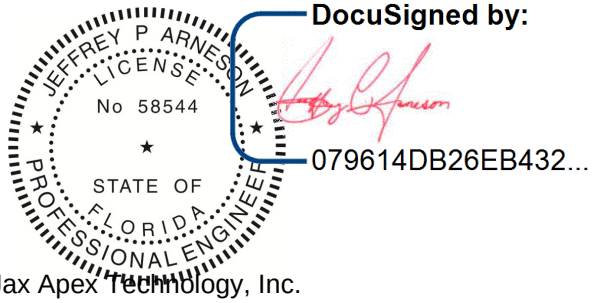
R101.2.1	Scope
R4405	HVHZ Concrete
R4407	HVHZ Masonry
R4408	HVHZ Steel
R4409	HVHZ Wood

Standards

AISI S100	2016
ANSI/AWC NDS	2018
ASTM D7147	2011

12. IDENTIFICATION:

Each connector covered by this report shall be stamped with the manufacturer's name and/or trademark and the product name.



Jax Apex Technology, Inc.
Jeffrey P. Arneson, P.E.
P.E. No. 58544
October 13, 2020

Jax Apex Technology, Inc.
9000 Regency Square Boulevard, Suite 100
Jacksonville, FL 32211

Evaluation reports are the opinion of the engineer who prepared the report, based on the findings, and in no way constitute or imply approval by a local building authority. The engineer, in review of the data submitted, finds that, in his opinion, the product, material, system, or method of construction specifically identified in this report conforms with or is a suitable alternate to that specified in the Florida Building Code, SUBJECT TO THE LIMITATIONS IN THIS REPORT

Jeffrey P. Arneson, P.E., a licensed Florida professional engineer and employee of Jax Apex Technology, Inc. (Apex Technology) has reviewed the data submitted for compliance with the Florida Building Code. Neither Jeffrey P. Arneson, nor Apex Technology, are responsible for any errors or omissions to any documents, calculations, drawings, specifications, tests, or summaries prepared and submitted by the design professional or preparer of record who are listed in the Substantiating Data section of this report.

REPORT NO: SIM202003

CATEGORY: Structural Components

SUBCATEGORY: Wood Connectors

SUBMITTED BY:

SIMPSON STRONG-TIE COMPANY, INC.
5956 W. LAS POSITAS BOULEVARD
PLEASANTON, CA 94588

1. CERTIFICATION OF INDEPENDENCE:

Jeffrey P. Arneson, the Florida engineer who prepared this report, and Apex Technology have no financial interest in the manufacturing, sales, or distribution of the products included in this report. Jeffrey P. Arneson and Apex Technology comply with all criteria as stated in Florida Administrative Code Chapter 61G20-3.

2. PRODUCT NAME:

Column Caps for CMU and Concrete Piers

CCQM, CCTQM, CCCQM, ECCLQM, ECCLLQMD, ECCLRQMD

Wood to Masonry Hangers

LGUM26-2, LGUM28-2, LGUM210-2, LGUM26-3, LGUM28-3, LGUM210-3, LGUM26-4, LGUM28-4, LGUM210-4, LGUM46, LGUM48, LGUM410, HGUM5.25, HGUM5.50, HGUM7.00, HGUM7.25, HGUM9.00

MBHU3.56/9.25KT, MBHU3.56/11.25KT, MBHU3.56/11.88KT, MBHU3.56/14KT, MBHU3.56/16KT, MBHU3.56/18KT, MBHU5.50/9.25KT, MBHU5.50/11.25KT, MBHU5.50/11.88KT, MBHU5.50/14KT, MBHU5.50/16KT, MBHU5.50/18KT

MBHA3.12/9.25, MBHA3.12/11.25, MBHA3.56/7.25, MBHA3.56/9.25, MBHA3.56/11.25, MBHA3.56/11.88, MBHA3.56/14, MBHA3.56/16, MBHA3.56/18, MBHA5.50/7.25, MBHA5.50/9.25, MBHA5.50/11.25, MBHA5.50/11.88, MBHA5.50/14, MBHA5.50/16, MBHA5.50/18

3. SCOPE OF EVALUATION:

Load evaluation as a structural component using the requirements of the 7*th Edition (2020) Florida Building Code, Building* and 7*th Edition (2020) Florida Building Code, Residential*.

4. DESCRIPTION:

4.1 CCQM-SDSHDG, CCTQM-SDSG, CCCQM-SDSG, ECCLQM-KT and ECCLQMD-KT Column Caps for CMU and Concrete Piers. The CCQM, CCTQM, CCCQM, and ECCLQM are used to connect wood beams to the top of CMU or concrete piers, primarily for raised pier foundations. The beams can be oriented in a "straight" configuration (CCQM), "T" configuration (CCTQM), "cross" configuration (CCCQM), or an "L" configuration (ECCLQM). The ECCLQMD has an extended main bucket to support a deck joist framing into the corner. The ECCLQMD is available in left (ECCLLQMD) and right (ECCLRQMD) versions. The column caps fasten to the beams with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws. Specially shaped anchor bolts are factory welded to the bottom of the buckets, and become embedded in the top of the pier. The ECCLQMG-KT and ECCLQMD-KT both include two MSTQM straps, which extend up to provide a holdown for corner framing above. The CCQM, CCTQM, CCCQM, ECCLQM and ECCLQMD are manufactured from 7 gauge steel meeting ASTM A1011 GRADE 33 with a hot-dipped galvanized finish in accordance with ASTM A153. The MSTQM strap is manufactured from 12 gauge steel meeting ASTM A653 SS GRADE 40 with a G90 galvanized finish. Column cap fastener schedule and allowable loads are shown in Table 1. Column cap dimensions are shown in Table 2. MSTQM strap fastener schedule and allowable loads are specified in Note 8 of Table 1. See Figures 1, 2, and 3 for additional details.

4.2 LGUM / HGUM Masonry Girder Hangers. The LGUM and HGUM series of high capacity hangers are used to connect wood girders and beams to masonry or concrete walls. Both series use Simpson Strong-Tie® Titen HD® Heavy-Duty Screw Anchors to attach to the masonry or concrete wall, and Strong-Drive® SDS Heavy-Duty Connector screws to attach the beam to the hanger. The LGUM is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. The HGUM is manufactured from 7 gauge galvanized steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Girder hanger fastener schedule, dimensions, and allowable loads are shown in Table 3. See Figure 4 for additional details.

4.3 MBHU Masonry Beam Hangers. The MBHU series of hangers are used to anchor wood beams, joists, trusses, or rafters to masonry or concrete walls. The MBHU fastens to the wood member with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws, and fastens to the masonry or concrete wall with two ¾"×5" Titen HD® Heavy-Duty Screw Anchors. These connectors are manufactured from 10 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. MBHU fastener schedules, dimensions, and allowable loads are shown in Table 4. See Figures 5 and 6 for additional details.

4.4 MBHA Masonry Hangers. The MBHA series of hangers are used to anchor wood beams, joists, trusses, or rafters to masonry or concrete walls. The MBHA fastens to the wood member with 0.148"×3" nails and fastens to the masonry or concrete with two lengths of threaded rod embedded in the wall using either Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® Epoxy Adhesive. These connectors are manufactured from 10 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Hanger fastener schedules, dimensions, and allowable loads are shown in Table 5. See Figure 7 for additional details.

5. MATERIALS:

5.1 Steel. Steel specifications for each product listed in this evaluation report shall be as indicated in the previous section. In addition to the standard G90 coating, some models (designated with a model number ending with Z) are available with a G185 zinc coating specification in accordance with ASTM A653. Some models (designated with a model number ending with HDG or G) are available with a hot-dip galvanization, also known as "batch" galvanization, in accordance with ASTM A123, with a minimum specified coating weight of 2.0 ounces of zinc per square foot of surface area, total for both sides. Some models (designated with a model number ending with SS) are available in Type 316L stainless material manufactured in accordance with ASTM A240 sheet, strip or plate and ASTM A480 (General Requirements) in the following designation: UNS designation S31603, AISI Type 316L. Model numbers in this report may not include the Z, HDG, or SS ending, but the information shown applies.

5.2 Wood. Wood members to which these connectors are fastened shall be solid sawn lumber, glued-laminated lumber, or structural composite lumber having dimensions consistent with the connector dimensions shown in Tables 1 through 6. Unless otherwise noted, lumber shall be Southern Pine (SP) or Douglas Fir-Larch (DF) having a minimum specific gravity of 0.55 or 0.50, respectively. Where indicated by SPF, lumber shall be Spruce-Pine-Fir having a minimum specific gravity of 0.42.

5.3 Concrete/Masonry. Concrete and Masonry design specifications shall be the stricter of the specifications by the engineer of record, the Florida Building Code minimum standards, the following, or as noted in the report:

Material	Specification	Minimum Compressive Strength
Concrete, f'_c	-	2,500 psi[1]
Masonry, f'_m	ASTM C1314	1,500 psi
Masonry Unit	ASTM C90	2,000 psi
Mortar	ASTM C270 Type S	1,800 psi (or by proportions)
Grout	ASTM C476	2,000 psi (or by proportions)

1. Where noted for CCQM models, 3,000 psi concrete is required.

5.4 Nails. Unless noted otherwise, nails shall be common nails. Nails shall comply with ASTM F1667 and shall have the minimum bending yield strengths F_{yb}:

Nail Pennyweight	Nail Shank Diameter (inch)	Nail Length (inch)	F_{yb} (psi)
10d	0.148	3	90,000
16d×2½	0.162	2½	90,000

Fasteners for galvanized connectors in pressure-preservative treated wood shall be hot-dipped zinc coated galvanized steel with coating weights in accordance with

ASTM A153 or steel mechanically galvanized in accordance with ASTM B695, Class 55. Fasteners for stainless steel connectors shall be stainless steel except where otherwise permitted by the treatment manufacturer.

The allowable loads of stainless-steel connectors match those of carbon-steel connectors when installed with Simpson Strong-Tie stainless-steel, SCNR ring-shank nails, unless noted otherwise.

5.5 Strong-Drive® SDS Heavy-Duty Connector Screws. Fasteners used with the connectors described in Tables 1, 3, and 4 of the report must be Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector Screws as recognized in FL9589. SDS screws used in contact with preservative-treated or fire-retardant-treated lumber must, at a minimum, comply with FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.6 Titen HD® Heavy-Duty Screw Anchors. Anchors shown in Tables 3 and 4 and Figures 4 and 6 shall be Simpson Strong-Tie® Titen HD Heavy-Duty Screw Anchors for masonry or concrete as recognized in FL15730 and FL16230. Installation shall be as specified in FL15730 and FL16230.

5.7 Epoxy Anchoring Adhesives. Anchors shown in Table 5 and Figure 7 shall be all-thread rod installed with either Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® High-Strength Epoxy Adhesive for masonry or concrete as recognized in FL15730 and FL16230. Installation shall be as specified in FL15730 and FL16230.

6. INSTALLATION:

Installation shall be in accordance with this report and the most recent edition of the Simpson Strong-Tie *Wood Construction Connectors* catalog. Information in this report supersedes any conflicting information between information provided in this report and the catalog.

7. SUBSTANTIATING DATA:

Test data submitted by Testing Engineers Inc. and Simpson Strong-Tie, and signed and sealed calculations performed by Bryan Wert, P.E., in accordance with the *7th Edition (2020) Florida Building Code, Building* and *Residential* codes.

Model	Test Lab	Test Number
ECCLQM	Testing Engineers, Inc.	P002, P004, P005, Q375
ECCLQMD	Simpson Strong-Tie, Inc.	S944, S945
CCQM, CCTQM	Testing Engineers, Inc.	P006, P007, P008, P009
LGUM	Testing Engineers, Inc.	M202, M203, M204, M211, M212, M213, M221, M222, M224
HGUM	Testing Engineers, Inc.	M207, M209, M216, M217, Q690, Q691, R058, R059
MBHU	Testing Engineers, Inc.	N447, N448, N449, N450, O356, O357, O360, O361, P773, P774, Q848, Q849
MBHA	Testing Engineers, Inc.	F391, F392, F393, F394, F964, F965

8. FINDINGS:

The connectors listed in this evaluation report comply with the *7th Edition (2020) Florida Building Code, Building*, and *7th Edition (2020) Florida Building Code, Residential* when installed in accordance with this report.

9. LIMITATIONS:

1. Maximum allowable loads shall not exceed the allowable loads listed in this report. Allowable loads listed in this report are based on allowable stress design. The loads in this report are not applicable to Load and Resistance Factor Design.
2. Capacity of wood members is not covered by this report. Capacity of wood members must be checked by the building designer.
3. Connectors in this report having a galvanized coating thickness less than G185 shall not be installed in contact with preservative-treated wood products that are exposed to rainfall or ground moisture.
4. Allowable loads for more than one direction for a single connection cannot be added together. A design load that can be divided into components in the directions given must be evaluated as follows:

   (Design Uplift / Allowable Uplift) + (Design Lateral / Allowable Lateral) < 1.0

10. ALLOWABLE LOADS:

The tables that follow reference the allowable loads for the aforementioned products.

TABLE 1: CCQM / CCCQM / CCTQM / ECCLQM Fasteners and Allowable Loads

Model No.	No. of ¼"×2½" SDS Screws			16" Square Grout-Filled CMU Pier[3,6]				16" Square CMU Shell Filled with 3,000 psi Concrete[3,7]			
				Uplift (160)			Lateral (160)	Uplift (160)			Lateral (160)
	Main Beam	Side Beam	Deck Beam	Main Beam	Side Beam	Total		Main Beam	Side Beam	Total	
CCQM-SDSHDG	12	—	—	6,750	—	6,750	2,460	6,495	—	6,495	2,650
CCTQM-SDSG	12	8	—	6,750	5,375	6,750	2,460	6,495	5,375	6,495	2,650
CCCQM-SDSG	12	8	—	6,750	5,375	6,750	2,460	6,495	5,375	6,495	2,650
ECCLQMG-KT[8]	16	16	—	6,240	6,240	7,340	2,220	6,240	6,240	7,830	2,565
ECCLQMDG-KT[9]	16	16	6	6,240	6,240	7,340	2,220	6,240	6,240	7,830	2,565

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Total uplift load and lateral load is based on tested anchor failure in the pier.
3. Allowable loads are based on either a 16" square grout-filled CMU pier with f'_m of 1,500 psi or a 16" square CMU shell filled with 3,000 psi concrete. A minimum of (4) #7 vertical rebars or equivalent steel area is required. The Designer shall design and detail the GFCMU/concrete pier to resist all forces including uplift, shear, and moment.
4. Pier height per Building Designer.
5. Side beam and main beam uplift loads assume DF or SP members and are not additive.
6. The allowable loads listed for grout-filled CMU apply to solid concrete piers of 2,500 psi concrete a minimum of 16" square.
7. The allowable loads listed for CMU shell filled with 3,000 psi concrete apply to solid concrete piers of 3,000 psi concrete a minimum of 14" square.
8. The ECCLQM-KT is a kit packaged with (2) MSTQM straps and (32) SDS ¼"×2½" screws. One strap may be installed on each face of the ECCLQM (as shown), using the SDS screws into the beams and (26) 0.162"x2½" nails (not provided) into the wall framing. The MSTQM strap's allowable tension load is 6,240 lbs. If straps are not installed, table uplift applies to beam only.
9. The ECCLQMD-KT is a kit packaged with (2) MSTQM straps and (32) SDS ¼"×2½" screws. The allowable download for the deck joist connection is 5,475 lb. (2-2X joist) or 5,240 lb. (2X joist) and the allowable uplift is 2,010 lb.

TABLE 2: CCQM / CCTQM / CCCQM / ECCLQM Dimensions				
Model No.	**Main Channel Width, W_1 (in.)**	**Side Stirrup Width, W_3 and W_4 (in.)**	**Main Channel Length, L_1 (in.)**	**Side Stirrup Length, L_2 (in.)**
CCQM3.62-SDSHDG	3 5/8	—	11	—
CCQM4.62-SDSHDG	4 5/8	—	11	—
CCQM5.50-SDSHDG	5 ½	—	11	—
CCTQM3.62-SDSG	3 5/8	3 5/8	11 ½	4
CCTQM4.62-SDSG	4 5/8	4 5/8	13 ½	4
CCTQM5.50-SDSG	5 ½	5 ½	13 ½	4
CCCQM3.62-SDSG	3 5/8	3 5/8	11 ½	4
CCCQM4.62-SDSG	4 5/8	4 5/8	13 ½	4
CCCQM5.50-SDSG	5 ½	5 ½	13 ½	4
ECCLQM3.62G-KT[1]	3 5/8	3 5/8	11 ½ [2]	7 ¾
ECCLQM4.62G-KT[1]	4 5/8	4 5/8	11 ½ [2]	7 ¾
ECCLQM5.50G-KT[1]	5 ½	5 ½	11 ½ [2]	7 ¾

Notes:
1. The MSTQM is a component of the ECCLQM kits. It is 12 ga.; 3" wide and 48" long.
2. The ECCLL/RQMDG-KT has the same dimensions as ECCLQMG-KT, except W1 = 16½".

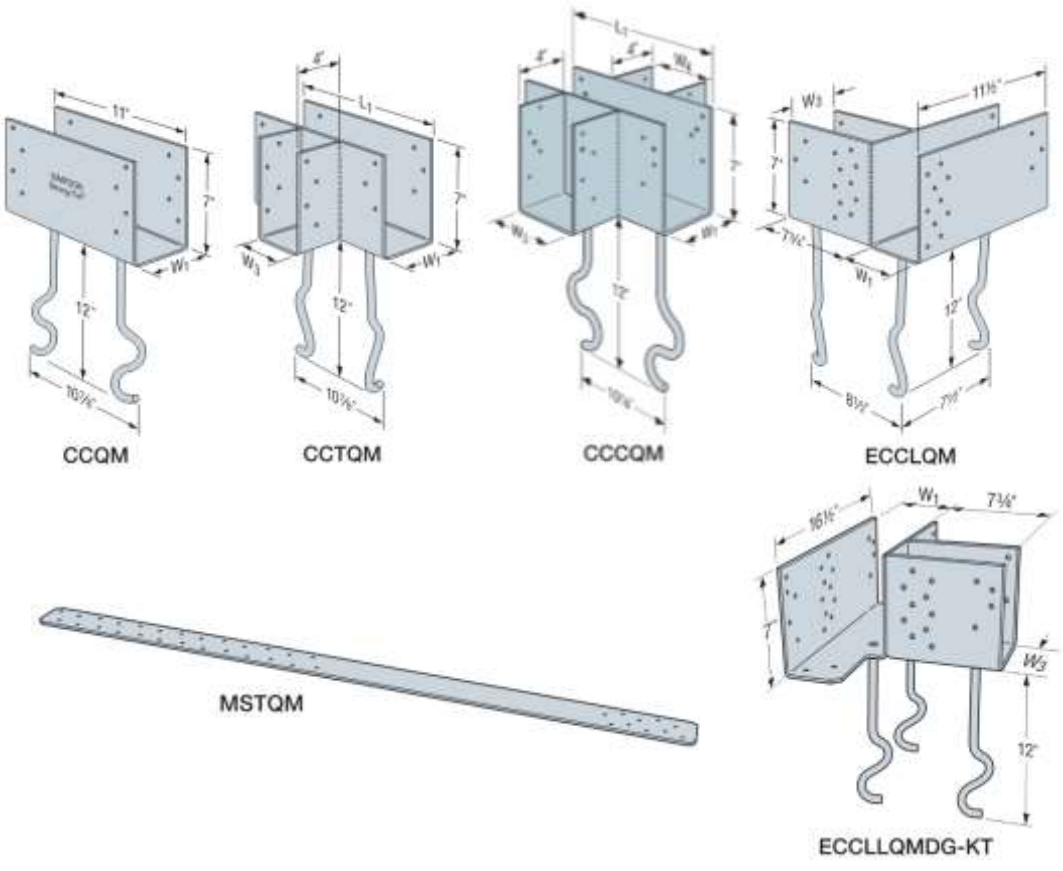


FIGURE 1: CCQM / CCTQM / CCCQM / ECCLQM / ECCLQMD / MSTQM Dimensions

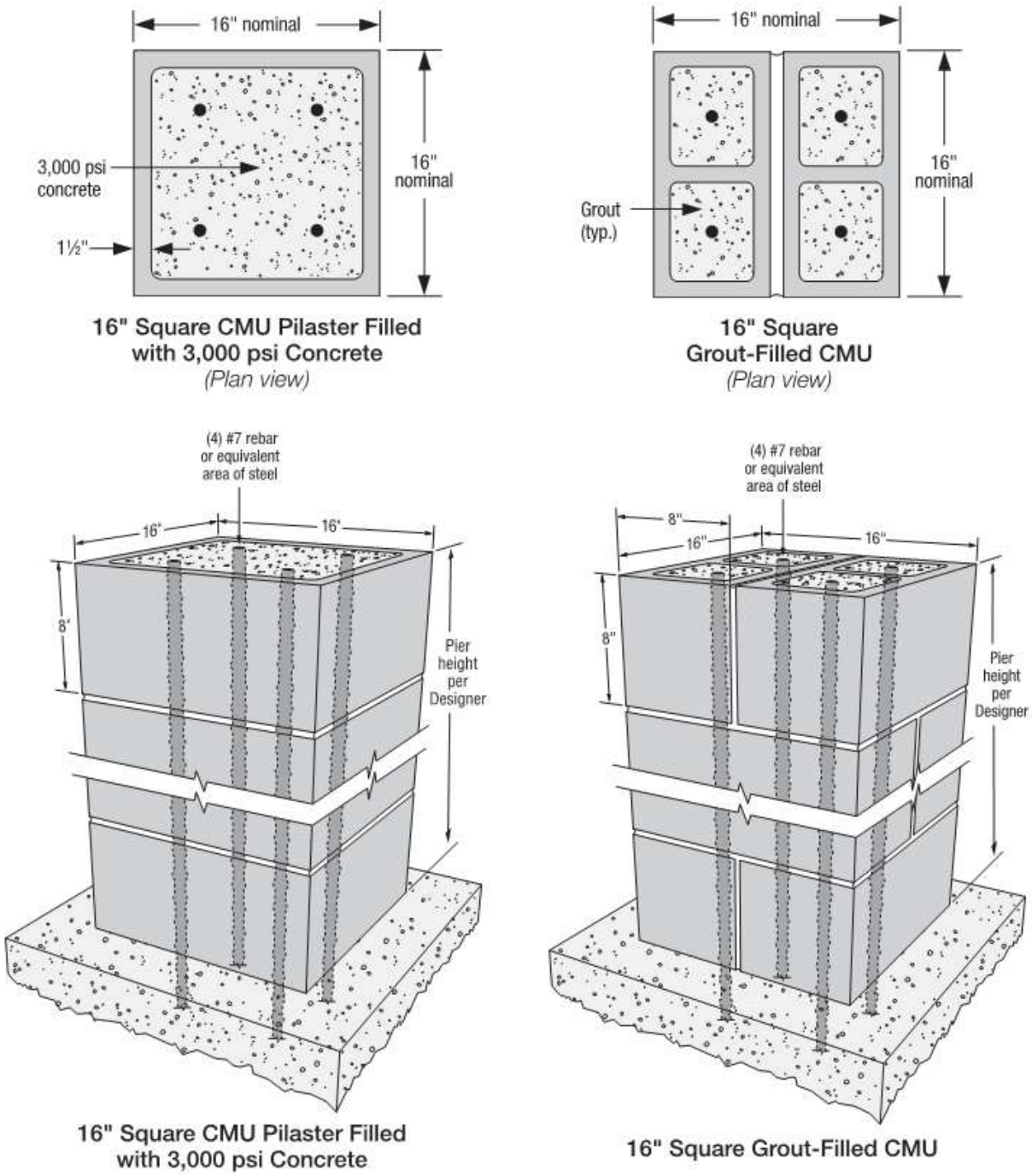
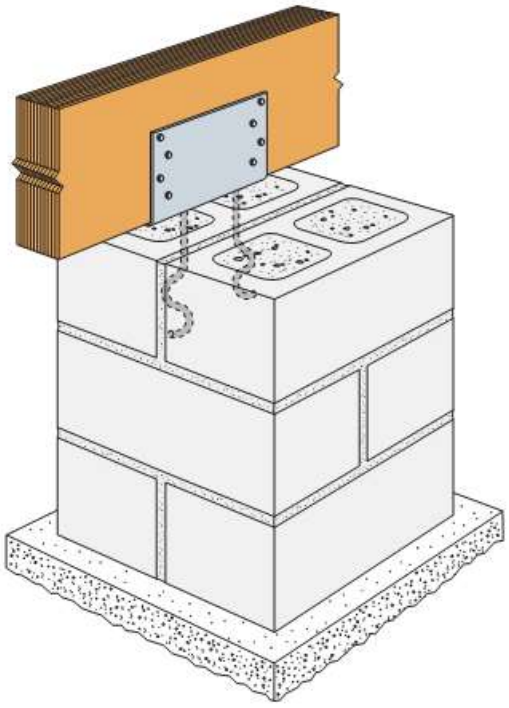
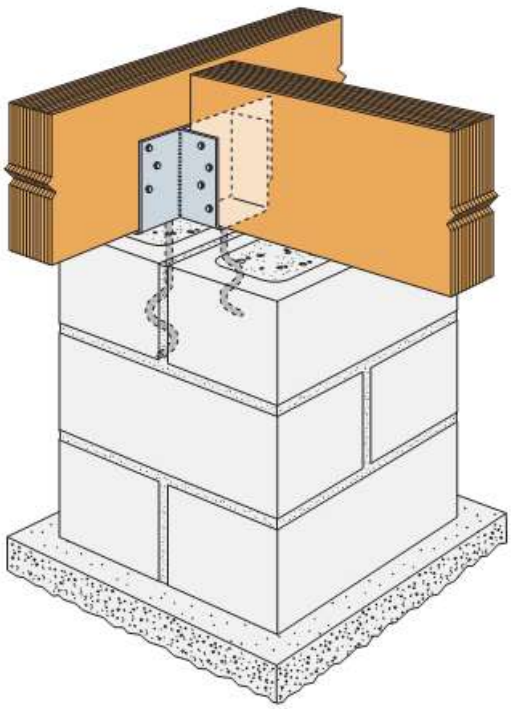


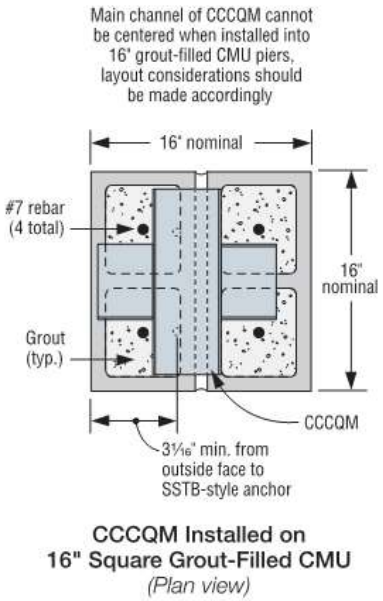
FIGURE 2: Pier Typical Dimensions and Minimum Requirements



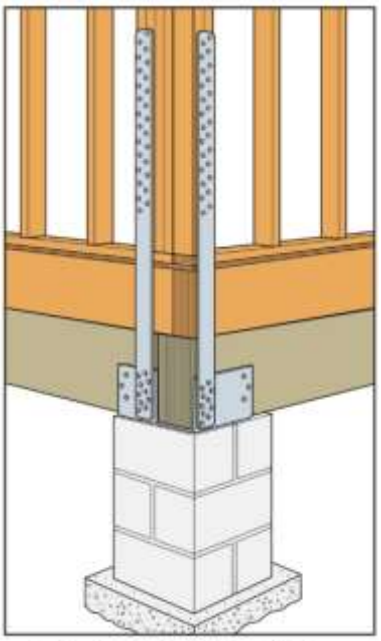
Typical CCQM Installation



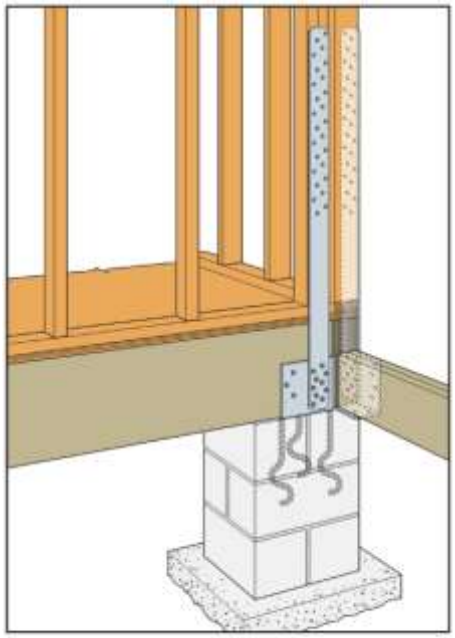
Typical CCTQM Installation



CCCQM Installed on 16" Square Grout-Filled CMU
(Plan view)



Typical ECCLQM Installation



Typical ECCLRQMD-KT Installation

FIGURE 3: CCQM / CCTQM / CCCQM / ECCLQM / ECCLQMD / MSTQM Installation Details

TABLE 3 – LGUM and HGUM Dimensions, Fasteners and Allowable Loads										
Model No.	Ga.	Dimensions (in.)			Fasteners		DF/SP Allowable Loads (lb.)			
		W	H	B	GFCMU and Concrete	Joist	Uplift[2] (160)		Download (100/115/125)	
					Titen HD® Anchors[7]	Strong-Drive® SDS Screws	GFCMU	Concrete	GFCMU	Concrete
Double 2x Sizes										
LGUM26-2	12	3 7/16	5	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,595	
LGUM28-2	12	3 7/16	7	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,250	
LGUM210-2	12	3 7/16	9	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,575	
Triple 2x Sizes										
LGUM26-3	12	5 ⅛	5 ⅛	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,610	
LGUM28-3	12	5 ⅛	7 ⅛	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,290	
LGUM210-3	12	5 ⅛	9 ⅛	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,715	
Quadruple 2x Sizes										
LGUM26-4	12	6 11/16	5 5/16	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,625	
LGUM28-4	12	6 11/16	7 5/16	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,335	
LGUM210-4	12	6 11/16	9 5/16	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,860	
4x Sizes										
LGUM46	12	3 ⅝	4 ⅞	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,600	
LGUM48	12	3 ⅝	6 ⅞	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,260	
LGUM410	12	3 ⅝	8 ⅞	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,620	
Engineered Wood & Structural Composite Lumber Sizes (Heavy Duty)										
HGUM5.25	7	5 ¼	11 to 30	5 ¼	(8) ⅝"×5"	(24) ¼"×2½"	4,105[2]	5,075[2]	14,025	14,770
HGUM5.50	7	5 ½		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			14,000	14,915
HGUM7.00	7	7		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,840	14,915
HGUM7.25	7	7 ¼		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,810	14,915
HGUM9.00	7	9		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,625	14,915

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Tabulated uplift loads are based on a 4" min. distance to top of wall. For HGUM installations with a 15" min. distance to top of wall, uplift loads may be increased to 6,180 lb. for GFCMU and 6,585 lb. for concrete.
3. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
4. LGUM must be installed on minimum 6" thick wall and HGUM on minimum 8" thick wall (nominal values for GFCMU).
5. Titen HD anchors may be installed into the head or bed joints.
6. Structural composite lumber (SCL) shall have a minimum specific gravity of 0.50.
7. For stainless steel LGUM and HGUM models, use ⅜"×5" and ⅝"×6" Stainless-Steel Titen HD anchors, respectively, to achieve tabulated loads.

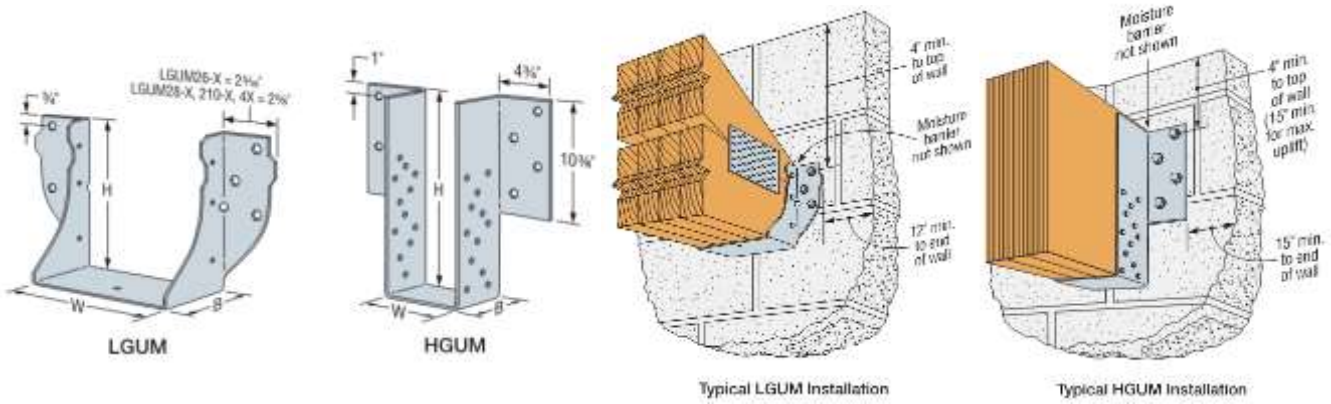


FIGURE 4: LGUM / HGUM Dimensions and Installation Details

TABLE 4: MBHU Dimensions, Fasteners, and Allowable Loads												
Model No.	Dimensions (in.)			Fasteners		DF/SP Allowable Loads (lb.)					
				GFCMU and Concrete	Joist	End of Wall / Outside Corner				Away From Edge	
						GFCMU		Concrete		GFCMU/Concrete	
	W	H	B	Titen HD® Anchors[5]	SDS Screws	Uplift (160)	Download (100)	Uplift (160)	Download (100)	Uplift (160)	Download (100)
MBHU3.56/9.25	3 9/16	9 ¼	3 ½	(2) ¾"×5"	(12) ¼"×2½"	1,610	2,440	2,715	4,190	2,210	4,005
MBHU3.56/11.25		11 ¼									
MBHU3.56/11.88		11 ⅞									
MBHU3.56/14		14				1,610	2,440	2,715	4,190	3,345	6,065
MBHU3.56/16		16									
MBHU3.56/18		18									
MBHU5.50/9.25	5 ½	9 ¼	3 ½	(2) ¾"×5"	(12) ¼"×2½"	1,610	2,440	2,715	4,190	2,210	4,005
MBHU5.50/11.25		11 ¼									
MBHU5.50/11.88		11 ⅞									
MBHU5.50/14		14				2,240	3,260[4]	3,485	6,970	3,345	6,065
MBHU5.50/16		16									
MBHU5.50/18		18									

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
3. Structural composite lumber (SCL) shall have a minimum specific gravity of 0.50.
4. Where noted in the table, download is for End of Wall condition. Download for Outside Corner is 3,640 lb.
5. For stainless steel MBHU models, use ¾"×6" Stainless-Steel Titen HD® anchors to achieve tabulated loads.

