DEI
DIBBLE ENGINEERS INC

SUPPLEMENTAL STRUCTURAL CALCULATIONS

PEYREE REMODEL

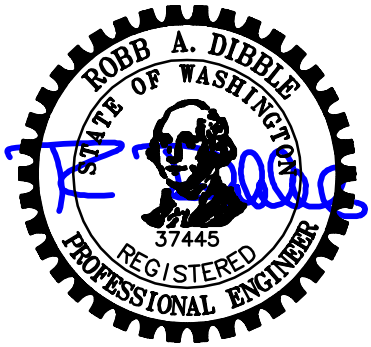
6059 77th Ave SE

Mercer Island, WA

Prepared for Gelotte Hommas

Project # 17-291

May 15, 2019



05/24/2019

DEI
DIBBLE ENGINEERS INC

SUPPLEMENTAL STRUCTURAL CALCULATIONS
TABLE OF CONTENTS

Peyree DEI Job #17-291

- Design Criteria and Loads — SD1 – SD2
- Gravity — SG1-SG83
  - Roof Framing
  - Upper Floor Framing
  - Main Floor Framing
- Lateral — SL1 – SL34
  - Moment Frame Foundation
    - Grade Beam
    - Pin Piles
- Appendix — A1-A95
  - Simpson Moment Frame
    - Calculations
    - Drawings

DEI
DIBBLE ENGINEERS INC

DESIGN CRITERIA AND LOADS

Peyree Remodel #17-291

SD1
Date: 06/2017
By: JCW

DEI
DIBBLE ENGINEERS INC

Design Criteria

Project Name:	**Peyree Remodel**
Project #:	**17-291**
Location:	**6059 77th Ave SE, Mercer Island, WA 98040**
Building Code:	**2015 IBC**
Permitting Authority:	**Mercer Island, WA**

Gravity Design Loads

Roof Dead Load	21	psf
Roof Live Load	20	psf
Exterior Wall Load	17	psf
Floor Dead Load	15	psf
Floor Live Load	40	psf
LL Reducible?	Y	Y/N
Partition Load	5	psf
Composite Floor	N/A	psf

Foundation

Soil Bearing Pressure	3000	psf
Pile Capacity	N/A	tons
Frost Depth	18	inch

Lateral Earth Pressures

Restrained	35	pcf
Unrestrained:	35	pcf
Passive Pressure:	350	pcf
Friction Coefficient	0.5	

Snow

Ground Snow Load p_g	20	psf
Importance Factor I_S	1	
Snow Load p_s	25	psf
Snow Exposure C_e	1.00	
Thermal Factor C_t	1.00	

Concrete

	f'c	fy	
Footing:	3	60	ksi
Slab-on-grade:	3	60	ksi
Concrete Beams :	3	60	ksi
Elevated Slabs:	N/A	60	ksi
Concrete Columns:	N/A	60	ksi
Concrete Walls:	4.5	60	ksi
Piles:	N/A	N/A	ksi

Masonry

	f'm	fy	
Reinforced Masonry	N/A	N/A	ksi

Structural Steel

	Fy	
W-shapes: ASTM A992, Gr. 50	50	ksi
Tube/Pipe: ASTM A500, Gr. B.	46	ksi

Lumber

Item: Species & Grade	Fb	Fv	
2x4 : HF #2	850	150	psi
2x6 & larger 2x 's : HF #2	850	150	psi
4x Beams & Post: HF #2	850	180	psi
6x Beams & Post : DF #1	1300	170	psi

Engineered Wood Products

SWWJ / OWWJ	TJI/TJL	
PSL / LSL / LVL	2900	psi
Glulam: 24F-V4 / 24F-V8	2400	psi
Sheathing (roof)	15/32	inch
Sheathing (floor)	23/32	inch
Sheathing (walls)	15/32	inch

DEI
DIBBLE ENGINEERS INC

Project Name: Peyree Remodel
Project #: 17-291
Subject:Design Weights

Sheet # 2
Date: 08/2018
By: JCW

Roof Dead Loads

Roofing Material - Concrete Tile	12.0 psf
1/2" Ply	1.5 psf
Framing Member - Pre-MFR Trusses at 24" o/c	3 psf
Batt Insulation	1 psf
5/8" Gyp Board	2.8 psf
Misc. Other	0.7 psf
	21.0 psf

Assume Roof DL of 21 psf

Typical Exterior Wall Dead Loads

Stucco Siding	10.0 psf
1/2" Ply/OSB	1.7 psf
Framing Member (2x6 Studs @ 16" oc)	1.4 psf
Batt Insulation	0.5 psf
1/2" Gyp Board	2.2 psf
Misc. Other	1.2 psf
	17.0 psf

Assume Wall DL of 17 psf

Typical Floor Dead Loads

Finish Flooring Material	3.0 psf
-	0 psf
-	0 psf
3/4" Ply	2.3 psf
Framing Member (11-7/8" TJI Joists at 16" o/c	3 psf
Batt Insulation	1 psf
5/8" Gyp Board	2.8 psf
Misc. Other	2.9 psf
	15.0 psf

Assume Floor DL of 15 psf

Deck Dead Loads

Deck Boards - Trex Decking	2.7 psf
- 1/2" Ply	1.7 psf
- 1/2" Concrete Pavers	18 psf
Framing Member (11-7/8" TJI at 16" o/c)	3 psf
- Perimeter Railing	2 psf
Misc. Other	2.6 psf
	30.0 psf

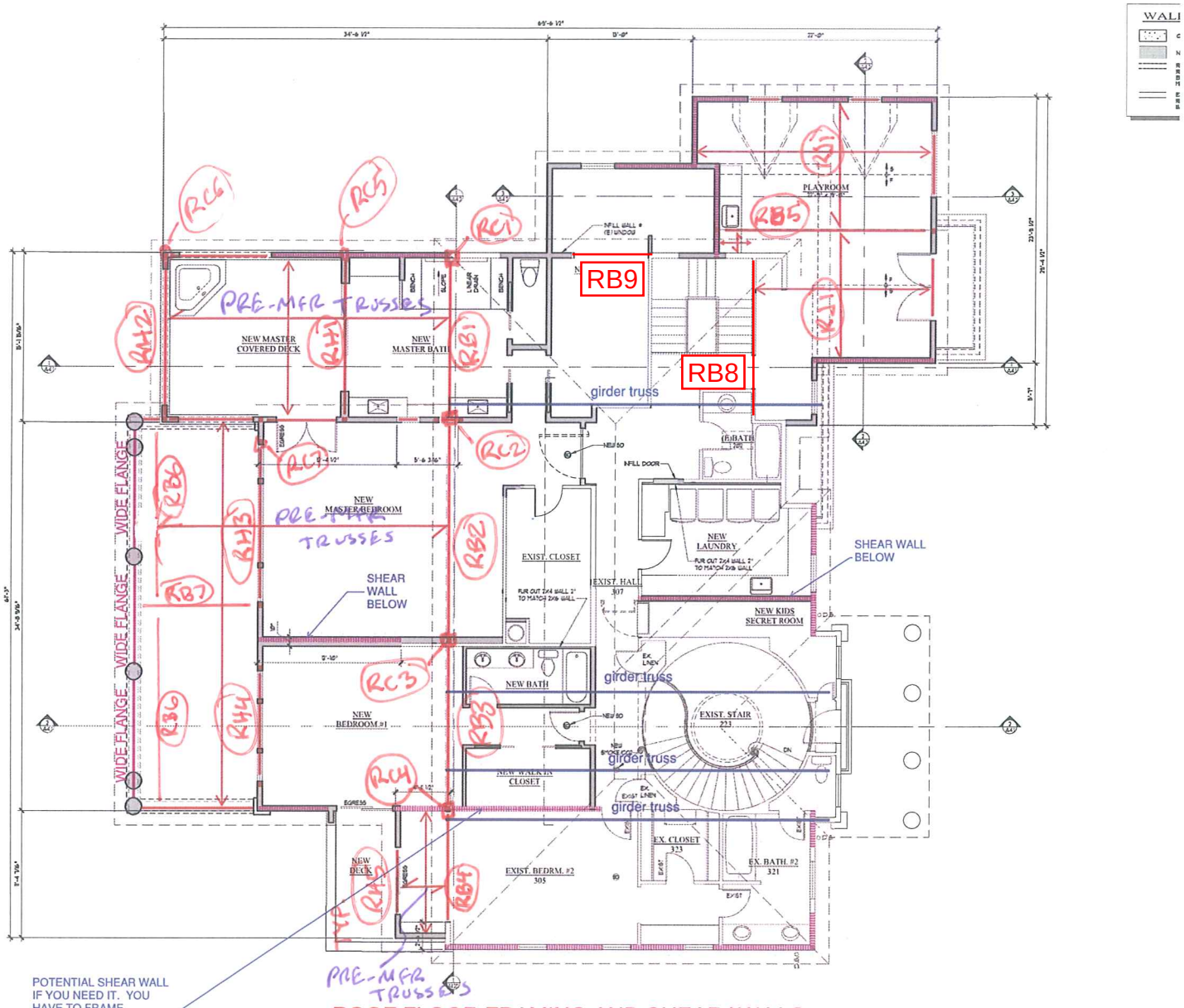
Assume DL of 30 psf

DEI
DIBBLE ENGINEERS INC

GRAVITY

Peyree Remodel #17-291

GRAVITY KEY PLAN – ROOF FRAMING



ROOF FLOOR FRAMING AND SHEAR WALLS BELOW

1 PROPOSED UPPER FLOOR PLAN
SCALE: 1/4" = 1'-0"

0 1 2 4 8 12 FT.

NORTH

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG2

Printed: 23 AUG 2018, 9:51AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RJ1 - Roof Rafter east

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Hem Fir
Wood Grade : No.2

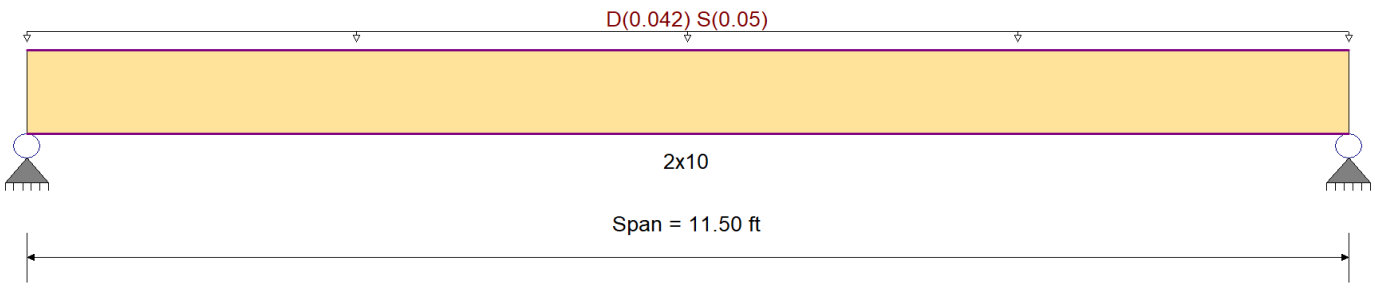
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0 psi
Fb -	850.0 psi
Fc - Prll	1,300.0 psi
Fc - Perp	405.0 psi
Fv	150.0 psi
Ft	525.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	470.0 ksi
Density	26.830 pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.709 : 1	Maximum Shear Stress Ratio	=	0.296 : 1
Section used for this span		2x10	Section used for this span		2x10
fb : Actual	=	877.18 psi	fv : Actual	=	51.07 psi
FB : Allowable	=	1,236.54 psi	Fv : Allowable	=	172.50 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	5.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.154 in	Ratio =	896	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.291 in	Ratio =	474	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.544	0.544
Overall MINimum	0.288	0.288
+D+H	0.256	0.256
+D+L+H	0.256	0.256
+D+Lr+H	0.256	0.256
+D+S+H	0.544	0.544
+D+0.750Lr+0.750L+H	0.256	0.256
+D+0.750L+0.750S+H	0.472	0.472
+D+0.60W+H	0.256	0.256
+D+0.70E+H	0.256	0.256
+D+0.750Lr+0.750L+0.450W+H	0.256	0.256
+D+0.750L+0.750S+0.450W+H	0.472	0.472

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG3

www.dibbleengineers.com

Printed: 23 AUG 2018, 9:51AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RJ1 - Roof Rafter east

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	0.472	0.472
+0.60D+0.60W+0.60H	0.154	0.154
+0.60D+0.70E+0.60H	0.154	0.154
D Only	0.256	0.256
Lr Only		
L Only		
S Only	0.288	0.288
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG4

Printed: 23 AUG 2018, 11:36AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : RB1 - Roof Beam north

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

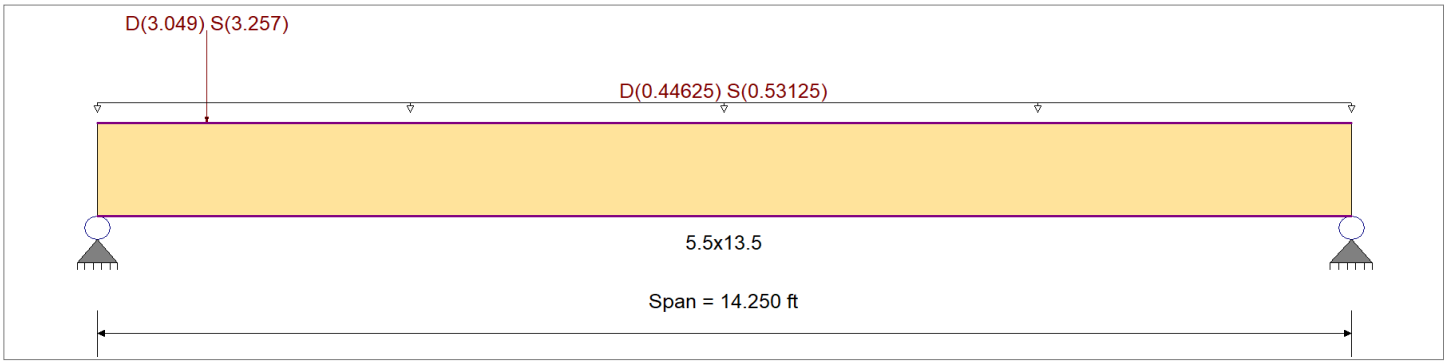
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 21.250 ft, (Roof)
Point Load : D = 3.049, S = 3.257 k @ 1.250 ft, (Girder Truss RXN)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.763 : 1	Maximum Shear Stress Ratio	=	0.779 : 1
Section used for this span		5.5x13.5	Section used for this span		5.5x13.5
fb : Actual	=	2,105.69 psi	fv : Actual	=	237.31 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	6.553 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.288 in	Ratio =	593	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.542 in	Ratio =	315	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	12.832	7.632
Overall MINimum	6.756	4.071
+D+H	6.076	3.562
+D+L+H	6.076	3.562
+D+Lr+H	6.076	3.562
+D+S+H	12.832	7.632
+D+0.750Lr+0.750L+H	6.076	3.562
+D+0.750L+0.750S+H	11.143	6.615
+D+0.60W+H	6.076	3.562
+D+0.70E+H	6.076	3.562
+D+0.750Lr+0.750L+0.450W+H	6.076	3.562