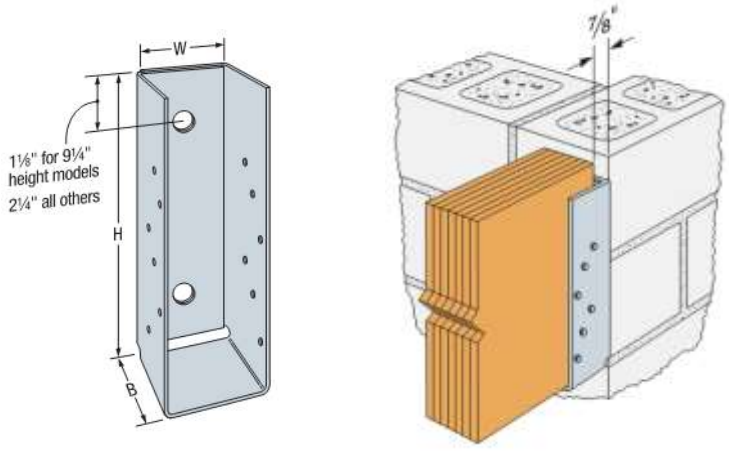


FIGURE 5: MBHU Dimensions and Installation Detail

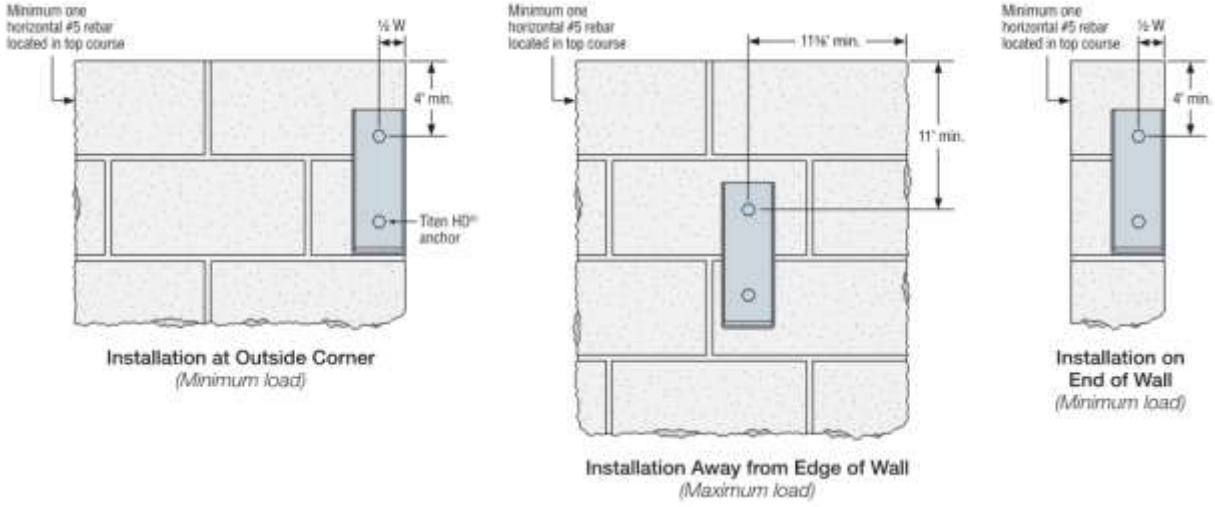


FIGURE 6: MBHU Installation Location Options

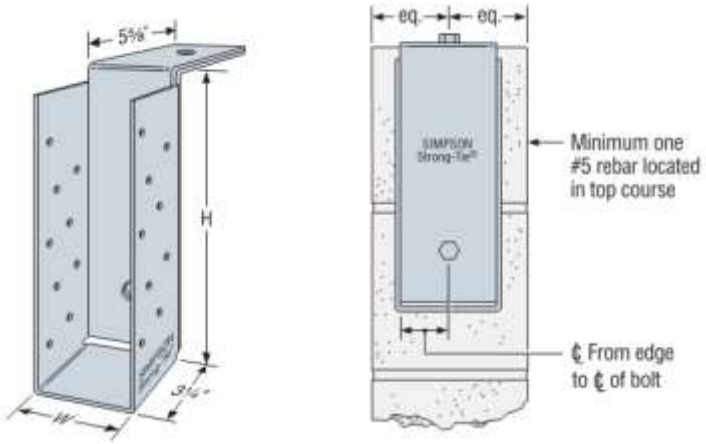


FIGURE 7: MBHA Dimensions and Installation Detail

TABLE 5: MBHA Dimensions, Fasteners, and Allowable Loads								
Model No.	Dimensions (in.)			Fasteners			DF/SP Allowable Loads (lb.)	
				GFCMU / Concrete[2,3]		Joist	GFCMU / Concrete[6,7]	
	W	H	B	Top	Face		Uplift (160)	Download (100)
MBHA3.56/7.25	3 9/16	7 ¼	3 ¼	(1) ATR¾ [4]	(1) ATR¾ [5]	(18) 0.148"×3"	1,885	4,380
MBHA5.50/7.25	5 ½	7 ¼	3 ¼					
MBHA3.12/9.25	3 1/8	9 ¼	3 ¼	(1) ATR¾ [4]	(1) ATR¾ [4]	(18) 0.148"×3"	3,475 [7]	6,275 [7]
MBHA3.12/11.25	3 1/8	11 ¼	3 ¼					
MBHA3.56/9.25	3 9/16	9 ¼	3 ¼					
MBHA3.56/11.25	3 9/16	11 ¼	3 ¼					
MBHA3.56/11.88	3 9/16	11 7/8	3 ¼					
MBHA3.56/14	3 9/16	14	3 ¼					
MBHA3.56/16	3 9/16	16	3 ¼					
MBHA3.56/18	3 9/16	18	3 ¼					
MBHA5.50/9.25	5 ½	9 ¼	3 ¼					
MBHA5.50/11.25	5 ½	11 ¼	3 ¼					
MBHA5.50/11.88	5 ½	11 7/8	3 ¼					
MBHA5.50/14	5 ½	14	3 ¼					
MBHA5.50/16	5 ½	16	3 ¼					
MBHA5.50/18	5 ½	18	3 ¼					

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
3. ATR is all-thread rod, ¾" diameter, grade A307 or better.
4. Loads for MBHA are based on installation using Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® High-Strength Epoxy Adhesive with 6¾" minimum embedment required.
5. MBHA hangers with height of 7¼" require a 3½" minimum embedment of the face bolt using ET-HP Epoxy Adhesive or Simpson Strong-Tie SET-XP High-Strength Epoxy Adhesive.
6. Alternate anchorage permitted when designed by others to have equivalent or better capacity.
7. MBHA hangers with height of 9¼" and greater installed into concrete achieve 3,980 lb. uplift (160) and 6,790 lb. download (100).

11. REFERENCES:

Florida Building Code, Building 7th Edition (2020)

Section 104.11	Alternative materials, design, and methods of construction and equipment
Chapter 19	Concrete
Chapter 21	Masonry
Chapter 22	Steel
Chapter 23	Wood

Florida Building Code, Residential 7th Edition (2020)

R101.2	Scope
R4405	HVHZ Concrete
R4407	HVHZ Masonry
R4408	HVHZ Steel
R4409	HVHZ Wood

Standards

AISI S100	2016
ANSI/AWC NDS	2018
ASTM D7147	2011

12. IDENTIFICATION:

Each connector covered by this report shall be stamped with the manufacturer's name and/or trademark and the product name.

DocuSigned by:



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Jax Apex Technology, Inc.
Jeffrey P. Arneson, P.E.
P.E. No. 58544
October 13, 2020

Jax Apex Technology, Inc.
9000 Regency Square Boulevard, Suite 100
Jacksonville, FL 32211

Evaluation reports are the opinion of the engineer who prepared the report, based on the findings, and in no way constitute or imply approval by a local building authority. The engineer, in review of the data submitted, finds that, in his opinion, the product, material, system, or method of construction specifically identified in this report conforms with or is a suitable alternate to that specified in the Florida Building Code, SUBJECT TO THE LIMITATIONS IN THIS REPORT

Jeffrey P. Arneson, P.E., a licensed Florida professional engineer and employee of Jax Apex Technology, Inc. (Apex Technology) has reviewed the data submitted for compliance with the Florida Building Code. Neither Jeffrey P. Arneson, nor Apex Technology, are responsible for any errors or omissions to any documents, calculations, drawings, specifications, tests, or summaries prepared and submitted by the design professional or preparer of record who are listed in the Substantiating Data section of this report.

REPORT NO: SIM202001

CATEGORY: Structural Components

SUBCATEGORY: Wood Connectors

SUBMITTED BY:

SIMPSON STRONG-TIE COMPANY, INC.
5956 W. LAS POSITAS BOULEVARD
PLEASANTON, CA 94588

1. CERTIFICATION OF INDEPENDENCE:

Jeffrey P. Arneson, the Florida engineer who prepared this report, and Apex Technology have no financial interest in the manufacturing, sales, or distribution of the products included in this report. Jeffrey P. Arneson and Apex Technology comply with all criteria as stated in Florida Administrative Code Chapter 61G20-3.

2. PRODUCT NAMES:

Truss to Wall Connectors
MTSM16, MTSM20, HTSM16, HTSM20, HM9, HGAM10, H16, H16S, H16-2, H16-2S

Embedded Truss Anchors
META12, META16, META18, META20, META24, META40, HETA12, HETA16, HETA20, HETA24, HETA40, HHETA16, HHETA20, HHETA24, HHETA40, HETAL12, HETAL16, HETAL20, LTA2, DETAL20

Wood to Masonry Straps
MSTAM24, MSTAM36, MSTCM40, MSTCM60

Girder Tiedowns
LGT2, LGT3, LGT4, FGTR, FGTRHL, FGTRHR

3. SCOPE OF EVALUATION:

Load evaluation as a structural component using the requirements of the 7*th Edition (2020) Florida Building Code, Building* and *7th Edition (2020) Florida Building Code, Residential*.

4. DESCRIPTION:

4.1 MTSM and HTSM Twist Straps. The MSTM and HTSM twist straps are used to anchor wood trusses, rafters, or beams to masonry or concrete walls. They fasten to the wood member with 0.148"×3" or 0.148"×1½" nails, and fasten to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. MTSM twist straps are manufactured from 16 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. HTSM twist straps are manufactured from 14 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Twist strap fastener schedules, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.2 HM9 Hurricane Tie. The HM9 is used to anchor wood trusses, rafters, or joists to masonry or concrete. The HM9 fastens to the wood member with ¼"×1½" long Strong-Drive® SDS Heavy-Duty Connector screws and fastens to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The HM9 is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Hurricane tie fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.3 HGAM10 Hurricane Gusset Angle. The HGAM10 is used to anchor wood trusses, rafters, joists, or beams to masonry or concrete. The HGAM10 fastens to the wood member with ¼"×1½" long Strong-Drive® SDS Heavy-Duty Connector screws and fastens to masonry with ¼"×2¾" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The HGAM10 is manufactured from 14 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Angle fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.4 LTA2 Lateral Truss Anchor. The LTA2 is used to anchor wood trusses, rafters, or beams to masonry or concrete. The LTA2 fastens to the wood member with 0.148"×1½" nails and has legs which are embedded into the wall system. The LTA2 is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Lateral truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 1. See Figures 1 and 2 for additional details.

4.5 H16, H16S, H16-2, and H16-2S Hurricane Ties. The H16, H16S, H16-2, and H16-2S are used to anchor wood trusses or rafters to masonry or concrete walls. These connectors wrap over the top of the rafter or truss. The straps are fastened to a masonry wall with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. These connectors are manufactured from No. 18 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 1. See Figures 1 and 2 for additional details.

4.6 META, HETA, HHETA, and HETAL Embedded Truss Anchors. Embedded truss anchors are used to anchor a wood member (usually a truss) to masonry or concrete. Embedded truss anchors fasten to a single-ply wood truss with 0.148"×1½" nails or

to a multi-ply truss with 0.162"×3½" nails. META, HETA, and HHETA models can be installed in a single application or doubled to achieve additional capacity. They are embedded in the masonry or concrete to a depth indicated on the side of the anchor (4" for META, HETA, and HHETA, and $5^{1}/_{16}$" for HETAL). The strap portion of the anchor is 1⅛" wide. The anchors are manufactured from steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish, with the exception of the HETAL truss seat which is manufactured from steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Steel thickness is as specified in Table 2. Embedded truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 2 for single installations and Table 3 for double installations. See Figures 3 and 4 for additional details of single and double embedded truss anchors.

4.7 DETAL20 Double Embedded Truss Anchor. The DETAL20 is a high capacity connector used to anchor single-ply wood trusses or rafters to masonry or concrete. The DETAL fastens to the wood member with 0.148"×1½" nails. It is embedded in the masonry or concrete to a depth of 4½". The strap portion of the anchor is 1⅛" wide and is manufactured from 16 gauge steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish. The truss seat is manufactured from 18 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Embedded truss anchor fastener schedule, dimensions, and allowable loads are shown in Table 3. See Figure 4 for additional details.

4.8 MSTAM and MSTCM Strap Ties. The MSTAM and MSTCM strap ties are used to provide a tension connection between wood members and a masonry or concrete structure. The MSTAM straps are 1¼" wide for use on single 2× and thicker members. They fasten to the wood member with 0.148"×3" nails and fasten to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The MSTCM straps are 3" wide for use on double 2× or single 4× and larger members. They fasten to the wood member with 0.148"×3¼" nails and fasten to masonry with ¼"×2¼" Titen Turbo or Titen 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The MSTCM straps have countersunk nail slots for a lower nailing profile and coined edges for safer handling. MSTAM and MSTCM straps are manufactured from steel meeting ASTM A653 SS Grade 50, Class 1, with a G90 galvanized finish. Steel thickness is as specified in Table 4. Masonry strap fastener schedule, dimensions, and allowable loads are shown in Table 4. See Figure 5 for additional details.

4.9 FGTR, FGTRHL, and FGTRHR Face Mount Girder Tiedowns. The FGTR is a non-pitch specific girder tie down that can be used in new construction or retrofit applications to tie down a girder truss or beam to a concrete or masonry. The FGTR can be installed in a single application or doubled to achieve additional uplift capacity. The FGTR plate fastens to the wood member with Strong-Drive® SDS Heavy-Duty Connector screws and the strap fastens to masonry or concrete with ½"×5" Titen HD® Heavy-Duty Screw Anchors. The FGTRHL and FGTRHR are designed with the strap at a 45 degree angle for corner hip applications. The FGTR straps are manufactured from 7 gauge ASTM A1011 Grade 33 steel and the plates are made from 3 gauge ASTM A1011 Grade 33 steel. Both components have a powder coat painted finish. Girder tie down fastener schedule, dimensions, and allowable loads are shown in Table 5. See Figure 6 for additional details.

4.10 LGT2 Light Truss/Girder Tiedown. The LGT2 is used to anchor a 2-ply wood truss or beam (maximum 3" wide) to wood, concrete, or masonry. The LGT2 fastens to the wood truss or beam with 0.148"×3¼" nails or #9×1½" Strong-Drive® SD Connector screws. For wood wall applications, it is fastened with 0.148"×3¼" nails

or #9×1½" Strong-Drive SD Connector screws. For masonry or concrete applications, it fastens to masonry with ¼"×2¼" Titen Turbo™ or Titen® 2 screw anchors, or to concrete with ¼"×1¾" Titen Turbo or Titen 2 screw anchors. The LGT2 is manufactured from 14 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

4.11 LGT3-SDS2.5 Light Truss/Girder Tiedown. The LGT3-SDS2.5 is used to anchor a 3-ply wood truss or beam (maximum 5" wide) to wood, concrete, or masonry. The LGT3-SDS2.5 fastens to the wood truss or beam with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws. For wood wall applications, it is fastened to wood studs and plates with 0.148"×3¼" nails. For masonry or concrete applications, it is fastened with ⅜"×5" Titen HD® Heavy-Duty Screw Anchors. The LGT3-SDS2.5 is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

4.12 LGT4-SDS3 Light Truss/Girder Tiedown. The LGT4-SDS3 is used to anchor a 4-ply wood truss or beam (maximum 6½" wide) to wood, concrete, or masonry. The LGT4-SDS3 fastens to the wood truss or beam with ¼"×3" long Strong-Drive® SDS Heavy-Duty Connector screws. For wood wall applications, it is fastened to wood studs and plates with 0.148"×3¼" nails. For masonry or concrete applications, it is fastened with ⅜"×5" Titen HD® Heavy-Duty Screw Anchors. The LGT4-SDS3 is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. Allowable loads and fastener schedule are shown in Table 6. See Figure 7 for additional details.

5. MATERIALS:

5.1 Steel. Steel specifications for each product listed in this evaluation report shall be as indicated in the previous section. In addition to the standard G90 coating, some models (designated with a model number ending with Z) are available with a G185 zinc coating specification in accordance with ASTM A653. Some models (designated with a model number ending with HDG) are available with a hot-dip galvanization, also known as “batch” galvanization, in accordance with ASTM A123, with a minimum specified coating weight of 2.0 ounces of zinc per square foot of surface area, total for both sides. Some models (designated with a model number ending with SS) are available in Type 316L stainless material manufactured in accordance with ASTM A240 sheet, strip or plate and ASTM A480 (General Requirements) in the following designation: UNS designation S31603, AISI Type 316L. Model numbers in this report may not include the Z, HDG, or SS ending, but the information shown applies.

5.2 Wood. Wood members to which these connectors are fastened shall be solid sawn lumber, glued-laminated lumber, or structural composite lumber having dimensions consistent with the connector dimensions shown in Tables 1 through 6. Unless otherwise noted, lumber shall be Southern Pine (SP) or Douglas Fir-Larch (DF) having a minimum specific gravity of 0.55 or 0.50, respectively. Where indicated by SPF, lumber shall be Spruce-Pine-Fir having a minimum specific gravity of 0.42.

5.3 Concrete/Masonry. Concrete and masonry design specifications shall be the stricter of the specifications by the engineer of record, the Florida Building Code minimum standards, the following, or as noted in the report:

Material	Specification	Minimum Compressive Strength
Concrete, f'_c	-	2,500 psi
Masonry, f'_m	ASTM C1314	1,500 psi
Masonry Unit	ASTM C90	2,000 psi
Mortar	ASTM C270 Type S	1,800 psi (or by proportions)
Grout	ASTM C476	2,000 psi (or by proportions)

5.4 Nails. Unless noted otherwise, nails shall be common nails. Nails shall comply with ASTM F1667 and shall have the minimum bending yield strengths F_{yb}:

Nail Pennyweight	Nail Shank Diameter (inch)	Nail Length (inch)	F_{yb} (psi)
10d×1½	0.148	1½	90,000
10d Common	0.148	3	90,000
16d sinker	0.148	3¼	90,000
16d Common	0.162	3½	90,000

Fasteners for galvanized connectors in pressure-preservative treated wood shall be hot-dipped zinc coated galvanized steel with coating weights in accordance with ASTM A153 or steel mechanically galvanized in accordance with ASTM B695, Class 55. Fasteners for stainless steel connectors shall be stainless steel except where otherwise permitted by the treatment manufacturer.

The allowable loads of stainless-steel connectors match those of carbon-steel connectors when installed with Simpson Strong-Tie stainless-steel, SCNR ring-shank nails, unless noted otherwise.

5.5 Strong-Drive® SDS Heavy-Duty Connector Screws. Fasteners used with the connectors described in Tables 1, 5, and 6, and shown in Figures 2, 6, 8, and 10 of the report must be Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector wood screws recognized in FL9589. Model numbers shown in this report may not include the full SDS model number after the connector model number (e.g., LGT3-SDS2.5), but the information shown applies. SDS screws used in contact with preservative-treated or fire-retardant-treated lumber must, as a minimum, comply with FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.6 Strong-Drive® SD Connector Screws. Certain connectors described in this report may be installed with Simpson Strong-Tie® Strong-Drive SD Connector wood screws recognized in FL9589. Installation shall be as specified in FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.7 Titen Turbo™ and Titen® 2 Concrete and Masonry Screw Anchors. Titen Turbo™ screw anchors shown in Tables 1, 4, and 6 and Figures 2, 5, and 7 are recognized in FL16230. Alternately, Titen® 2 screw anchors of the same diameter and length may be substituted for Titen Turbo models shown with no change to the

tabulated allowable loads. Titen 2 screw anchors are recognized in FL16230. Installation for both screw anchors shall be as specified in FL16230.

5.8 Titen HD® Heavy-Duty Screw Anchors. Anchors shown in Tables 5 and 6 and Figure 6 shall be Simpson Strong-Tie® Titen HD Heavy-Duty Screw Anchors for masonry or concrete as recognized in FL15730. Installation shall be as specified in FL15730.

6. INSTALLATION:

Installation shall be in accordance with this report and the most recent edition of the Simpson Strong-Tie *Wood Construction Connectors* catalog. Information in this report supersedes any conflicting information between information provided in this report and the catalog.

7. SUBSTANTIATING DATA:

Test data submitted by Testing Engineers Inc., Product Testing, Inc., and Simpson Strong-Tie, and signed and sealed calculations performed by Bryan Wert, P.E., in accordance with the 7th *Edition (2020) Florida Building Code, Building* and *Residential* codes.

Product	Test Lab	Test Number
MTSM, HTSM	Testing Engineers, Inc.	H756, I841, O555, O564
HM9	Product Testing, Inc.	02-3793
HGAM10	Product Testing, Inc.	L007, L018
	Testing Engineers, Inc.	H046, H141
H16, H16S, H16-2, H16-2S	Testing Engineers, Inc.	H591, I376
META, HETA, HHETA, HETAL	Product Testing, Inc.	K966, K997, L008, L011, L012, L013, L014, L015, L019, L020, L021, L022, L023, P670, Q219, Q220, Q221
	Testing Engineers, Inc.	D792, D793, D844, N218
DETAL	Testing Engineers, Inc.	O795, O796, O797, O798, O799
LTA2	Testing Engineers, Inc.	P966, P967, P968, Q018, Q019, Q020, Q856
MSTCM	Testing Engineers, Inc.	N471
FGTR	Product Testing, Inc.	K930, K931, K987
FGTRHL/R	Product Testing, Inc.	K932
LGT2	Simpson Strong-Tie	23-19086, 23-19111
	Testing Engineers, Inc.	I839, L921, L922
LGT3	Testing Engineers, Inc.	L233, L234, L431, L918
LGT4	Testing Engineers, Inc.	O113, O393, O394, O587

8. FINDINGS:

The connectors listed in this evaluation report comply with the 7th *Edition (2020) Florida Building Code, Building*, and 7th *Edition (2020) Florida Building Code, Residential* when installed in accordance with this report.

9. LIMITATIONS:

1. Maximum allowable loads shall not exceed the allowable loads listed in this report. Allowable loads listed in this report are based on allowable stress design. The loads in this report are not applicable to Load and Resistance Factor Design.

2. Capacity of wood members is not covered by this report. Capacity of wood members must be checked by the building designer.

3. Connectors in this report having a galvanized coating thickness less than G185 shall not be installed in contact with preservative-treated wood products that are exposed to rainfall or ground moisture.

4. Allowable loads for more than one direction for a single connection cannot be added together. A design load that can be divided into components in the directions given must be evaluated as follows:

   (Design Uplift / Allowable Uplift) + (Design Lateral Parallel to Plate / Allowable Lateral Parallel to Plate) + (Design Lateral Perpendicular to Plate / Allowable Lateral Perpendicular to Plate) < 1.0

   As an alternate, the embedded truss anchors in Tables 2 and 3 can be evaluated using the following: The design load in each direction shall not exceed the published allowable load in that direction multiplied by 0.75.

10. ALLOWABLE LOADS:

The tables that follow reference the allowable loads for the aforementioned products.

TABLE 1: MTSM, HTSM, HM9, HGAM10, H16, and LTA2 Dimensions, Fasteners, and Allowable Loads												
Model No.	Ga.	L (in.)	Fasteners			Allowable Loads (lb.)					
			Rafter / Truss	GFCMU (Titen Turbo™)[6]	Concrete (Titen Turbo™)[6]	DF/SP (160)			SPF/HF (160)		
						Uplift	F_1	F_2	Uplift	F_1	F_2
MTSM16	16	16	(7) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	830	120[3]	90[3]	715	120[3]	90[3]
MTSM20	16	20	(7) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	830			715		
HTSM16	14	16	(8) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	1,110			955		
HTSM20	14	20	(10) 0.148"×1½"	(4) ¼"×2¼"	(4) ¼"×1¾"	1,110			955		
HM9[9]	18	–	(4) ¼"×1½" SDS	(5) ¼"×2¼"	(5) ¼"×1¾"	760	670	190	760	670	190
HGAM10[9]	14	–	(4) ¼"×1½" SDS	(4) ¼"×2¾"	(4) ¼"×1¾"	810	875	1,105[5]	585	630	795[5]
H16[10]	18	18¾"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370	–	–	1,180	–	–
H16S[10]	18	11 11/16"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
H16-2[10]	18	18¾"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
H16-2S[10]	18	11 11/16"	(2) 0.148"×1½"	(6) ¼"×2¼"	(6) ¼"×1¾"	1,370			1,180		
LTA2 Perp.-to-Wall Installation	18	–	(10) 0.148"×1½"	Embed	Embed	1,180[7]	415	875	990	415	735
LTA2 Parallel-to-Wall Installation	18	–	(10) 0.148"×1½"	Embed	Embed	1,180[7]	950	220	990	800	220

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. MTSM and HTSM do not have to be wrapped over the truss/rafter to achieve tabulated loads.
3. Lateral loads apply for the MTSM and HTSM twist straps when the first seven nail holes near the bend line are filled on the truss/rafter side. Any other fasteners required can be installed in any open hole.
4. Allowable loads for the HGAM10 are for one connector. A minimum truss/rafter thickness of 2½" must be used when framing anchors are installed on each side of the joist or truss.
5. HGAM10 F_2 loads are for forces into the connector. F_2 loads away from the connector are 640 lb. (DF/SP) and 460 lb. (SPF/HF).
6. Hex-head model required. Minimum edge distance for Titen Turbo™ screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.
7. LTA2 allowable uplift on SP is 1,350 lb. for perpendicular-to-wall installation and parallel-to-wall installation.
8. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
9. The HM9KT and HGAM10KTA are kits with (20) HM9 and (10) HGAM10 connectors packaged with Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws and 2¼" and 2¾" Titen® 2 screw anchors, respectively. (1¾" Titen 2 screw anchors for concrete installations sold separately)
10. H16 models are pre-sloped at a 5:12 pitch and can be used on pitches from 3:12 to 7:12. The minimum heel height for H16 models is 4" and the maximum heel height is 13½" (Use H16 or H16-2).

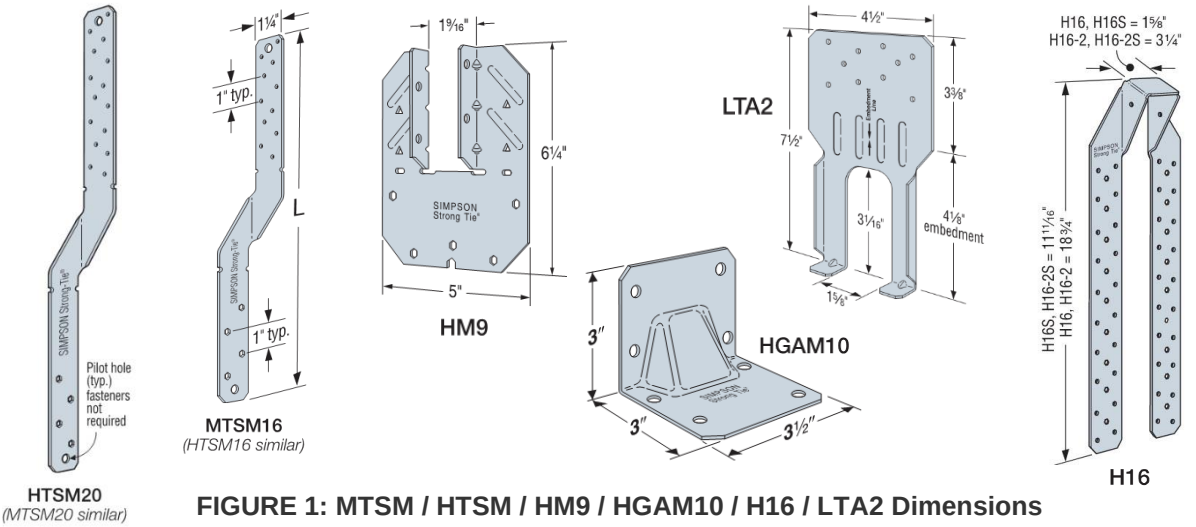


FIGURE 1: MTSM / HTSM / HM9 / HGAM10 / H16 / LTA2 Dimensions

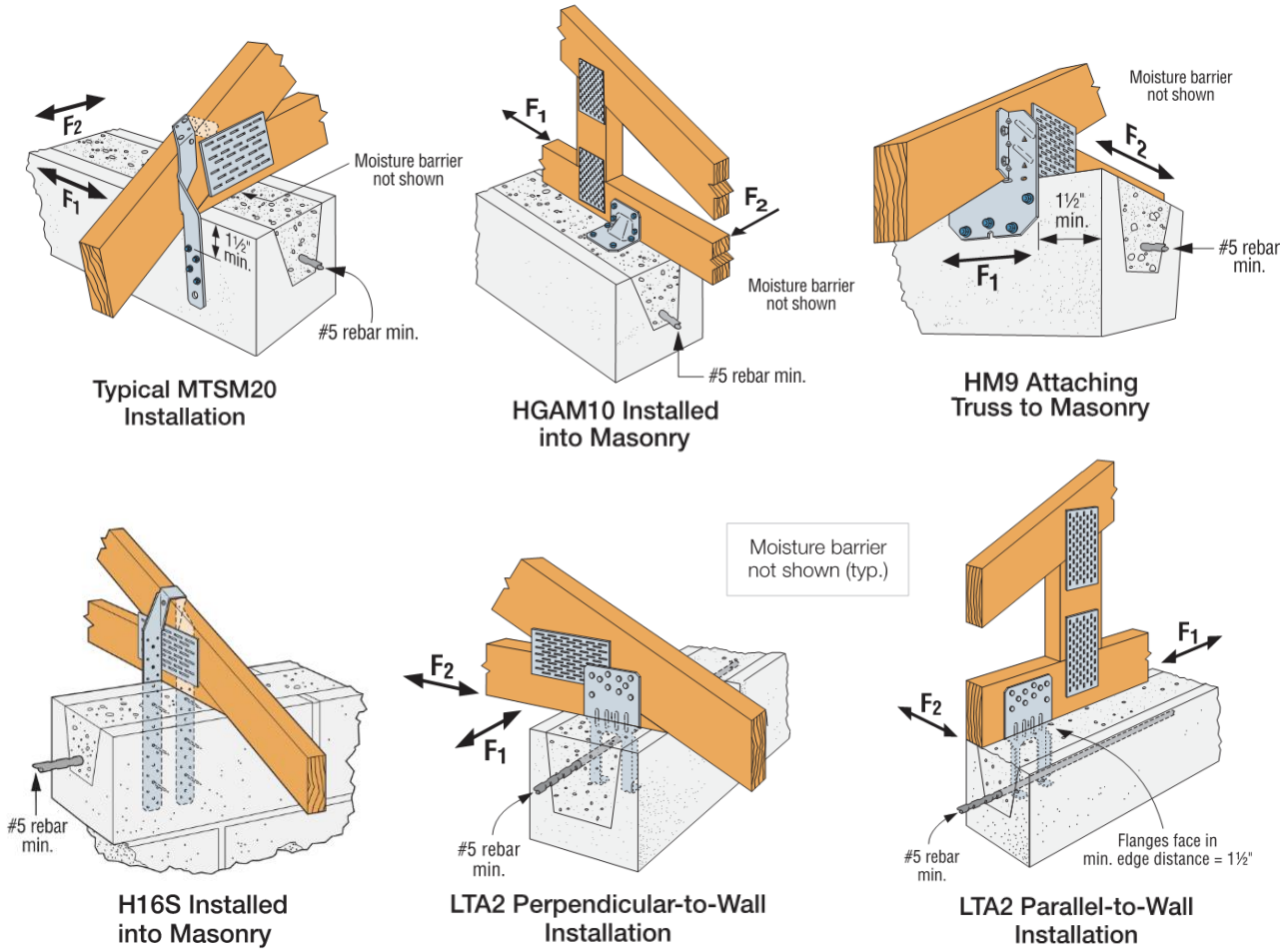


FIGURE 2: MTSM / HTSM / HM9 / HGAM10 / H16 / LTA2 Installation Details

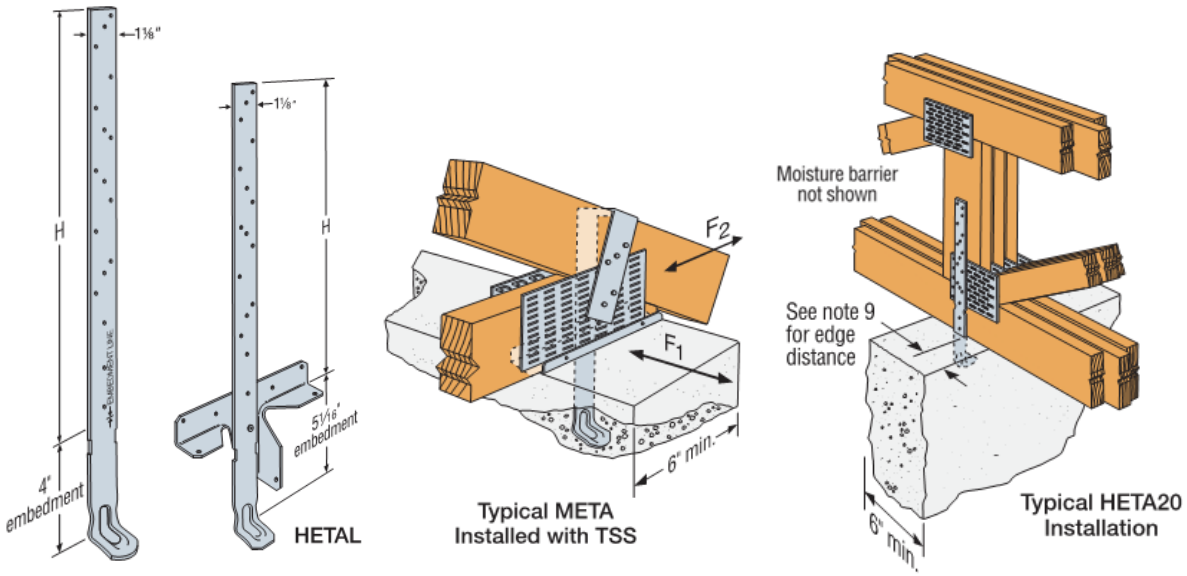


FIGURE 3: META / HETA / HHETA / HETAL Dimensions and Installation Details

TABLE 2: META, HETA, HHETA, and HETAL Dimensions, Fasteners, and Allowable Loads								
Model No.	Ga.	H (in.)	SP Allowable Loads (lb.)					
			1-ply Rafter / Truss (0.148"×1½" Nails)		2- or 3-ply Rafter / Truss (0.162"×3½" Nails)		F_1 (160)	F_2 (160)
			Qty.[2]	Uplift (160)	Qty.[2]	Uplift (160)		
META12	18	8	7	1,420	6	1,450	340	770
META16		12	8	1,450	6	1,450		
META18		14	8	1,450	6	1,450		
META20		16	7	1,420	5	1,210		
			8	1,450	6	1,450		
META24		20	8	1,450	6	1,450		
META40		36	8	1,450	6	1,450		
HETA12	16	8	7	1,455	7	1,730	340	770
HETA16		12	9	1,810	8	1,810		
HETA20		16	8	1,655	7	1,730		
			9	1,810	8	1,810		
HETA24		20	9	1,810	8	1,810		
HETA40		36	9	1,810	8	1,810		
HHETA16	14	12	10	2,120	9	2,120	340[7]	770
HHETA20		16	9	1,935	8	2,030		
			10	2,120	9	2,120		
HHETA24		20	10	2,120	9	2,120		
HHETA40		36	10	2,120	9	2,120		
HETAL12	16	7	10[3]	1,040	10[3]	1,235	390[4]	1,040
HETAL16		11	14[3]	1,810	13[3]	1,810		
HETAL20		15	14[3]	1,810	13[3]	1,810		

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. The META, HETA, and HHETA, require the lowest four nail holes be filled.
3. Five nails must be installed into the truss seat of the HETAL.
4. F_1 load towards face of HETAL is 1,870 lb.
5. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
6. It is acceptable to use a reduced number of fasteners in any model listed provided that there is a reduction in uplift load capacity. The load per nail can be approximated by dividing the allowable load by the number of nails. There should be a minimum of 4 nails installed in the strap. Lateral loads do not apply when fewer than 7 fasteners are used with the HETA and HHETA anchors or less than (6) 0.162"×3½" or (7) 0.148"×1½" fasteners are used with the META anchor.
7. The HHETA allowable F_1 load can be increased to 435 lb. if the strap is wrapped over the truss and a minimum of 12 nails are installed.
8. Straps do not have to be wrapped over the truss/rafter to achieve tabulated loads unless specifically noted otherwise.
9. Minimum edge distance for META, HETA, and HHETA is 1½" for concrete and 2" for masonry.