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG5

Printed: 23 AUG 2018, 11:36AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB1 - Roof Beam north

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W+H	11.143	6.615
+D+0.750L+0.750S+0.5250E+H	11.143	6.615
+0.60D+0.60W+0.60H	3.645	2.137
+0.60D+0.70E+0.60H	3.645	2.137
D Only	6.076	3.562
Lr Only		
L Only		
S Only	6.756	4.071
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG6

Printed: 23 AUG 2018, 11:41AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB2 - Roof Beam center

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

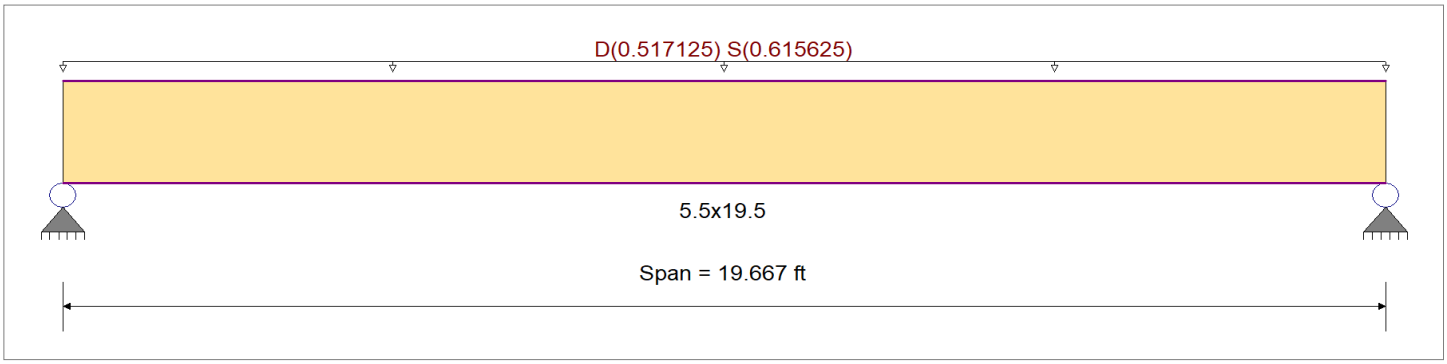
Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0	psi	*E : Modulus of Elasticity*		
Fb -	1,850.0	psi	Ebend- xx	1,800.0	ksi
Fc - Prll	1,650.0	psi	Eminbend - xx	950.0	ksi
Fc - Perp	650.0	psi	Ebend- yy	1,600.0	ksi
Fv	265.0	psi	Eminbend - yy	850.0	ksi
Ft	1,100.0	psi	Density	31.20	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 24.625 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.732 : 1	Maximum Shear Stress Ratio	=	0.438 : 1
Section used for this span		5.5x19.5	Section used for this span		5.5x19.5
fb : Actual	=	1,924.15 psi	fv : Actual	=	133.45 psi
FB : Allowable	=	2,627.88 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	9.834 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.341	in	Ratio =	692 >=360
Max Upward Transient Deflection	0.000	in	Ratio =	0 <360
Max Downward Total Deflection	0.640	in	Ratio =	368 >=240.
Max Upward Total Deflection	0.000	in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.367	11.367
Overall MINimum	6.054	6.054
+D+H	5.314	5.314
+D+L+H	5.314	5.314
+D+Lr+H	5.314	5.314
+D+S+H	11.367	11.367
+D+0.750Lr+0.750L+H	5.314	5.314
+D+0.750L+0.750S+H	9.854	9.854
+D+0.60W+H	5.314	5.314
+D+0.70E+H	5.314	5.314
+D+0.750Lr+0.750L+0.450W+H	5.314	5.314
+D+0.750L+0.750S+0.450W+H	9.854	9.854

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG7

www.dibbleengineers.com

Printed: 23 AUG 2018, 11:41AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : RB2 - Roof Beam center

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	9.854	9.854
+0.60D+0.60W+0.60H	3.188	3.188
+0.60D+0.70E+0.60H	3.188	3.188
D Only	5.314	5.314
Lr Only		
L Only		
S Only	6.054	6.054
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG8

Printed: 23 AUG 2018, 11:43AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : RB3 - Roof Beam south

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

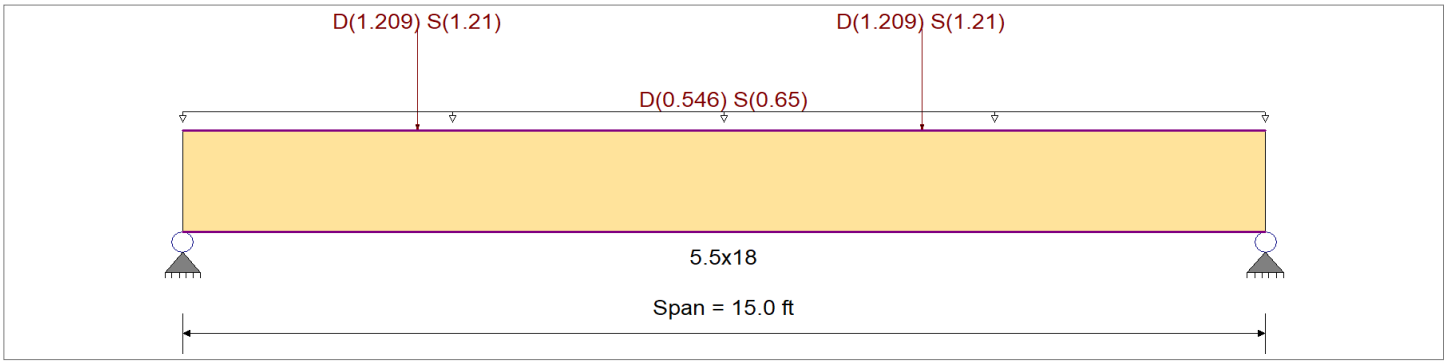
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 26.0 ft, (Roof)
Point Load : D = 1.209, S = 1.210 k @ 3.250 ft, (Girder Truss RXN)
Point Load : D = 1.209, S = 1.210 k @ 10.250 ft, (Girder Truss RXN)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.652 : 1	Maximum Shear Stress Ratio	=	0.497 : 1
Section used for this span		5.5x18	Section used for this span		5.5x18
fb : Actual	=	1,775.38 psi	fv : Actual	=	151.40 psi
FB : Allowable	=	2,721.74 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	7.719 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.199 in	Ratio =	905 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.378 in	Ratio =	476 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.792	11.308
Overall MINimum	6.206	5.964
+D+H	5.586	5.344
+D+L+H	5.586	5.344
+D+Lr+H	5.586	5.344
+D+S+H	11.792	11.308
+D+0.750Lr+0.750L+H	5.586	5.344
+D+0.750L+0.750S+H	10.240	9.817
+D+0.60W+H	5.586	5.344
+D+0.70E+H	5.586	5.344

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG9

Printed: 23 AUG 2018, 11:43AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB3 - Roof Beam south

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+0.450W+H	5.586	5.344
+D+0.750L+0.750S+0.450W+H	10.240	9.817
+D+0.750L+0.750S+0.5250E+H	10.240	9.817
+0.60D+0.60W+0.60H	3.351	3.206
+0.60D+0.70E+0.60H	3.351	3.206
D Only	5.586	5.344
Lr Only		
L Only		
S Only	6.206	5.964
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG10

Printed: 23 AUG 2018, 11:44AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB4 - Roof Beam south

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

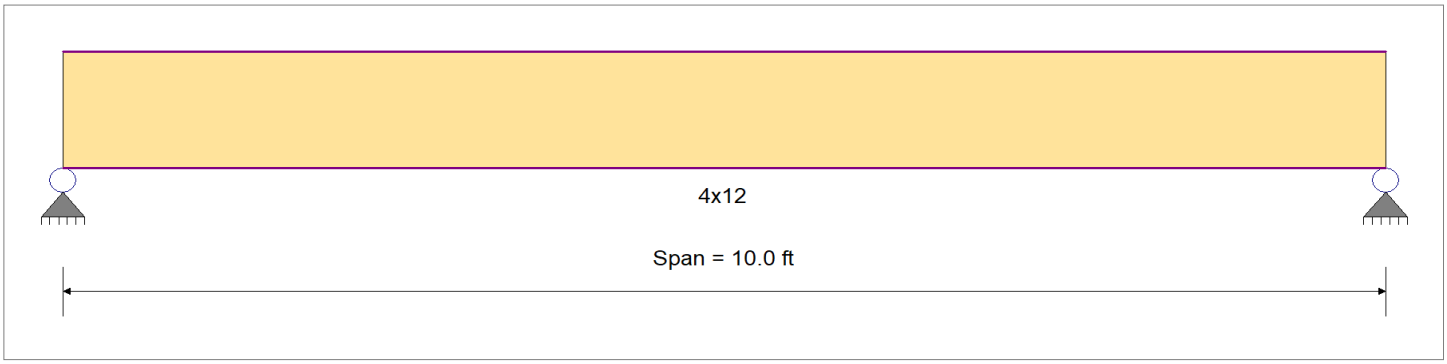
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Hem Fir
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0 psi	*E : Modulus of Elasticity*	
Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Fc - Perp	405.0 psi		
Fv	150.0 psi		
Ft	525.0 psi	Density	26.830 pcf



Applied Loads

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

Beam self weight calculated and added to loads

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.951 : 1	Maximum Shear Stress Ratio	=	0.327 : 1
Section used for this span		4x12	Section used for this span		4x12
fb : Actual	=	1,022.77 psi	fv : Actual	=	56.32 psi
FB : Allowable	=	1,075.25 psi	Fv : Allowable	=	172.50 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	6.934 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.109 in	Ratio =	1100	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.208 in	Ratio =	575	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.631	7.365
Overall MINimum	0.854	3.825
+D+H	0.777	3.540
+D+L+H	0.777	3.540
+D+Lr+H	0.777	3.540
+D+S+H	1.631	7.365
+D+0.750Lr+0.750L+H	0.777	3.540
+D+0.750L+0.750S+H	1.418	6.409
+D+0.60W+H	0.777	3.540
+D+0.70E+H	0.777	3.540
+D+0.750Lr+0.750L+0.450W+H	0.777	3.540
+D+0.750L+0.750S+0.450W+H	1.418	6.409
+D+0.750L+0.750S+0.5250E+H	1.418	6.409

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG11

www.dibbleengineers.com

Printed: 23 AUG 2018, 11:44AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB4 - Roof Beam south

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+0.60D+0.60W+0.60H	0.466	2.124
+0.60D+0.70E+0.60H	0.466	2.124
D Only	0.777	3.540
Lr Only		
L Only		
S Only	0.854	3.825
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG12

Printed: 23 AUG 2018, 10:51AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB5 - Roof Ridge Beam north east

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

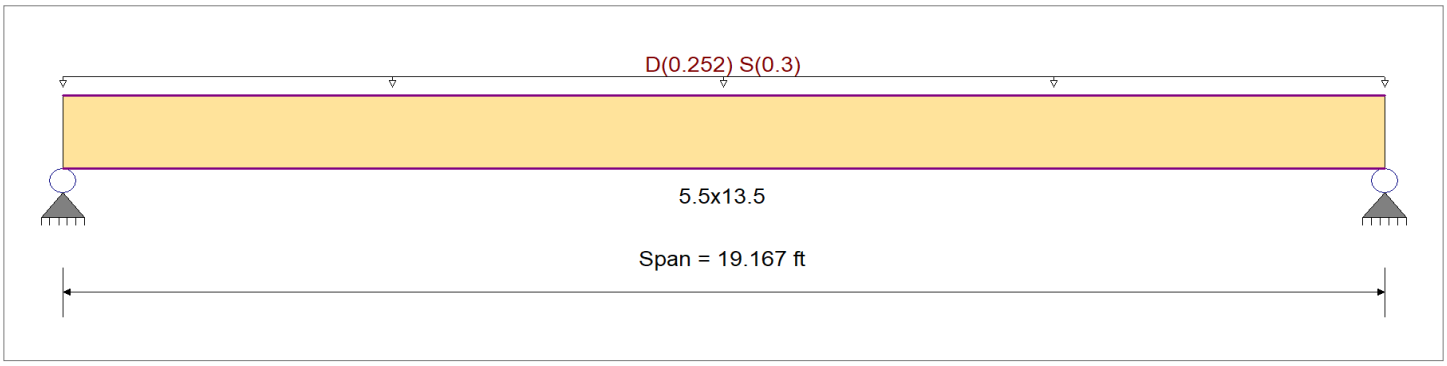
Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Ebend- yy	1,600.0 ksi
Eminbend - yy	850.0 ksi
Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 12.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.686 : 1	Maximum Shear Stress Ratio	=	0.319 : 1
Section used for this span		5.5x13.5	Section used for this span		5.5x13.5
fb : Actual	=	1,873.85 psi	fv : Actual	=	97.14 psi
FB : Allowable	=	2,733.34 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	9.584 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.451 in	Ratio =	509	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.855 in	Ratio =	269	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.444	5.444
Overall MINimum	2.875	2.875
+D+H	2.569	2.569
+D+L+H	2.569	2.569
+D+Lr+H	2.569	2.569
+D+S+H	5.444	5.444
+D+0.750Lr+0.750L+H	2.569	2.569
+D+0.750L+0.750S+H	4.726	4.726
+D+0.60W+H	2.569	2.569
+D+0.70E+H	2.569	2.569
+D+0.750Lr+0.750L+0.450W+H	2.569	2.569
+D+0.750L+0.750S+0.450W+H	4.726	4.726