TABLE 3: DETAL20 and Double META, HETA, and HHETA Fasteners and Allowable Loads										
Model No.	Qty.	Application	SP Allowable Loads (lb.)							
			1-ply Rafter / Truss				2- or 3-ply Rafter / Truss			
			0.148"×1½" Nail Qty.[4]	Uplift	F_1	F_2	0.162"×3½" Nail Qty.[4]	Uplift	F_1	F_2
DETAL20	1	GFCMU	18	2,480	2,000[5]	1,370	—	—	—	—
		Concrete	18	2,480	2,000	1,465	—	—	—	—
META	2	GFCMU	10	1,875	Refer to Table 2[6]		14	1,795	1,285[7]	1,080
		Concrete	10	1,875			14	2,435	1,285[7]	1,080
HETA	2	GFCMU	10	1,920			12	2,365	1,350[7]	1,430
		Concrete	10	1,920			12	2,560	1,350[7]	1,430
HHETA	2	GFCMU	10	1,920			12	2,365	1,350[7]	1,430
		Concrete	10	1,920			14	3,180	1,350[7]	1,430

Notes:

1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
3. Install with spoons facing outward and spaced no more than ⅛" wider than the truss width.
4. The DETAL20 requires (6) 0.148"×1½" nails in the truss seat and (6) 0.148"×1½" nails into each strap. For all other models, install half of the required number of fasteners in each strap.
5. F_1 loads for the DETAL20 in GFCMU may cause an additional 1/32" deflection beyond the standard ⅛" limit.
6. Lateral loads for double META, HETA, and HHETA apply only to 2- or 3-ply applications with anchors spaced a minimum of 3" apart. For 1-ply applications, use lateral loads in Table 2. DETAL lateral loads apply to 1-ply application.
7. F_1 loads for double META, HETA, and HHETA may cause an additional 1/16" deflection beyond the standard ⅛" limit when the straps are installed not wrapped over the heel as shown.

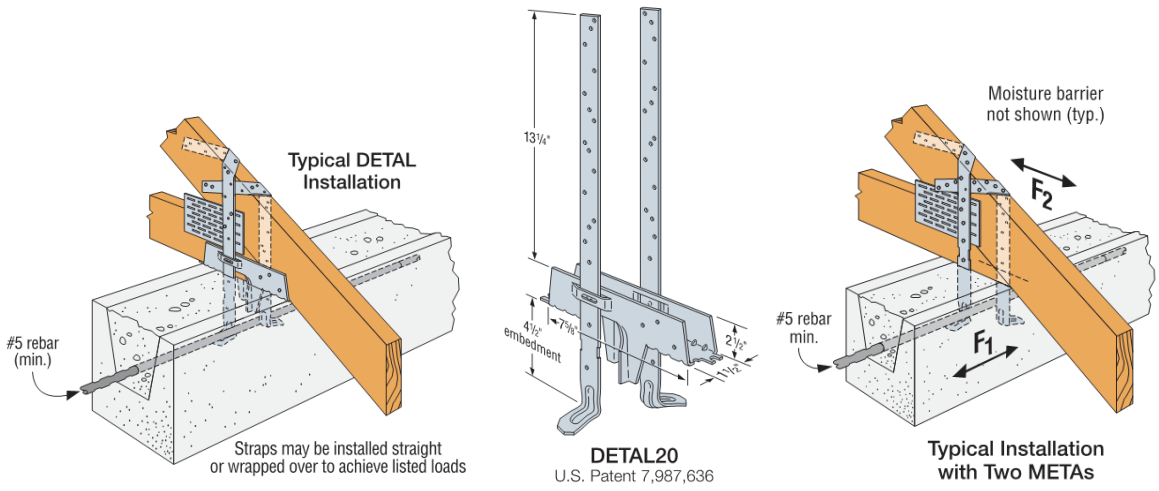


FIGURE 4: DETAL20 Dimensions and Double META / HETA / HHETA Installation Details

TABLE 4: MSTAM and MSTCM Dimensions, Fasteners, and Allowable Loads								
Model No.	Ga.	W (in.)	L (in.)	Fasteners			Allowable Tension / Uplift (lb.)	
				Rafter / Truss	GFCMU (Titen Turbo™)[3]	Concrete (Titen Turbo™)[3]	DF/SP/SPF/HF (160)	
							GFCMU	Concrete
MSTAM24	18	1¼	24	(9) 0.148"×3"	(5) ¼"×2¼"	(5) ¼"×1¾"	1,425	1,450
MSTAM36	16	1¼	36	(13) 0.148"×3"	(8) ¼"×2¼"	(8) ¼"×1¾"	1,870	1,870
MSTCM40	16	3	40¼	(26) 0.148"×3¼"	(14) ¼"×2¼"	(14) ¼"×1¾"	4,220	4,220
MSTCM60	16	3	59½	(26) 0.148"×3¼"	(14) ¼"×2¼"	(14) ¼"×1¾"	4,220	4,220

Notes:

1. Tension loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.
3. Hex-head model required. Minimum edge distance for Titen Turbo™ screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.

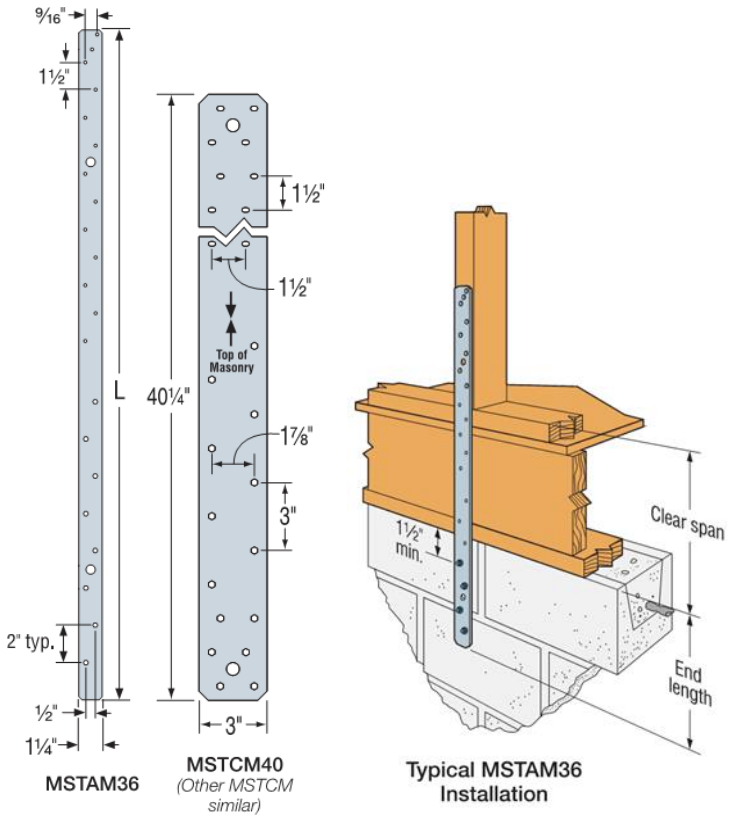


FIGURE 5: MSTAM / MSTCM Dimensions and Installation Details

TABLE 5: FGTR, FGTRHL, and FGTRHR Fasteners and Allowable Loads					
Model No.	Qty.	Fasteners		Allowable Uplift Load (lb.)	
		Rafter / Truss (Strong-Drive® SDS)	GFCMU / Concrete (Titen HD®)	DF/SP (160)	SPF/HF (160)
FGTR	1	(18) ¼"×3"	(2) ½"×5"	4,725[6]	3,400
	2	(36) ¼"×3"	(4) ½"×5"	8,885	6,395
FGTRHL/R	1	(18) ¼"×3"	(2) ½"×5"	3,635	2,615

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Products shall be attached to grouted and reinforced CMU walls or reinforced concrete walls that are designed to transfer the uplift loads to the foundation.
3. Minimum edge distance for the Titen HD anchor is 4".
4. Titen HD anchors should be spaced in every other hole on the part.
5. Attached members must be designed to resist the applied loads.
6. Uplift for FGTR corner installations shall be limited to 4,425 lb.
7. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.

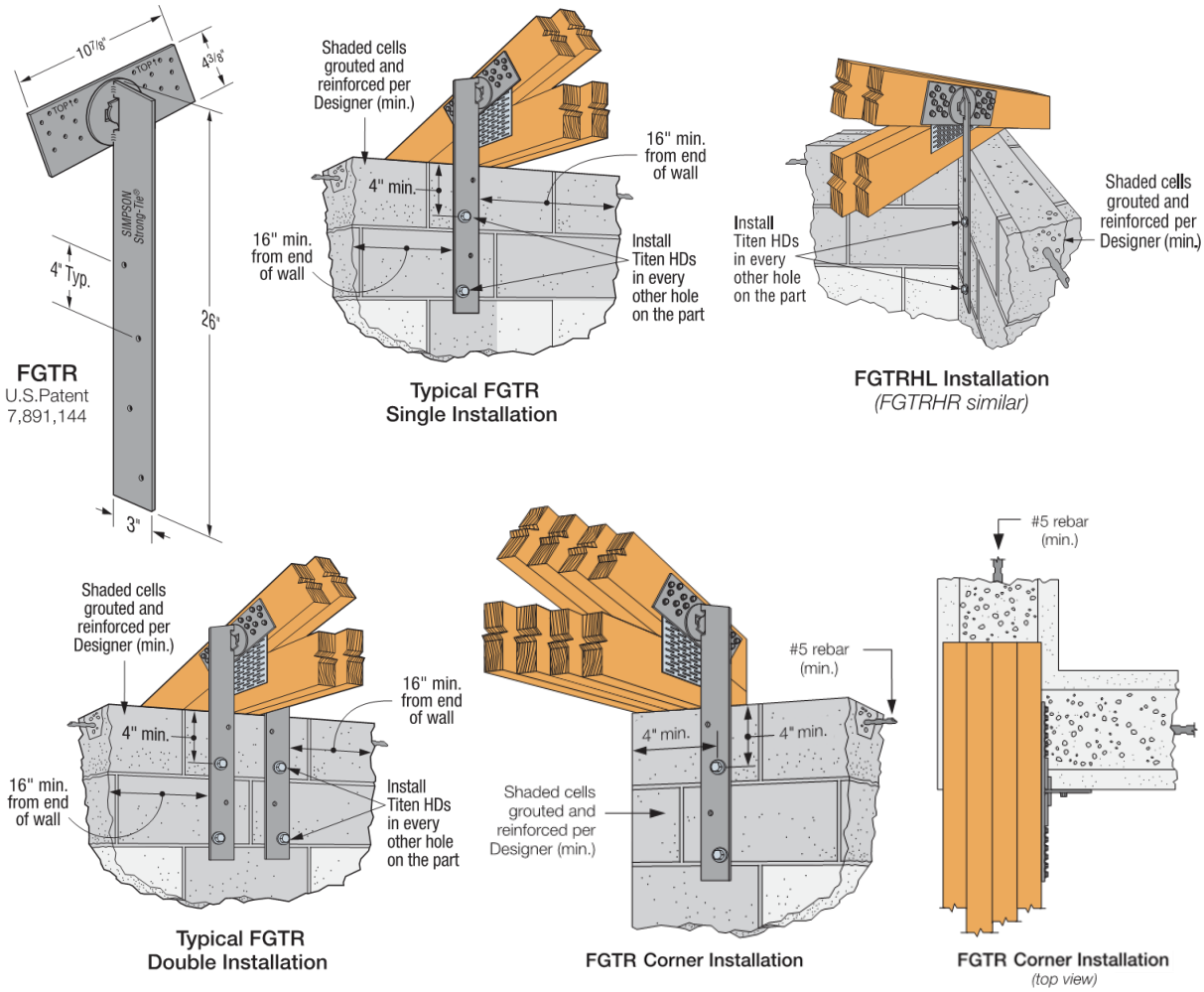


FIGURE 6: FGTR / FGTRHL / FGTRHR Dimensions and Installation Details

TABLE 6: LGT Fasteners and Allowable Loads										
Model No.	Qty.	No. of Plies	Applica-tion	Fasteners			Allowable Loads (lb.)			
							Uplift (160)		Lateral (160)	
				Rafter / Truss	Wood Studs / Framing	GFCMU / Concrete	DF/SP	SPF/HF	DF/SP/SPF/HF	
									F_1	F_2
LGT2	1	2	Wood	(16) 0.148"×3¼"	(14) 0.148"×3¼"	—	2,040	1,755	700[3]	170[3]
				(16) #9×1½" SD	(14) #9×1½" SD	—	2,465	2,125		
			GFCMU or Conc.	(16) 0.148"×3¼"	—	(7) ¼"×2¼" Titen Turbo[5]	2,030	1,750		
LGT3-SDS2.5	1	3	Wood	(12) ¼"×2½" SDS	(21) 0.148"×3¼"	—	3,480	2,505	795	385
			GFCMU or Conc.	(12) ¼"×2½" SDS	—	(4) ⅜"×5" Titen HD	3,285	2,365		
LGT4-SDS3	1	4	Wood	(16) ¼"×3" SDS	(30) 0.148"×3¼"	—	4,060	2,920	2,000[4]	675[4]
			GFCMU or Conc.	(16) ¼"×3" SDS	—	(4) ⅜"×5" Titen HD	3,285	2,365		

Notes:
1. Loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Tabulated LGT2 uplift with SD screws is for DF lumber. LGT2 uplift using SD screws with SP lumber is 2,670 lb.
3. LGT2 lateral loads require installation of optional (4) 0.148"×3¼" in triangle fastener holes into top plates.
4. LGT4 lateral loads require installation of optional (7) 0.148"×3¼" in triangle fastener holes into top plates.
5. Hex-head model required. For concrete applications, use ¼"×1¾" Titen Turbo™. Minimum edge distance for Titen Turbo screw anchor is 1½". Titen® 2 screw anchors of the same diameter and length may be substituted for the tabulated Titen Turbo screw anchor size with no change in allowable load.
6. Minimum f'_c = 2,500 psi. Minimum f'_m = 1,500 psi.

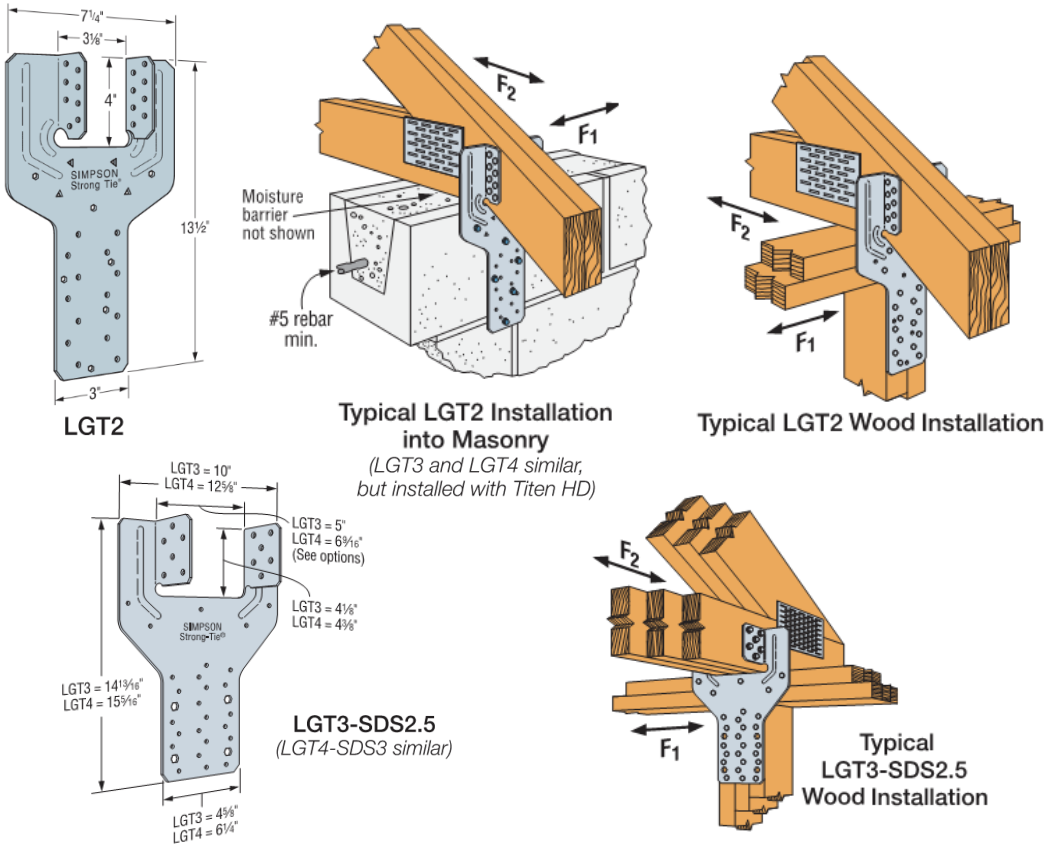


FIGURE 7: LGT2 / LGT3 / LGT4 Dimensions and Installation Details

11. REFERENCES:

Florida Building Code, Building 7th Edition (2020)

Section 104.11	Alternative materials, design, and methods of construction and equipment
Chapter 19	Concrete
Chapter 21	Masonry
Chapter 22	Steel
Chapter 23	Wood

Florida Building Code, Residential 7th Edition (2020)

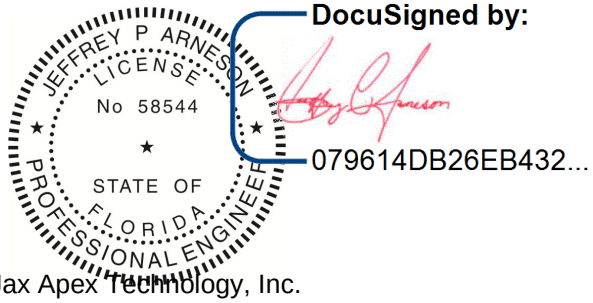
R101.2.1	Scope
R4405	HVHZ Concrete
R4407	HVHZ Masonry
R4408	HVHZ Steel
R4409	HVHZ Wood

Standards

AISI S100	2016
ANSI/AWC NDS	2018
ASTM D7147	2011

12. IDENTIFICATION:

Each connector covered by this report shall be stamped with the manufacturer's name and/or trademark and the product name.



Jax Apex Technology, Inc.
Jeffrey P. Arneson, P.E.
P.E. No. 58544
October 13, 2020

Jax Apex Technology, Inc.
9000 Regency Square Boulevard, Suite 100
Jacksonville, FL 32211

Evaluation reports are the opinion of the engineer who prepared the report, based on the findings, and in no way constitute or imply approval by a local building authority. The engineer, in review of the data submitted, finds that, in his opinion, the product, material, system, or method of construction specifically identified in this report conforms with or is a suitable alternate to that specified in the Florida Building Code, SUBJECT TO THE LIMITATIONS IN THIS REPORT

Jeffrey P. Arneson, P.E., a licensed Florida professional engineer and employee of Jax Apex Technology, Inc. (Apex Technology) has reviewed the data submitted for compliance with the Florida Building Code. Neither Jeffrey P. Arneson, nor Apex Technology, are responsible for any errors or omissions to any documents, calculations, drawings, specifications, tests, or summaries prepared and submitted by the design professional or preparer of record who are listed in the Substantiating Data section of this report.

REPORT NO: SIM202003

CATEGORY: Structural Components

SUBCATEGORY: Wood Connectors

SUBMITTED BY:

SIMPSON STRONG-TIE COMPANY, INC.
5956 W. LAS POSITAS BOULEVARD
PLEASANTON, CA 94588

1. CERTIFICATION OF INDEPENDENCE:

Jeffrey P. Arneson, the Florida engineer who prepared this report, and Apex Technology have no financial interest in the manufacturing, sales, or distribution of the products included in this report. Jeffrey P. Arneson and Apex Technology comply with all criteria as stated in Florida Administrative Code Chapter 61G20-3.

2. PRODUCT NAME:

Column Caps for CMU and Concrete Piers

CCQM, CCTQM, CCCQM, ECCLQM, ECCLLQMD, ECCLRQMD

Wood to Masonry Hangers

LGUM26-2, LGUM28-2, LGUM210-2, LGUM26-3, LGUM28-3, LGUM210-3, LGUM26-4, LGUM28-4, LGUM210-4, LGUM46, LGUM48, LGUM410, HGUM5.25, HGUM5.50, HGUM7.00, HGUM7.25, HGUM9.00

MBHU3.56/9.25KT, MBHU3.56/11.25KT, MBHU3.56/11.88KT, MBHU3.56/14KT, MBHU3.56/16KT, MBHU3.56/18KT, MBHU5.50/9.25KT, MBHU5.50/11.25KT, MBHU5.50/11.88KT, MBHU5.50/14KT, MBHU5.50/16KT, MBHU5.50/18KT

MBHA3.12/9.25, MBHA3.12/11.25, MBHA3.56/7.25, MBHA3.56/9.25, MBHA3.56/11.25, MBHA3.56/11.88, MBHA3.56/14, MBHA3.56/16, MBHA3.56/18, MBHA5.50/7.25, MBHA5.50/9.25, MBHA5.50/11.25, MBHA5.50/11.88, MBHA5.50/14, MBHA5.50/16, MBHA5.50/18

3. SCOPE OF EVALUATION:

Load evaluation as a structural component using the requirements of the 7*th Edition (2020) Florida Building Code, Building* and 7*th Edition (2020) Florida Building Code, Residential*.

4. DESCRIPTION:

4.1 CCQM-SDSHDG, CCTQM-SDSG, CCCQM-SDSG, ECCLQM-KT and ECCLQMD-KT Column Caps for CMU and Concrete Piers. The CCQM, CCTQM, CCCQM, and ECCLQM are used to connect wood beams to the top of CMU or concrete piers, primarily for raised pier foundations. The beams can be oriented in a "straight" configuration (CCQM), "T" configuration (CCTQM), "cross" configuration (CCCQM), or an "L" configuration (ECCLQM). The ECCLQMD has an extended main bucket to support a deck joist framing into the corner. The ECCLQMD is available in left (ECCLLQMD) and right (ECCLRQMD) versions. The column caps fasten to the beams with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws. Specially shaped anchor bolts are factory welded to the bottom of the buckets, and become embedded in the top of the pier. The ECCLQMG-KT and ECCLQMD-KT both include two MSTQM straps, which extend up to provide a holdown for corner framing above. The CCQM, CCTQM, CCCQM, ECCLQM and ECCLQMD are manufactured from 7 gauge steel meeting ASTM A1011 GRADE 33 with a hot-dipped galvanized finish in accordance with ASTM A153. The MSTQM strap is manufactured from 12 gauge steel meeting ASTM A653 SS GRADE 40 with a G90 galvanized finish. Column cap fastener schedule and allowable loads are shown in Table 1. Column cap dimensions are shown in Table 2. MSTQM strap fastener schedule and allowable loads are specified in Note 8 of Table 1. See Figures 1, 2, and 3 for additional details.

4.2 LGUM / HGUM Masonry Girder Hangers. The LGUM and HGUM series of high capacity hangers are used to connect wood girders and beams to masonry or concrete walls. Both series use Simpson Strong-Tie® Titen HD® Heavy-Duty Screw Anchors to attach to the masonry or concrete wall, and Strong-Drive® SDS Heavy-Duty Connector screws to attach the beam to the hanger. The LGUM is manufactured from 12 gauge steel meeting ASTM A653 SS Grade 40 with a G90 galvanized finish. The HGUM is manufactured from 7 gauge galvanized steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Girder hanger fastener schedule, dimensions, and allowable loads are shown in Table 3. See Figure 4 for additional details.

4.3 MBHU Masonry Beam Hangers. The MBHU series of hangers are used to anchor wood beams, joists, trusses, or rafters to masonry or concrete walls. The MBHU fastens to the wood member with ¼"×2½" long Strong-Drive® SDS Heavy-Duty Connector screws, and fastens to the masonry or concrete wall with two ¾"×5" Titen HD® Heavy-Duty Screw Anchors. These connectors are manufactured from 10 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. MBHU fastener schedules, dimensions, and allowable loads are shown in Table 4. See Figures 5 and 6 for additional details.

4.4 MBHA Masonry Hangers. The MBHA series of hangers are used to anchor wood beams, joists, trusses, or rafters to masonry or concrete walls. The MBHA fastens to the wood member with 0.148"×3" nails and fastens to the masonry or concrete with two lengths of threaded rod embedded in the wall using either Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® Epoxy Adhesive. These connectors are manufactured from 10 gauge steel meeting ASTM A653 SS Grade 33 with a G90 galvanized finish. Hanger fastener schedules, dimensions, and allowable loads are shown in Table 5. See Figure 7 for additional details.

5. MATERIALS:

5.1 Steel. Steel specifications for each product listed in this evaluation report shall be as indicated in the previous section. In addition to the standard G90 coating, some models (designated with a model number ending with Z) are available with a G185 zinc coating specification in accordance with ASTM A653. Some models (designated with a model number ending with HDG or G) are available with a hot-dip galvanization, also known as "batch" galvanization, in accordance with ASTM A123, with a minimum specified coating weight of 2.0 ounces of zinc per square foot of surface area, total for both sides. Some models (designated with a model number ending with SS) are available in Type 316L stainless material manufactured in accordance with ASTM A240 sheet, strip or plate and ASTM A480 (General Requirements) in the following designation: UNS designation S31603, AISI Type 316L. Model numbers in this report may not include the Z, HDG, or SS ending, but the information shown applies.

5.2 Wood. Wood members to which these connectors are fastened shall be solid sawn lumber, glued-laminated lumber, or structural composite lumber having dimensions consistent with the connector dimensions shown in Tables 1 through 6. Unless otherwise noted, lumber shall be Southern Pine (SP) or Douglas Fir-Larch (DF) having a minimum specific gravity of 0.55 or 0.50, respectively. Where indicated by SPF, lumber shall be Spruce-Pine-Fir having a minimum specific gravity of 0.42.

5.3 Concrete/Masonry. Concrete and Masonry design specifications shall be the stricter of the specifications by the engineer of record, the Florida Building Code minimum standards, the following, or as noted in the report:

Material	Specification	Minimum Compressive Strength
Concrete, f'_c	-	2,500 psi[1]
Masonry, f'_m	ASTM C1314	1,500 psi
Masonry Unit	ASTM C90	2,000 psi
Mortar	ASTM C270 Type S	1,800 psi (or by proportions)
Grout	ASTM C476	2,000 psi (or by proportions)

1. Where noted for CCQM models, 3,000 psi concrete is required.

5.4 Nails. Unless noted otherwise, nails shall be common nails. Nails shall comply with ASTM F1667 and shall have the minimum bending yield strengths F_{yb}:

Nail Pennyweight	Nail Shank Diameter (inch)	Nail Length (inch)	F_{yb} (psi)
10d	0.148	3	90,000
16d×2½	0.162	2½	90,000

Fasteners for galvanized connectors in pressure-preservative treated wood shall be hot-dipped zinc coated galvanized steel with coating weights in accordance with

ASTM A153 or steel mechanically galvanized in accordance with ASTM B695, Class 55. Fasteners for stainless steel connectors shall be stainless steel except where otherwise permitted by the treatment manufacturer.

The allowable loads of stainless-steel connectors match those of carbon-steel connectors when installed with Simpson Strong-Tie stainless-steel, SCNR ring-shank nails, unless noted otherwise.

5.5 Strong-Drive® SDS Heavy-Duty Connector Screws. Fasteners used with the connectors described in Tables 1, 3, and 4 of the report must be Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector Screws as recognized in FL9589. SDS screws used in contact with preservative-treated or fire-retardant-treated lumber must, at a minimum, comply with FL9589. The lumber treater or Simpson Strong-Tie Company should be contacted for recommendations on minimum corrosion resistance and connection capacities of fasteners used with the specific proprietary preservative-treated or fire retardant-treated lumber.

5.6 Titen HD® Heavy-Duty Screw Anchors. Anchors shown in Tables 3 and 4 and Figures 4 and 6 shall be Simpson Strong-Tie® Titen HD Heavy-Duty Screw Anchors for masonry or concrete as recognized in FL15730 and FL16230. Installation shall be as specified in FL15730 and FL16230.

5.7 Epoxy Anchoring Adhesives. Anchors shown in Table 5 and Figure 7 shall be all-thread rod installed with either Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® High-Strength Epoxy Adhesive for masonry or concrete as recognized in FL15730 and FL16230. Installation shall be as specified in FL15730 and FL16230.

6. INSTALLATION:

Installation shall be in accordance with this report and the most recent edition of the Simpson Strong-Tie *Wood Construction Connectors* catalog. Information in this report supersedes any conflicting information between information provided in this report and the catalog.

7. SUBSTANTIATING DATA:

Test data submitted by Testing Engineers Inc. and Simpson Strong-Tie, and signed and sealed calculations performed by Bryan Wert, P.E., in accordance with the *7th Edition (2020) Florida Building Code, Building* and *Residential* codes.

Model	Test Lab	Test Number
ECCLQM	Testing Engineers, Inc.	P002, P004, P005, Q375
ECCLQMD	Simpson Strong-Tie, Inc.	S944, S945
CCQM, CCTQM	Testing Engineers, Inc.	P006, P007, P008, P009
LGUM	Testing Engineers, Inc.	M202, M203, M204, M211, M212, M213, M221, M222, M224
HGUM	Testing Engineers, Inc.	M207, M209, M216, M217, Q690, Q691, R058, R059
MBHU	Testing Engineers, Inc.	N447, N448, N449, N450, O356, O357, O360, O361, P773, P774, Q848, Q849
MBHA	Testing Engineers, Inc.	F391, F392, F393, F394, F964, F965

8. FINDINGS:

The connectors listed in this evaluation report comply with the *7th Edition (2020) Florida Building Code, Building*, and *7th Edition (2020) Florida Building Code, Residential* when installed in accordance with this report.

9. LIMITATIONS:

1. Maximum allowable loads shall not exceed the allowable loads listed in this report. Allowable loads listed in this report are based on allowable stress design. The loads in this report are not applicable to Load and Resistance Factor Design.
2. Capacity of wood members is not covered by this report. Capacity of wood members must be checked by the building designer.
3. Connectors in this report having a galvanized coating thickness less than G185 shall not be installed in contact with preservative-treated wood products that are exposed to rainfall or ground moisture.
4. Allowable loads for more than one direction for a single connection cannot be added together. A design load that can be divided into components in the directions given must be evaluated as follows:

   (Design Uplift / Allowable Uplift) + (Design Lateral / Allowable Lateral) < 1.0

10. ALLOWABLE LOADS:

The tables that follow reference the allowable loads for the aforementioned products.

TABLE 1: CCQM / CCCQM / CCTQM / ECCLQM Fasteners and Allowable Loads

Model No.	No. of ¼"×2½" SDS Screws			16" Square Grout-Filled CMU Pier[3,6]				16" Square CMU Shell Filled with 3,000 psi Concrete[3,7]			
				Uplift (160)			Lateral (160)	Uplift (160)			Lateral (160)
	Main Beam	Side Beam	Deck Beam	Main Beam	Side Beam	Total		Main Beam	Side Beam	Total	
CCQM-SDSHDG	12	—	—	6,750	—	6,750	2,460	6,495	—	6,495	2,650
CCTQM-SDSG	12	8	—	6,750	5,375	6,750	2,460	6,495	5,375	6,495	2,650
CCCQM-SDSG	12	8	—	6,750	5,375	6,750	2,460	6,495	5,375	6,495	2,650
ECCLQMG-KT[8]	16	16	—	6,240	6,240	7,340	2,220	6,240	6,240	7,830	2,565
ECCLQMDG-KT[9]	16	16	6	6,240	6,240	7,340	2,220	6,240	6,240	7,830	2,565

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Total uplift load and lateral load is based on tested anchor failure in the pier.
3. Allowable loads are based on either a 16" square grout-filled CMU pier with f'_m of 1,500 psi or a 16" square CMU shell filled with 3,000 psi concrete. A minimum of (4) #7 vertical rebars or equivalent steel area is required. The Designer shall design and detail the GFCMU/concrete pier to resist all forces including uplift, shear, and moment.
4. Pier height per Building Designer.
5. Side beam and main beam uplift loads assume DF or SP members and are not additive.
6. The allowable loads listed for grout-filled CMU apply to solid concrete piers of 2,500 psi concrete a minimum of 16" square.
7. The allowable loads listed for CMU shell filled with 3,000 psi concrete apply to solid concrete piers of 3,000 psi concrete a minimum of 14" square.
8. The ECCLQM-KT is a kit packaged with (2) MSTQM straps and (32) SDS ¼"×2½" screws. One strap may be installed on each face of the ECCLQM (as shown), using the SDS screws into the beams and (26) 0.162"x2½" nails (not provided) into the wall framing. The MSTQM strap's allowable tension load is 6,240 lbs. If straps are not installed, table uplift applies to beam only.
9. The ECCLQMD-KT is a kit packaged with (2) MSTQM straps and (32) SDS ¼"×2½" screws. The allowable download for the deck joist connection is 5,475 lb. (2-2X joist) or 5,240 lb. (2X joist) and the allowable uplift is 2,010 lb.

TABLE 2: CCQM / CCTQM / CCCQM / ECCLQM Dimensions				
Model No.	**Main Channel Width, W_1 (in.)**	**Side Stirrup Width, W_3 and W_4 (in.)**	**Main Channel Length, L_1 (in.)**	**Side Stirrup Length, L_2 (in.)**
CCQM3.62-SDSHDG	3 5/8	—	11	—
CCQM4.62-SDSHDG	4 5/8	—	11	—
CCQM5.50-SDSHDG	5 ½	—	11	—
CCTQM3.62-SDSG	3 5/8	3 5/8	11 ½	4
CCTQM4.62-SDSG	4 5/8	4 5/8	13 ½	4
CCTQM5.50-SDSG	5 ½	5 ½	13 ½	4
CCCQM3.62-SDSG	3 5/8	3 5/8	11 ½	4
CCCQM4.62-SDSG	4 5/8	4 5/8	13 ½	4
CCCQM5.50-SDSG	5 ½	5 ½	13 ½	4
ECCLQM3.62G-KT[1]	3 5/8	3 5/8	11 ½ [2]	7 ¾
ECCLQM4.62G-KT[1]	4 5/8	4 5/8	11 ½ [2]	7 ¾
ECCLQM5.50G-KT[1]	5 ½	5 ½	11 ½ [2]	7 ¾

Notes:
1. The MSTQM is a component of the ECCLQM kits. It is 12 ga.; 3" wide and 48" long.
2. The ECCLL/RQMDG-KT has the same dimensions as ECCLQMG-KT, except W1 = 16½".

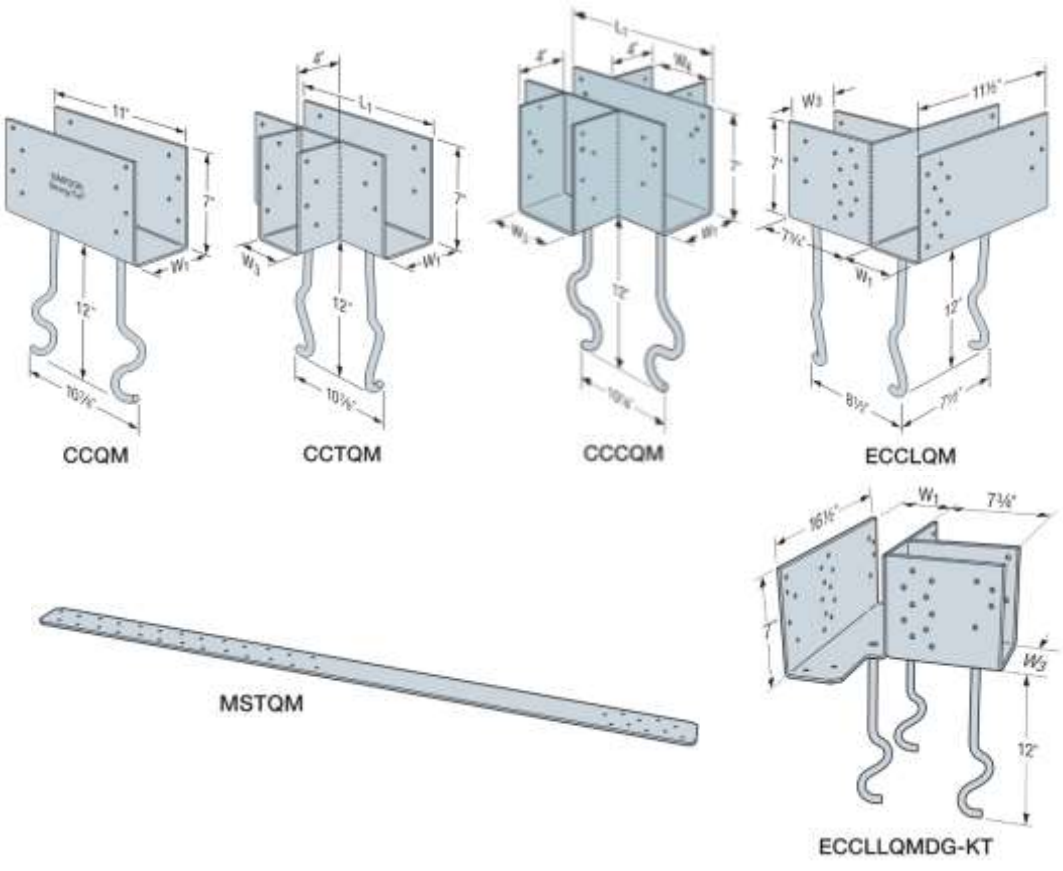


FIGURE 1: CCQM / CCTQM / CCCQM / ECCLQM / ECCLQMD / MSTQM Dimensions

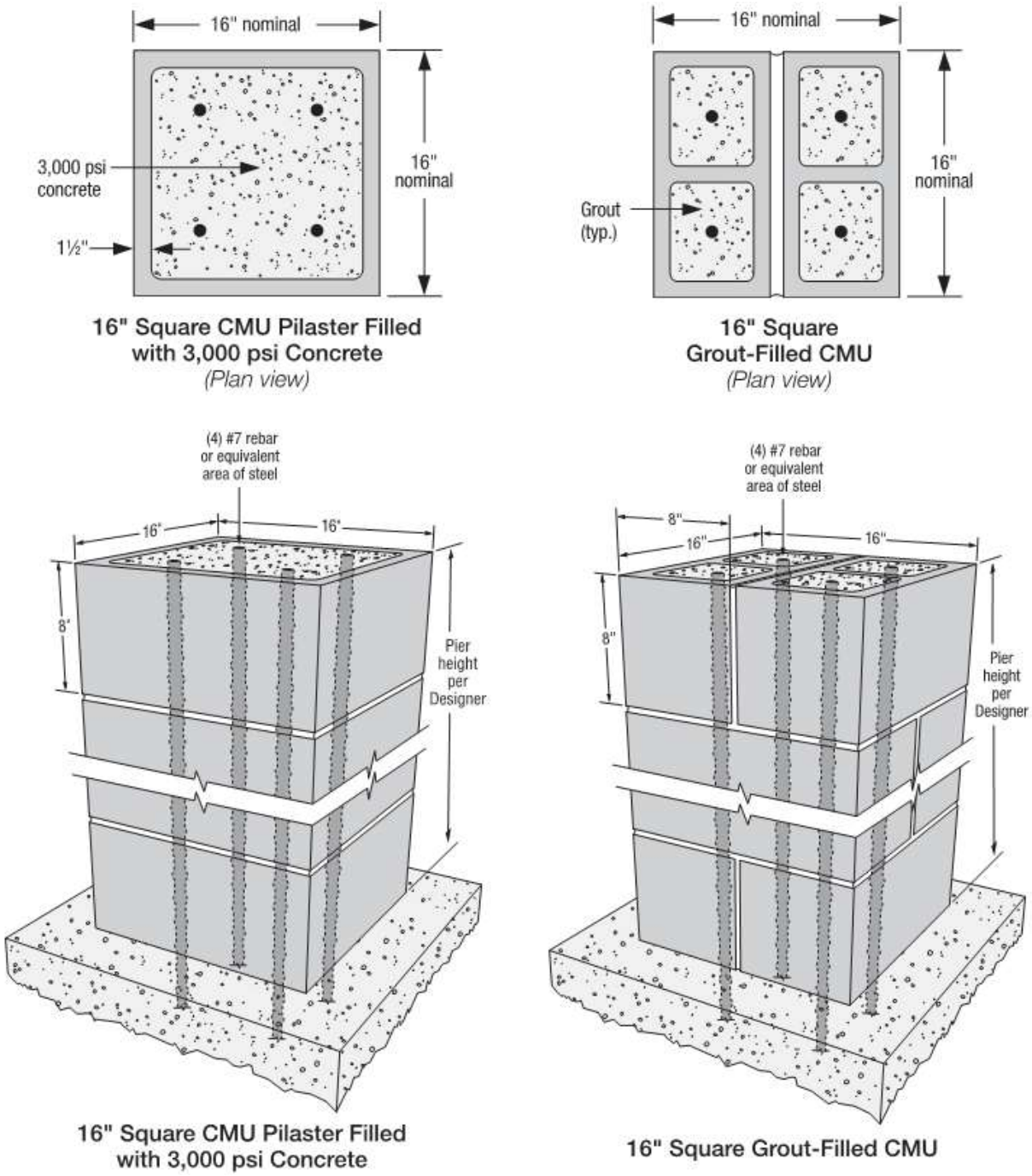
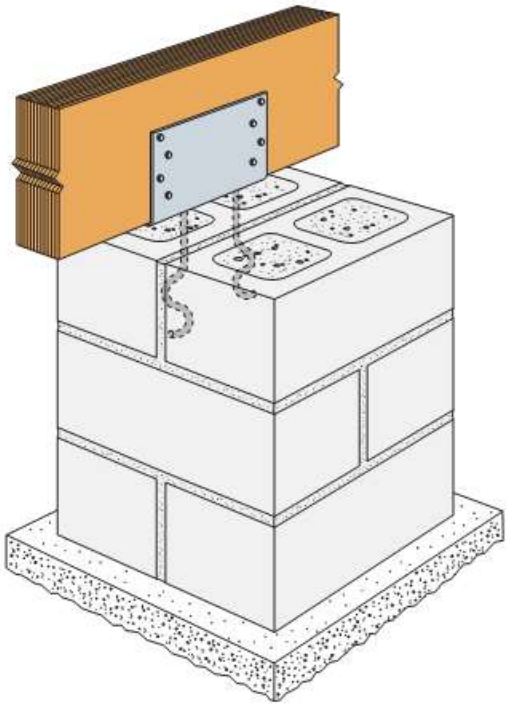
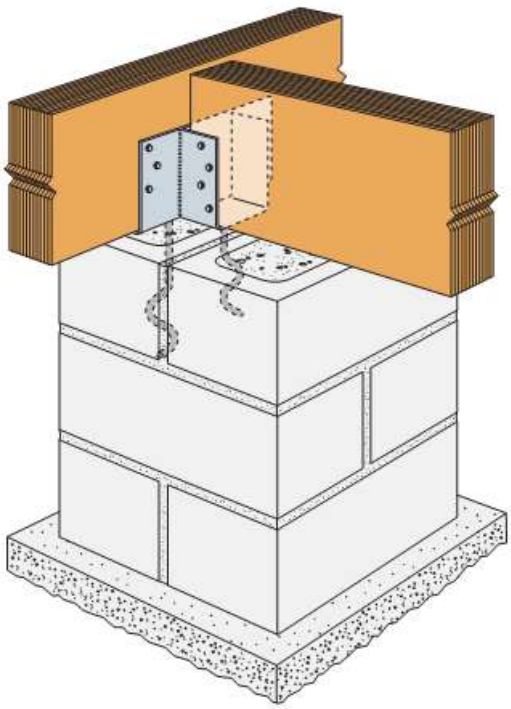


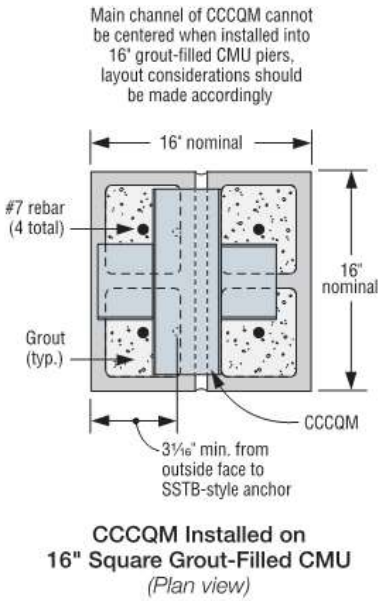
FIGURE 2: Pier Typical Dimensions and Minimum Requirements



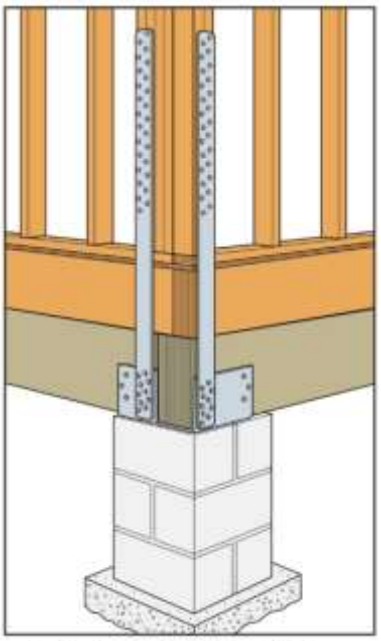
Typical CCQM Installation



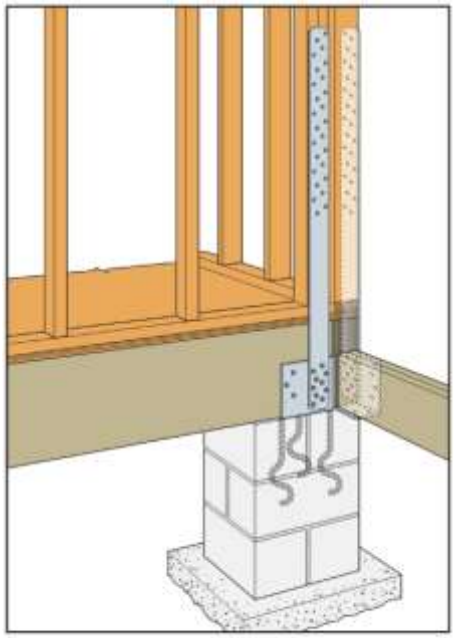
Typical CCTQM Installation



CCCQM Installed on 16" Square Grout-Filled CMU
(Plan view)



Typical ECCLQM Installation



Typical ECCLRQMD-KT Installation

FIGURE 3: CCQM / CCTQM / CCCQM / ECCLQM / ECCLQMD / MSTQM Installation Details

TABLE 3 – LGUM and HGUM Dimensions, Fasteners and Allowable Loads										
Model No.	Ga.	Dimensions (in.)			Fasteners		DF/SP Allowable Loads (lb.)			
		W	H	B	GFCMU and Concrete	Joist	Uplift[2] (160)		Download (100/115/125)	
					Titen HD® Anchors[7]	Strong-Drive® SDS Screws	GFCMU	Concrete	GFCMU	Concrete
Double 2x Sizes										
LGUM26-2	12	3 7/16	5	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,595	
LGUM28-2	12	3 7/16	7	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,250	
LGUM210-2	12	3 7/16	9	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,575	
Triple 2x Sizes										
LGUM26-3	12	5 ⅛	5 ⅛	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,610	
LGUM28-3	12	5 ⅛	7 ⅛	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,290	
LGUM210-3	12	5 ⅛	9 ⅛	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,715	
Quadruple 2x Sizes										
LGUM26-4	12	6 11/16	5 5/16	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,625	
LGUM28-4	12	6 11/16	7 5/16	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,335	
LGUM210-4	12	6 11/16	9 5/16	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,860	
4x Sizes										
LGUM46	12	3 ⅝	4 ⅞	4	(4) ⅜"×4"	(4) ¼"×2½"	1,430		5,600	
LGUM48	12	3 ⅝	6 ⅞	4	(6) ⅜"×4"	(6) ¼"×2½"	2,435		8,260	
LGUM410	12	3 ⅝	8 ⅞	4	(8) ⅜"×4"	(8) ¼"×2½"	3,575		9,620	
Engineered Wood & Structural Composite Lumber Sizes (Heavy Duty)										
HGUM5.25	7	5 ¼	11 to 30	5 ¼	(8) ⅝"×5"	(24) ¼"×2½"	4,105[2]	5,075[2]	14,025	14,770
HGUM5.50	7	5 ½		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			14,000	14,915
HGUM7.00	7	7		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,840	14,915
HGUM7.25	7	7 ¼		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,810	14,915
HGUM9.00	7	9		5 ¼	(8) ⅝"×5"	(24) ¼"×2½"			13,625	14,915

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Tabulated uplift loads are based on a 4" min. distance to top of wall. For HGUM installations with a 15" min. distance to top of wall, uplift loads may be increased to 6,180 lb. for GFCMU and 6,585 lb. for concrete.
3. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
4. LGUM must be installed on minimum 6" thick wall and HGUM on minimum 8" thick wall (nominal values for GFCMU).
5. Titen HD anchors may be installed into the head or bed joints.
6. Structural composite lumber (SCL) shall have a minimum specific gravity of 0.50.
7. For stainless steel LGUM and HGUM models, use ⅜"×5" and ⅝"×6" Stainless-Steel Titen HD anchors, respectively, to achieve tabulated loads.

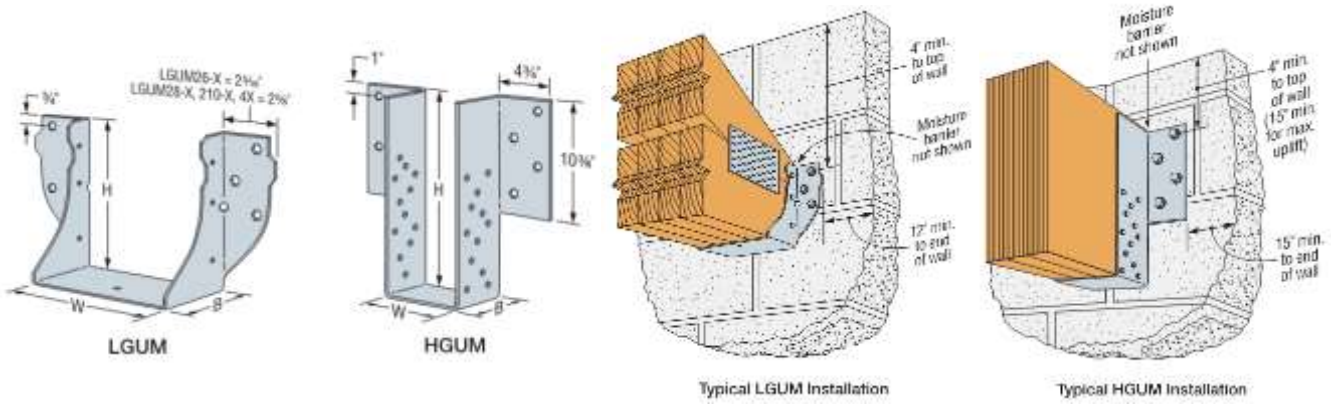


FIGURE 4: LGUM / HGUM Dimensions and Installation Details

TABLE 4: MBHU Dimensions, Fasteners, and Allowable Loads												
Model No.	Dimensions (in.)			Fasteners		DF/SP Allowable Loads (lb.)					
				GFCMU and Concrete	Joist	End of Wall / Outside Corner				Away From Edge	
						GFCMU		Concrete		GFCMU/Concrete	
	W	H	B	Titen HD® Anchors[5]	SDS Screws	Uplift (160)	Download (100)	Uplift (160)	Download (100)	Uplift (160)	Download (100)
MBHU3.56/9.25	3 9/16	9 ¼	3 ½	(2) ¾"×5"	(12) ¼"×2½"	1,610	2,440	2,715	4,190	2,210	4,005
MBHU3.56/11.25		11 ¼									
MBHU3.56/11.88		11 ⅞									
MBHU3.56/14		14				1,610	2,440	2,715	4,190	3,345	6,065
MBHU3.56/16		16									
MBHU3.56/18		18									
MBHU5.50/9.25	5 ½	9 ¼	3 ½	(2) ¾"×5"	(12) ¼"×2½"	1,610	2,440	2,715	4,190	2,210	4,005
MBHU5.50/11.25		11 ¼									
MBHU5.50/11.88		11 ⅞									
MBHU5.50/14		14				2,240	3,260[4]	3,485	6,970	3,345	6,065
MBHU5.50/16		16									
MBHU5.50/18		18									

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
3. Structural composite lumber (SCL) shall have a minimum specific gravity of 0.50.
4. Where noted in the table, download is for End of Wall condition. Download for Outside Corner is 3,640 lb.
5. For stainless steel MBHU models, use ¾"×6" Stainless-Steel Titen HD® anchors to achieve tabulated loads.

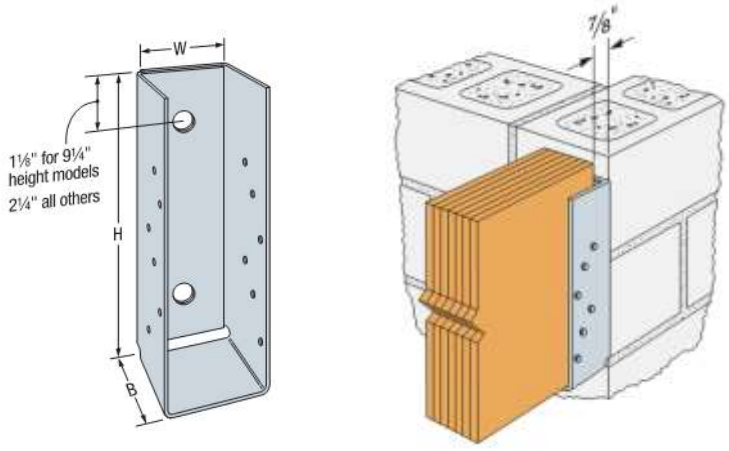


FIGURE 5: MBHU Dimensions and Installation Detail

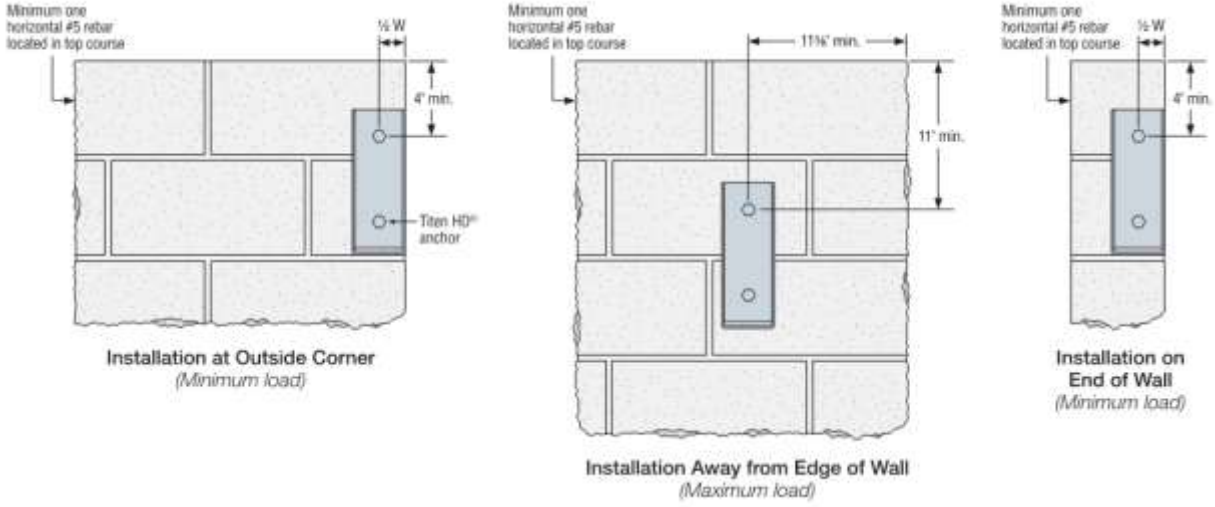


FIGURE 6: MBHU Installation Location Options

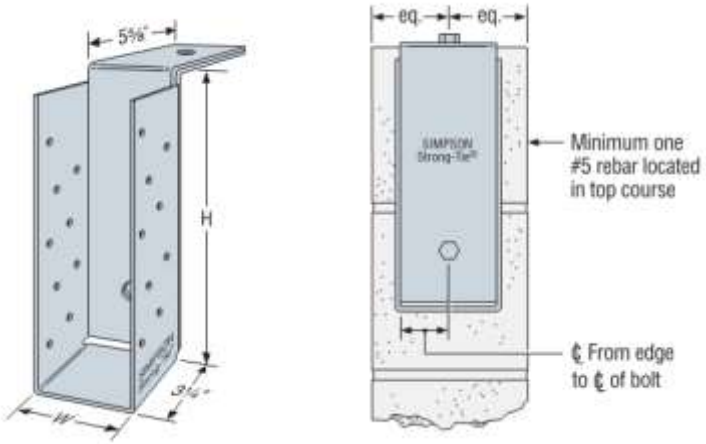


FIGURE 7: MBHA Dimensions and Installation Detail

TABLE 5: MBHA Dimensions, Fasteners, and Allowable Loads								
Model No.	Dimensions (in.)			Fasteners			DF/SP Allowable Loads (lb.)	
				GFCMU / Concrete[2,3]		Joist	GFCMU / Concrete[6,7]	
	W	H	B	Top	Face		Uplift (160)	Download (100)
MBHA3.56/7.25	3 9/16	7 ¼	3 ¼	(1) ATR¾ [4]	(1) ATR¾ [5]	(18) 0.148"×3"	1,885	4,380
MBHA5.50/7.25	5 ½	7 ¼	3 ¼					
MBHA3.12/9.25	3 1/8	9 ¼	3 ¼	(1) ATR¾ [4]	(1) ATR¾ [4]	(18) 0.148"×3"	3,475 [7]	6,275 [7]
MBHA3.12/11.25	3 1/8	11 ¼	3 ¼					
MBHA3.56/9.25	3 9/16	9 ¼	3 ¼					
MBHA3.56/11.25	3 9/16	11 ¼	3 ¼					
MBHA3.56/11.88	3 9/16	11 7/8	3 ¼					
MBHA3.56/14	3 9/16	14	3 ¼					
MBHA3.56/16	3 9/16	16	3 ¼					
MBHA3.56/18	3 9/16	18	3 ¼					
MBHA5.50/9.25	5 ½	9 ¼	3 ¼					
MBHA5.50/11.25	5 ½	11 ¼	3 ¼					
MBHA5.50/11.88	5 ½	11 7/8	3 ¼					
MBHA5.50/14	5 ½	14	3 ¼					
MBHA5.50/16	5 ½	16	3 ¼					
MBHA5.50/18	5 ½	18	3 ¼					

Notes:

1. Uplift loads have been increased 60% for wind loading as permitted by the code. No further increase permitted. Reduce where other loads govern.
2. Minimum f'_m = 1,500 psi and f'_c = 2,500 psi.
3. ATR is all-thread rod, ¾" diameter, grade A307 or better.
4. Loads for MBHA are based on installation using Simpson Strong-Tie® ET-HP® Epoxy Adhesive or Simpson Strong-Tie SET-XP® High-Strength Epoxy Adhesive with 6¾" minimum embedment required.
5. MBHA hangers with height of 7¼" require a 3½" minimum embedment of the face bolt using ET-HP Epoxy Adhesive or Simpson Strong-Tie SET-XP High-Strength Epoxy Adhesive.
6. Alternate anchorage permitted when designed by others to have equivalent or better capacity.
7. MBHA hangers with height of 9¼" and greater installed into concrete achieve 3,980 lb. uplift (160) and 6,790 lb. download (100).

11. REFERENCES:

Florida Building Code, Building 7th Edition (2020)

Section 104.11	Alternative materials, design, and methods of construction and equipment
Chapter 19	Concrete
Chapter 21	Masonry
Chapter 22	Steel
Chapter 23	Wood

Florida Building Code, Residential 7th Edition (2020)

R101.2	Scope
R4405	HVHZ Concrete
R4407	HVHZ Masonry
R4408	HVHZ Steel
R4409	HVHZ Wood

Standards

AISI S100	2016
ANSI/AWC NDS	2018
ASTM D7147	2011

12. IDENTIFICATION:

Each connector covered by this report shall be stamped with the manufacturer's name and/or trademark and the product name.

DocuSigned by:



079614DB26EB432...

Jax Apex Technology, Inc.
Jeffrey P. Arneson, P.E.
P.E. No. 58544
October 13, 2020

MIAMI-DADE COUNTY
DEPARTMENT OF REGULATORY AND ECONOMIC RESOURCES (RER)
BOARD AND CODE ADMINISTRATION DIVISION

MIAMI-DADE COUNTY
PRODUCT CONTROL SECTION
11805 SW 26 Street, Room 208
Miami, Florida 33175-2474
T (786) 315-2590 F (786) 315-2599
www.miamidade.gov/economy

NOTICE OF ACCEPTANCE (NOA)

Cast-Crete USA, LLC
6324 County Road 579
Seffner, Florida 33584

SCOPE:
This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed and accepted by Miami-Dade County RER- Product Control Section to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Section (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. RER reserves the right to revoke this acceptance, if it is determined by Miami-Dade County Product Control Section that this product or material fails to meet the requirements of the applicable building code.

This product is approved as described herein, and has been designed to comply with the High Velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: "Cast-Crete" 8 inch High Strength Precast and Pre-stressed Concrete Lintels

APPROVAL DOCUMENT: Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheets 1 through 3 of 3, prepared by Craig Parrino, P.E., dated June 7, 1996, last revision dated June 12, 2014, signed and sealed by Craig Parrino, P.E., on January 22, 2019, bearing the Miami-Dade County Product Control Revision stamp with the Notice of Acceptance number and the expiration date by the Miami-Dade County Product Control Section.

MISSILE IMPACT RATING: Large and Small Missile Impact Resistant

LABELING: Each lintel shall bear a permanent label with the manufacturer's name or logo and the Miami-Dade County logo.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.