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG13

www.dibbleengineers.com

Printed: 23 AUG 2018, 10:51AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB5 - Roof Ridge Beam north east

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	4.726	4.726
+0.60D+0.60W+0.60H	1.542	1.542
+0.60D+0.70E+0.60H	1.542	1.542
D Only	2.569	2.569
Lr Only		
L Only		
S Only	2.875	2.875
W Only		
E Only		
H Only		

Project Title:
Engineer:
Project ID:
Project Descr:

SG14

Printed: 16 MAY 2019, 11:14AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB8 - Roof Pickup Existing Trusses along Grid 9

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

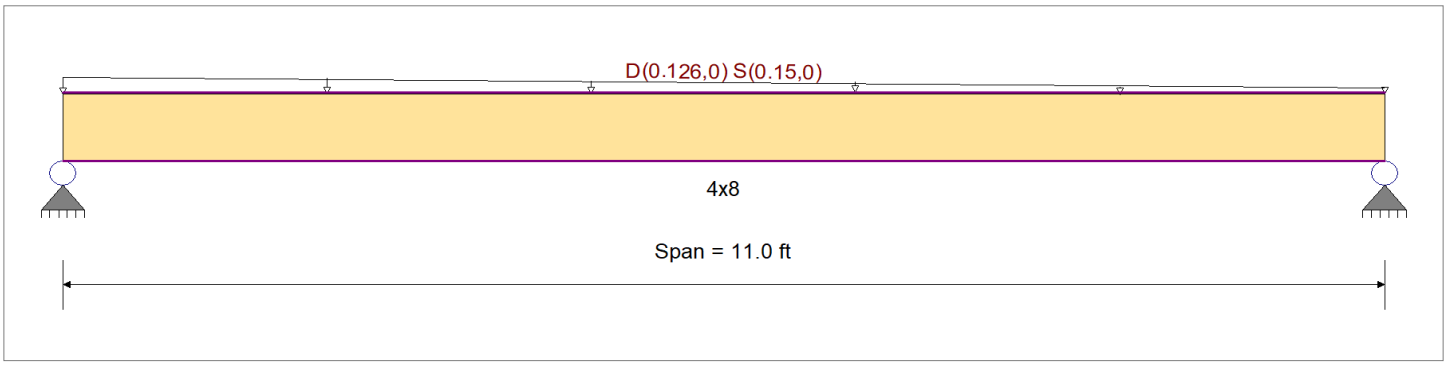
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Hem Fir
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0 psi	*E : Modulus of Elasticity*	
Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Fc - Perp	405.0 psi		
Fv	150.0 psi		
Ft	525.0 psi	Density	26.840 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Varying Uniform Load : D= 0.0210->0.0210, S= 0.0250->0.0250 ksf, Extent = 0.0 -->> 11.0 ft, Trib Width = 6.0->0.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.681 : 1	Maximum Shear Stress Ratio	=	0.299 : 1
Section used for this span		4x8	Section used for this span		4x8
fb : Actual	=	865.79 psi	fv : Actual	=	51.64 psi
FB : Allowable	=	1,270.75 psi	Fv : Allowable	=	172.50 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	4.657 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.172 in	Ratio =	765	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.328 in	Ratio =	402	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.412	0.180	0.90	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	994.50	0.41	24.32	135.00
+D+L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.371	0.162	1.00	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1105.00	0.41	24.32	150.00
+D+Lr+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.297	0.130	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1381.25	0.41	24.32	187.50
+D+S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.681	0.299	1.15	1.300	1.00	1.00	1.00	1.00	1.00	2.21	865.79	1270.75	0.87	51.64	172.50

Project Title:
Engineer:
Project ID:
Project Descr:

SG15

Printed: 16 MAY 2019, 11:14AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB8 - Roof Pickup Existing Trusses along Grid 9

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+0.750Lr+0.750L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.297	0.130	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1381.25	0.41	24.32	187.50
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.592	0.260	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.92	751.87	1270.75	0.76	44.81	172.50
+D+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.232	0.101	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1768.00	0.41	24.32	240.00
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.232	0.101	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1768.00	0.41	24.32	240.00
+D+0.750Lr+0.750L+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.232	0.101	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.05	410.14	1768.00	0.41	24.32	240.00
+D+0.750L+0.750S+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.425	0.187	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.92	751.87	1768.00	0.76	44.81	240.00
+D+0.750L+0.750S+0.5250E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.425	0.187	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.92	751.87	1768.00	0.76	44.81	240.00
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.139	0.061	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.63	246.08	1768.00	0.25	14.59	240.00
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.139	0.061	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.63	246.08	1768.00	0.25	14.59	240.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3280	5.299		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.038	0.532
Overall MINimum	0.550	0.275
+D+H	0.488	0.257
+D+L+H	0.488	0.257
+D+Lr+H	0.488	0.257
+D+S+H	1.038	0.532
+D+0.750Lr+0.750L+H	0.488	0.257
+D+0.750L+0.750S+H	0.901	0.463
+D+0.60W+H	0.488	0.257
+D+0.70E+H	0.488	0.257
+D+0.750Lr+0.750L+0.450W+H	0.488	0.257
+D+0.750L+0.750S+0.450W+H	0.901	0.463
+D+0.750L+0.750S+0.5250E+H	0.901	0.463
+0.60D+0.60W+0.60H	0.293	0.154
+0.60D+0.70E+0.60H	0.293	0.154
D Only	0.488	0.257
Lr Only		
L Only		
S Only	0.550	0.275
W Only		
E Only		
H Only		

Project Title:
Engineer:
Project ID:
Project Descr:

SG16

Printed: 16 MAY 2019, 11:14AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB9 - Roof Pickup Existing Trusses along Grid C.2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

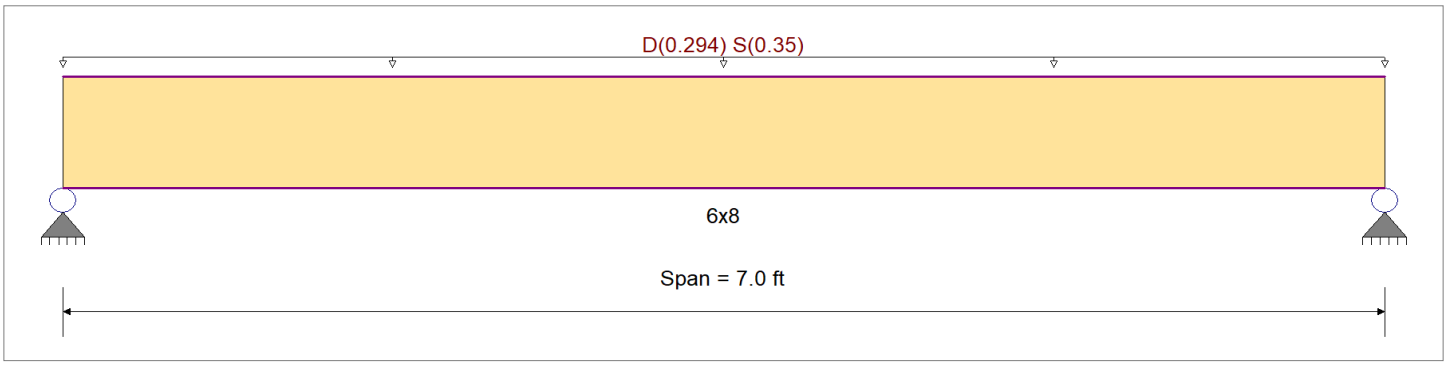
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi	*E : Modulus of Elasticity*	
Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
Fc - Prll	1,500.0 psi	Eminbend - xx	620.0 ksi
Fc - Perp	625.0 psi		
Fv	180.0 psi		
Ft	675.0 psi	Density	31.210 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 14.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.809 : 1	Maximum Shear Stress Ratio	=	0.331 : 1
Section used for this span		6x8	Section used for this span		6x8
fb : Actual	=	930.74 psi	fv : Actual	=	68.54 psi
FB : Allowable	=	1,150.00 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.387 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.058 in	Ratio =	1451 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.108 in	Ratio =	778 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.480	0.196	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	900.00	0.87	31.80	162.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.432	0.177	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1000.00	0.87	31.80	180.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.345	0.141	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1250.00	0.87	31.80	225.00
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.809	0.331	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.00	930.74	1150.00	1.88	68.54	207.00
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.345	0.141	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1250.00	0.87	31.80	225.00
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Project Title:
Engineer:
Project ID:
Project Descr:

SG17

Printed: 16 MAY 2019, 11:14AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RB9 - Roof Pickup Existing Trusses along Grid C.2

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
Length = 7.0 ft	1	0.701	0.287	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.46	806.01	1150.00	1.63	59.36	207.00
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.270	0.110	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1600.00	0.87	31.80	288.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.270	0.110	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1600.00	0.87	31.80	288.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.270	0.110	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.86	431.83	1600.00	0.87	31.80	288.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.504	0.206	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.46	806.01	1600.00	1.63	59.36	288.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.504	0.206	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.46	806.01	1600.00	1.63	59.36	288.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.162	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.11	259.10	1600.00	0.52	19.08	288.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.162	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.11	259.10	1600.00	0.52	19.08	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.1079	3.526		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.285	2.285
Overall MINimum	1.225	1.225
+D+H	1.060	1.060
+D+L+H	1.060	1.060
+D+Lr+H	1.060	1.060
+D+S+H	2.285	2.285
+D+0.750Lr+0.750L+H	1.060	1.060
+D+0.750L+0.750S+H	1.979	1.979
+D+0.60W+H	1.060	1.060
+D+0.70E+H	1.060	1.060
+D+0.750Lr+0.750L+0.450W+H	1.060	1.060
+D+0.750L+0.750S+0.450W+H	1.979	1.979
+D+0.750L+0.750S+0.5250E+H	1.979	1.979
+0.60D+0.60W+0.60H	0.636	0.636
+0.60D+0.70E+0.60H	0.636	0.636
D Only	1.060	1.060
Lr Only		
L Only		
S Only	1.225	1.225
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG18

Printed: 23 AUG 2018, 11:46AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH1 - Header west, north sliding door

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

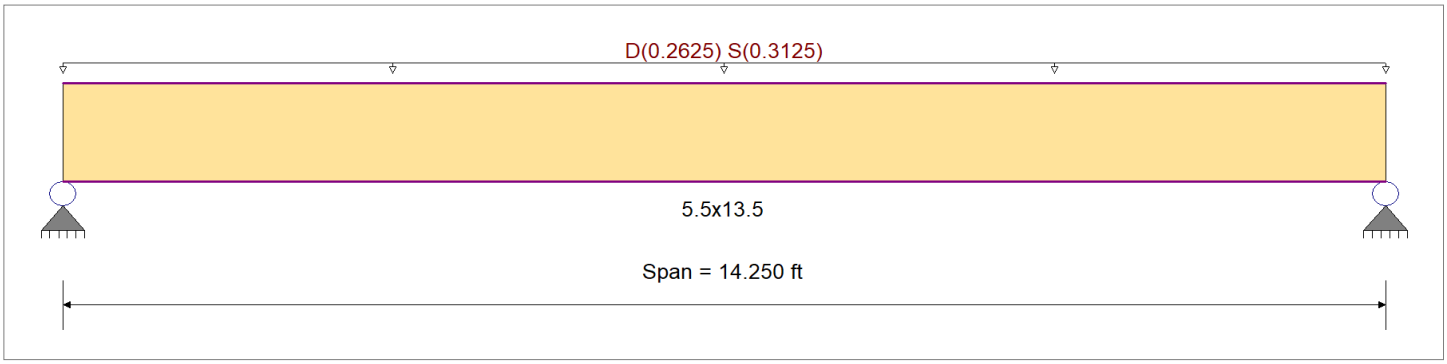
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 12.50 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.390 : 1	Maximum Shear Stress Ratio	=	0.236 : 1
Section used for this span		5.5x13.5	Section used for this span		5.5x13.5
fb : Actual	=	1,077.69 psi	fv : Actual	=	72.04 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	7.125 ft	Location of maximum on span	=	13.158 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.144 in	Ratio =	1190	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.272 in	Ratio =	629	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.211	4.211
Overall MINimum	2.227	2.227
+D+H	1.985	1.985
+D+L+H	1.985	1.985
+D+Lr+H	1.985	1.985
+D+S+H	4.211	4.211
+D+0.750Lr+0.750L+H	1.985	1.985
+D+0.750L+0.750S+H	3.655	3.655
+D+0.60W+H	1.985	1.985
+D+0.70E+H	1.985	1.985
+D+0.750Lr+0.750L+0.450W+H	1.985	1.985
+D+0.750L+0.750S+0.450W+H	3.655	3.655

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG19

Printed: 23 AUG 2018, 11:46AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH1 - Header west, north sliding door

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	3.655	3.655
+0.60D+0.60W+0.60H	1.191	1.191
+0.60D+0.70E+0.60H	1.191	1.191
D Only	1.985	1.985
Lr Only		
L Only		
S Only	2.227	2.227
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG20

Printed: 23 AUG 2018, 11:47AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH2 - Header north west

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 14.250 ft
(Used for non-slender calculations)

Wood Species DF/DF
Wood Grade 24F - V4

Fb +	2,400.0 psi	Fv	265.0 psi
Fb -	1,850.0 psi	Ft	1,100.0 psi
Fc - Prll	1,650.0 psi	Density	31.20 pcf
Fc - Perp	650.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,800.0	1,600.0	1,800.0	ksi
Minimum	950.0	850.0		

Wood Section Name **5.5x7.5**
Wood Grading/Manuf. Western
Wood Member Type GLB

Exact Width	**5.50**	in
Exact Depth	**7.50**	in
Area	41.250	in^2
Ix	193.359	in^4
Iy	103.984	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 14.250 ft, K = 1.0

Applied Loads

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

Column self weight included : 127.359 lbs * Dead Load Factor
BENDING LOADS . . .
Roof - 9.5ft trib: Lat. Uniform Load creating Mx-x, D = 0.1995, S = 0.2375 k/ft
C&C Wind - 4.25ft trib: Lat. Uniform Load creating My-y, W = 0.1550 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9372** : 1
Load Combination +D+S+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 7.173 ft
At maximum location values are . . .
Applied Axial 0.1274 k
Applied Mx 11.092 k-ft
Applied My 0.0 k-ft
Fc : Allowable 1,256.16 psi