This NOA **renews NOA #20-0630.16** and consists of this page 1, evidence submitted pages E-1, E-2, E-3, E-4, E-5, E-6 & E-7 as well as approval document mentioned above.
The submitted documentation was reviewed by **Helmy A. Makar, P.E., M.S.**

MIAMI-DADE COUNTY APPROVED

Helmy A. Makar
03/10/2022

NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

Cast-Crete USA, LLC

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

1. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #96-0613.01

A. DRAWINGS

1. *Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", prepared by Craig Parrino, P.E., dated 06/07/96, revision #1 dated 11/08/96, sheet 1 of 1, and Drawing No. LT8, titled "Cast-Crete 8" Lintel Safe Load Tables", prepared by Craig Parrino, P.E., dated 06/07/97, revision #4 dated 04/28/98, sheet 1 of 1, both drawings signed and sealed by Craig Parrino, P.E.*

B. TESTS

1. *Test report on flexural testing on Precast and Pre-stressed Concrete Lintels Filled Models, total of (36) concrete lintel specimens, per ASTM E-529-94, prepared by Applied Research Laboratories, Report No. 29702B, dated 07/30/97, signed and sealed by Christopher A. Hamon, P.E.*

C. CALCULATIONS

1. *Calculations for Cast-Crete 8-inch Lintels, dated 06/06/96, 349 pages, prepared by Craig Parrino, P.E., signed and sealed by Craig Parrino, P.E.*
2. *Calculations for Cast-Crete 8-inch Lintels, dated 11/11/96, 45 pages, prepared by Craig Parrino, P.E., signed and sealed by Craig Parrino, P.E.*

D. MATERIAL CERTIFICATIONS

1. *Mill Certified Inspection Report Silo #4 dated April 1996 for cement by Southdown Brooksville Cement Laboratory.*
2. *Mill Certified Inspection Report #20-8275 dated 06/20/95 for pre-stressing strand by The Crispin Company.*
3. *Mill Certified Inspection Report #20-8345 dated 11/13/95 for pre-stressing strand by The Crispin Company.*
4. *Mill Certified Inspection Report #2111-078240 dated 04/25/96 for #3 rebar by Ameristeel.*
5. *Mill Certified Inspection Report #84-M19614 dated 02/29/96 for #5 rebar by Birmingham Steel Corp.*
6. *Mill Certified Inspection Report #58-A66332 dated 01/28/96 for #4 rebar by Birmingham Steel Corp.*
7. *Mill Certified Inspection Report #84-M20749 dated 04/05/96 for #5 rebar by Birmingham Steel Corp.*
8. *Mill Certified Inspection Report #84-M19805 dated 03/06/96 for #4 rebar by Birmingham Steel Corp.*
9. *Mill Certified Inspection Report #84-M21208 dated 04/21/96 for #4 rebar by Birmingham Steel Corp.*
10. *Mill Certified Inspection Report #84-M20652 dated 04/03/96 for #5 rebar by Birmingham Steel Corp.*

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

11. *Mill Certified Inspection Report #84-M20749 dated 04/05/96 for #5 rebar by Birmingham Steel Corp.*
12. *Quality Control Manual for Cast-Crete Machine Made Precast Lintels dated 05/14/96.*
13. *Quality Control Manual for Pre-stressed Products dated 03/20/95.*

2. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #01-0209.03

A. DRAWINGS

1. *None.*

B. TESTS

1. *None.*

C. CALCULATIONS

1. *None.*

D. MATERIAL CERTIFICATIONS

1. *None.*

3. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL # 03-0605.05

A. DRAWINGS:

1. *Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheet 1 of 1, prepared by Craig Parrino, P.E., dated June 7, 1996, revision #2 dated January 28, 2003, and Drawing No. LT8, titled "Cast-Crete 8" Lintel Safe Load Tables", sheets 1 & 2 of 2, prepared by Craig Parrino, P.E., dated June 7, 1996, revision #5 dated March 24, 2003, all drawings are signed and sealed by Craig Parrino, P.E.*

B. TESTS:

1. *Test report on flexural testing on Precast Concrete Lintels Filled Models, total of (12) concrete lintel specimens, per ASTM E-529-94, prepared by Applied Research Laboratories, Report No. 30404B, dated 04/23/03, signed and sealed by Christopher A. Hamon, P.E.*

C. CALCULATIONS:

1. *Calculations for Cast-Crete High Strength Precast and Pre-stressed 8-inch Lintels, dated May 29, 2003, sheets 1.1 through 1.493 and 2.1 through 2.336, prepared by Craig Parrino, P.E., signed and sealed by Craig Parrino, P.E.*

D. MATERIAL CERTIFICATION:

1. *Quality System Manual for Cast-Crete Machine Made Precast Lintels revised on May 01, 2002.*
2. *Cast-Crete Quality System Manual for Pre-stressed Products dated 2002.*

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

4. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #06-0222.01

A. DRAWINGS

1. *Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheets 1 through 3 of 3, prepared by Craig Parrino, P.E., dated June 7, 1996, last revision dated February 17, 2006, signed and sealed by Craig Parrino, P.E.*

B. TESTS

1. *None.*

C. CALCULATIONS

1. *None.*

D. QUALITY ASSURANCE

1. *By Miami-Dade Building Code Compliance Office.*

E. MATERIAL CERTIFICATION

1. *None.*

F. OTHER

1. *Letter from Cast-Crete Corporation, dated 02/20/2006, signed by Craig Parrino, P.E.*

5. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL # 11-0121.04

A. DRAWINGS

1. *None.*

B. TESTS

1. *None.*

C. CALCULATIONS

1. *None.*

D. QUALITY ASSURANCE

1. *By Miami-Dade Building and Neighborhood Compliance Department (BNC).*

E. MATERIAL CERTIFICATION

1. *None.*

F. OTHER

1. *Letter from Cast-Crete Corporation, dated January 18, 2011, signed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2007 Edition.*

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

6. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #12-0209.13

A. DRAWINGS

1. *Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheets 1 through 3 of 3, prepared by Craig Parrino, P.E., dated June 7, 1996, last revision dated February 17, 2006, signed and sealed by Craig Parrino, P.E., on March 22, 2012.*

B. TESTS

1. *None.*

C. CALCULATIONS

1. *None.*

D. QUALITY ASSURANCE

1. *By Miami-Dade County Department of Permitting, Environment, and Regulatory Affairs.*

E. MATERIAL CERTIFICATION

1. *None.*

F. OTHER

1. *Letter from Cast-Crete Corporation, dated 02/03/2012, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2010 Edition.*

7. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #14-0903.02

A. DRAWINGS

1. *Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheets 1 through 3 of 3, prepared by Craig Parrino, P.E., dated June 7, 1996, last revision dated June 12, 2014, signed and sealed by Craig Parrino, P.E., on August 19, 2014.*

B. TESTS

1. *None.*

C. CALCULATIONS

1. *Calculations for High Strength Precast and Pre-stressed 8-inch Lintels, dated August 19, 2014, sheets 14.101 through 14.124, prepared, signed, sealed by Craig Parrino, P.E.*

D. QUALITY ASSURANCE

1. *By Miami-Dade County Department of Regulatory and Economic Resources (RER).*

E. MATERIAL CERTIFICATION

1. *None.*

F. OTHER

1. *Letter from Cast-Crete USA, Inc., dated August 19, 2014, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2014 Edition.*

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

8. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #16-0126.03

A. DRAWINGS

1. None.

B. TESTS

1. None.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

F. OTHER

1. Letter from Cast-Crete USA, Inc., dated January 13, 2016, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2014 Edition.

9. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #17-0227.15

A. DRAWINGS

1. None.

B. TESTS

1. Verification Test report on flexural testing on Precast Concrete Lintels Filled Models, total of (8) concrete lintel specimens, per ASTM E-529-94, prepared by FTL, Report No. 16-0425.08, dated 01/26/17, signed and sealed by Idalmis Ortega, P.E.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

10. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #17-0821.19

A. DRAWINGS

1. None.

B. TESTS

1. None.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

F. OTHER

1. Letter from Cast-Crete USA, Inc., dated August 08, 2017, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2017 Edition.

11. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #19-0130.13

A. DRAWINGS

1. Drawing No. FD8, titled "Cast-Crete 8" Lintel Fabrication Details", sheets 1 through 3 of 3, prepared by Craig Parrino, P.E., dated June 7, 1996, last revision dated June 12, 2014, signed and sealed by Craig Parrino, P.E., on January 22, 2019.

B. TESTS

1. None.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

F. OTHER

1. Article of Conversion for a name change.

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

12. EVIDENCE SUBMITTED UNDER PREVIOUS APPROVAL #20-0630.16

A. DRAWINGS

1. None.

B. TESTS

1. None.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

F. OTHER

1. Letter from Cast-Crete USA, Inc., dated August 08, 2017, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2017 Edition.

2. Letter from Cast-Crete USA, Inc., dated June 11, 2020, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2020 Edition.

13. NEW EVIDENCE SUBMITTED

A. DRAWINGS

1. None.

B. TESTS

1. None.

C. CALCULATIONS

1. None.

D. QUALITY ASSURANCE

1. By Miami-Dade County Department of Regulatory and Economic Resources (RER).

E. MATERIAL CERTIFICATION

1. None.

F. OTHER

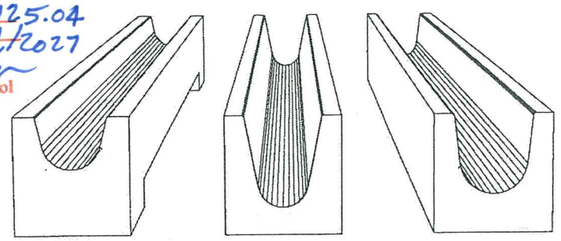
1. Letter from Cast-Crete USA, Inc., dated January 12, 2022, signed and sealed by Craig Parrino, P.E., certifying compliance with the Florida Building Code, 2020 Edition.

Helmy A. Makar, P.E., M.S.
Product Control Section Supervisor
NOA No. 22-0125.04
Expiration Date: 05/21/2027
Approval Date: 03/10/2022

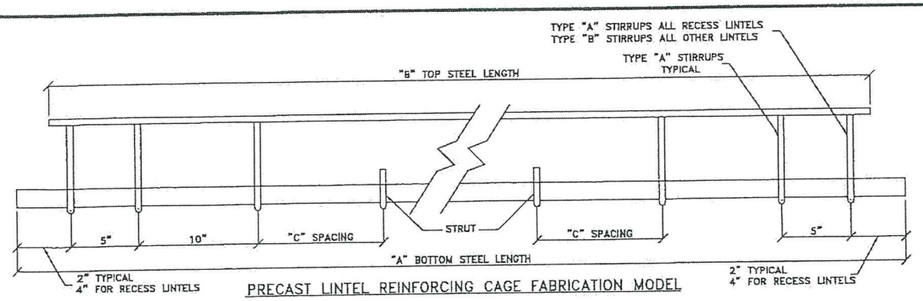
CAST-CRETE® HIGH STRENGTH PRECAST AND PRESTRESSED CONCRETE LINTELS

PRODUCT RENEWED
as complying with the Florida
Building Code
Acceptance No 22-0125.04
Expiration Date 05/21/2027
By Hedy A. Mesa
Miami Dade Product Control

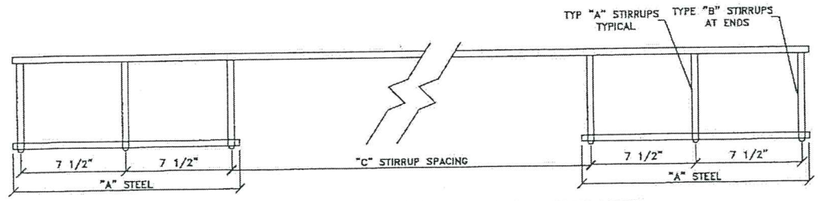
PRODUCT REVISED
as complying with the Florida
Building Code
Acceptance No 20-0630.16
Expiration Date 05/21/2022
By Hedy A. Mesa
Miami Dade Product Control



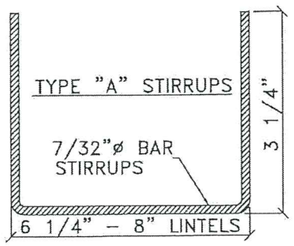
8" PRECAST W/ 2" RECESS DOOR U-LINTEL | 8" PRECAST U-LINTEL | 8" PRESTRESSED U-LINTEL



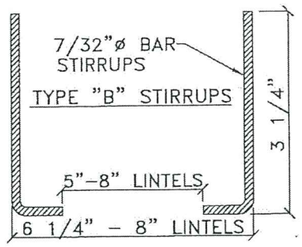
PRECAST LINTEL REINFORCING CAGE FABRICATION MODEL



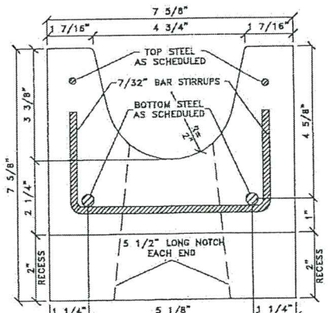
PRESTRESSED LINTEL REINFORCING CAGE FABRICATION MODEL



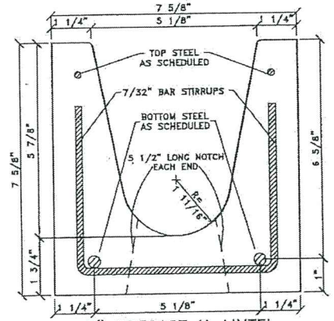
STIRRUP DETAILS
SCALE: N.T.S.



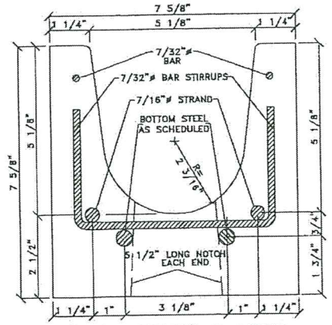
STIRRUP DETAILS
SCALE: N.T.S.



8" PRECAST W/ 2" RECESS DOOR U-LINTEL
SCALE: N.T.S.



8" PRECAST U-LINTEL
SCALE: N.T.S.



8" PRESTRESSED U-LINTEL
SCALE: N.T.S.

8" PRESTRESSED U-LINTEL DATA TABLE

LINTEL SIZE	VOLUME CY	WEIGHT LBS	" A " STEEL TYPE	" A " STEEL LENGTH	" B " TOP STEEL TYPE	" B " TOP STEEL LENGTH	" C " SPACING	NO. OF STIRRUPS
14'-8"	0.134	542	7/32 BAR	1'-3"	7/32 BAR	14'-6"	4 @ 3'-0"	9
15'-4"	0.140	567	7/32 BAR	1'-3"	7/32 BAR	15'-2"	4 @ 3'-2"	9
17'-4"	0.158	639	7/32 BAR	1'-3"	7/32 BAR	17'-2"	4 @ 3'-8"	9
19'-4"	0.176	713	7/32 BAR	1'-3"	7/32 BAR	19'-2"	4 @ 4'-2"	9
21'-4"	0.194	787	7/32 BAR #4 REBAR MIDSPAN	1'-3" 15'-0"	7/32 BAR	21'-2"	5 @ 3'-8 3/4"	10
22'-0"	0.200	811	7/32 BAR #4 REBAR MIDSPAN	1'-3" 16'-0"	7/32 BAR	21'-10"	5 @ 3'-10 3/8"	10
24'-0"	0.219	885	7/32 BAR #4 REBAR MIDSPAN	1'-3" 18'-0"	7/32 BAR	23'-10"	5 @ 4'-3 1/8"	10

8" PRECAST W/ 2" RECESS DOOR U-LINTEL DATA TABLE

LINTEL SIZE	OVERALL LENGTH	RECESS LENGTH	VOLUME CY	WEIGHT LBS	" A " BOTTOM STEEL TYPE	" A " BOTTOM STEEL LENGTH	" B " TOP STEEL TYPE	" B " TOP STEEL LENGTH	"C" SPACING	NO. OF STRUTS/ STIRRUPS
2'-8"	4'-4"	3'-0"	0.036	146	#3 REBAR	3'-10" PLUS HOOK	7/32 BAR	3'-8"	N/A	4
3'-0"	4'-6"	3'-4"	0.037	150	#3 REBAR	4'-0" PLUS HOOK	7/32 BAR	3'-10"	N/A	4
4'-0"	5'-8"	4'-4"	0.047	190	#3 REBAR	5'-0" PLUS HOOK	7/32 BAR	5'-0"	N/A	6
4'-4"	5'-10"	4'-8"	0.047	190	#3 REBAR	5'-2" PLUS HOOK	7/32 BAR	5'-2"	N/A	6
5'-0"	6'-8"	5'-4"	0.054	219	#4 REBAR	5'-4" PLUS HOOK	#3 REBAR	6'-0"	N/A	6
6'-0"	7'-6"	6'-4"	0.060	243	#4 REBAR	6'-4" PLUS HOOK	#3 REBAR	6'-10"	N/A	6
8'-0"	9'-8"	8'-4"	0.078	316	#4 REBAR	8'-4" PLUS HOOK	#3 REBAR	9'-0"	2'-0"	8

8" PRECAST U-LINTEL DATA TABLE

LINTEL SIZE	VOLUME CY	WEIGHT LBS	" A " BOTTOM STEEL TYPE	" A " BOTTOM STEEL LENGTH	" B " TOP STEEL TYPE	" B " TOP STEEL LENGTH	"C" SPACING	NO. OF STRUTS/ STIRRUPS
3'-6"	0.032	126	#3 REBAR	3'-5"	7/32 BAR	3'-2"	N/A	4
4'-0"	0.037	144	#3 REBAR	3'-11"	7/32 BAR	3'-8"	N/A	4
4'-6"	0.041	162	#3 REBAR	4'-5"	7/32 BAR	4'-2"	N/A	4
5'-4"	0.049	192	#3 REBAR	5'-3"	7/32 BAR	5'-0"	N/A	6
5'-10"	0.054	210	#3 REBAR	5'-9"	7/32 BAR	5'-6"	N/A	6
6'-6"	0.060	234	#4 REBAR	6'-5"	7/32 BAR	6'-2"	N/A	6
7'-6"	0.069	270	#4 REBAR	7'-5"	7/32 BAR	7'-2"	N/A	6
9'-4"	0.086	336	#4 REBAR	9'-3"	#3 REBAR	9'-0"	2'-0"	8
10'-6"	0.096	378	#4 REBAR	10'-5"	#3 REBAR	10'-2"	2'-0"	8
11'-4"	0.104	408	#5 REBAR	11'-3"	#3 REBAR	11'-0"	3'-0"	8
12'-0"	0.110	432	#5 REBAR	11'-11"	#3 REBAR	11'-8"	3'-0"	8
13'-4"	0.122	480	#5 REBAR	13'-3"	#3 REBAR	13'-0"	3'-0"	8
14'-0"	0.129	504	#5 REBAR	13'-11"	#3 REBAR	13'-8"	3'-0"	8

GENERAL NOTES:

1. REINFORCING STEEL: 7/32 BAR ASTM A510; #3, #4, AND #5 REBAR GRADE 60 ASTM A615; 7/16 INCH DIAMETER 270KSI LOW-RELAXATION PRESTRESSING SRTAND ASTM A416.
2. CONCRETE STRENGTHS: f'c = 6000 PSI FOR 8" PRESTRESSED LINTELS; f'c = 4000 PSI FOR ALL PRECAST 8 INCH LINTELS.
3. INITIAL PRESTRESS FORCE = 0.70 x fpu.
4. STRIPPING STRENGTH: PRESTRESSED LINTELS = 3500 PSI.
5. NOTE: THE TOTAL NUMBER OF STIRRUPS REQUIRED IS COUNTED STARTING FROM EACH END OF THE REINFORCING CAGE; FOR EXAMPLE - NO. OF STIRRUPS = 4, THEN THE REINFORCING CAGE MODEL SHOWS 2 STIRRUPS FROM EACH END.
6. 2.5 POUNDS OF POLYPROPYLENE FIBERS PER CU.YD. OF CONCRETE MAY BE USED IN LIEU OF PRESTRESSED LINTEL REINFORCING CAGE TO RESIST LIFTING AND HANDLING FORCES.

PRODUCT REVISED
as complying with the Florida
Building Code
Acceptance No 19-0130.13
Expiration Date 05/21/2022
By Hedy A. Mesa
Miami Dade Product Control

CRAIG PARRINO
6324 CR 579
SEFFNER, FL 33584
FL. P.E. LIC. NO. 44756

CRAIG PARRINO No. 44756 STATE OF FLORIDA PROFESSIONAL ENGINEER 1-22-19

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NO.	BY	REVISION	DATE
1	CP	GENERAL REVISION NUMBER ONE	11/08/96
2	TO	GENERAL REVISION NUMBER TWO	1/28/03
3	CP	NAME CHANGE & SYNTHETIC FIBER OPTION	2/17/06
4	CP	GENERAL REVISION	6/12/14
5			

MEMBER OF NATIONAL CONCRETE MASONRY ASSOCIATION

CAST-CRETE® USA, L.L.C.
HIGH STRENGTH PRECAST AND PRESTRESSED CONCRETE LINTELS AND SILLS
6324 County Road 579
Seffner, Florida 33584
Phone: 813-621-4641

TITLE: CAST-CRETE 8" LINTEL FABRICATION DETAILS

FILE NAME: DBSUB1-14	DRAWN BY: CRAIG PARRINO, P.E. - REG. NO. 44756	SCALE: NOT TO SCALE
DATE: 06/07/96	CHECKED BY: CRAIG PARRINO, P.E. - REG. NO. 44756	DRAWING No. FDB SHEET 1 OF 3

SAFE GRAVITY LOADS FOR 8" PRECAST & PRESTRESSED U-LINTELS

CAST-CRETE	SAFE LOAD - POUNDS PER LINEAR FOOT							
TYPE / LENGTH	8U8	8F8–0B	8F12–0B	8F16–0B	8F20–0B	8F24–0B	8F28–0B	8F32–0B
		8F8–1B	8F12–1B	8F16–1B	8F20–1B	8F24–1B	8F28–1B	8F32–1B
3'–6" (42") PRECAST	2231	3069	3719	5163	6607	8054	9502	10951
		3069	4605	6113	7547	8974	10394	11809
4'–0" (48") PRECAST	1966	2561	2751	3820	4890	5961	7034	8107
		2693	4605	6113	7547	8974	10394	11809
4'–6" (54") PRECAST	1599	1969	2110	2931	3753	4576	5400	6224
		2189	4375	6113	7547 (7)	8672	10294	11809
5'–4" (64") PRECAST	1217	1349	1438	1999	2560	3123	3686	4249
		1663	3090	5365	7547 (36)	7342 (19)	8733 (19)	10127 (19)
5'–10" (70") PRECAST	1062	1105	1173	1631	2090	2549	3009	3470
		1451	2622	4360	7168 (45)	6036 (19)	7181 (19)	8328 (20)
6'–6" (78") PRECAST	908	1238	2177	3480	3031	3707	4383	5061
		1238	2177	3480	5381	8360	10394 (37)	8825 (14)
7'–6" (90") PRECAST	743	1011	1729	2632	2205	2698	3191	3685
		1011	1729	2661	3898	5681	8467 (44)	6472 (15)
9'–4" (112") PRECAST	554	699	1160	1625	2564	3486	2818	3302
		752	1245	1843	2564	3486	4705 (37)	6390 (47)
10'–6" (126") PRECAST	475	535	890	1247	2093	2777	2163	2536
		643	1052	1533	2093	2781	3643 (38)	4754 (45)
11'–4" (136") PRECAST	362	582	945	1366	1846	2423	3127	4006
		582	945	1366	1846	2423	3127	4006
12'–0" (144") PRECAST	337	540	873	1254	1684	2193	2805	3552
		540	873	1254	1684	2193	2805	3552
13'–4" (160") PRECAST	296	471	755	1075	1428	1838	2316	2883
		471	755	1075	1428	1838	2316	2883
14'–0" (168") PRECAST	279	424	706	1002	1326	1697	2127	2630
		442	706	1002	1326	1697	2127	2630
14'–8" (176") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		458	783	1370	1902	2245	2517	2712
15'–4" (184") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		412	710	1250	1733	2058	2320	2513
17'–4" (208") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		300	548	950	1326	1609	1849	2047
19'–4" (232") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		235	420	750	1037	1282	1515	1716
21'–4" (256") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		180	340	598	845	1114	1359	1468
22'–0" (264") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		165	315	550	784	1047	1285	1399
24'–0" (288") PRESTRESSED	N.R.	NR	NR	NR	NR	NR	NR	NR
		129	250	450	654	884	1092	1222

(#) THE NUMBERS IN PARENTHESIS ARE PERCENT REDUCTIONS FOR GRADE 40 FIELD ADDED REBAR. SEE GENERAL NOTE NO. 4 ON SHEET 3 OF 3.

SAFE LATERAL LOADS FOR 8" PRECAST & PRESTRESSED U-LINTELS

CAST-CRETE	SAFE LOAD PLF		
TYPE / LENGTH	8U8	8F8	RCMU *
3'–6" (42") PRECAST	1025	1024	1598
4'–0" (48") PRECAST	765	763	1309
4'–6" (54") PRECAST	592	591	1073
5'–4" (64") PRECAST	411	411	745
5'–10" (70") PRECAST	340	339	616
6'–6" (78") PRECAST	507	721	490
7'–6" (90") PRECAST	424	534	363
9'–4" (112") PRECAST	326	512	230
10'–6" (126") PRECAST	284	401	180
11'–4" (136") PRECAST	260	452	154
12'–0" (144") PRECAST	244	402	137
13'–4" (160") PRECAST	217	324	110
14'–0" (168") PRECAST	205	293	100
14'–8" (176") PRESTRESSED	N.R.	284	91
15'–4" (184") PRESTRESSED	N.R.	259	83
17'–4" (208") PRESTRESSED	N.R.	194	64
19'–4" (232") PRESTRESSED	N.R.	148	52
21'–4" (256") PRESTRESSED	N.R.	125	42
22'–0" (264") PRESTRESSED	N.R.	116	40
24'–0" (288") PRESTRESSED	N.R.	91	33

* SEE GENERAL NOTE NO. 18 ON SHEET 3 OF 3.

SAFE UPLIFT LOADS FOR 8" PRECAST & PRESTRESSED U-LINTELS

CAST-CRETE	SAFE LOAD - POUNDS PER LINEAR FOOT						
TYPE / LENGTH	8F8–1T	8F12–1T	8F16–1T	8F20–1T	8F24–1T	8F28–1T	8F32–1T
	8F8–2T	8F12–2T	8F16–2T	8F20–2T	8F24–2T	8F28–2T	8F32–2T
3'–6" (42") PRECAST	1569	2655	3524	4394	5263	6132	7001
	1569	2655	3524	4394	5263	6132	7001
4'–0" (48") PRECAST	1363	2305	3060	3815	4570	5325	6079
	1363	2305	3060	3815	4570	5325	6079
4'–6" (54") PRECAST	1207	2040	2707	3375	4043	4711	5379
	1207	2040	2707	3375	4043	4711	5379
5'–4" (64") PRECAST	1016	1715 (11)	2276 (4)	2838	3399	3961	4522
	1016	1715	2276	2838	3399	3961	4522
5'–10" (70") PRECAST	909	1567 (18)	2080 (13)	2593 (9)	3107 (6)	3620 (4)	4133 (3)
	929	1567	2080	2593	3107	3620	4133
6'–6" (78") PRECAST	835 (12)	1407 (26)	1868 (21)	2329 (18)	2790 (16)	3251 (14)	3712 (12)
	835	1407	1868	2329	2790	3251	3712
7'–6" (90") PRECAST	727 (23)	1065 (26)	1624 (31)	2025 (28)	2426 (26)	2827 (25)	3228 (24)
	727	1224	1624	2025	2426	2827	3228
9'–4" (112") PRECAST	591	708 (25)	1136 (34)	1474 (34)	1815 (34)	2157 (34)	2500 (34)
	591	862	1318	1643	1969	2294	2619
10'–6" (126") PRECAST	530	575 (24)	916 (33)	1188 (33)	1461 (33)	1736 (33)	2011 (33)
	530	695	1180 (8)	1472 (4)	1763 (2)	2055	2346
11'–4" (136") PRECAST	474	504 (23)	800 (32)	1037 (32)	1274 (32)	1513 (32)	1753 (32)
	494	607	1042 (10)	1372 (11)	1643 (8)	1915 (6)	2187 (5)
12'–0" (144") PRECAST	470 (9)	458 (23)	724 (31)	938 (32)	1153 (32)	1369 (32)	1585 (32)
	470	550	940 (10)	1302 (15)	1560 (13)	1818 (11)	2075 (10)
13'–4" (160") PRECAST	418 (15)	386 (22)	607 (30)	785 (30)	964 (30)	1143 (31)	1323 (31)
	428	460	780 (9)	1159 (21)	1418 (20)	1653 (19)	1887 (17)
14'–0" (168") PRECAST	384 (15)	358 (21)	560 (29)	724 (30)	889 (30)	1054 (30)	1220 (30)
	410	425	717 (9)	1063 (21)	1358 (23)	1582 (22)	1807 (21)
14'–8" (176") PRESTRESSED	239	334 (21)	520 (29)	672 (29)	825 (29)	978 (29)	1131 (30)
	246	395	663 (9)	980 (20)	1303 (26)	1518 (25)	1734 (24)
15'–4" (184") PRESTRESSED	224	313 (20)	486 (28)	627 (29)	769 (29)	911 (29)	1054 (29)
	230	368	616 (9)	908 (20)	1240 (28)	1460 (27)	1668 (26)
17'–4" (208") PRESTRESSED	187	263 (19)	405 (28)	521 (27)	638 (27)	756 (27)	873 (27)
	192	306	506 (8)	740 (19)	1005 (27)	1271 (31)	1473 (31)
19'–4" (232") PRESTRESSED	162	229 (17)	348 (25)	447 (25)	547 (25)	647 (25)	747 (25)
	166	263	429 (8)	623 (18)	841 (28)	1059 (30)	1227 (30)
21'–4" (256") PRESTRESSED	142	204 (16)	307 (23)	393 (23)	480 (23)	568 (24)	655 (24)
	142	232	373 (7)	537 (17)	721 (25)	905 (29)	1048 (29)
22'–0" (264") PRESTRESSED	137	197 (15)	295 (22)	379 (23)	462 (23)	546 (23)	630 (23)
	137	223	358 (7)	513 (17)	688 (24)	863 (28)	999 (28)
24'–0" (288") PRESTRESSED	124	179 (14)	267 (21)	342 (21)	416 (21)	490 (21)	567 (22)
	124	202	319 (7)	455 (16)	606 (22)	757 (27)	907 (27)

(#) THE NUMBERS IN PARENTHESIS ARE PERCENT REDUCTIONS FOR GRADE 40 FIELD ADDED REBAR. SEE GENERAL NOTE NO. 4 ON SHEET 3 OF 3.

PRODUCT RENEWED
as complying with the Florida Building Code
Acceptance No 22-0125.04
Expiration Date 05/21/2027
By Miami Dade Product Control

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No 20-0630.16
Expiration Date 05/21/2022
By Miami Dade Product Control

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No 19-0130.13
Expiration Date 05/21/2022
By Miami Dade Product Control

CRAIG PARRINO
6324 CR 579
SEFFNER, FL 33584
FL. P.E. LIC. NO. 44756

CRAIG PARRINO LICENSE No. 44756 STATE OF FLORIDA PROFESSIONAL ENGINEER
1-22-19

SEE GENERAL NOTES – SHEET 3 OF 3

NO.	BY	REVISION	DATE
1	CP	GENERAL REVISION NUMBER ONE	11/08/96
2	CP	RECESS TYPE DESIGNATION	12/30/96
3	CP	8U8 11'–4" THRU 14'–0"	11/08/96
4	CP	GENERAL REVISION	4/28/98
5	CP	GENERAL REVISION	3/24/03
6	CP	NAME CHANGE	2/17/06
7	CP	GENERAL REVISION	6/12/14

MEMBER OF CMA NATIONAL CONCRETE MASONRY ASSOCIATION

CAST-CRETE® USA, LLC.
HIGH STRENGTH PRECAST AND PRESTRESSED CONCRETE LINTELS AND SILLS
6324 County Road 579
Seffner, Florida 33584
Phone: 813-621-4641

TITLE: CAST–CRETE 8" LINTEL FABRICATION DETAILS

FILE NAME: DBSUB2-14	DRAWN BY: CRAIG PARRINO, P.E. – REG. NO. 44756	SCALE: NOT TO SCALE
DATE: 06/07/96	CHECKED BY: CRAIG PARRINO, P.E. – REG. NO. 44756	DRAWING No. FD8 SHEET 2 OF 3

SAFE GRAVITY LOADS FOR 8" PRECAST w/ 2" RECESS DOOR U-LINTELS

CAST-CRETE		SAFE LOAD - POUNDS PER LINEAR FOOT						
LENGTH \ TYPE	8RU6	8RF6-0B	8RF10-0B	8RF14-0B	8RF18-0B	8RF22-0B	8RF26-0B	8RF30-0B
		8RF6-1B	8RF10-1B	8RF14-1B	8RF18-1B	8RF22-1B	8RF26-1B	8RF30-1B
4'-4" (52") PRECAST	1635	1749	3355	3280	4349	5421	6493	7567
		1891	3699	5206	6639	8060	9479	10893
4'-6" (54") PRECAST	1494	1596	3063	2992	3968	4946	5924	6904
		1756	3699	5206	6639	8060	9479	10893
5'-8" (68") PRECAST	866	920	1770	1716	2277	2839	3402	3966
		1167	2481	4567	6389	8060 (34)	7917 (19)	9311 (19)
5'-10" (70") PRECAST	810	859	1653	1600	2124	2649	3174	3700
		1113	2342	4242	6639 (10)	8060 (38)	7402 (19)	8706 (19)
6'-8" (80") PRECAST	797	901	1825	3120	5048	7747	9448	7360
		901	1825	3120	5048	7915	9479	10893 (32)
7'-6" (90") PRECAST	669	755	1490	2459	3776	5743	7239	5623
		755	1490	2459	3776	5743	8998 (19)	10893 (48)
9'-8" (116") PRECAST	411	466	999	1568	2253	3129	4091	3146
		526	999	1568	2253	3129	4150	5891 (47)

(#) THE NUMBERS IN PARENTHESIS ARE PERCENT REDUCTIONS FOR GRADE 40 FIELD ADDED REBAR. SEE GENERAL NOTE NO. 4

SAFE LATERAL LOADS FOR 8" PRECAST w/ 2" RECESS DOOR U-LINTELS

CAST-CRETE	SAFE LOAD PLF		
LENGTH \ TYPE	8RU6	8RF6	RCMU *
4'-4" (52") PRECAST	758	757	1164
4'-6" (54") PRECAST	694	693	1073
5'-8" (68") PRECAST	408	407	655
5'-10" (70") PRECAST	382	381	616
6'-8" (80") PRECAST	595	788	464
7'-6" (90") PRECAST	509	674	363
9'-8" (116") PRECAST	370	490	214

* SEE GENERAL NOTE 18

SAFE UPLIFT LOADS FOR 8" PRECAST w/ 2" RECESS DOOR U-LINTELS

CAST-CRETE	SAFE LOAD - POUNDS PER LINEAR FOOT						
LENGTH \ TYPE	8RF6-1T	8RF10-1T	8RF14-1T	8RF18-1T	8RF22-1T	8RF26-1T	8RF30-1T
	8RF6-2T	8RF10-2T	8RF14-2T	8RF18-2T	8RF22-2T	8RF26-2T	8RF30-2T
4'-4" (52") PRECAST	905	1668	2362	3056	3751	4445	5140
	905	1668	2362	3056	3751	4445	5140
4'-6" (54") PRECAST	867	1604	2272	2939	3607	4275	4943
	867	1604	2272	2939	3607	4275	4943
5'-8" (68") PRECAST	675	1269 (16)	1797	2326 (5)	2854 (2)	3382	3911
	675	1269	1797	2326	2854	3382	3911
5'-10" (70") PRECAST	655	1207 (16)	1746 (11)	2259 (7)	2773 (5)	3286 (3)	2799 (1)
	655	1233	1746	2259	2773	3286	2799
6'-8" (80") PRECAST	570	929 (16)	1530 (22)	1980 (19)	2429 (16)	2879 (15)	3329 (14)
	570	1080	1530	1980	2429	2879	3329
7'-6" (90") PRECAST	506	742 (16)	1364 (30)	1765 (27)	2166 (25)	2567 (24)	2968 (23)
	506	898	1364	1765	2166	2567	2968
9'-8" (116") PRECAST	395	468 (15)	884 (32)	1222 (33)	1540 (34)	1859 (34)	2180 (34)
	395	560	1072	1386	1701	2016	2331

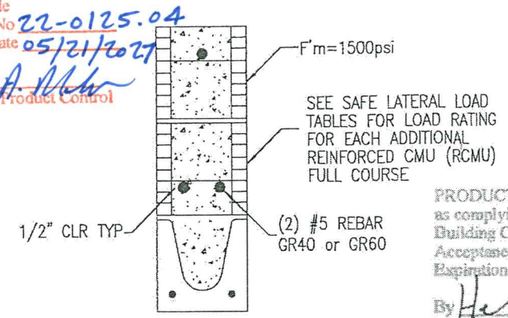
(#) THE NUMBERS IN PARENTHESIS ARE PERCENT REDUCTIONS FOR GRADE 40 FIELD ADDED REBAR. SEE GENERAL NOTE NO. 4

GENERAL NOTES

1) Materials:
   - f'c 8" precast lintel = 4000 psi
   - f'c prestressed lintel = 6000 psi
   - f'c grout = 3000 psi with maximum 3/8 inch aggregate
   - concrete masonry units (CMU) per ASTM C90 with minimum net area compressive strength = 1900 psi
   - rebar per ASTM A615 Grade 60
   - prestressing strand per ASTM A416 Grade 270 low relaxation
   - mortar per ASTM C270 Type M or S
2) All safe load tables based on minimum 4 inch nominal bearing.
   Exception: Safe loads for lintel type 8U8 and 8RU6 must be reduced by 20% if bearing length is less than 6 1/2 inches.
3) Provide full mortar bed and head joints.
4) The number in parenthesis indicates the percent reduction for grade 40 field added rebar.
   Example: 7'-6" Lintel 8F32-1B safe gravity load = 6472 (15) w/ 15% reduction → 6472 (.85) = 5501 plf
5) All lintels meet or exceed L/360 deflection.
   Exception: Prestress lintel type 8F8 17'-4" and longer meets or exceeds L/180 deflection. Use is limited to conditions meeting the Florida Building Code Sections 1604.3.1 as applicable
6) Shore filled lintels as required.
7) One #7 rebar may be substituted for two #5 rebars.
8) Installation/attachment of lintel must comply with the architectural and/or structural documents.
9) The designer may evaluate concentrated loads from the safe load tables by calculating the maximum resisting moment and shear at d-away from the face of support.
10) N.R. = Not Rated
11) Bottom field added rebar to be located at the bottom of the lintel cavity.
12) Lintels are manufactured with 5-1/2" long notches at the ends to accomodate vertical cell reinforcing and grouting.
13) The exterior surface of lintels installed in exterior concrete masonry walls shall have a coating of stucco applied in accordance with ASTM C-926 or other approved coating.
14) Lintels loaded simultaneously with vertical (gravity or uplift) and horizontal (lateral) loads should be checked for the combined loading with the following equation:

$$\frac{\text{Applied vertical load}}{\text{Safe vertical load}} + \frac{\text{Applied horizontal load}}{\text{Safe horizontal load}} \leq 1.0$$

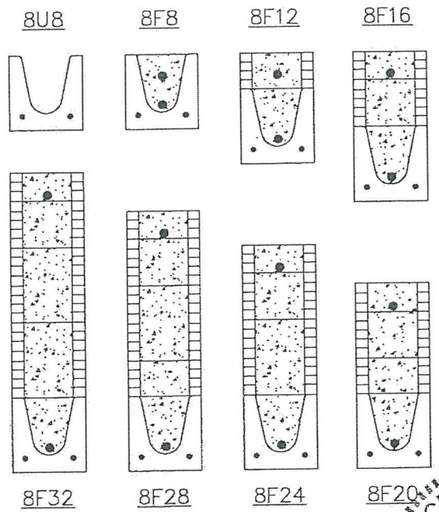
15) For composite lintel heights not shown, use safe load from next lower height shown.
16) For lintel lengths not shown, use safe load from next longest length shown.
17) Safe loads are superimposed allowable loads.
18) Safe loads based on rational design analyis per ACI 530 and ACI 318
19) Additional lateral load capacity can be obtained by the designer by providing additional reinforced concrete masonry units (RCMU) above the lintel. See sketch below:



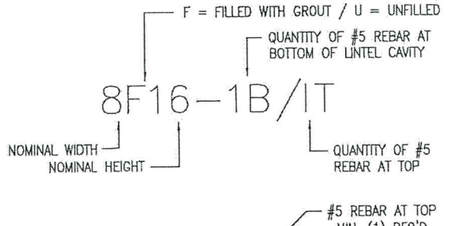
PRODUCT RENEWED
as complying with the Florida Building Code
Acceptance No 22-0125.04
Expiration Date 05/21/2027
By Miami Dade Product Control

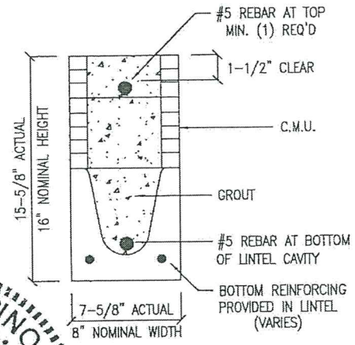
PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No 20-0630.16
Expiration Date 05/21/2022
By Miami Dade Product Control

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No 19-0130.13
Expiration Date 05/21/2022
By Miami Dade Product Control



TYPE DESIGNATION





CRAIG PARRINO
LICENSE
No. 44756
STATE OF FLORIDA
PROFESSIONAL ENGINEER
1-22-19

CRAIG PARRINO
6324 CR 579
SEFFNER, FL 33584
FL. P.E. LIC. NO. 44756

NO.	BY	REVISION	DATE
1	CP	GENERAL REVISION NUMBER ONE	11/08/96
2	CP	RECESS TYPE DESIGNATION	12/30/96
3	CP	8U8 11'-4" THRU 14'-0"	11/08/96
4	CP	GENERAL REVISION	4/28/98
5	CP	GENERAL REVISION	3/24/03
6	CP	NAME CHANGE	2/17/06
7	CP	GENERAL REVISION	6/12/14

MEMBER OF NATIONAL CONCRETE MASONRY ASSOCIATION

CAST-CRETE USA, LLC.
HIGH STRENGTH PRECAST AND PRESTRESSED CONCRETE LINTELS AND SILLS
6324 County Road 579
Seffner, Florida 33584
Phone: 813-621-4641

TITLE: CAST-CRETE 8" LINTEL FABRICATION DETAILS

FILE NAME: DBSUB3-14	DRAWN BY: CRAIG PARRINO, P.E. - REG. NO. 44756	SCALE: NOT TO SCALE	
DATE: 06/07/96	CHECKED BY: CRAIG PARRINO, P.E. - REG. NO. 44756	DRAWING No. FD8	SHEET 3 OF 3

VIPIN N. TOLAT, P.E.
15123 Lantern Creek Lane,
Houston, TX. 77068-3831
TEL: 281-444-9183
Fax: 281-444-9184
Email: vtolat@sbcglobal.net

Engineering - Inspections & Product Approvals

ENGINEER'S EVALUATION REPORT # NU0413C dated 10/30/2020

CATEGORY: Structural Components
SUB CATEGORY: Metal Connectors

REPORT HOLDER:

NuVue Industries Inc;
1055 E. 29th Street,
Hialeah, FL. 33013
www.nu-vueindustries.com
nuvue@bellsouth.net
Phone: 305-694-0397
Fax: 305-694-0398

1.0 EVALUATION SCOPE:

Compliance with 2020 Florida Building Code- Building and Residential

2.0 PRODUCT DESCRIPTION:

Refer to tables 1 through 21 of this report for Product name, size, size and number of fasteners, fastening details shown in the diagrams and the allowable loads.

3.0 STRUCTURAL SPECIFICATIONS :

1. Steel shall conform to ASTM A653, SS grade 33, min. yield 33 ksi, min. tensile strength 45 ksi and min. galvanized coating of G 60 per ASTM A653.
2. Allowable loads and fasteners are based on NDS 2018.
3. Design loads are for S. Pine, specific gravity 0.55. Design loads for other species shall be adjusted per NDS 2018.
4. Allowable uplift loads have been adjusted for load duration factor CD of 1.6. Allowable gravity loads have been adjusted for CD values of 1.0, 1.15 and 1.25 per NDS 2018. Design loads do not include 33% increase for steel and concrete.
5. Concrete in Tie beams shall be min. of 2500 psi. Concrete Masonry, Grout and mortar in concrete masonry shall be min. of 1500 psi. Concrete masonry shall comply with ASTM C90.
6. Combined load of Uplift, L1 and L2 shall satisfy the following equation.

$$\frac{\text{Actual Uplift}}{\text{Allowable Uplift}} + \frac{\text{Actual L1}}{\text{Allowable L1}} + \frac{\text{Actual L2}}{\text{Allowable L2}} \leq 1.0$$

4.0 INSTALLATION

Installation shall be in accordance with this report and the latest edition of Nu-Vue Industries Catalog. The information in this report supercedes any conflicting information in the catalog.

5.0 EVIDENCE SUBMITTED:

Test reports submitted by Product testing Inc, (PT) Atec Associates Inc(Atec) and PSI Inc and signed and sealed calculations in conformance with FBC 2020 by Vipin N. Tolat, P.E. Tests conducted do conform to ASTM D 1761-2006, ASTM D7147-2011 and AISI S100-2016 and calculations for design loads conform to NDS 2018.

Product tested	**Test #/Test lab**	**Date Tested**
NVTA/NVTAS	02-3938/PT	8/6/02
NVTA/NVTAS	02-4073/PT	11/6/02
NVTA/NVTAS	02-4074/PT	11/6/02
NVTA/NVTAS	02-4075/PT	11/6/02
NVTA/NVTAS	31.22456.0002/ATEC	7/6/02
NVBH 24	02-4096/PT	12/3/02
NVUH 26	02-4095/PT	1/17/03
NVRT	03-4177/PT	2/3/03
NVRT	03-4202/PT	2/19/03
NVRT	03/4270,4271/PT	3/27/03
NVTH/NVTHS	04-4698/PT	4/15/04
NVSNP3	03-4482/PT	9/15/03
NV358	03/4543/PT	12/19/03
NV458	03-4590/PT	12/31/03
NVHCL/R	03-4625/PT	1/21/04
NVSTA/NVHTA/NVSTA-H/NVHTA-H	04-4641,4642/PT	3/17&3/22/04
NVJH24,26,28	03-4385,86,87/PT	5/30/03
NVSO236	03-4349,57,58/PT	5/13,5/19,5/20/03
NVTP4	03-4303,44/PT	4/21, 5/1/03
NVTP4H	03-4345,43/PT	5/2, 5/5/03
NVHC43,43/2	70-02-94-00381/ATEC	11/27/95
AB5	05-5195,95A/PT	2/15/08
AB7	05-5196,96A/PT	2/15/08
NVHC37	03697.0001/ATEC	11/27/96
IKE1	05-5612/PT	3/20/06

IKE2	06-5622/PT	5/1/06
NVTHJ26	04-4995/PT &138-96013-01/PSI	1/31/05&2/7/89
NVTHJ28	04-4996/PT& 138-96013-05/PSI	1/31/05&12/2/89
NVHGA10	11041/FTL	7/6/2020
NVWS	12272/FTL	8/26/2020
NVTT	03-4631/PT	6/21/2004
NVTT	04-4908/PT	7/21/2004

6.0 DESIGN:

1 Maximum allowable loads shall not exceed the allowable loads listed in this report. Allowable loads are based on allowable stress design per NDS.

2. Capacity of wood members is not covered by this report. Allowable loads shall not exceed the capacity of wood members. Capacity of wood members shall be checked by Engineer/Architect of record.

3. Wood members with which the connectors are used must be nominal dimension lumber or approved structural composite lumber.

7.0 CODITIONS OF USE:

1. NuVue Industries metal structural connectors described in this report comply with or are suitable alternative to what is specified in section 1.0 of this report.
2. Design loads must be less than the allowable loads shown in all the tables of this report.
3. The connectors must be manufactured, identified and installed in accordance with this report and the manufacturer's instructions.
4. Products covered by this report are manufactured by NuVue industries Inc in Hialeah, Florida under a quality control program with inspections by NAMI Inc having State of Florida license # QUA 1789.

Vipin N. Tolat, P.E.
Florida P.E. # 12847
10/30/2020

vnt/nu0413C

VIPIN N. TOLAT
LICENSE
No. 12847
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

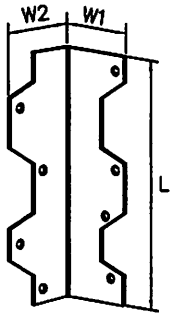
18 Gauge Angle Clips.

Table 1

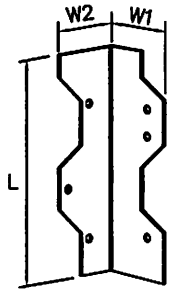
Product Code	Dimensions (inches)			Fastener Schedule		Allowable Loads (lbs)	
	W1	W2	L	Header	Joist	F1	F2
AB5	1½"	2 5/16"	5	3-10dx1½"	3-10dx1½"	511 .	595
AB7	1½"	2 5/16"	7	4-10dx1½"	4-10dx1½"	582	794

Notes: Nail wider angle leg to Joist and Shorter leg to Header.

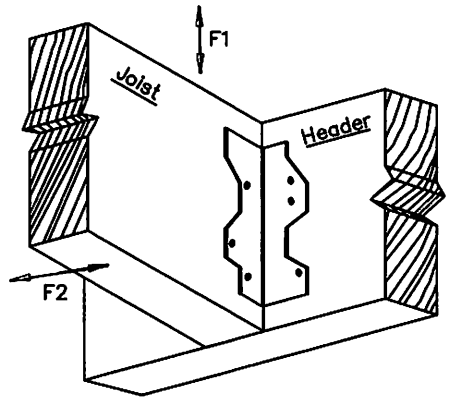
CD = 1.6 for F1 & F2.



AB-7



AB-5



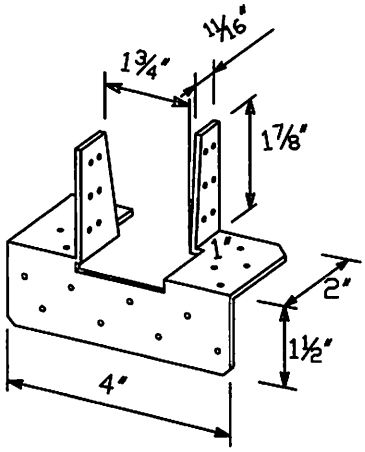
Typical Installation

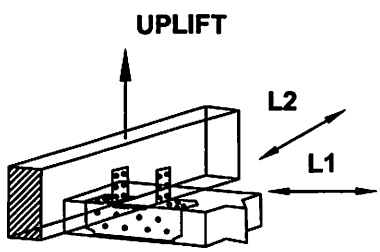
18 Gauge NVHC 37 5WAY Grip Clips (520)

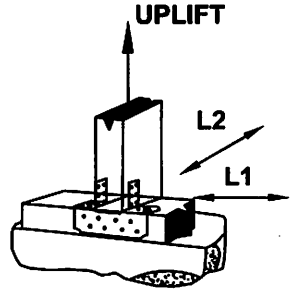
Table 2

Product Code	Description	Nail Schedule		Allowable Design Loads (lbs)		
		Header or Plate	Joist or Stud	Uplift	L1	L2
NVHC 37	5 Way Clip	16-8d or 16-10d	12-8d or 12-10d	702	560	637

CD = 1.6 for F1 & F2.





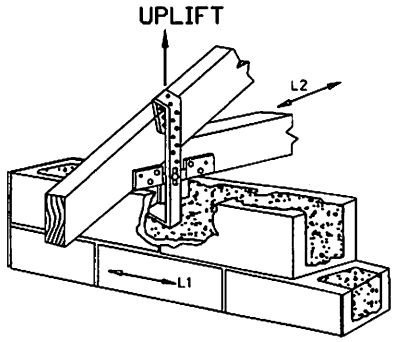


Deep Seat Truss Anchor. They are designed to resist lateral and uplift forces. The strap is made of 14 gauge steel and the seats of 20 gauge steel.

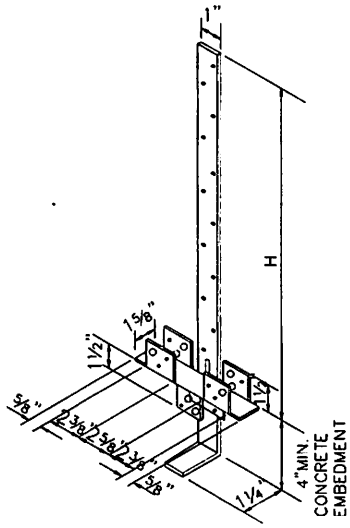
Table 3

Assembly Product Code	Dimension H (inches)	Total No. of Fasteners in Strap 10d x 1½"	Total No. of Fasteners in 20 GA. Seat 10d x 1½"	Allowable Loads (lbs) F'c = 2500 psi		
				Uplift	L1	L2
NVSTA12	12	5	6	1046	700	1049
NVSTA16	16	6	6	1141	760	1144
NVSTA20	20	7	6	1236	823	1239
NVSTA22	22	8	6	1331	887	1335
NVSTA24	24	9	6	1426	950	1430

CD = 1.6 for L1 & L2.



NVSTA

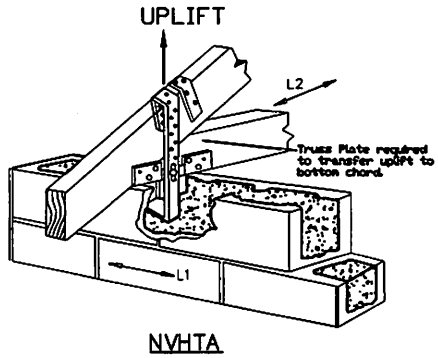


Holden Double Strap Riveted Truss Anchor. They are designed of 14 gauge steel plates to resist lateral and uplift forces. The seats are made of 20 gauge steel.

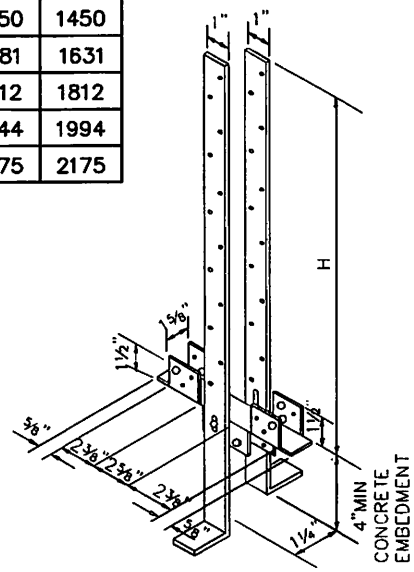
Table 4

Assembly Product Code	Dimension H (inches)	Total No. of Fasteners in two Straps 10d x 1½"	Total No. of Fasteners in 20 GA. Seat 10d x 1½"	Allowable Loads (lbs) F'c = 2500 psi (unless otherwise noted)			
				Uplift	Uplift F'c = 3000 psi	L1	L2
NVHTA12	12	10	6	1506	1766	1050	1450
NVHTA16	16	12	6	1695	1987	1181	1631
NVHTA20	20	14	6	1883	2208	1312	1812
NVHTA22	22	16	6	2071	2429	1444	1994
NVHTA24	24	18	6	2259	2649	1575	2175

CD = 1.6 for L1 & L2.



NVHTA

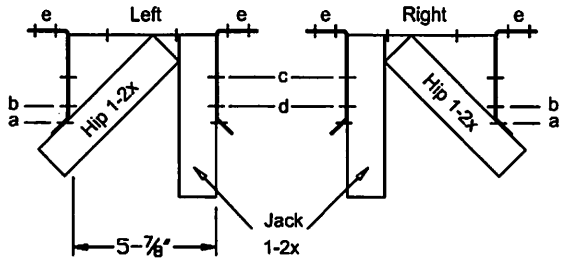


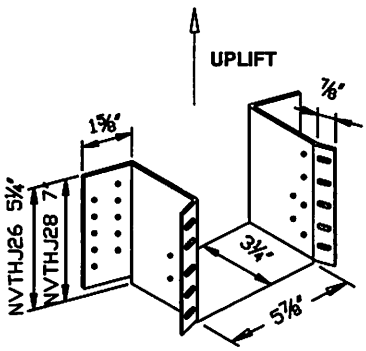
12 Gauge NVTHJ Truss Hip & Jack Hanger

Table 5

Product Code	Allowable Loads (lbs) S. Pine				Nail Schedule						
	Uplift Loads 160%	Gravity			Header Nails 16d (e)	Hip Nail 10d			Jack Nail 10d		
		100%	115%	125%		a	b	total	c	d	total
NVTHJ26	1478	2444	2444	2444	16	4	3	7	2	3	5
NVTHJ28	1931	3229	3333	3333	20	5	4	9	2	3	5

Note: For 1–2x members 10dx1½" nails can be used





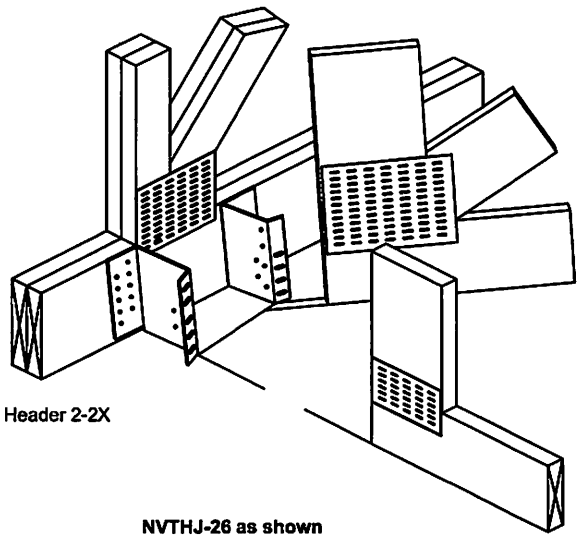
NVTHJ-28 as shown
NVTHJ-26 similar

20G Stud Plate Ties

Table 6

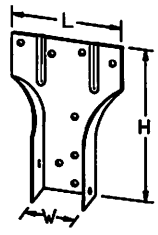
Product Code	Dimensions (inches)			Fasteners		Allowable Loads (lbs)		
	W	H	L	Stud	Plate	Uplift	L1	L2
IKE 1	1½	5	3½	6–10d	4–10d	787	337	337
IKE 2	1½	6⅝	3½	6–10d	7–10d	932	451	318

CD = 1.6 for L1 & L2.

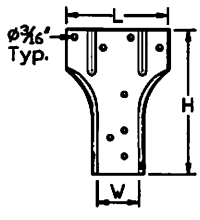


NVTHJ-26 as shown
NVTHJ-28 similar

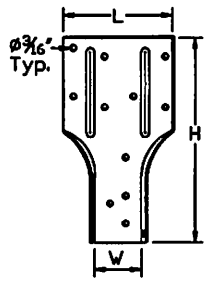
U.S. Patent No. 4,964,253



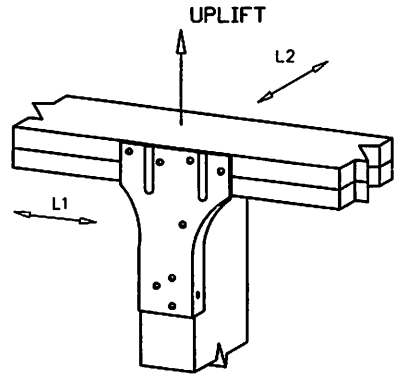
IKE-1
(ISOMETRIC)



IKE-1



IKE-2



TYPICAL IKE1
INSTALATION

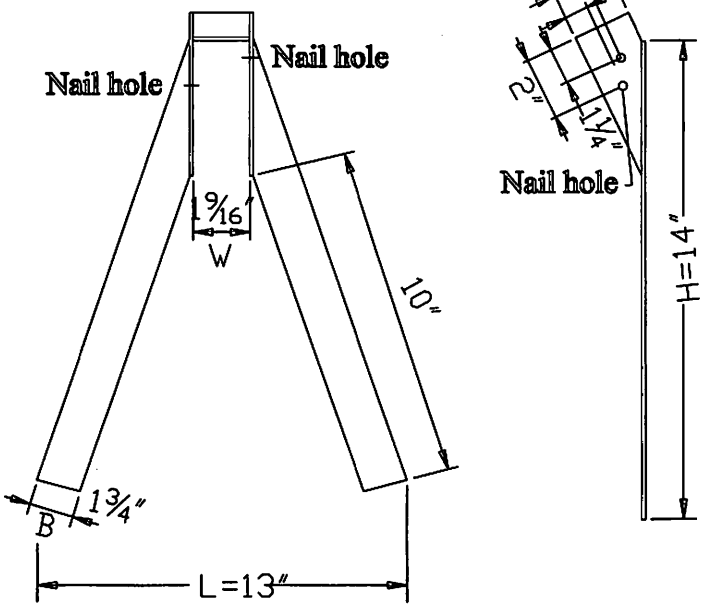
18 Gauge NVTT Sanibel Truss Strap

Table 7

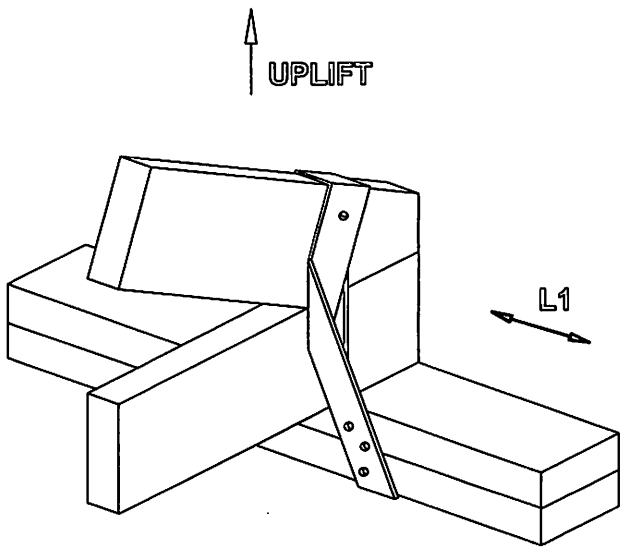
Product Code	Dimensions (inches)				Fasteners Schedule			Allowable Loads (lbs)	
	W	B	H	L	Truss	Top Plates	Hollow Concrete Masonry	Uplift	L1
NVTT–1	1 9/16	1 3/4	14	13	2–10dx1½"	6–10d	—	983	543
NVTT–2	1 9/16	1 3/4	14	13	2–10dx1½"	—	6–¼"øx1 ½" Tapcons	1584	465

1. 1–10dx1½" nail is placed on each side of the Truss and 3–10dnails in each leg are placed in two top plates.

2. 3–¼"dia. x 1½" long, 1¼" embedment tapcons are placed in each leg and into the hollow concrete masonry. Maintain 2½" edge distance from top of the block and spacing of 3" between the tapcons.



NVTT 1 and 2

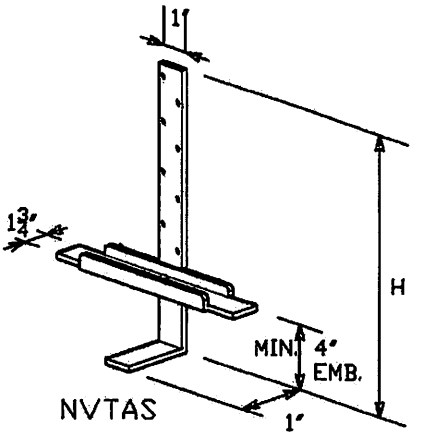


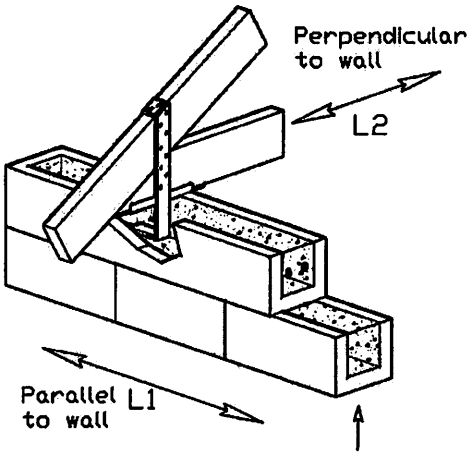
NVTT-1 As Shown
NVTT-2 Similar except concrete masonry with tapcon anchors instead of Wood Plates and nails.

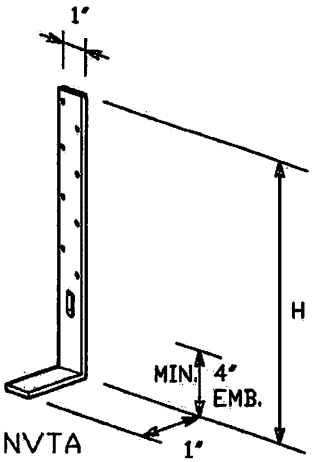
Table 8

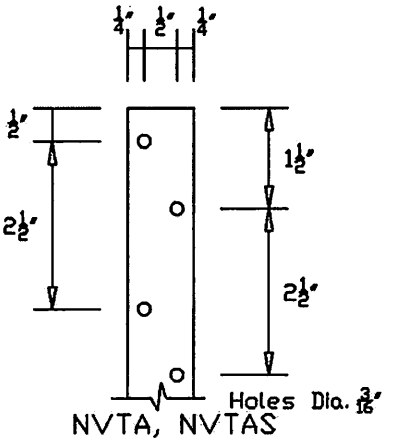
Truss Anchors NVTA and Riveted Truss Anchors with Seat NVTAS
14 G Straps, 20 G Seats

H Length (in)	Product Code	
16	NVTA-16	NVTAS 212
18	NVTA-18	NVTAS 214
20	NVTA-20	NVTAS 216
22	NVTA-22	NVTAS 218
24	NVTA-24	NVTAS 220
26	NVTA-26	NVTAS 222
28	NVTA-28	NVTAS 224
30	NVTA-30	NVTAS 226
36	NVTA-36	NVTAS 232
48	NVTA-48	NVTAS 244









No. of Fasteners each strap 10d	Maximum Allowable Load (lbs)			
	Uplift Single Strap	Uplift Double Straps	L1 Single & Double Straps	L2 Single & Double Straps
5	757	1514	250	500
6	805	1610	250	500
7	854	1708	250	500
8	902	*1804	250	500
9	951	*1902	250	500
10	999	*1998	250	500
11	1048	*2096	250	500
12	1096	*2192	250	500
13	1145	*2290	250	500
14	1193	*2290	250	500

*Note: For 8 or more nails per strap, use double truss for double straps.