PASS Maximum Shear Stress Ratio = **0.3715** : 1
Load Combination +D+S+H
Location of max.above base 14.250 ft
Applied Design Shear 113.223 psi
Allowable Shear 304.750 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	3.114 k	Bottom along Y-Y	3.114 k
Top along X-X	1.104 k	Bottom along X-X	1.104 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 1.177 in at 7.173 ft above base
for load combination : +D+S+H
Along X-X 0.8737 in at 7.173 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				1.421	1.421	0.127					
+D+L+H				1.421	1.421	0.127					
+D+Lr+H				1.421	1.421	0.127					
+D+S+H				3.114	3.114	0.127					
+D+0.750Lr+0.750L+H				1.421	1.421	0.127					
+D+0.750L+0.750S+H				2.691	2.691	0.127					
+D+0.60W+H	0.663	0.663		1.421	1.421	0.127					
+D+0.70E+H				1.421	1.421	0.127					
+D+0.750Lr+0.750L+0.450W+H	0.497	0.497		1.421	1.421	0.127					
+D+0.750L+0.750S+0.450W+H	0.497	0.497		2.691	2.691	0.127					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG21

Printed: 23 AUG 2018, 11:47AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH2 - Header north west

Maximum Reactions

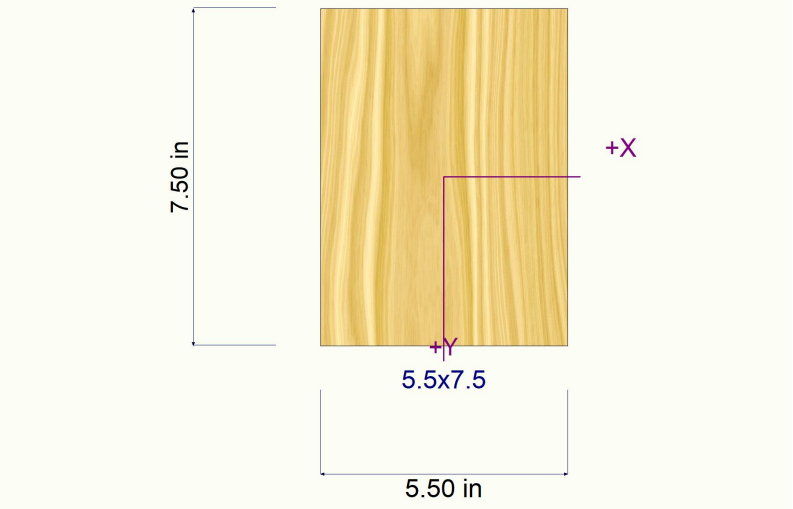
Note: Only non-zero reactions are listed.

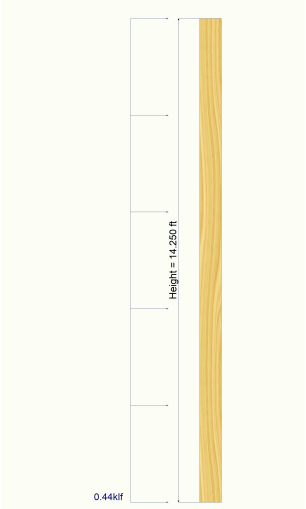
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H				2.691	2.691	0.127					
+0.60D+0.60W+0.60H	0.663	0.663		0.853	0.853	0.076					
+0.60D+0.70E+0.60H				0.853	0.853	0.076					
D Only				1.421	1.421	0.127					
Lr Only											
L Only											
S Only				1.692	1.692						
W Only	1.104	1.104									
E Only											
H Only											

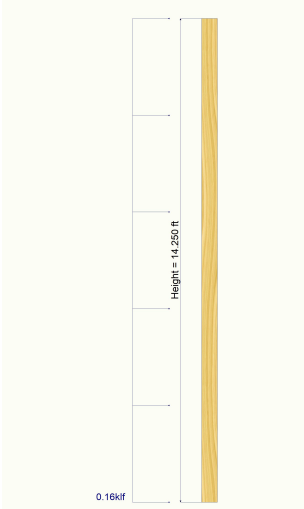
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.538 in	7.173 ft
+D+L+H	0.0000 in	0.000 ft	0.538 in	7.173 ft
+D+Lr+H	0.0000 in	0.000 ft	0.538 in	7.173 ft
+D+S+H	0.0000 in	0.000 ft	1.177 in	7.173 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.538 in	7.173 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	1.017 in	7.173 ft
+D+0.60W+H	0.5242 in	7.173 ft	0.538 in	7.173 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.538 in	7.173 ft
+D+0.750Lr+0.750L+0.450W+H	0.3932 in	7.173 ft	0.538 in	7.173 ft
+D+0.750L+0.750S+0.450W+H	0.3932 in	7.173 ft	1.017 in	7.173 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	1.017 in	7.173 ft
+0.60D+0.60W+0.60H	0.5242 in	7.173 ft	0.323 in	7.173 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.323 in	7.173 ft
D Only	0.0000 in	0.000 ft	0.538 in	7.173 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.640 in	7.173 ft
W Only	0.8737 in	7.173 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG22

Printed: 23 AUG 2018, 11:53AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : RH3 - Header west

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

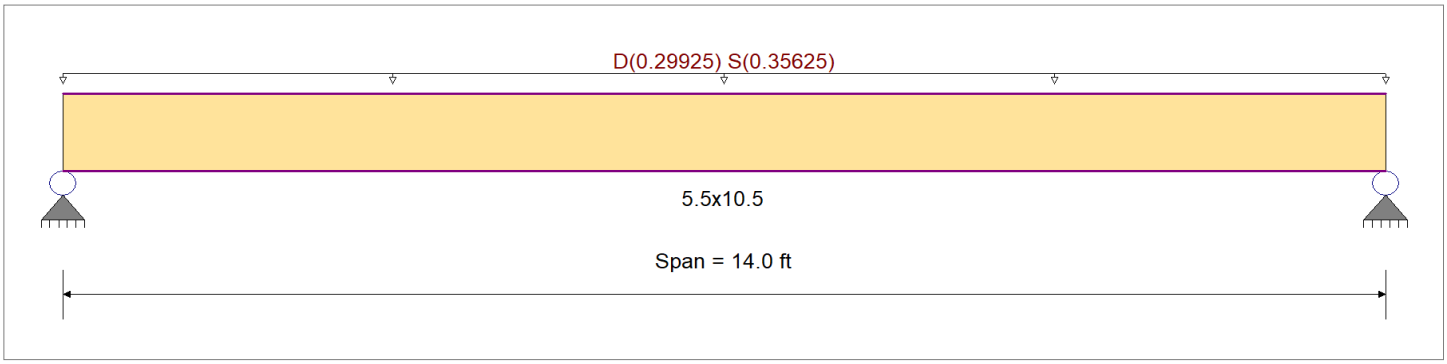
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 14.250 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.704 : 1	Maximum Shear Stress Ratio	=	0.349 : 1
Section used for this span		5.5x10.5	Section used for this span		5.5x10.5
fb : Actual	=	1,943.31 psi	fv : Actual	=	106.39 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	7.000 ft	Location of maximum on span	=	13.131 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.324 in	Ratio =	518	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.608 in	Ratio =	276	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.676	4.676
Overall MINimum	2.494	2.494
+D+H	2.182	2.182
+D+L+H	2.182	2.182
+D+Lr+H	2.182	2.182
+D+S+H	4.676	4.676
+D+0.750Lr+0.750L+H	2.182	2.182
+D+0.750L+0.750S+H	4.053	4.053
+D+0.60W+H	2.182	2.182
+D+0.70E+H	2.182	2.182
+D+0.750Lr+0.750L+0.450W+H	2.182	2.182
+D+0.750L+0.750S+0.450W+H	4.053	4.053

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG23

Printed: 23 AUG 2018, 11:53AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH3 - Header west

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	4.053	4.053
+0.60D+0.60W+0.60H	1.309	1.309
+0.60D+0.70E+0.60H	1.309	1.309
D Only	2.182	2.182
Lr Only		
L Only		
S Only	2.494	2.494
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG24

Printed: 23 AUG 2018, 11:30AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH4 - Header south west

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

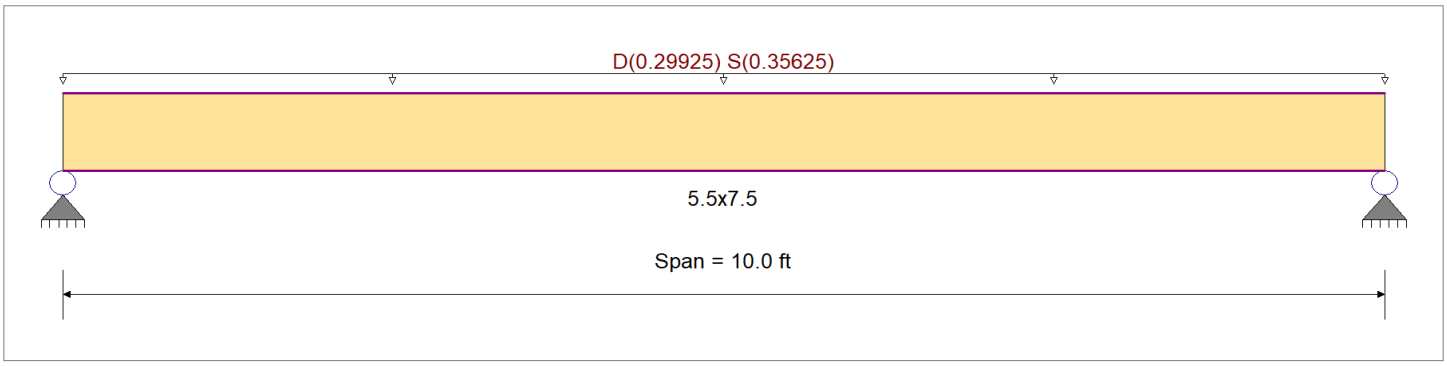
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0210, S = 0.0250 ksf, Tributary Width = 14.250 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.700 : 1	Maximum Shear Stress Ratio	=	0.347 : 1
Section used for this span		5.5x7.5	Section used for this span		5.5x7.5
fb : Actual	=	1,932.91 psi	fv : Actual	=	105.82 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.232 in	Ratio =	518	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.432 in	Ratio =	277	>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.322	3.322
Overall MINimum	1.781	1.781
+D+H	1.541	1.541
+D+L+H	1.541	1.541
+D+Lr+H	1.541	1.541
+D+S+H	3.322	3.322
+D+0.750Lr+0.750L+H	1.541	1.541
+D+0.750L+0.750S+H	2.877	2.877
+D+0.60W+H	1.541	1.541
+D+0.70E+H	1.541	1.541
+D+0.750Lr+0.750L+0.450W+H	1.541	1.541
+D+0.750L+0.750S+0.450W+H	2.877	2.877

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG25

www.dibbleengineers.com

Printed: 23 AUG 2018, 11:30AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH4 - Header south west

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H	2.877	2.877
+0.60D+0.60W+0.60H	0.925	0.925
+0.60D+0.70E+0.60H	0.925	0.925
D Only	1.541	1.541
Lr Only		
L Only		
S Only	1.781	1.781
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG26

Printed: 23 AUG 2018, 11:33AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH5 - Header south, typ

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 6.0 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name **2-2x8**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**3.0**	in
Exact Depth	**7.250**	in
Area	21.750	in^2
Ix	95.270	in^4
Iy	16.313	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 6.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 24.315 lbs * Dead Load Factor
BENDING LOADS . . .
Roof - 4ft trib: Lat. Uniform Load creating Mx-x, D = 0.0840, S = 0.10 k/ft
C&C Wind - 4.25ft trib: Lat. Uniform Load creating My-y, W = 0.1550 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4124** : 1
Load Combination +D+0.750L+0.750S+0.450W+H
Governing NDS Forumlanp + Mxx + Myy, NDS Eq. 3.9-
Location of max.above base 3.020 ft
At maximum location values are . . .
Applied Axial 0.02432 k
Applied Mx 0.7155 k-ft
Applied My 0.3139 k-ft
Fc : Allowable 1,851.90 psi

PASS Maximum Shear Stress Ratio = **0.2207** : 1
Load Combination +D+S+H
Location of max.above base 6.0 ft
Applied Design Shear 38.069 psi
Allowable Shear 172.50 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.5520 k	Bottom along Y-Y	0.5520 k
Top along X-X	0.4650 k	Bottom along X-X	0.4650 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.04379 in at 3.020 ft above base
for load combination : +D+S+H
Along X-X 0.2154 in at 3.020 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				0.252	0.252	0.024					
+D+L+H				0.252	0.252	0.024					
+D+Lr+H				0.252	0.252	0.024					
+D+S+H				0.552	0.552	0.024					
+D+0.750Lr+0.750L+H				0.252	0.252	0.024					
+D+0.750L+0.750S+H				0.477	0.477	0.024					
+D+0.60W+H	0.279	0.279		0.252	0.252	0.024					
+D+0.70E+H				0.252	0.252	0.024					
+D+0.750Lr+0.750L+0.450W+H	0.209	0.209		0.252	0.252	0.024					
+D+0.750L+0.750S+0.450W+H	0.209	0.209		0.477	0.477	0.024					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG27

www.dibbleengineers.com

Printed: 23 AUG 2018, 11:33AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RH5 - Header south, typ

Maximum Reactions

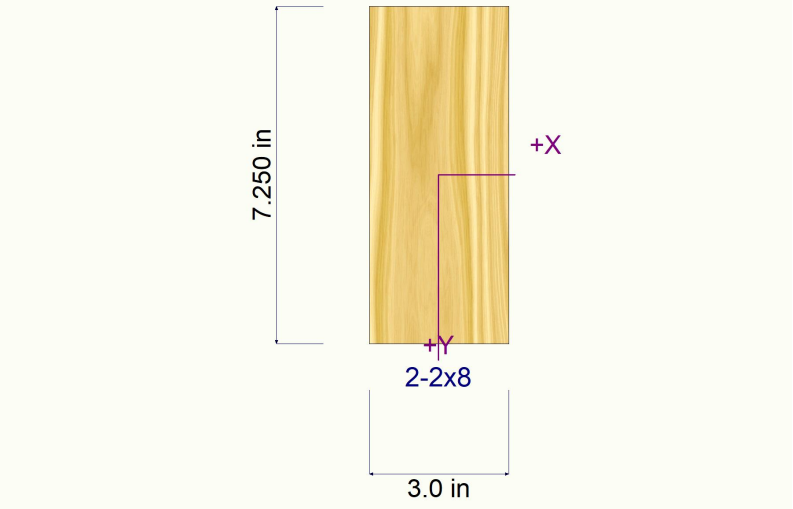
Note: Only non-zero reactions are listed.

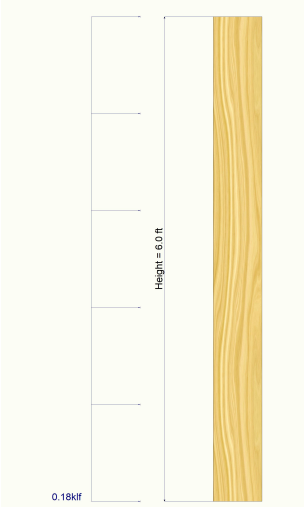
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H				0.477	0.477	0.024					
+0.60D+0.60W+0.60H	0.279	0.279		0.151	0.151	0.015					
+0.60D+0.70E+0.60H				0.151	0.151	0.015					
D Only				0.252	0.252	0.024					
Lr Only											
L Only											
S Only				0.300	0.300						
W Only	0.465	0.465									
E Only											
H Only											

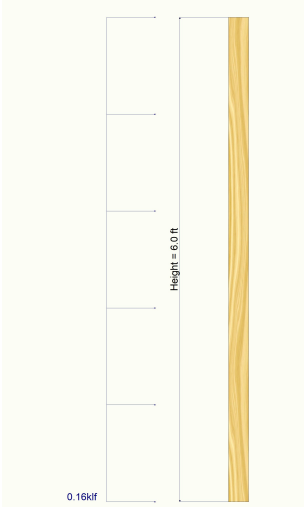
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.020 in	3.020 ft
+D+L+H	0.0000 in	0.000 ft	0.020 in	3.020 ft
+D+Lr+H	0.0000 in	0.000 ft	0.020 in	3.020 ft
+D+S+H	0.0000 in	0.000 ft	0.044 in	3.020 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.020 in	3.020 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.038 in	3.020 ft
+D+0.60W+H	0.1293 in	3.020 ft	0.020 in	3.020 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.020 in	3.020 ft
+D+0.750Lr+0.750L+0.450W+H	0.0969 in	3.020 ft	0.020 in	3.020 ft
+D+0.750L+0.750S+0.450W+H	0.0969 in	3.020 ft	0.038 in	3.020 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.038 in	3.020 ft
+0.60D+0.60W+0.60H	0.1293 in	3.020 ft	0.012 in	3.020 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.012 in	3.020 ft
D Only	0.0000 in	0.000 ft	0.020 in	3.020 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.024 in	3.020 ft
W Only	0.2154 in	3.020 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG28

Printed: 23 AUG 2018, 12:04PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC1 - Column north RB1 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 8.50 ft
(Used for non-slender calculations)

Wood Species : Hem Fir
Wood Grade : No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name : **4x6**
Wood Grading/Manuf. : Graded Lumber
Wood Member Type : Sawn

Exact Width	**3.50**	in
Exact Depth	**5.50**	in
Area	19.250	in^2
Ix	48.526	in^4
Iy	19.651	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 8.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 30.487 lbs * Dead Load Factor
AXIAL LOADS . . .
beam RB1 RXN: Axial Load at 8.50 ft, D = 3.562, S = 4.071 k
BENDING LOADS . . .
C&C Wjnd - 1.33ft trib: Lat. Uniform Load creating Mx-x, W = 0.0490 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9362** : 1
Load Combination: +D+S+H
Governing NDS Forumla: Comp Only, fc/Fc'
Location of max.above base: 0.0 ft
At maximum location values are . . .
Applied Axial: 7.663 k
Applied Mx: 0.0 k-ft
Applied My: 0.0 k-ft
Fc : Allowable: 425.229 psi

PASS Maximum Shear Stress Ratio = **0.04057** : 1
Load Combination: +D+0.60W+H
Location of max.above base: 0.0 ft
Applied Design Shear: 9.736 psi
Allowable Shear: 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.2083 k	Bottom along Y-Y	0.2083 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y: 0.09221 in at 4.279 ft above base
for load combination : W Only
Along X-X: 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						3.592					
+D+L+H						3.592					
+D+Lr+H						3.592					
+D+S+H						7.663					
+D+0.750Lr+0.750L+H						3.592					
+D+0.750L+0.750S+H						6.646					
+D+0.60W+H				0.125	0.125	3.592					
+D+0.70E+H						3.592					
+D+0.750Lr+0.750L+0.450W+H				0.094	0.094	3.592					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG29

Printed: 23 AUG 2018, 12:04PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC1 - Column north RB1 support

Maximum Reactions

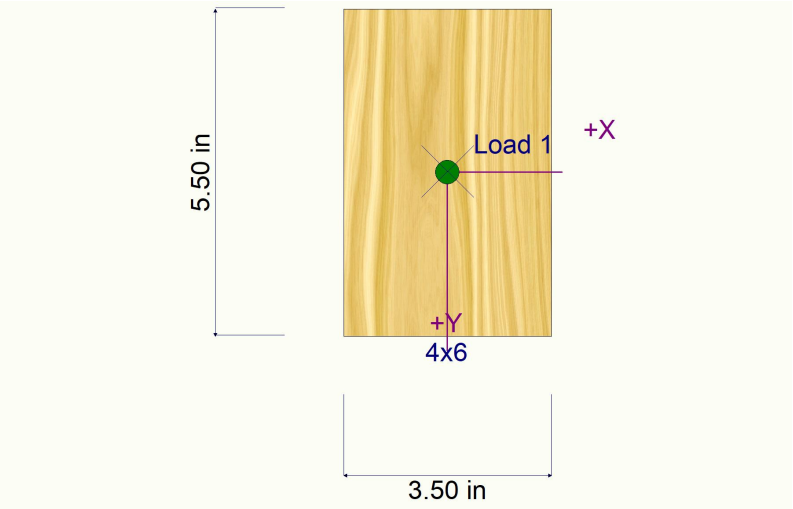
Note: Only non-zero reactions are listed.

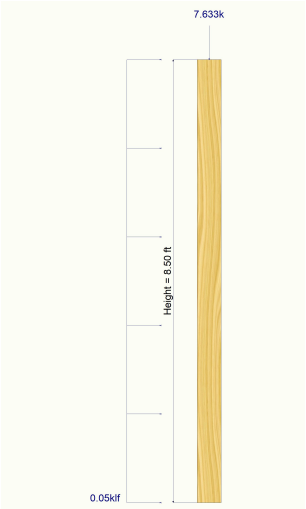
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.450W+H				0.094	0.094	6.646					
+D+0.750L+0.750S+0.5250E+H						6.646					
+0.60D+0.60W+0.60H				0.125	0.125	2.155					
+0.60D+0.70E+0.60H						2.155					
D Only						3.592					
Lr Only											
L Only											
S Only						4.071					
W Only				0.208	0.208						
E Only											
H Only											

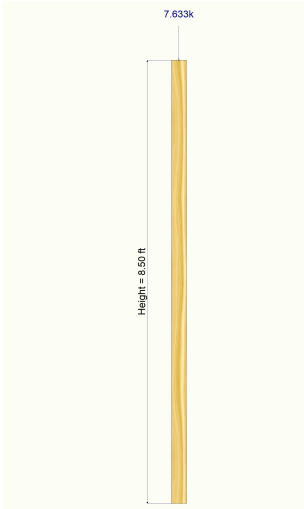
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.055 in	4.279 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.041 in	4.279 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.041 in	4.279 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.055 in	4.279 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.092 in	4.279 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG30

Printed: 23 AUG 2018, 12:04PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC2 - Column RB1, RB2 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Douglas Fir - Larch
Wood Grade No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,600.0	1,600.0	1,600.0	ksi
Minimum	580.0	580.0		

Wood Section Name **6x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width **5.50** in
Exact Depth **5.50** in
Area 30.250 in^2
Ix 76.255 in^4
Iy 76.255 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 55.710 lbs * Dead Load Factor
AXIAL LOADS . . .
beam RB1 RXN: Axial Load at 8.50 ft, D = 6.076, S = 6.756 k
beam RB2 RXN: Axial Load at 8.50 ft, D = 5.314, S = 6.054 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9280** : 1
Load Combination +D+S+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 24.256 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 864.04 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.60D+0.70E+0.60H
Location of max.above base 8.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						11.446					
+D+L+H						11.446					
+D+Lr+H						11.446					
+D+S+H						24.256					
+D+0.750Lr+0.750L+H						11.446					
+D+0.750L+0.750S+H						21.053					
+D+0.60W+H						11.446					
+D+0.70E+H						11.446					
+D+0.750Lr+0.750L+0.450W+H						11.446					
+D+0.750L+0.750S+0.450W+H						21.053					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG31

Printed: 23 AUG 2018, 12:04PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC2 - Column RB1, RB2 support

Maximum Reactions

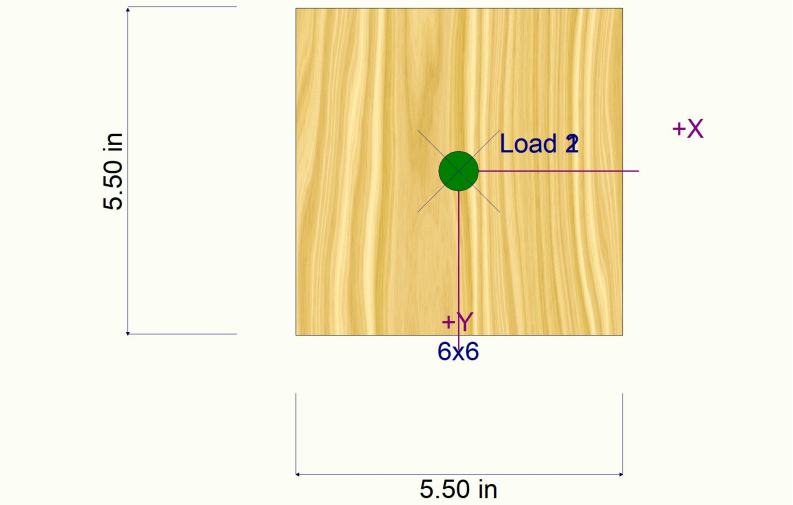
Note: Only non-zero reactions are listed.

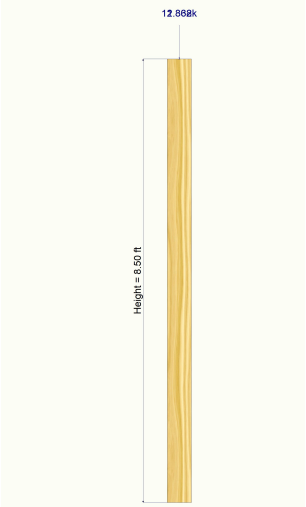
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H						21.053					
+0.60D+0.60W+0.60H						6.867					
+0.60D+0.70E+0.60H						6.867					
D Only						11.446					
Lr Only											
L Only											
S Only						12.810					
W Only											
E Only											
H Only											

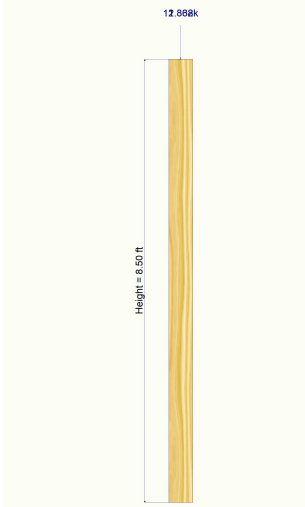
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG32

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC3 - Column RB2, RB3 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Douglas Fir - Larch
Wood Grade No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,600.0	1,600.0	1,600.0	ksi
Minimum	580.0	580.0		

Wood Section Name **6x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**5.50**	in
Exact Depth	**5.50**	in
Area	30.25	in^2
Ix	76.255	in^4
Iy	76.255	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 55.710 lbs * Dead Load Factor
AXIAL LOADS . . .
beam RB3 RXN: Axial Load at 8.50 ft, D = 5.344, S = 5.964 k
beam RB2 RXN: Axial Load at 8.50 ft, D = 5.314, S = 6.054 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8697** : 1
Load Combination +D+S+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 22.732 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 864.04 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.60D+0.70E+0.60H
Location of max.above base 8.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						10.714					
+D+L+H						10.714					
+D+Lr+H						10.714					
+D+S+H						22.732					
+D+0.750Lr+0.750L+H						10.714					
+D+0.750L+0.750S+H						19.727					
+D+0.60W+H						10.714					
+D+0.70E+H						10.714					
+D+0.750Lr+0.750L+0.450W+H						10.714					
+D+0.750L+0.750S+0.450W+H						19.727					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG33

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC3 - Column RB2, RB3 support

Maximum Reactions

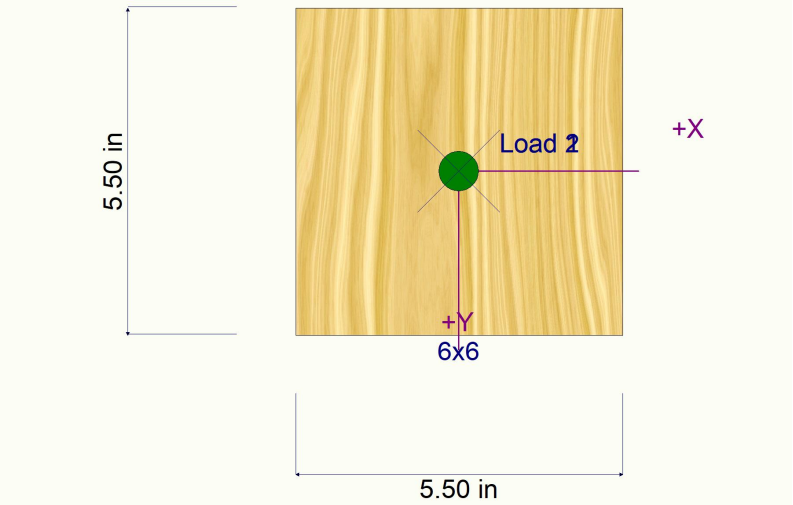
Note: Only non-zero reactions are listed.