No. of Fasteners each strap 10d x 1.5"	Maximum Allowable Load (lbs)			
	Uplift Single Strap	Uplift Double Straps	L1 Single & Double Straps	L2 Single & Double Straps
5	1032	2236	250	500
6	1127	2254	385	565
7	1136	2272	520	630
8	1144	*2288	520	630
9	1153	*2306	520	630
10	1161	*2322	520	630
11	1170	*2340	520	630
12	1178	*2356	520	630
13	1187	*2374	520	630

TABLE 9

NVBH 24 BUTTERFLY HANGER

SIZE	PRODUCT CODE	GAUGE	FASTENER SCHEDULE		ALLOWABLE LOADS (lbs.)	
			HEADER 8d	JOIST 8d	DOWNWARD GRAVITY LOADS C_D=1.0	WIND[2] UPLIFT LOAD C_D=1.6
2x4	NVBH24	18	12	6	1113	364

Notes:

1. Values are based on $1\frac{1}{2}$" header and Joist thickness.
2. Can only be used in Non-HVHZ.

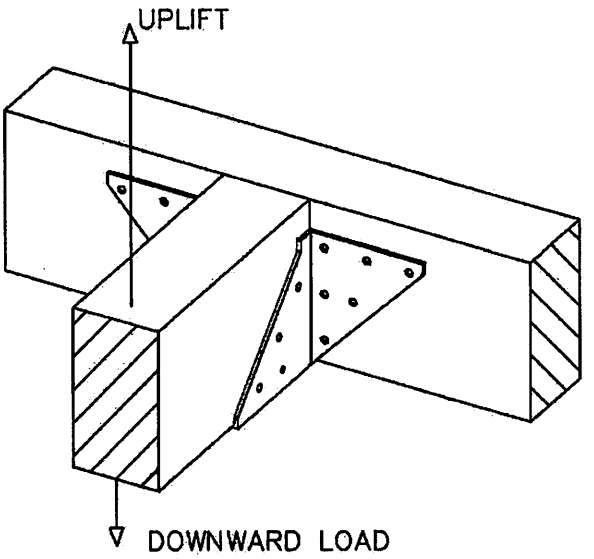
TABLE 10

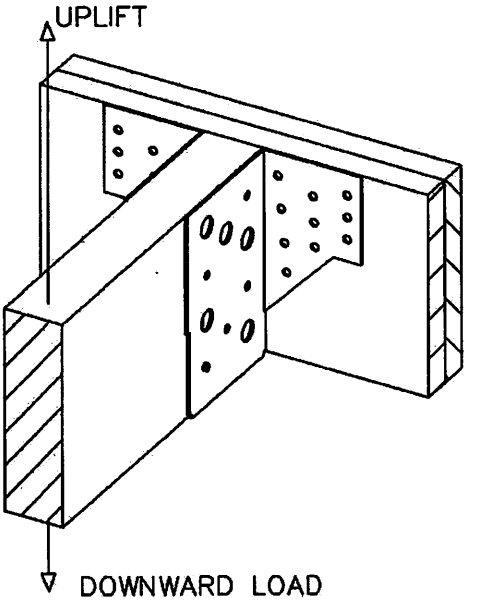
NVUH 26 JOIST HANGER

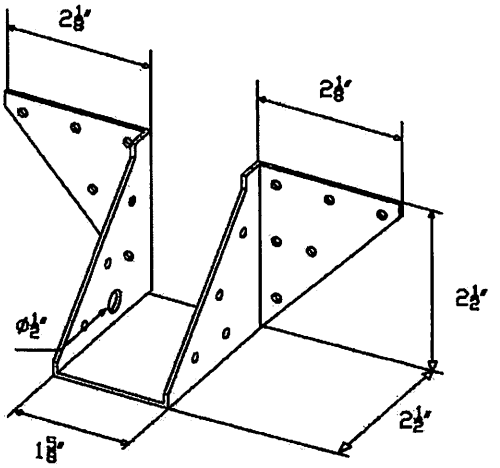
SIZE	PRODUCT CODE	GAUGE	FASTENER SCHEDULE		ALLOWABLE LOADS (lbs.)	
			HEADER 16d	JOIST 10d x $1\frac{1}{2}$"	DOWNWARD GRAVITY LOADS C_D=1.0	WIND UPLIFT LOAD C_D=1.6
2x6	NVUH26	14	20	10	2233	1213

Notes:

1. Values are based on 3" header thickness and $1\frac{1}{2}$" Joist thickness.







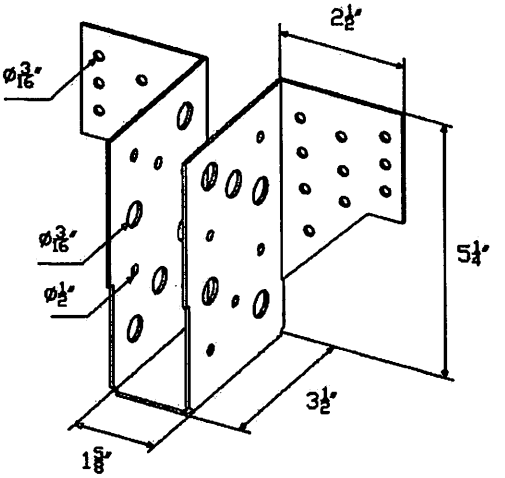


TABLE 11

NVRT Flat and Twisted Rafter Ties 1"X14 G

Length (in)	Product Code	Gauge
12	NVRT–12	14
16	NVRT–16	14
18	NVRT–18	14
20	NVRT–20	14
22	NVRT–22	14
24	NVRT–24	14
30	NVRT–30	14
36	NVRT–36	14
48	NVRT–48	14

NVRT Wood to Wood

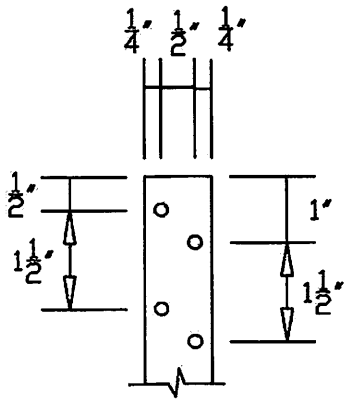
16d Fasteners or 10dx1½"		Maximum Uplift Load (lbs)	
TOTAL	In each member*	Flat Ties	Twisted Ties
6	3	588 [5]	588 [5]
8	4	725	724
10	5	861	860
12	6	998	996
14	7	1135	1132

NVRT Wood to Concrete

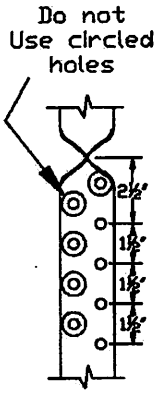
No. of 16d nails to Wood Framing	No. of ¼" diameter Tapcons to Concrete	Maximum Uplift Load (lbs)
3	3	588 [5]
4	4	722
5	4	856
6	5	991
7	5	1125

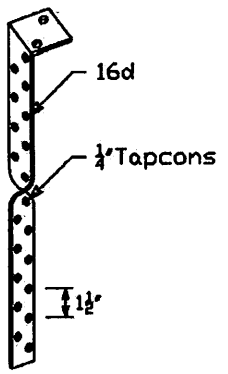
Notes:

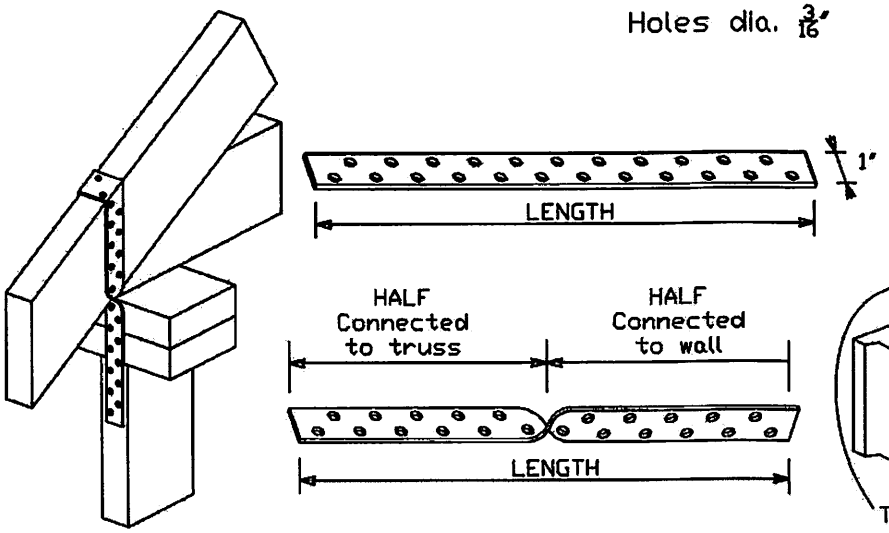
1. Specify "F" for Flat and "T" for Twisted when ordering.
2. Fastener values are based on a minimum 1½" thick wood members.
3. * Indicates no. of nails in each connected wood member.
4. ITW tapcons shall be embedded a minimum of 1¼" into the concrete tie beam formed with concrete filled masonry. ITW tapcons shall have a minimum edge distance of 2½" and minimum staggered spacing of ¾" as shown.
5. Use only in Non–HVHZ.
6. For Uplift loads C_D=1.6



NVRT Anchor
Holes dia. 3/16"







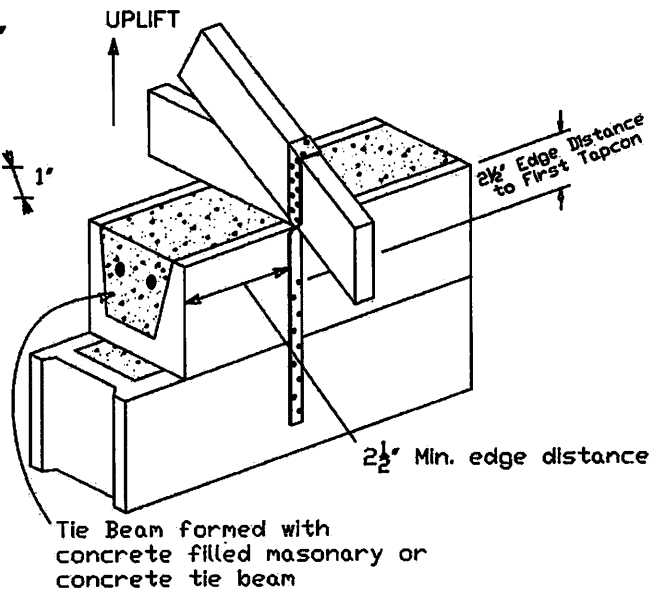


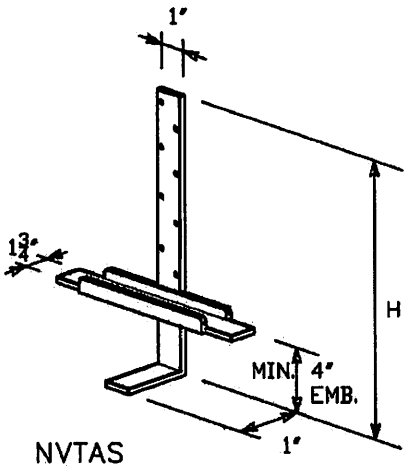
TABLE 12

Truss Anchors NVTH 1¼"x14G, Seat 18G

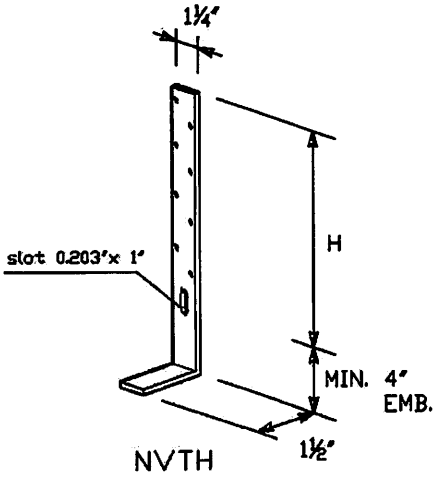
H Length (in)	Product Code	
12	NVTH-16	NVTHS-212
14	NVTH-18	NVTHS-214
16	NVTH-20	NVTHS-216
18	NVTH-22	NVTHS-218
20	NVTH-24	NVTHS-220
22	NVTH-26	NVTHS-222
24	NVTH-28	NVTHS-224
26	NVTH-30	NVTHS-226
32	NVTH-36	NVTHS-232
44	NVTH-48	NVTHS-244

No. of Fasteners in each Strap 10d x 1.5"	Maximum Uplift Loads (lbs)		Maximum Lateral Loads (lbs)			
	Single Strap on Single Truss	Double Straps on Double Truss	Single Strap		Double Straps	
			L1	L2	L1	L2
5	1032	2064	560	525	1120	1050
6	1222	2444	671	630	1342	1260
7	1275	2550	783	735	1566	1470
8	1329	2658	783	735	1566	1470
9	1383	2766	783	735	1566	1470
10	1437	2874	783	735	1566	1470
11	1490	2980	783	735	1566	1470
12	1544	3088	783	735	1566	1470
13	1598	3196	783	735	1566	1470

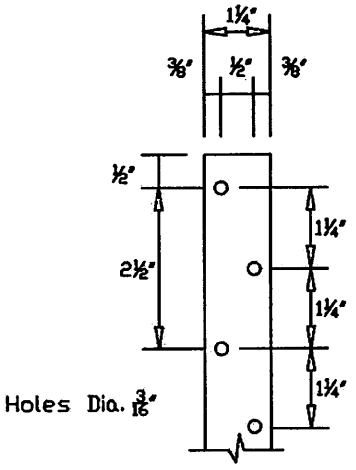
C_D=1.6 for Uplift and Lateral loads.



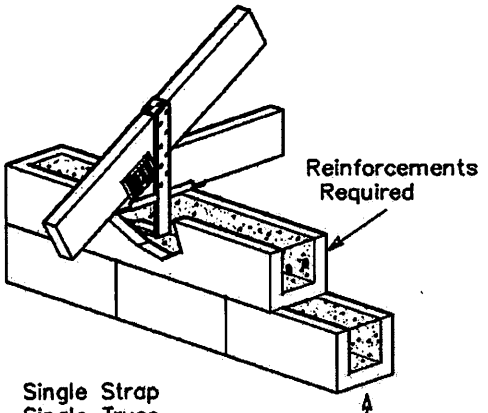
NVTAS



NVTH



NVTH, NVTHS



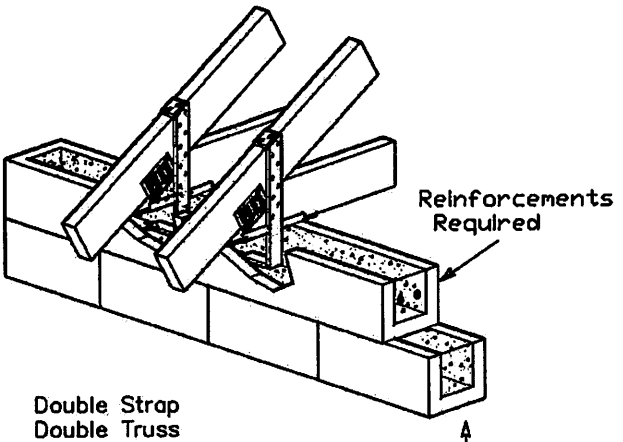
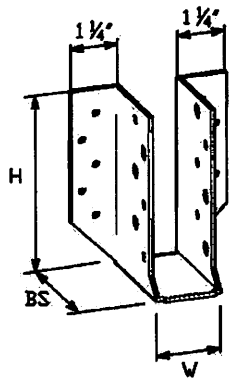


TABLE 13

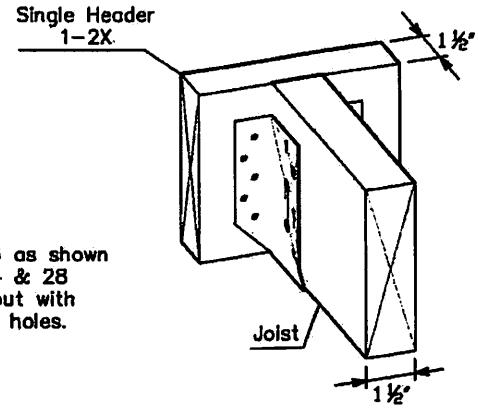
JOIST SUPPORTS

18 G NVJH JOIST SUPPORTS									Allowable Loads (lbs)		
Product Code	Dimension (inches)			Joist Size	Double Header Size	Single Header Size	Fasteners			Gravity Loads 100%	Uplift Loads 160%
	W	H	BS				Double Header	Single Header	Joists	Double Header	Single Header	Single & Double Headers
NVJH24	1 5/8	3 5/8	3	2x4 2x6	2-2x4 2-2x6	2x4 2x6	6-10d	6-10d x 1½"	4-10d x 1½"	774	774	493*
NVJH26	1 5/8	5	3	2x6 2x8	2-2x6 2-2x8	2x6 2x8	10-10d	10-10d x 1½"	6-10d x 1½"	1290	1290	821
NVJH28	1 5/8	6 3/4	3	2x8 2x10 2x12	2-2x8 2-2x10 2-2x12	2x8 2x10 2x12	14-10d	14-10d x 1½"	7-10d x 1½"	1806	1806	1079

* Use only in non-HVHZ



NVJH 26 as shown
NVJH 24 & 28
similar but with
different holes.



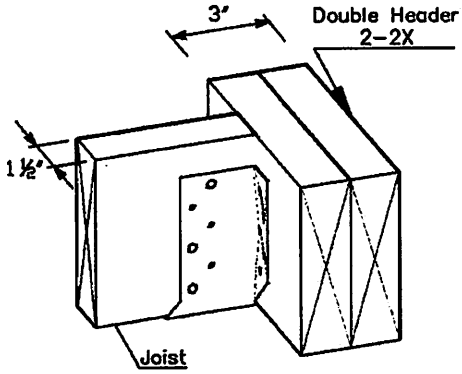
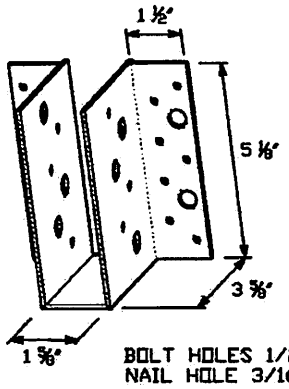


TABLE 14

NVSO 236, 16 GAUGE, HEAVY DUTY
FACE MOUNT JOIST HANGER

Joist Size	Header Size	Fasteners		Allowable Loads (Lbs.)	
		Header	Joist	GRAVITY 100%	Uplift 160%
2x6-8	2-2x8 2-2x10 2-2x12	14-10d	6-10d	1688	1064
		14-16d	6-16d	1800	1228
		4-⅜" x 3" Leg Screws	6-16d	1228	1168



BOLT HOLES 1/2" DIA.
NAIL HOLE 3/16" DIA.

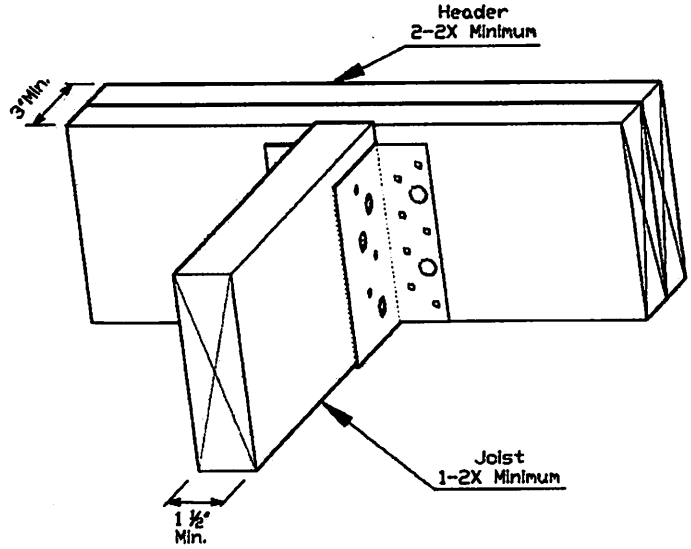
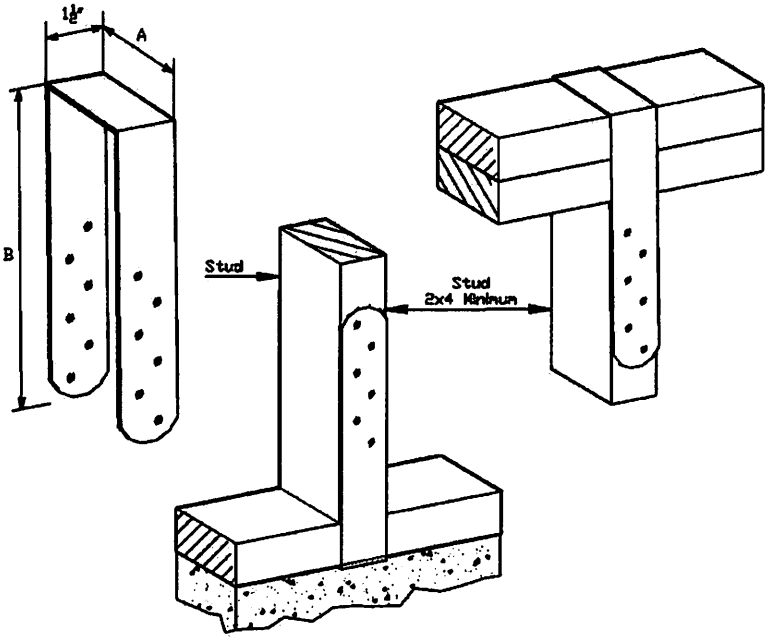


TABLE 15
TOP PLATE ANCHORS
NVTP & NVTPH

Size	Product Code	Gauge	Dimensions (in)	
			A	B
2x4/4x4	NVTP4	20	3 5/8"	8"
2x6/4x6	NVTP6	20	5 5/8"	8"
2x8/4x8	NVTP8	20	7 3/8"	8"
2x4/4x4	NVTP4H	18	3 5/8"	8"
2x6/4x6	NVTP6H	18	5 5/8"	8"
2x8/4x8	NVTP8H	18	7 3/8"	8"

Product code	Total number of fasteners 10d x 1 ½" Max. Uplift Capacity (lbs)			
	6	8	10	12
NVTP 4,6,8	828	1087	1346	1605
NVTP 4H,6H,8H	938	1207	1476	1745



Notes:

1. One half of all specified fasteners shall be used on each side of the stud to achieve tabulated values.
2. C_D=1.6 for Uplift.

TABLE 16
18 Gauge NVHC 43 & NVHC 43/2 HURRICANE CLIP.

PRODUCT CODE	DESCRIPTION	FASTENERS		DESIGN LOADS (LBS)		
		HEADER	JOIST	UPLIFT	L1	L2
NVHC 43	Hurricane Clip – Wide	9–10d	9–10d	687*	407	308
NVHC 43/2	Hurricane Clip – Widex2	10–10d	10–10d	917	547	432

* For Uplift, use two clips, one on each side to comply with section 2321.7 of the FBC

C_D=1.6 for Uplift L1 & L2.

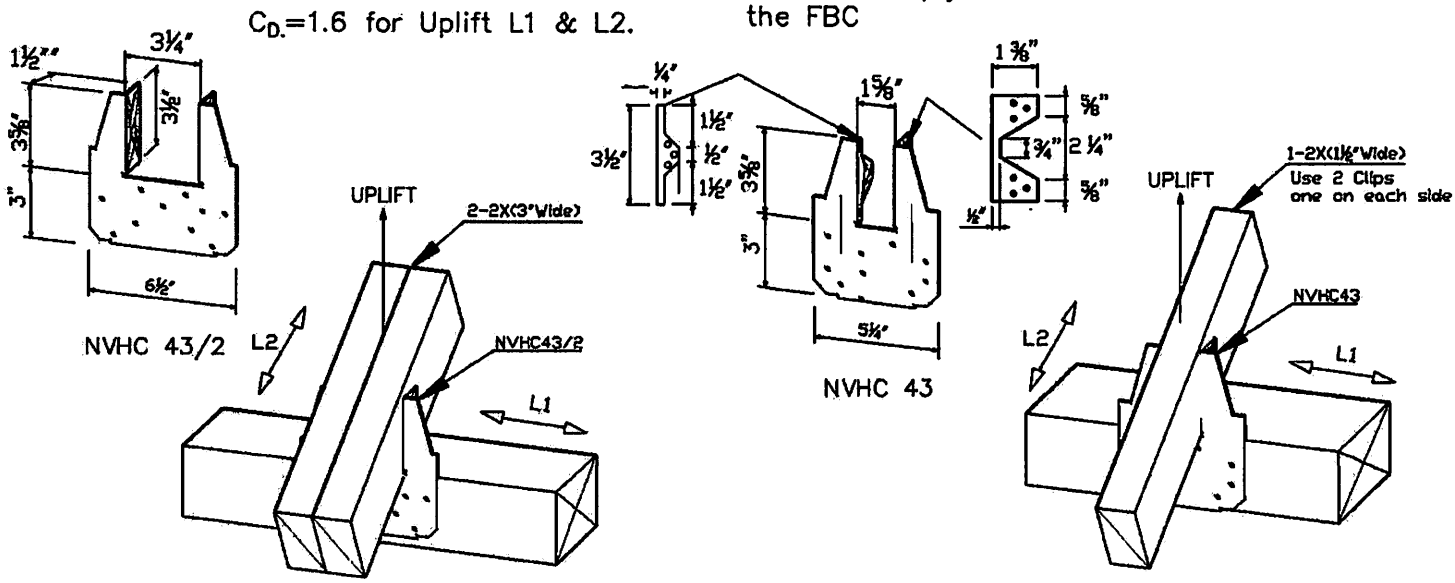
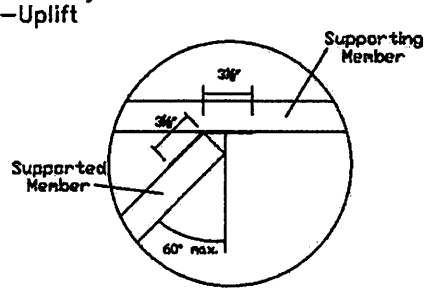
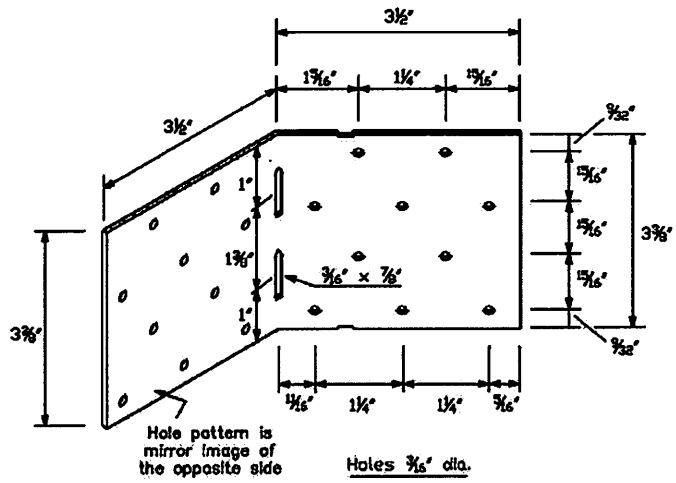


TABLE 17 SKEWED NAIL PLATE				
Product Code	Steel Gauge	Fastener Schedule	Allowable Loads (lbs)	
		Each End	Gravity	Uplift
NVSNP3	16	(6) 8d x 1½"	570	570

For Uplift, use two NVSNP3, one at top chord and one at bottom chord of the supporting and supported Trusses in compliance with section 2321.7 of the FBC.
C_D=1.0–Gravity
C_D=1.6–Uplift



NVSNP3 Installation

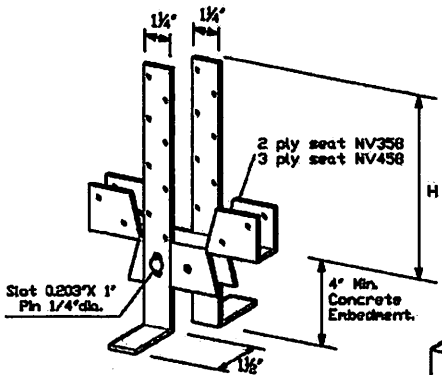


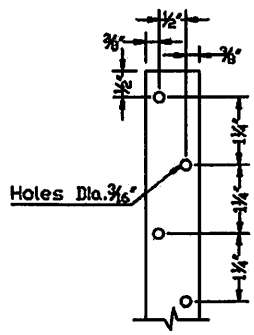
14G Straps NVTH358/NVTH458		
Assembly Product Code	14G Strap Product Code	Dimension H (inches)
NV358/NV458-12	NVTH16	12
NV358/NV458-14	NVTH18	14
NV358/NV458-16	NVTH20	16
NV358/NV458-20	NVTH24	20
NV358/NV458-22	NVTH26	22
NV358/NV458-24	NVTH28	24
NV358/NV458-26	NVTH30	26
NV358/NV458-32	NVTH36	32
NV358/NV458-44	NVTH48	44

TABLE 18 – NVTH358–2 Ply Seat				
Total No. of Fasteners in 2 Straps 10d x 3"	Total No. of Fasteners in Seat 10d x 3"	Allowable Loads (lbs)		
		Uplift	L1	L2
8	8	2245	1961	1839
10	8	2525	2206	2068
12	8	2806	2452	2298
14	8	3086	2697	2528
16	8	3367	2942	2758

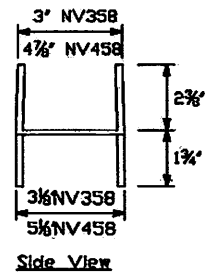
C_D=1.6 for Uplift L1 & L2.

TABLE 19 – NVTH458–2 Ply Seat				
Total No. of Fasteners in 2 Straps 10d x 3"	Total No. of Fasteners in Seat 10d x 3"	Allowable Loads (lbs)		
		Uplift	L1	L2
8	8	2245	2783	2078
10	8	2525	3131	2338
12	8	2806	3479	2597
14	8	3086	3827	2857
16	8	3367	4175	3117

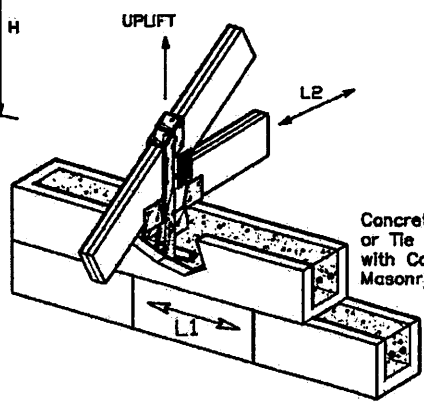


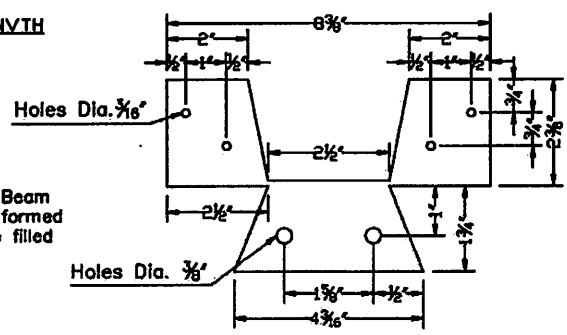


14G NVTH



Side View



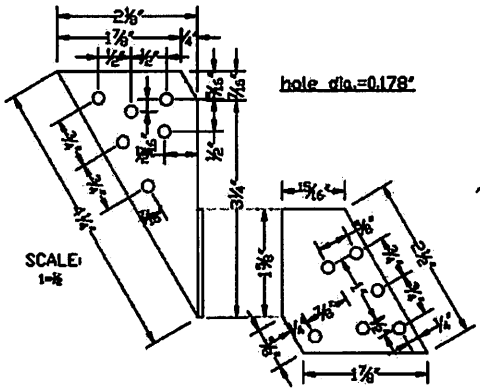


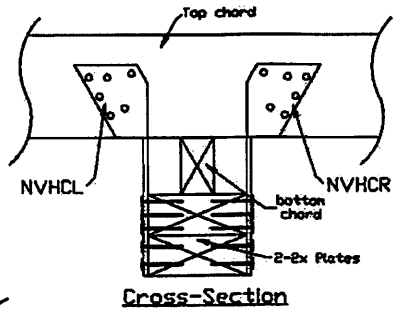
18G Seat Detail

TABLE 20
HURRICANE CLIPS

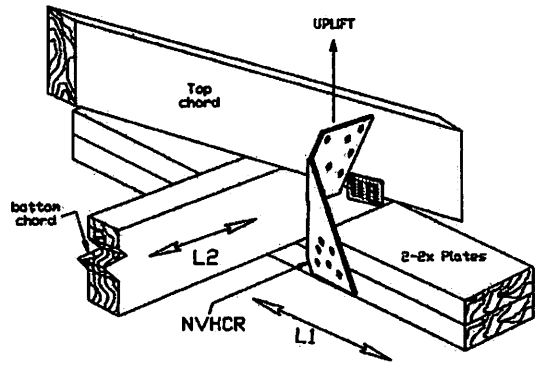
Product Code	Description	Gauge	Fasteners 10d x 1½"		Allowable Loads (lbs)		
			Header	Joist	Uplift	L1	L2
NVHCR	HURRICANE CLIP – RIGHT	18	6	6	525	253	333
NVHCL	HURRICANE CLIP – LEFT	18	6	6	525	253	333

For Uplift, use two clips, one on each side to comply with section 2321.7 of the FBC.
C_D=1.6 for Uplift, L1 & L2.





Cross-Section



14G NVTH Straps NVSTH/NVHTH		
Assembly Product Code	14G Strap Product Code	Dimension H (inches)
NVSTA-12H NVHTA-12H	NVTH16	12
NVSTA-14H NVHTA-14H	NVTH18	14
NVSTA-16H NVHTA-16H	NVTH20	16
NVSTA-20H NVHTA-20H	NVTH24	20
NVSTA-22H NVHTA-22H	NVTH26	22
NVSTA-24H NVHTA-24H	NVTH28	24
NVSTA-26H NVHTA-26H	NVTH30	26
NVSTA-32H NVHTA-32H	NVTH36	32
NVSTA-44H NVHTA-44H	NVTH48	44

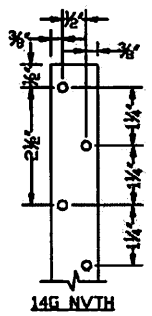
TABLE 21 NVSTA Single Strap

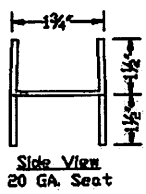
Total No. of Fasteners in Straps 10d x 1½"	Total No.of Fasteners in 20 GA. Seat 10d x 1½"	Allowable Loads (lbs) F'c=2500 Psi		
		Uplift	L1	L2
5	6	1308	700	1049
6	6	1426	760	1144
7	6	1545	823	1239
8	6	1664	887	1335
9	6	1783	950	1430

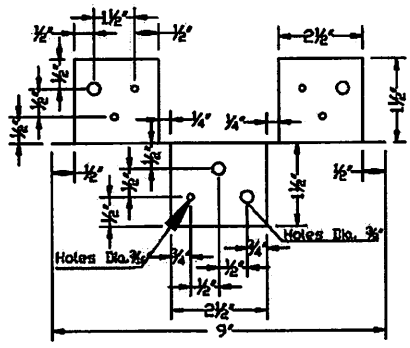
TABLE 22 NVSTA–Double Strap

Total No. of Fasteners in 2 Straps 10d x 3"	Total No. of Fasteners in Seat 10d x 3"	Allowable Loads (lbs) Fc'=2500 Psi Unless Noted			
		Uplift	Uplift Fc'=3000	L1	L2
8	8	1772	2078	1050	1450
10	8	1994	2338	1181	1631
12	8	2215	2598	1312	1812
14	8	2437	2858	1444	1994
16	8	2658	3117	1575	2175

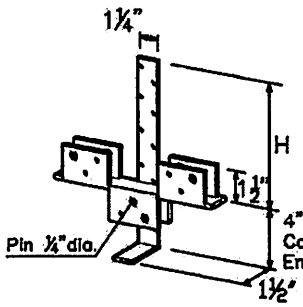
C_D=1.6 for Uplift L1 & L2.



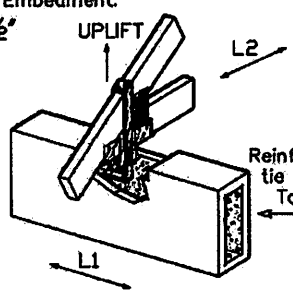


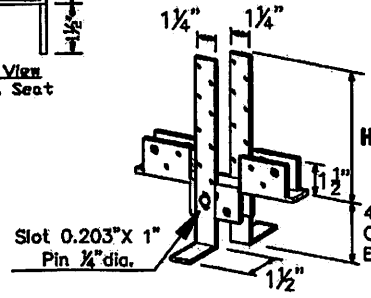


20G Seat



NVSTA 12H 44H





NVHTA 12H 44H

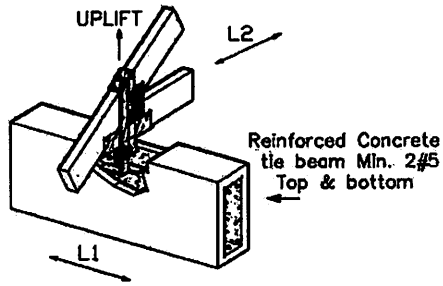
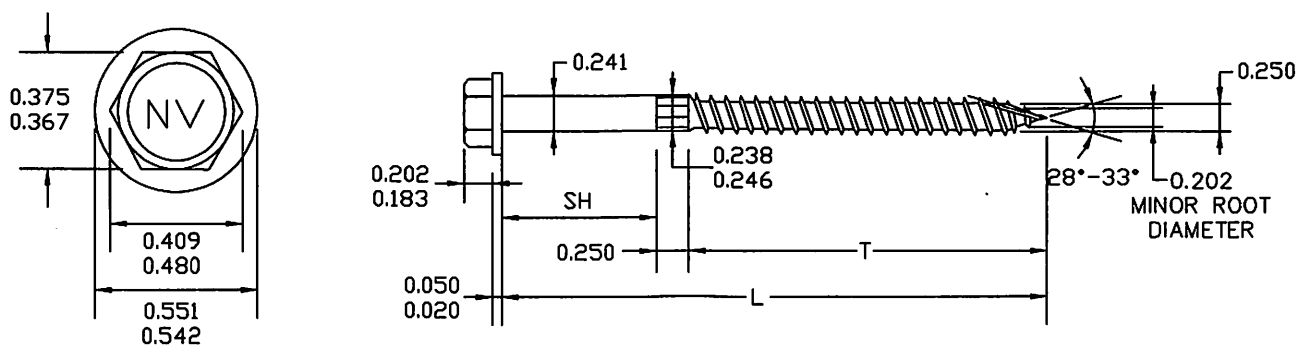


TABLE 23A

NVWS–WOOD SCREW SPECIFICATIONS, BENDING YIELD STRENGTH, AND ALLOWABLE SCREW STRENGTH

Fastener Designation	Screw Dimensions (inches)			Bending Yield 1 Strength F_{yb}(psi)	Screw Ultimate Strength(lbs)		Allowable[2] Screw design Strength(lbs)	
	Overall L Length	SH(in)	Thread T Length		Tension	Shear	Tension	Shear
NVWS1.5	1½"	¼"	1¼"	174,906	4452	2754	1484	918
NVWS3	3"	¾"	2¼"	156,893	4563	2715	1521	905

1. Do not use Bending yield Strength to calculate Lateral design values per NDS. Use Lateral design values as shown in table 3.
2. Allowable design loads are ultimate loads divided by a factor of safety of 3.