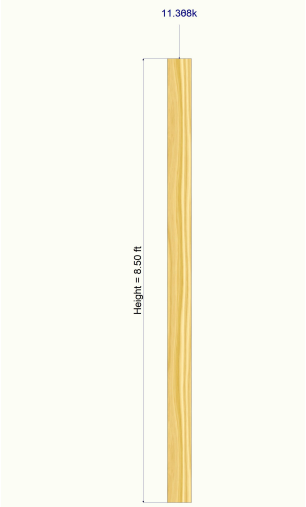
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H						19.727					
+0.60D+0.60W+0.60H						6.428					
+0.60D+0.70E+0.60H						6.428					
D Only						10.714					
Lr Only											
L Only											
S Only						12.018					
W Only											
E Only											
H Only											

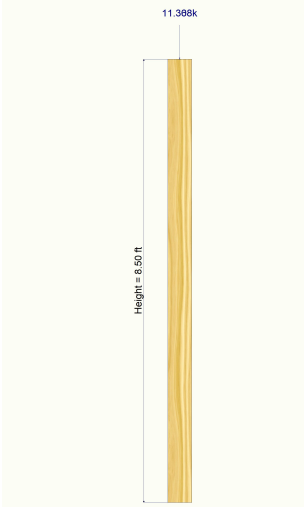
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG34

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC4 - Column RB3, RB4 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Douglas Fir - Larch
Wood Grade No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,600.0	1,600.0	1,600.0	ksi
Minimum	580.0	580.0		

Wood Section Name **6x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**5.50**	in
Exact Depth	**5.50**	in
Area	30.250	in^2
Ix	76.255	in^4
Iy	76.255	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0	
Cf or Cv for Compression	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Factor	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 55.710 lbs * Dead Load Factor
AXIAL LOADS . . .
beam RB3 RXN: Axial Load at 8.50 ft, D = 5.586, S = 6.206 k
beam RB4 RXN: Axial Load at 8.50 ft, D = 3.540, S = 3.825 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7351** : 1
Load Combination +D+S+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 19.213 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 864.04 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.60D+0.70E+0.60H
Location of max.above base 8.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						9.182					
+D+L+H						9.182					
+D+Lr+H						9.182					
+D+S+H						19.213					
+D+0.750Lr+0.750L+H						9.182					
+D+0.750L+0.750S+H						16.705					
+D+0.60W+H						9.182					
+D+0.70E+H						9.182					
+D+0.750Lr+0.750L+0.450W+H						9.182					
+D+0.750L+0.750S+0.450W+H						16.705					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG35

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC4 - Column RB3, RB4 support

Maximum Reactions

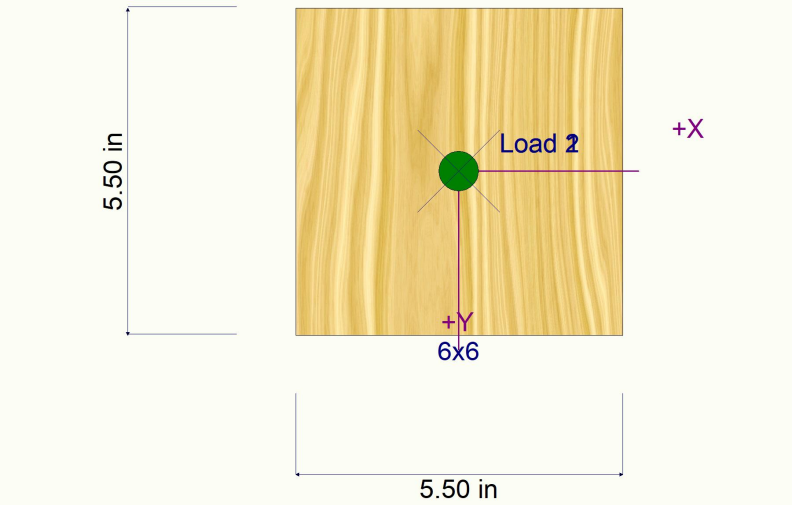
Note: Only non-zero reactions are listed.

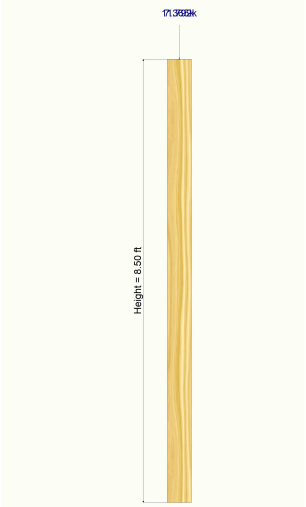
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H						16.705					
+0.60D+0.60W+0.60H						5.509					
+0.60D+0.70E+0.60H						5.509					
D Only						9.182					
Lr Only											
L Only											
S Only						10.031					
W Only											
E Only											
H Only											

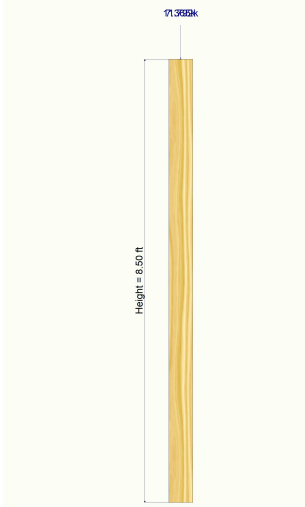
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG36

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC5 - Column RH1 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name **4x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**3.50**	in
Exact Depth	**5.50**	in
Area	19.250	in^2
Ix	48.526	in^4
Iy	19.651	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 8.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 30.487 lbs * Dead Load Factor
AXIAL LOADS . . .
hdr RH1 RXN: Axial Load at 8.50 ft, D = 1.985, S = 2.227 k
BENDING LOADS . . .
C&C Wjnd - 7.67ft trib: Lat. Uniform Load creating Mx-x, W = 0.280 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7310** : 1
Load Combination +D+0.750L+0.750S+0.450W+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 4.221 ft
At maximum location values are . . .
Applied Axial 3.686 k
Applied Mx 1.138 k-ft
Applied My 0.0 k-ft
Fc : Allowable 434.516 psi

PASS Maximum Shear Stress Ratio = **0.2318** : 1
Load Combination +D+0.60W+H
Location of max.above base 0.0 ft
Applied Design Shear 55.636 psi
Allowable Shear 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.190	k	Bottom along Y-Y	1.190	k
Top along X-X	0.0	k	Bottom along X-X	0.0	k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.5269 in at 4.279 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						2.015					
+D+L+H						2.015					
+D+Lr+H						2.015					
+D+S+H						4.242					
+D+0.750Lr+0.750L+H						2.015					
+D+0.750L+0.750S+H						3.686					
+D+0.60W+H				0.714	0.714	2.015					
+D+0.70E+H						2.015					
+D+0.750Lr+0.750L+0.450W+H				0.536	0.535	2.015					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG37

Printed: 23 AUG 2018, 12:05PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC5 - Column RH1 support

Maximum Reactions

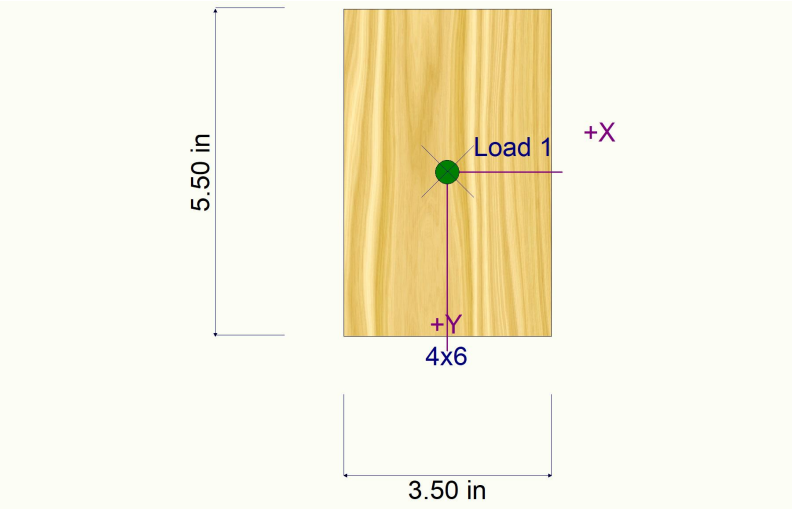
Note: Only non-zero reactions are listed.

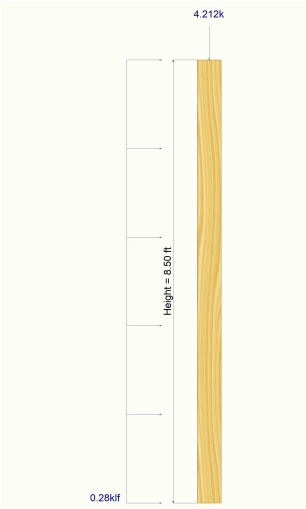
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.450W+H				0.536	0.535	3.686					
+D+0.750L+0.750S+0.5250E+H						3.686					
+0.60D+0.60W+0.60H				0.714	0.714	1.209					
+0.60D+0.70E+0.60H						1.209					
D Only						2.015					
Lr Only											
L Only											
S Only						2.227					
W Only				1.190	1.190						
E Only											
H Only											

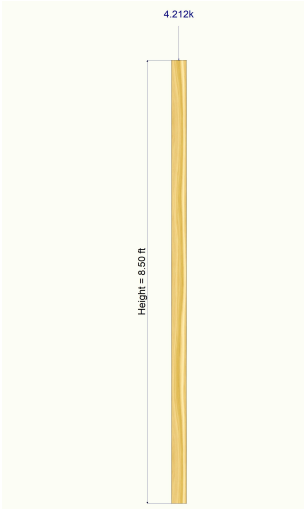
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.316 in	4.279 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.237 in	4.279 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.237 in	4.279 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.316 in	4.279 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.527 in	4.279 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG38

Printed: 23 AUG 2018, 12:06PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC6 - Column RH2 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name **2-2x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**3.0**	in
Exact Depth	**5.50**	in
Area	16.50	in^2
Ix	41.594	in^4
Iy	12.375	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 8.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 26.131 lbs * Dead Load Factor
AXIAL LOADS . . .
hdr RH2 RXN: Axial Load at 8.50 ft, D = 1.421, S = 1.692 k
Header north: Axial Load at 8.50 ft, D = 0.1960, S = 0.2333 k
BENDING LOADS . . .
C&C Wjnd - 5.67ft trib: Lat. Uniform Load creating Mx-x, W = 0.2070 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7995** : 1
Load Combination +D+0.750L+0.750S+0.450W+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 4.279 ft
At maximum location values are . . .
Applied Axial 3.087 k
Applied Mx 0.8412 k-ft
Applied My 0.0 k-ft
Fc : Allowable 323.547 psi

PASS Maximum Shear Stress Ratio = **0.1999** : 1
Load Combination +D+0.60W+H
Location of max.above base 0.0 ft
Applied Design Shear 47.986 psi
Allowable Shear 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.8798 k	Bottom along Y-Y	0.8798 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.4545 in at 4.279 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
	k				k-ft				
+D+H					1.643				
+D+L+H					1.643				
+D+Lr+H					1.643				
+D+S+H					3.568				
+D+0.750Lr+0.750L+H					1.643				
+D+0.750L+0.750S+H					3.087				
+D+0.60W+H			0.528	0.528	1.643				
+D+0.70E+H					1.643				

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG39

Printed: 23 AUG 2018, 12:06PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC6 - Column RH2 support

Maximum Reactions

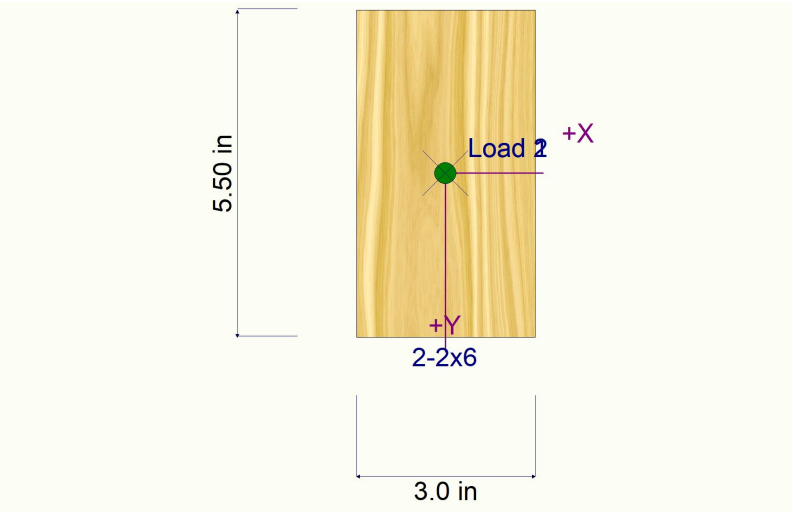
Note: Only non-zero reactions are listed.