NVWS SCREW

Structural Notes:

1. NVWS wood screws are manufactured from SAEC–1022 steel with a zinc yellow chromate finish.
2. Allowable loads are based on 1½" thick wood members.
3. All tests have been conducted in accordance with ASTM D1761, ASTM F1575, ASTM D1037 and AISI S904.
4. Allowable Lateral Loads are based on tests conducted and not based on NDS.
5. Design loads are for Douglas Fir and Southern Pine with a specific gravity of 0.50 and moisture content between 11% and 19%.
6. Loads shown in tables 23B, C, D can be increased by adjustment factor C_D=1.6 for uplift and lateral loads and other adjustment factors in accordance with NSD 2018/2015.
7. All designs conform to FBC 2020/2017, ASTM D7147–2005/2011 .

TABLE 23B

Withdrawal Design Values for NVWS Screws Wood Specific Gravity 0.5, Moisture Content > 11.0

	Screws Length (inches)	Thread Length (inches)	Withdrawal Design value lbs	Withdrawal Design value lbs/inch of Thread length
NVWS1.5	1½	1¼	175	141
NVWS3	3	2	296	148

TABLE 23C

Lateral Design Values for single shear NVWS Screws Wood Specific Gravity 0.5, Moisture Content > 11.0

Fastener Designation	14 G Steel to Wood lbs	Wood to Wood lbs
NVWS1.5	201	-
NVWS3	308	236

TABLE 23D

Pull Through Design Values for NVWS Screws Wood Specific Gravity 0.5, Moisture Content > 11.0

Fastener Designation	Pull Through Design Value lbs
NVWS1.5	313
NVWS3	351

TABLE 23E

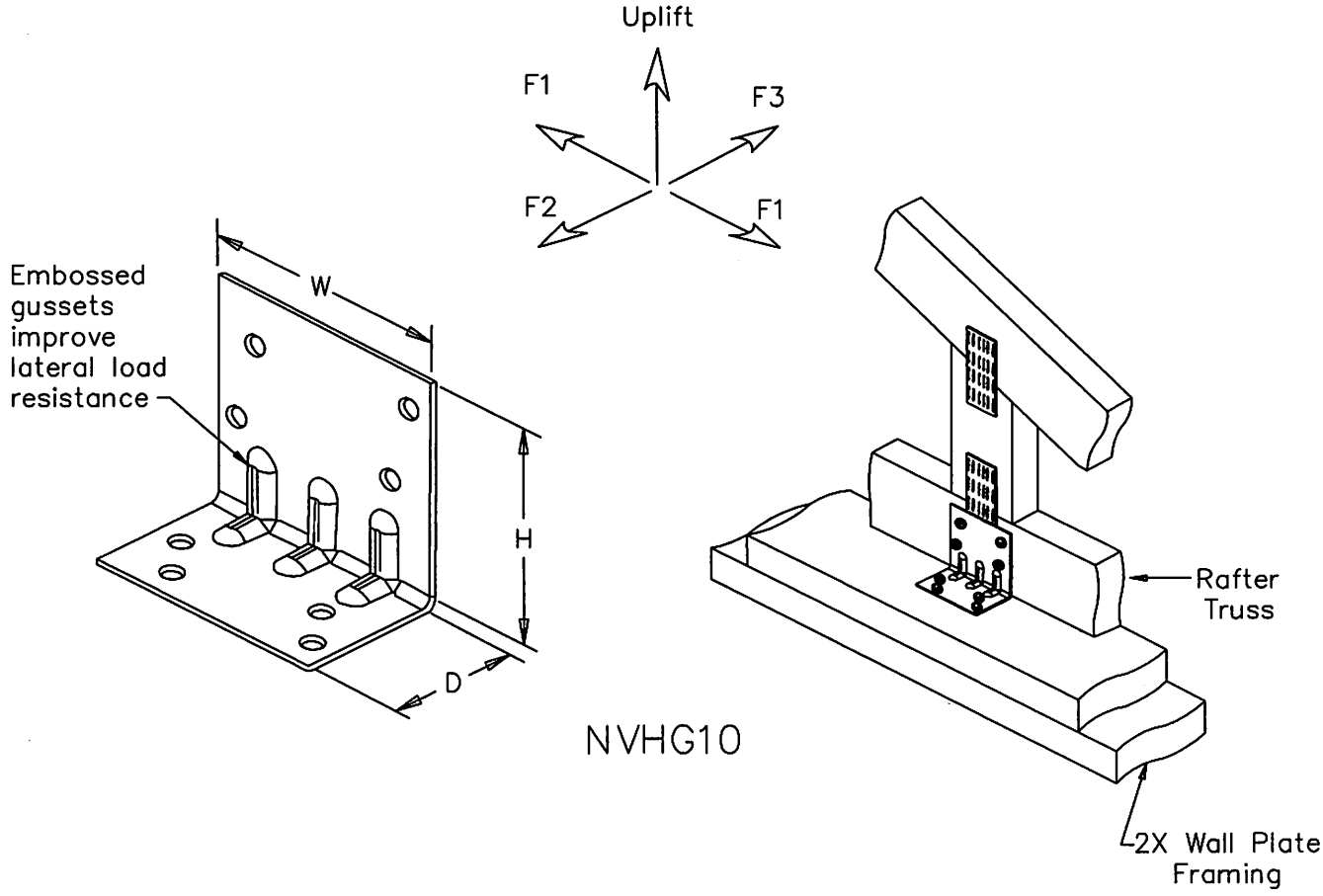
Spacing and Edge distance
Minimum Spacing between fasteners = 2"
Minimum Edge distance = ¾"

TABLE 24

NVHGA10 HURRICANE GUSSET ANGLE ALLOWABLE LOADS[1,2]

Product Designation	Steel Gage	Dimensions (in)			Fastener Schedule				Allowable Loads (lbs)			
					Plate		Rafter/Truss		F_1	F_2[1]	F_3	Uplift
		W	H	L	Qty.	Type	Qty.	Type	$C_{D=1.6}$	$C_{D=1.6}$	$C_{D=1.6}$	$C_{D=1.6}$
NVHGA10	14	3½"	3	2	4	NVWS3	4	NVWS1.5	1286	1091	1120	790

1. Allowable loads in the F_2 direction are based on compression perpendicular to grain design value, $F_{c\perp}$ of 565 psi or greater.



NVHG10

General Notes:

1. Steel shall conform to ASTM A653, SS grade 40 minimum yield 40 ksi, minimum tensile strength 55 ksi and minimum galvanized coating of G 60 per ASTM A653.
2. Allowable loads and fasteners are based on Tables 23.
3. Design loads are for S, Pine/D. Fir specific gravity 0.50.
4. Combined load of Uplift and Lateral Loads shall be satisfy the following equation.

$$\frac{\text{Actual Uplift}}{\text{Allowable Uplift}} + \frac{\text{Actual F1, F2, F3}}{\text{Allowable F1, F2, F3}} <= 1.0$$

5. Allowable loads are based on 1½" thick wood members unless otherwise noted.
6. All design Conform to FBC 2020/2017, ASTM D1761, ASTM D7147, and NDS 2018/2015.

MIAMI-DADE COUNTY
PRODUCT CONTROL SECTION
11805 SW 26 Street, Room 208
Miami, Florida 33175-2474
T (786) 315-2590 F (786) 315-2599
www.miamidade.gov/economy

DEPARTMENT OF REGULATORY AND ECONOMIC RESOURCES (RER)
BOARD AND CODE ADMINISTRATION DIVISION

NOTICE OF ACCEPTANCE (NOA)

ITW Buildex
1349 West Bryn Mawr Avenue
Itasca, IL 60143

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed and accepted by Miami-Dade County RER-Product Control Section to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Section (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. RER reserves the right to revoke this acceptance, if it is determined by Miami-Dade County Product Control Section that this product or material fails to meet the requirements of the applicable building code.

This product is approved as described herein, and has been designed to comply with the Florida Building Code, including the High Velocity Hurricane Zone.

DESCRIPTION: Tapcon® Concrete and Masonry Anchors with Advanced Threadform Technology™

APPROVAL DOCUMENT: Drawing No. **16-3844**, titled "Tapcon Concrete Anchors with Advanced Threadform Technology", sheets 1 and 2 of 2, dated 06/16/2006, with last revision dated 07/25/2014, prepared by Engineering Express, signed and sealed by Frank L. Bennardo, P.E. on 12/13/2016, bearing the Miami-Dade County Product Control revision stamp with the Notice of Acceptance number and expiration date by the Miami-Dade County Product Control Section.

MISSILE IMPACT RATING: None

LABELING: Each box shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.

This NOA **renews and revises NOA # 12-0816.06** and consists of this page 1 and evidence pages E-1 and E-2, as well as approval document mentioned above.

The submitted documentation was reviewed by **Carlos M. Utrera, P.E.**

MIAMI-DADE COUNTY APPROVED

01/18/2017

NOA No. 16-1222.06
Expiration Date: August 31, 2021
Approval Date: January 26, 2017

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

A. DRAWINGS

1. Drawing No. **16-3844**, titled "Tapcon Concrete Anchors with Advanced Threadform Technology", sheets 1 and 2 of 2, dated 06/16/2006, with last revision dated 07/25/2014, prepared by Engineering Express, signed and sealed by Frank L. Bennardo, P.E. on 12/13/2016.

B. TESTS *"Submitted under NOA # 07-0315.03"*

1. Tension and Shear Resistance Test of 3/16" SS Tapcon ATFT with Climashield per ASTM E 488, prepared by C.E.L Consulting, Report No. **71B153**, dated 01/31/2007, signed and sealed by Lee W. Mattis, P.E.
2. Corrosion Resistance Test per ASTM G 85 as noted in TAS 114(E), prepared by PRI Construction Materials Technologies, Report No. **ITW-004-02-01**, dated 01/05/2007, signed and sealed by Charles L. Thomas, P.E.

"Submitted under NOA # 05-1020.01"

3. Test report on Tension and Shear Resistance, Report No. **4I131** werc for "Wercs Tapcon Concrete/Masonry Screw Anchors" per ASTM E 488, prepared by Cel Consulting on 04/29/2005, signed and sealed by L. W. Mattis, P.E.
4. Test report on Tension and Shear Resistance, Report No. **4I131** werc Supplement 6519 for Tapcon with Advanced Threadform Technology per ASTM E 488 prepared by Cel Consulting on 05/19/2005, signed and sealed by L. W. Mattis, P.E.
5. Test report of corrosion resistance for fasteners, Project No. **ITW-003-02-01, 02, 03 & 04,** for fasteners with Tapcon Plus, Silver Climaseal, Silver Ultrashield and White Ultrashield per ASTM G85 as noted on TAS 114(E), prepared by Cel Consulting on 10/22/2004 and 05/16/2006, signed and sealed by C. L. Thomas, P.E.

C. CALCULATIONS *"Submitted under NOA # 12-0816.06"*

1. Anchors allowable loads calculations prepared by Engineering Express, dated 08/14/2012, signed and sealed by Frank L. Bennardo, P.E.

"Submitted under NOA # 07-0315.03"

2. Calculations addendum on new stainless steel anchors, prepared by Engineering Express, dated 03/08/2007, signed and sealed by Frank L. Bennardo, P.E.

01/18/2017

Carlos M. Utrera, P.E.
Product Control Examiner
NOA No. 16-1222.06
Expiration Date: August 31, 2021
Approval Date: January 26, 2017

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

C. CALCULATIONS (Continued)

"Submitted under NOA # 05-1020.01"

3. Calculations of allowable loads for Tapcon with Advanced Threadform Technology, prepared by Engineering Express, dated 06/22/2006, signed and sealed by Frank L. Bennardo, P.E.

D. QUALITY ASSURANCE

1. Miami-Dade Department of Regulatory and Economic Resources (RER)

E. MATERIAL CERTIFICATIONS

1. None.

F. STATEMENTS

1. Statement letter of code conformance to the 5th edition (2014) FBC issued by Engineering Express, dated 12/13/2016, signed and sealed by Frank L. Bennardo, P.E.
2. Statement letter of no financial interest issued by Engineering Express, dated 12/13/2016, signed and signed by Frank L. Bennardo, P.E.

"Submitted under NOA # 12-0816.06"

3. Drawing No. **06-ITW-0001** statement of code conformance to 2007 and 2010 FBC issued by Engineering Express, dated 08/14/2012, signed and sealed by Frank L. Bennardo, P.E.

"Submitted under NOA # 09-0121.19"

4. Statement letter of code conformance to 2007 FBC issued by Engineering Express, dated 01/05/2009, signed and sealed by Frank L. Bennardo, P.E.

"Submitted under NOA # 05-1020.01"

5. No interest letter issued by Engineering Express, dated 06/23/2006, signed and signed by Frank L. Bennardo, P.E.

01/18/2017

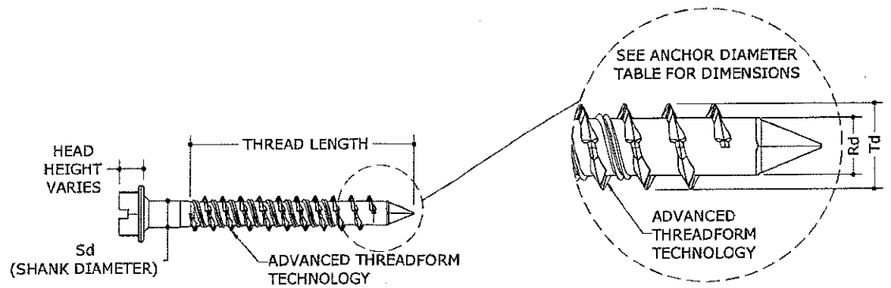
Carlos M. Utrera, P.E.
Product Control Examiner
NOA No. 16-1222.06
Expiration Date: August 31, 2021
Approval Date: January 26, 2017

ITW Buildex TAPCON® with ADVANCED THREADFORM TECHNOLOGY™

CONCRETE & MASONRY ANCHOR

GENERAL NOTES:

1. THIS PRODUCT HAS BEEN DESIGNED & TESTED IN ACCORDANCE WITH THE STRUCTURAL PROVISIONS OF THE 2010 FLORIDA BUILDING CODE & FBC FIFTH EDITION (2014), FOR USE WITHIN AND OUTSIDE THE HIGH VELOCITY HURRICANE ZONE, AND THE FOLLOWING STANDARDS: ASTM E488, ASTM G85, & TAS 114.
2. ANCHOR INSTALLATION SHALL BE MADE IN ACCORDANCE WITH MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS AND THIS NOTICE OF ACCEPTANCE.
3. CONCRETE SHALL CONFORM TO ACI 301 SPECIFICATIONS, WITH STRENGTH PROPERTIES AS SPECIFIED HEREIN. CONCRETE BLOCK SHALL CONFORM TO ASTM C-90.
4. ANCHORS REPRESENTED IN TABLES 1-4 ARE MANUFACTURED FROM HEAT-TREATED STEEL WITH AN ALTERNATING HIGH-LOW ADVANCED THREADFORM TECHNOLOGY. ANCHOR YIELD STRENGTH Fy=100 KSI, ULTIMATE STRENGTH Fu=125 KSI. HEAT TREATED STEEL ANCHORS SHALL HAVE THE CORROSION-RESISTANT COATINGS AS DESCRIBED BELOW IN COMPLIANCE WITH THE FLORIDA BUILDING CODE:
   - SILVER & WHITE ULTRASHIELD
   - BLUE & SILVER CLIMASEAL
5. ANCHORS REPRESENTED IN TABLE 5 ARE OF 410 STAINLESS STEEL ALLOY WITH AN ALTERNATING HIGH-LOW ADVANCED THREADFORM TECHNOLOGY AND SHALL HAVE A CLIMASHIELD CORROSION-RESISTANT COATING IN COMPLIANCE WITH THE FLORIDA BUILDING CODE . ANCHOR YIELD STRENGTH Fy=100 KSI, ULTIMATE STRENGTH Fu=125 KSI.
6. PILOT HOLES FOR ALL INSTALLATIONS SHALL HAVE A DEPTH 1/4" LONGER THAN INDICATED EMBEDMENT DEPTH WITH THE FOLLOWING PILOT HOLE SPECIFICATIONS:
   - 6.1. 3/16" ANCHORS: 5/32" TAPCON DRILL BIT
   - 6.2. 1/4" ANCHORS: 3/16" TAPCON DRILL BIT
7. ALLOWABLE LOAD SHOWN=ULTIMATE LOAD DIVIDED BY 4.0 FOR SOLID NON-CRACKED CONCRETE SUBSTRATES, 5.0 FOR HOLLOW BLOCK SUBSTRATES. NO ALLOWABLE STRESS INCREASE HAS BEEN USED IN PREPARATION OF THIS DOCUMENT.
8. ANCHORS SHALL NOT BE INSTALLED IN CRACKED CONCRETED SUBSTRATES, AS DEFINED IN ACI 355.2.
9. ANCHOR EDGE DISTANCES, EMBEDMENTS, AND SPACINGS BELOW THOSE SHOWN IN DESIGN TABLES HEREIN ARE NOT ACCEPTABLE.
10. ALLOWABLE LOAD CAPACITIES TO SUBSTRATES THAT ARE NOT SHOWN IN THE DESIGN TABLES LISTED HEREIN ARE OUTSIDE THE SCOPE OF THIS CERTIFICATION AND SHALL BE DETERMINED BY A LICENSED PROFESSIONAL ENGINEER.
11. ANCHOR VALUES LISTED HEREIN ARE DETERMINED THROUGH TESTING REPORT DATA AND CHECKED FOR CONSISTENCY WITH EACH TEST PERFORMED.
12. REFERENCE THE FOLLOWING TEST REPORTS:
   - CEL CONSULTING
     - #4I131werc (4/29/2005)
     - #4I131werc SUPPLEMENT 6519 (5/19/2006)
     - #71B153 (01/31/2007)
   - PRI ASPHALT TECHNOLOGIES, INC.
     - #ITW-002-02-01 (12/1/2004)
     - #ITW-003-02-02 (6/15/2006)
     - #ITW-003-02-03 (6/15/2006)
     - #ITW-003-02-04 (6/15/2006)
   - PRI CONSTRUCTION MATERIALS TECHNOLOGIES, INC.
     - #ITW-004-02-01 (11/14/2006)



1/1 TAPCON WITH ADVANCED THREADFORM TECHNOLOGY
1:1 SCALE — SIDE VIEW

ANCHOR DIAMETER					
ANCHOR TYPE	D (NOMINAL DIAMETER)	Sd (SHANK DIAMETER)	MINIMUM BASE PLATE CLEARANCE HOLE DIAMETER	Td (THREAD DIAMETER)	Rd (ROOT DIAMETER)
3/16" TAPCON	0.1875"	0.145"	0.250"	CONTACT ITW BUILDEX FOR SPECIFIC ANCHOR DIAMETER INFORMATION	
1/4" TAPCON	0.250"	0.190"	0.3125"		

HEAD STYLE OPTIONS

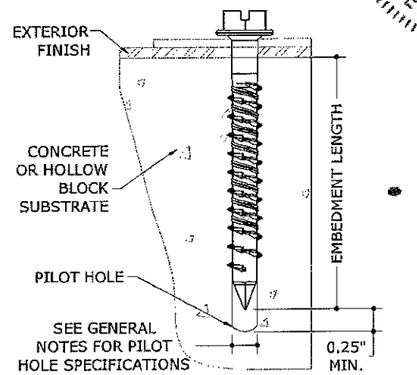
SLOTTED HEX WASHER HEAD — PHILLIPS FLAT HEAD — PHILLIPS PAN HEAD — HEX HEAD MARKINGS

MAXI-SET HEX WASHER HEAD — SCOTS STAINLESS STEEL HEAD WITH WASHER — PHILLIPS HEAD MARKINGS

ANCHOR HEAD DIMENSION		
HEAD STYLE	HEAD HEIGHT	HEAD DIAMETER
3/16" PFH	0.120"	0.357"
#12 PFH	0.148"	0.414"
1/4" PFH	0.148"	0.471"

ANCHOR HEAD DIMENSION			
HEAD STYLE	HEAD HEIGHT	ACROSS FLAT	WASHER DIAMETER
3/16" HWH	0.140"	0.246"	0.355"
1/4" HWH	0.180"	0.308"	0.415"
1/4" MAXISET	0.225"	0.308"	0.630"
1/4" SCOTS	0.280"	0.308"	0.587"

PRODUCT REVISED
as complying with the Florida Building Code
NOA–No. 16-1222.06
Expiration Date 08/31/2021
By
Miami–Dade Product Control



2/1 TYPICAL SECTION
1:1 SCALE — SIDE VIEW

ANCHOR LENGTH		
PART LENGTH	THREAD LENGTH	
	3/16" TAPCON	1/4" TAPCON
1-1/4"	FULL THREAD	FULL THREAD
1-3/4"	1-5/8"	1-5/8"
2-1/4"	1-5/8"	1-5/8"
2-3/4"	1-5/8"	1-5/8"
3-1/4"	1-5/8"	1-5/8"
3-3/4"	1-5/8"	1-5/8"
4"	1-5/8"	1-5/8"
5"		1-5/8"
6"		1-5/8"

LONG THREAD ANCHOR LENGTH		
PART LENGTH	THREAD LENGTH	
	3/16" TAPCON	1/4" TAPCON
1-3/4"	FULL THREAD	FULL THREAD
2-1/4"	2-1/8"	2-1/8"
2-3/4"	2-1/8"	2-1/8"
3-1/4"	2-1/8"	2-1/8"
3-3/4"	2-1/8"	2-1/8"
4"	2-1/8"	2-1/8"
5"		2-1/8"
6"		2-1/8"

- DESIGNATES UNAVAILABLE LENGTHS

FRANK L. BENNARDO, P.E. FL LIC. No. 46544
STATE OF FLORIDA — PROFESSIONAL ENGINEER
12/13/2016

ENGINEERING EXPRESS®
160 SW 12th AVENUE, #106
DEERFIELD BEACH, FL 33442-ER
PH: (954) 354-0660 FAX: (954) 354-0440
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CERT OF AUTH #9665
A FRANK L. BENNARDO, P.E., INC. INNOVATION

ITW Buildex
Buildex Division of Illinois Tool Works, Inc.
1349 West Bryn Mawr Avenue
Itasca, IL 60143

TAPCON CONCRETE ANCHORS WITH ADVANCED THREADFORM TECHNOLOGY
MIAMI-DADE NOTICE OF ACCEPTANCE

REMARKS	DRWN	CHKD	DATE
INIT ISSUE	CL	FLB	6/15/06
REVISE FOR 07 FBC	TSB	CL	12/30/08
REVISE FOR 10 FBC	CSL	TSB	08/10/12
REV. FBC 5TH ED (2014)	CSL	TSB	07/25/14
			-
			-

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16-3844
SCALE:
PAGE DESCRIPTION:
1 OF 2

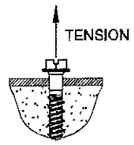
12/13/2016 - 2:18pm rickn V:\Projects\16-3844 ATT Tapcon NOA 12-0816.06 Revision and Renewal\WP\16-3844b Tapcon with ATT (NOA).dwg

STEEL ANCHORS TO NORMAL WEIGHT CONCRETE

1a TENSION CAPACITIES

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **SINGLE** ANCHORS

ANCHOR DIAM	EMBED	EDGE DIST	2000 PSI CONC (S=16D)	2000 PSI CONC (S>16D)	4000 PSI CONC (S≥16D)	5000 PSI CONC (S≥16D)
3/16"	1"	1"	125 lb	125 lb	141 lb	141 lb
		1 7/8"	136 lb	164 lb	168 lb	210 lb
	1 1/2"	1"	243 lb	243 lb	279 lb	318 lb
		1 7/8"	268 lb	278 lb	279 lb	327 lb
	1 3/4"	1"	327 lb	358 lb	360 lb	390 lb
		1 7/8"	327 lb	390 lb	401 lb	433 lb
1/4"	1"	1 1/4"	171 lb	174 lb	194 lb	237 lb
		2 1/2"	171 lb	188 lb	199 lb	237 lb
	1 1/2"	1 1/4"	313 lb	313 lb	449 lb	519 lb
		2 1/2"	346 lb	346 lb	454 lb	545 lb
	1 3/4"	1 1/4"	443 lb	465 lb	543 lb	543 lb
		2 1/2"	443 lb	508 lb	594 lb	707 lb

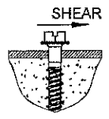


PRODUCT REVISED
as complying with the Florida Building Code
NOA-No. 16-1222.06
Expiration Date 08/31/2021
By
Miami-Dade Product Control

1b SHEAR CAPACITIES

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **SINGLE** ANCHORS

ANCHOR DIAM	EMBED	EDGE DIST	2000 PSI CONC (S=16D)	2000 PSI CONC (S>16D)	4000 PSI CONC (S≥16D)	5000 PSI CONC (S≥16D)
3/16"	1"	1 1/8"	126 lb	126 lb	150 lb	150 lb
		2 1/4"	176 lb	181 lb	181 lb	226 lb
	1 1/2"	1 1/8"	186 lb	186 lb	186 lb	186 lb
		2 1/4"	196 lb	215 lb	226 lb	226 lb
	1 3/4"	1 1/8"	204 lb	204 lb	204 lb	204 lb
		2 1/4"	218 lb	218 lb	251 lb	260 lb
1/4"	1"	1 1/2"	196 lb	196 lb	310 lb	310 lb
		3"	237 lb	237 lb	340 lb	359 lb
	1 1/2"	1 1/2"	232 lb	232 lb	346 lb	416 lb
		3"	351 lb	368 lb	417 lb	417 lb
	1 3/4"	1 1/2"	303 lb	303 lb	346 lb	500 lb
		3"	421 lb	421 lb	492 lb	500 lb

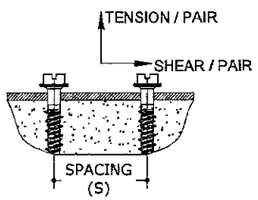


2 TENSION, SHEAR CAPACITIES (REDUCED)

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **PAIRS** OF ANCHORS*

ANCHOR DIAM	EMBED	SPACING (S)	EDGE DIST	2000 PSI CONC OR GREATER: TENSION	2000 PSI CONC OR GREATER: SHEAR
3/16"	1"	1 1/2"	3"	220 lb	352 lb
		2"		237 lb	352 lb
	1 1/2"	1 1/2"		443 lb	358 lb
		2"		474 lb	369 lb
	1 3/4"	1 1/2"		641 lb	445 lb
		2"		645 lb	455 lb
1/4"	1"	2"	4"	261 lb	447 lb
		3"		302 lb	474 lb
	1 1/2"	2"		533 lb	628 lb
		3"		637 lb	665 lb
	1 3/4"	2"		811 lb	1117 lb
		3"		848 lb	1131 lb

*REFER TO EDGE DISTANCE AND SPACING DETAIL FOR DEPICTION OF ANCHOR SPACING & EDGE DISTANCE REQUIREMENTS FOR PAIRS OF ANCHORS

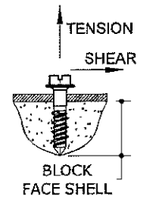


STEEL ANCHORS TO HOLLOW BLOCK

3 TENSION & SHEAR CAPACITIES

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **SINGLE** ANCHORS

ANCHOR DIAM	EMBED	EDGE DIST	LIGHT-WEIGHT CMU BLOCK (S≥16D): TENSION	LIGHT-WEIGHT CMU BLOCK (S≥16D): SHEAR	MEDIUM-WEIGHT CMU BLOCK (S≥16D): TENSION	MEDIUM-WEIGHT CMU BLOCK (S≥16D): SHEAR
3/16"	1"	2"	43 lb	83 lb	68 lb	135 lb
		4"	45 lb	83 lb	▨	147 lb
1/4"	1"	2"	43 lb	108 lb	118 lb	161 lb
		4"	56 lb	125 lb	▨	202 lb

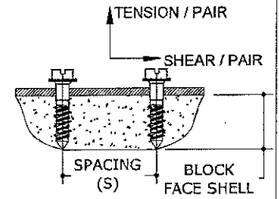


4 TENSION & SHEAR CAPACITIES

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **PAIRS** OF ANCHORS*

ANCHOR DIAM	EMBED	SPACING (S)	EDGE DIST	LIGHT-WEIGHT CMU BLOCK: TENSION	LIGHT-WEIGHT CMU BLOCK: SHEAR
3/16"	1"	1 1/2"	3"	103 lb	202 lb
		3"		103 lb	202 lb
1/4"	1"	2"	4"	84 lb	212 lb
		4"		136 lb	224 lb

*REFER TO EDGE DISTANCE AND SPACING DETAIL FOR DEPICTION OF ANCHOR SPACING & EDGE DISTANCE REQUIREMENTS FOR PAIRS OF ANCHORS



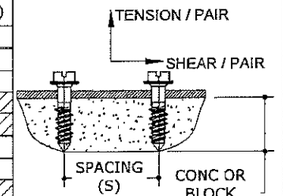
STAINLESS STEEL ANCHORS

5 TENSION & SHEAR CAPACITIES

BASED ON EMBEDMENT, EDGE DISTANCE & SPACING
FOR **PAIRS** OF ANCHORS*

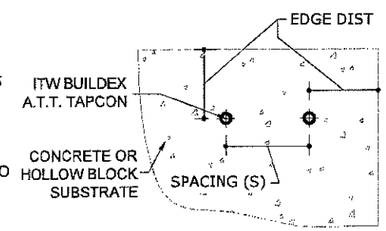
ANCHOR DIAM	EMBED	EDGE DIST	3000 PSI CONCRETE (S≥16D): TENSION	3000 PSI CONCRETE (S≥16D): SHEAR	MEDIUM-WEIGHT CMU BLOCK (S≥16D): TENSION	MEDIUM-WEIGHT CMU BLOCK (S≥16D): SHEAR
3/16"	1"	1"	164 lb	150 lb	54 lb	58 lb
		2 1/2"	164 lb	284 lb	54 lb	99 lb
	1 3/4"	1"	336 lb	169 lb	▨	▨
		2 1/2"	379 lb	364 lb	▨	▨
1/4"	1"	1"	187 lb	126 lb	68 lb	51 lb
		2 1/2"	220 lb	301 lb	77 lb	134 lb
	1 3/4"	1"	344 lb	214 lb	▨	▨
		2 1/2"	473 lb	516 lb	▨	▨

*REFER TO EDGE DISTANCE AND SPACING DETAIL FOR DEPICTION OF ANCHOR SPACING & EDGE DISTANCE REQUIREMENTS FOR PAIRS OF ANCHORS



CAPACITY TABLE NOTES:

1. ALLOWABLE LOAD SHOWN=ULTIMATE LOAD DIVIDED BY 4.0 FOR SOLID NON-CRACKED CONCRETE SUBSTRATES, 5.0 FOR CONCRETE MASONRY SUBSTRATES. EMBEDMENT VALUES LISTED CONSIDER FULL EMBEDMENT INTO THE CONCRETE OR HOLLOW BLOCK SUBSTRATE. EMBEDMENT DEPTHS DO NOT CONSIDER THE THICKNESS OF WOOD BUCKS, STUCCO OR ANY EXTERIOR FINISHES. ALL FINISHES SHALL BE BY OTHERS AND SHALL NOT EXCEED 1/8" MAXIMUM, OTHERWISE THEY SHALL BE SEPARATELY CERTIFIED TO TRANSFER ALL LOADING TO THE PROJECT SUPERSTRUCTURE.
2. ANCHORS SHALL BE INSTALLED WITH THE MINIMUM CENTER TO CENTER SPACING FOR ANCHOR CAPACITY AS LISTED IN TABLES. NO FURTHER SPACING REDUCTIONS MAY BE APPLIED TO THE VALUES LISTED HEREIN.
3. LINEAR INTERPOLATION MAY BE USED FOR EDGE DISTANCE AND SPACING BETWEEN MINIMUM AND MAXIMUM DISTANCES SHOWN IN TABLES.
4. HAMMER DRILL NOT ALLOWED FOR HOLLOW BLOCK INSTALLATIONS.
5. HOLLOW CONCRETE MASONRY UNITS (CMU) SHALL CONFORM TO ASTM C90 SPECIFICATIONS AND SHALL HAVE DENSITIES AS DEFINED BELOW:
   - "LIGHT WEIGHT" CMU: DENSITY > 95 PCF
   - "MEDIUM WEIGHT" CMU: DENSITY > 117 PCF
6. ▨ DESIGNATES CONDITIONS WHICH ARE NOT ACCEPTABLE FOR USE.



EDGE DIST & SPACING:
"PAIRS OF ANCHORS"

FRANK L. BENNARDO, P.E.
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STATE OF FLORIDA
12/13/2016

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TAPCON CONCRETE ANCHORS WITH
ADVANCED THREADFORM TECHNOLOGY
MIAMI-DADE NOTICE OF ACCEPTANCE

REMARKS	DRWN	CHKD	DATE
INIT ISSUE	CL	FLB	6/16/05
REVISE FOR 07 FBC	TSB	CL	12/30/08
REVISE FOR 10 FBC	CSL	TSB	08/10/12
REV. PBC-5TH ED (2014)	CSL	TSB	07/25/14

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