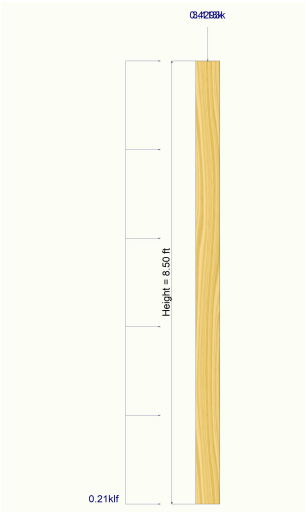
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750Lr+0.750L+0.450W+H				0.396	0.396	1.643					
+D+0.750L+0.750S+0.450W+H				0.396	0.396	3.087					
+D+0.750L+0.750S+0.5250E+H						3.087					
+0.60D+0.60W+0.60H				0.528	0.528	0.986					
+0.60D+0.70E+0.60H						0.986					
D Only						1.643					
Lr Only											
L Only											
S Only						1.925					
W Only				0.880	0.880						
E Only											
H Only											

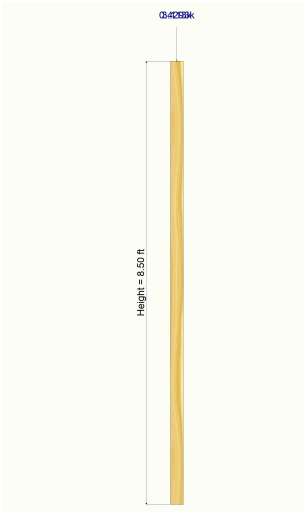
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.273 in	4.279 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.205 in	4.279 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.205 in	4.279 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.273 in	4.279 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.454 in	4.279 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG40

Printed: 23 AUG 2018, 12:06PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC7 - Column RH3 support

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 8.50 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name **2-2x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**3.0**	in
Exact Depth	**5.50**	in
Area	16.50	in^2
Ix	41.594	in^4
Iy	12.375	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8.50 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 8.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 26.131 lbs * Dead Load Factor
AXIAL LOADS . . .
hdr RH3 RXN: Axial Load at 8.50 ft, D = 2.182, S = 2.494 k
BENDING LOADS . . .
C&C Wjnd - 1.33ft trib: Lat. Uniform Load creating Mx-x, W = 0.0490 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8937** : 1
Load Combination +D+S+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 4.702 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 318.864 psi

PASS Maximum Shear Stress Ratio = **0.04733** : 1
Load Combination +D+0.60W+H
Location of max.above base 0.0 ft
Applied Design Shear 11.359 psi
Allowable Shear 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.2083	k	Bottom along Y-Y	0.2083	k
Top along X-X	0.0	k	Bottom along X-X	0.0	k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.1076 in at 4.279 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						2.208					
+D+L+H						2.208					
+D+Lr+H						2.208					
+D+S+H						4.702					
+D+0.750Lr+0.750L+H						2.208					
+D+0.750L+0.750S+H						4.079					
+D+0.60W+H				0.125	0.125	2.208					
+D+0.70E+H						2.208					
+D+0.750Lr+0.750L+0.450W+H				0.094	0.094	2.208					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG41

Printed: 23 AUG 2018, 12:06PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : RC7 - Column RH3 support

Maximum Reactions

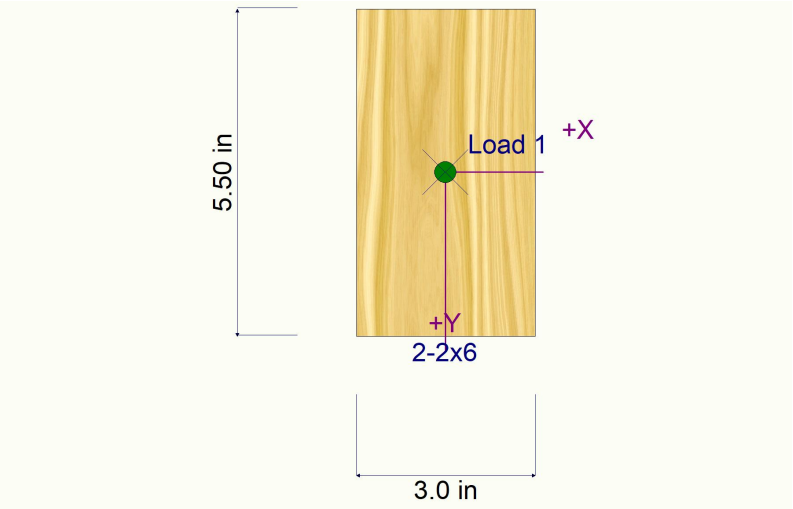
Note: Only non-zero reactions are listed.

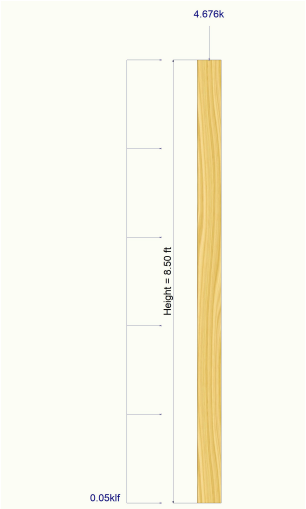
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.450W+H				0.094	0.094	4.079					
+D+0.750L+0.750S+0.5250E+H						4.079					
+0.60D+0.60W+0.60H				0.125	0.125	1.325					
+0.60D+0.70E+0.60H						1.325					
D Only						2.208					
Lr Only											
L Only											
S Only						2.494					
W Only				0.208	0.208						
E Only											
H Only											

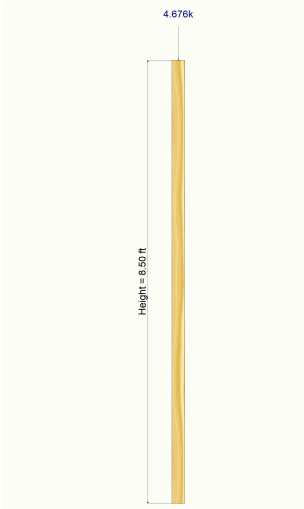
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.065 in	4.279 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.048 in	4.279 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.048 in	4.279 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.065 in	4.279 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.108 in	4.279 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

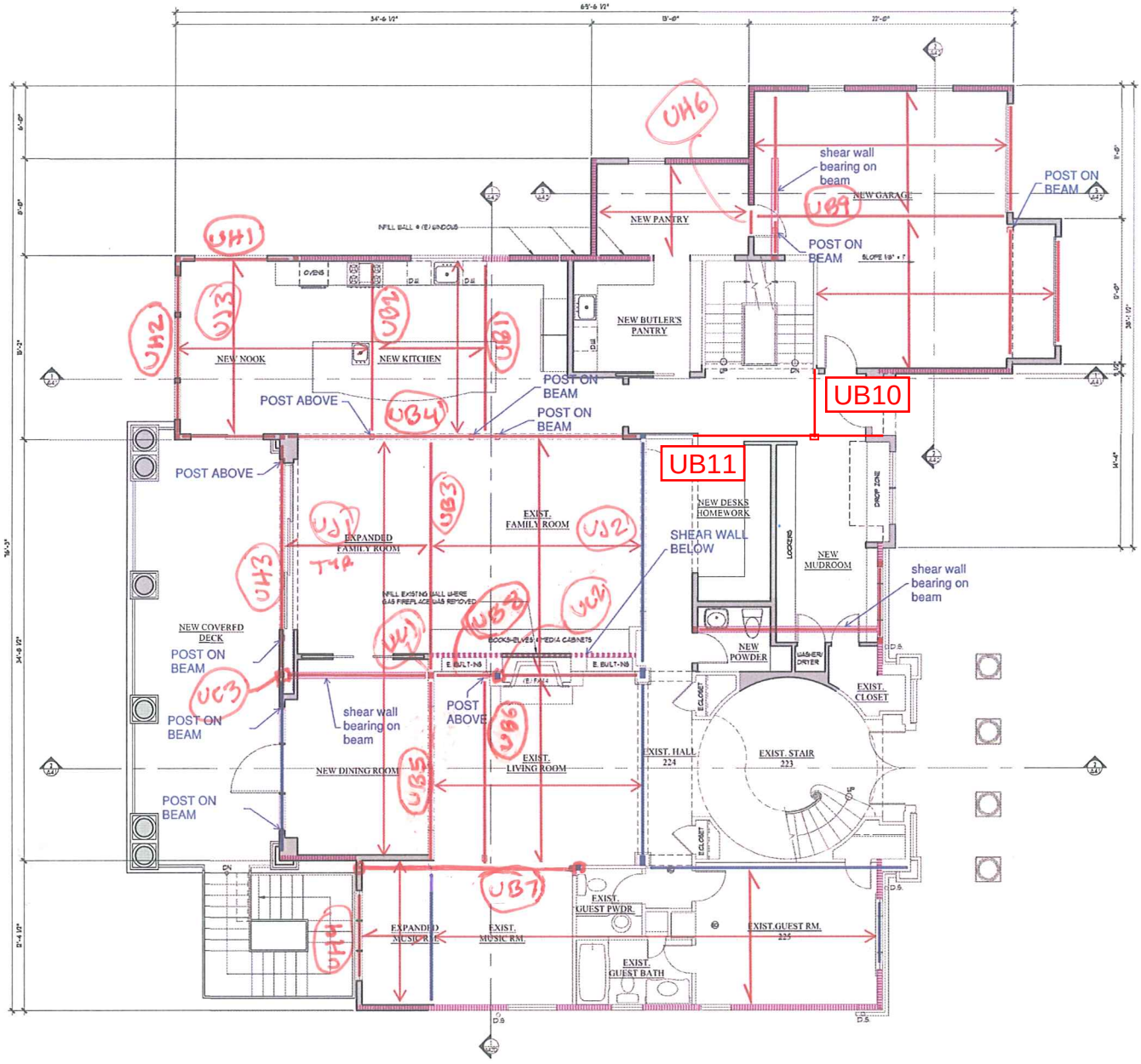
Sketches







GRAVITY KEY PLAN - UPPER FLOOR KEY PLAN



UPPER FLOOR FRAMING AND SHEAR WALLS BELOW

1 PROPOSED MAIN FLOOR PLAN
SCALE: 1/4" = 1'-0"

0 1 2 4 8 12 FT.

NORTH

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG43

Printed: 23 AUG 2018, 12:27PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB4 - Beam steel

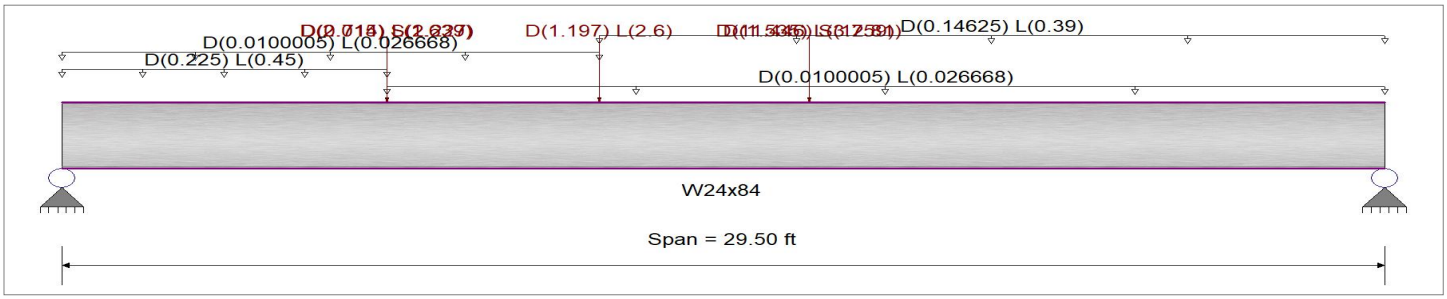
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 7.250 -->> 29.50 ft, Tributary Width = 0.6667 ft, (Floor)

Uniform Load : D = 0.030, L = 0.060 ksf, Extent = 0.0 -->> 7.250 ft, Tributary Width = 7.50 ft, (Deck)

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 0.0 -->> 12.0 ft, Tributary Width = 0.6667 ft, (Floor)

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 12.0 -->> 29.50 ft, Tributary Width = 9.750 ft, (Floor)

Point Load : D = 1.535, L = 3.759 k @ 16.667 ft, (UB1 Bm RXN)

Point Load : D = 0.7140, L = 1.639 k @ 7.250 ft, (UB2 Bm RXN)

Point Load : D = 1.197, L = 2.60 k @ 12.0 ft, (UB3 Bm RXN)

Point Load : D = 11.446, S = 12.810 k @ 16.667 ft, (RC2 Post Abv)

Point Load : D = 2.015, S = 2.227 k @ 7.250 ft, (RC5 Post Abv)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.484 : 1	Maximum Shear Stress Ratio =	0.109 : 1
Section used for this span	W24x84	Section used for this span	W24x84
Ma : Applied	270.425 k-ft	Va : Applied	24.798 k
Mn / Omega : Allowable	558.882 k-ft	Vn/Omega : Allowable	226.540 k
Load Combination	+D+0.750L+0.750S+H	Load Combination	+D+0.750L+0.750S+H
Location of maximum on span	16.689 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.189 in	Ratio =	1,869	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.547 in	Ratio =	647	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Maximum Forces & Stresses for Load Combinations

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG44

Printed: 23 AUG 2018, 12:27PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB4 - Beam steel

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 29.50 ft	1	0.237	0.053	132.68		132.68	933.33	558.88	1.00	1.00	12.02	339.81	226.54
+D+L+H													
Dsgn. L = 29.50 ft	1	0.388	0.096	216.63		216.63	933.33	558.88	1.00	1.00	21.80	339.81	226.54
+D+Lr+H													
Dsgn. L = 29.50 ft	1	0.237	0.053	132.68		132.68	933.33	558.88	1.00	1.00	12.02	339.81	226.54
+D+S+H													
Dsgn. L = 29.50 ft	1	0.416	0.087	232.41		232.41	933.33	558.88	1.00	1.00	19.68	339.81	226.54
+D+0.750Lr+0.750L+H													
Dsgn. L = 29.50 ft	1	0.350	0.085	195.62		195.62	933.33	558.88	1.00	1.00	19.36	339.81	226.54
+D+0.750L+0.750S+H													
Dsgn. L = 29.50 ft	1	0.484	0.109	270.43		270.43	933.33	558.88	1.00	1.00	24.80	339.81	226.54
+D+0.60W+H													
Dsgn. L = 29.50 ft	1	0.237	0.053	132.68		132.68	933.33	558.88	1.00	1.00	12.02	339.81	226.54
+D+0.70E+H													
Dsgn. L = 29.50 ft	1	0.237	0.053	132.68		132.68	933.33	558.88	1.00	1.00	12.02	339.81	226.54
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 29.50 ft	1	0.350	0.085	195.62		195.62	933.33	558.88	1.00	1.00	19.36	339.81	226.54
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 29.50 ft	1	0.484	0.109	270.43		270.43	933.33	558.88	1.00	1.00	24.80	339.81	226.54
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 29.50 ft	1	0.484	0.109	270.43		270.43	933.33	558.88	1.00	1.00	24.80	339.81	226.54
+0.60D+0.60W+0.60H													
Dsgn. L = 29.50 ft	1	0.142	0.032	79.61		79.61	933.33	558.88	1.00	1.00	7.21	339.81	226.54
+0.60D+0.70E+0.60H													
Dsgn. L = 29.50 ft	1	0.142	0.032	79.61		79.61	933.33	558.88	1.00	1.00	7.21	339.81	226.54

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E+H	1	0.5471	15.003		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	24.798	24.649
Overall MINimum	7.215	7.137
+D+H	12.025	11.896
+D+L+H	21.803	21.116
+D+Lr+H	12.025	11.896
+D+S+H	19.277	19.680
+D+0.750Lr+0.750L+H	19.359	18.811
+D+0.750L+0.750S+H	24.798	24.649
+D+0.60W+H	12.025	11.896
+D+0.70E+H	12.025	11.896
+D+0.750Lr+0.750L+0.450W+H	19.359	18.811
+D+0.750L+0.750S+0.450W+H	24.798	24.649
+D+0.750L+0.750S+0.5250E+H	24.798	24.649
+0.60D+0.60W+0.60H	7.215	7.137
+0.60D+0.70E+0.60H	7.215	7.137
D Only	12.025	11.896
Lr Only		
L Only	9.778	9.220
S Only	7.252	7.785
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title: SG45
Engineer:
Project ID:
Project Descr:

Printed: 23 AUG 2018, 12:30PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : UB6 - Beam center

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

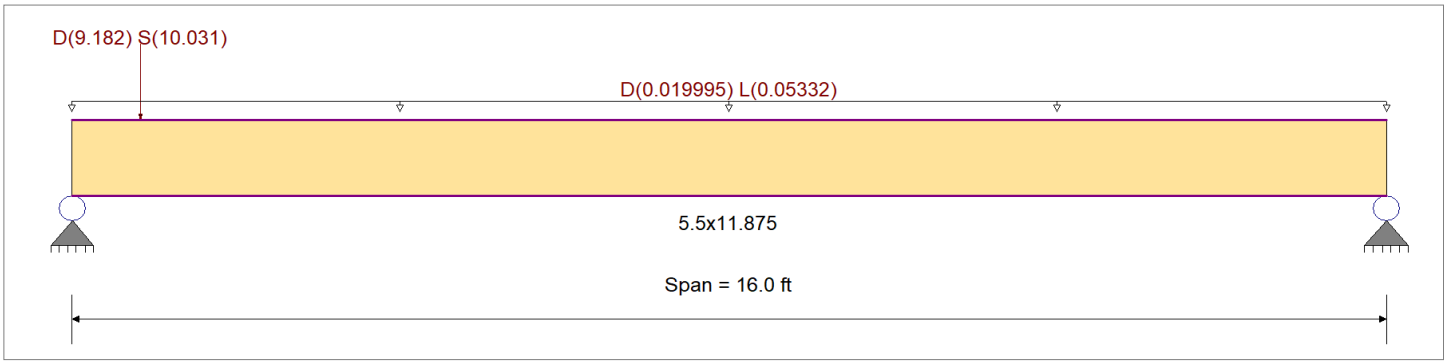
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft, (Floor)
Point Load : D = 9.182, S = 10.031 k @ 0.8333 ft, (RC4 Post RXN)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.517 : 1	Maximum Shear Stress Ratio	=	0.105 : 1
Section used for this span		5.5x11.875	Section used for this span		5.5x11.875
fb : Actual	=	1,425.91 psi	fv : Actual	=	32.01 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+0.750L+0.750S+H
Location of maximum on span	=	0.876 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.173 in	Ratio =	1112	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.366 in	Ratio =	524	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	18.486	1.463
Overall MINimum	9.509	0.522
+D+H	8.977	0.751
+D+L+H	9.404	1.178
+D+Lr+H	8.977	0.751
+D+S+H	18.486	1.274
+D+0.750Lr+0.750L+H	9.297	1.071
+D+0.750L+0.750S+H	16.428	1.463
+D+0.60W+H	8.977	0.751
+D+0.70E+H	8.977	0.751
+D+0.750Lr+0.750L+0.450W+H	9.297	1.071

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG46

Printed: 23 AUG 2018, 12:30PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB6 - Beam center

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W+H	16.428	1.463
+D+0.750L+0.750S+0.5250E+H	16.428	1.463
+0.60D+0.60W+0.60H	5.386	0.451
+0.60D+0.70E+0.60H	5.386	0.451
D Only	8.977	0.751
Lr Only		
L Only	0.427	0.427
S Only	9.509	0.522
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG47

Printed: 23 AUG 2018, 12:36PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB7 - Beam steel

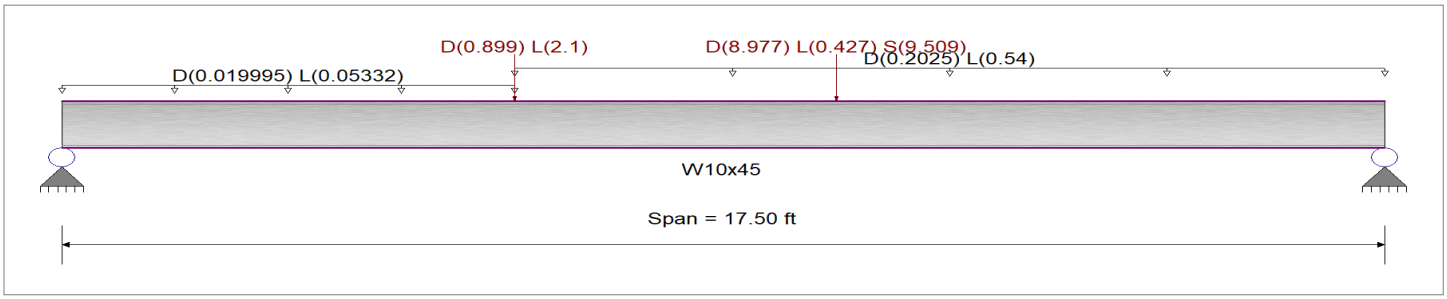
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 0.0 -->> 6.0 ft, Tributary Width = 1.333 ft, (Floor)

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 6.0 -->> 17.50 ft, Tributary Width = 13.50 ft, (Floor)

Point Load : D = 8.977, L = 0.4270, S = 9.509 k @ 10.250 ft, (UB6 Beam RXN)

Point Load : D = 0.8990, L = 2.10 k @ 6.0 ft, (UB5 Beam RXN)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.701 : 1	Maximum Shear Stress Ratio =	0.221 : 1
Section used for this span	W10x45	Section used for this span	W10x45
Ma : Applied	96.086 k-ft	Va : Applied	15.620 k
Mn / Omega : Allowable	136.976 k-ft	Vn/Omega : Allowable	70.70 k
Load Combination	+D+0.750L+0.750S+H	Load Combination	+D+0.750L+0.750S+H
Location of maximum on span	10.250 ft	Location of maximum on span	17.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.246 in	Ratio =	852	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.633 in	Ratio =	332	<360.0
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 17.50 ft	1	0.352	0.107	48.20		48.20	228.75	136.98	1.00	1.00	7.55	106.05	70.70
+D+L+H													
Dsgn. L = 17.50 ft	1	0.523	0.180	71.67		71.67	228.75	136.98	1.00	1.00	12.74	106.05	70.70
+D+Lr+H													
Dsgn. L = 17.50 ft	1	0.352	0.107	48.20		48.20	228.75	136.98	1.00	1.00	7.55	106.05	70.70
+D+S+H													
Dsgn. L = 17.50 ft	1	0.647	0.186	88.58		88.58	228.75	136.98	1.00	1.00	13.12	106.05	70.70
+D+0.750Lr+0.750L+H													
Dsgn. L = 17.50 ft	1	0.480	0.162	65.80		65.80	228.75	136.98	1.00	1.00	11.44	106.05	70.70
+D+0.750L+0.750S+H													
Dsgn. L = 17.50 ft	1	0.701	0.221	96.09		96.09	228.75	136.98	1.00	1.00	15.62	106.05	70.70
+D+0.60W+H													
Dsgn. L = 17.50 ft	1	0.352	0.107	48.20		48.20	228.75	136.98	1.00	1.00	7.55	106.05	70.70

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG48

www.dibbleengineers.com

Printed: 23 AUG 2018, 12:36PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB7 - Beam steel

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+0.70E+H													
Dsgn. L = 17.50 ft	1	0.352	0.107	48.20		48.20	228.75	136.98	1.00	1.00	7.55	106.05	70.70
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 17.50 ft	1	0.480	0.162	65.80		65.80	228.75	136.98	1.00	1.00	11.44	106.05	70.70
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 17.50 ft	1	0.701	0.221	96.09		96.09	228.75	136.98	1.00	1.00	15.62	106.05	70.70
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 17.50 ft	1	0.701	0.221	96.09		96.09	228.75	136.98	1.00	1.00	15.62	106.05	70.70
+0.60D+0.60W+0.60H													
Dsgn. L = 17.50 ft	1	0.211	0.064	28.92		28.92	228.75	136.98	1.00	1.00	4.53	106.05	70.70
+0.60D+0.70E+0.60H													
Dsgn. L = 17.50 ft	1	0.211	0.064	28.92		28.92	228.75	136.98	1.00	1.00	4.53	106.05	70.70

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E+H	1	0.6334	9.100		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.422	15.620
Overall MINimum	3.342	4.528
+D+H	5.571	7.546
+D+L+H	9.433	12.741
+D+Lr+H	5.571	7.546
+D+S+H	9.510	13.116
+D+0.750Lr+0.750L+H	8.467	11.442
+D+0.750L+0.750S+H	11.422	15.620
+D+0.60W+H	5.571	7.546
+D+0.70E+H	5.571	7.546
+D+0.750Lr+0.750L+0.450W+H	8.467	11.442
+D+0.750L+0.750S+0.450W+H	11.422	15.620
+D+0.750L+0.750S+0.5250E+H	11.422	15.620
+0.60D+0.60W+0.60H	3.342	4.528
+0.60D+0.70E+0.60H	3.342	4.528
D Only	5.571	7.546
Lr Only		
L Only	3.862	5.195
S Only	3.939	5.570
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG49

Printed: 23 AUG 2018, 12:38PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB8 - Beam center

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

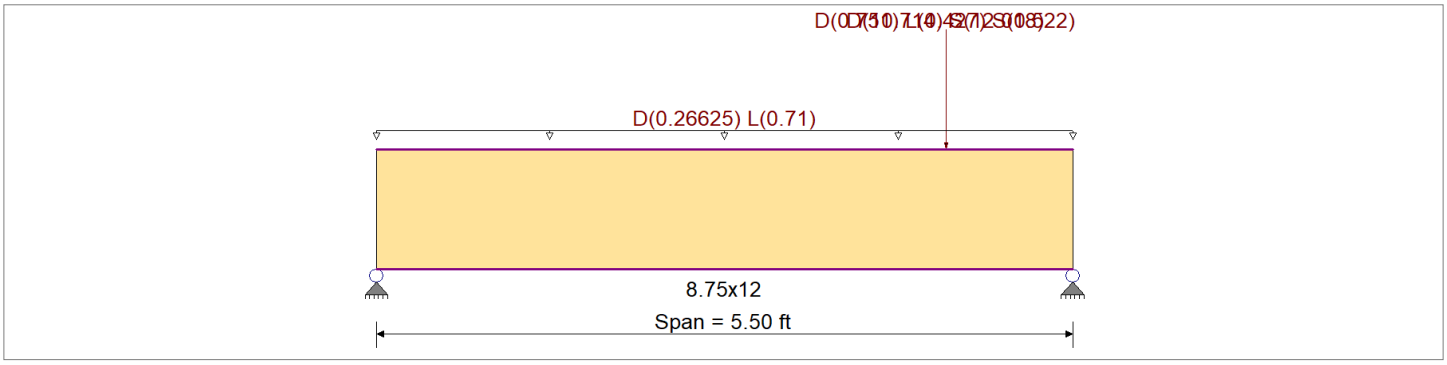
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 17.750 ft, (Floor)
Point Load : D = 10.714, S = 12.018 k @ 4.50 ft, (RC3 Post RXN)
Point Load : D = 0.7510, L = 0.4270, S = 0.5220 k @ 4.50 ft, (UB6 Beam RXN)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.420 : 1	Maximum Shear Stress Ratio	=	0.945 : 1
Section used for this span		8.75x12	Section used for this span		8.75x12
fb : Actual	=	1,158.66 psi	fv : Actual	=	287.87 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	4.496 ft	Location of maximum on span	=	4.516 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.018 in	Ratio =	3725	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.037 in	Ratio =	1769	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.112	20.435
Overall MINimum	2.280	10.260
+D+H	2.879	10.175
+D+L+H	4.909	12.477
+D+Lr+H	2.879	10.175
+D+S+H	5.159	20.435
+D+0.750Lr+0.750L+H	4.402	11.902
+D+0.750L+0.750S+H	6.112	19.597
+D+0.60W+H	2.879	10.175
+D+0.70E+H	2.879	10.175

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG50

www.dibbleengineers.com

Printed: 23 AUG 2018, 12:38PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB8 - Beam center

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+0.450W+H	4.402	11.902
+D+0.750L+0.750S+0.450W+H	6.112	19.597
+D+0.750L+0.750S+0.5250E+H	6.112	19.597
+0.60D+0.60W+0.60H	1.728	6.105
+0.60D+0.70E+0.60H	1.728	6.105
D Only	2.879	10.175
Lr Only		
L Only	2.030	2.302
S Only	2.280	10.260
W Only		
E Only		
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG51

Printed: 23 AUG 2018, 12:42PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB9 - Beam north east

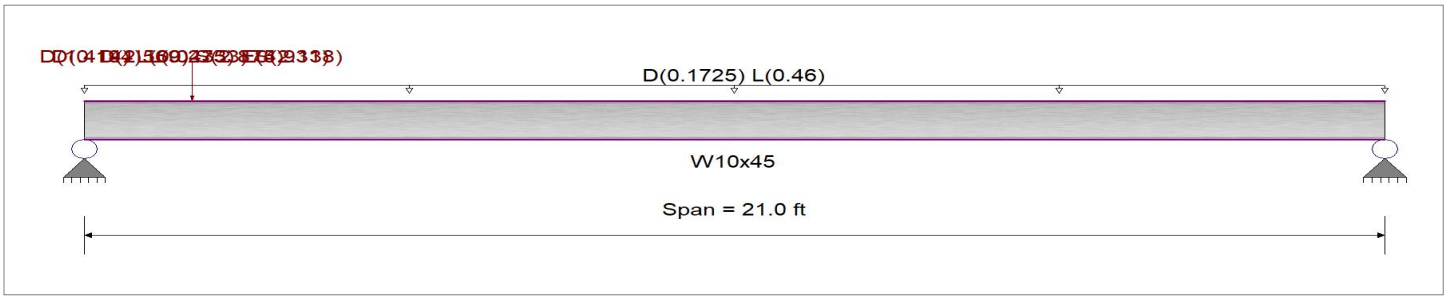
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 11.50 ft, (Floor)

Point Load : D = 0.1940, L = 0.2750, E = 4.931 k @ 1.750 ft, (Beam north RXN w/ SW HD)

Point Load : D = 1.414, L = 0.04333, S = 2.138 k @ 1.750 ft, (Beam south RXN)

Point Load : D = 2.569, S = 2.875 k @ 1.750 ft, (RB5 ridge bm rxn abv)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.302 : 1	Maximum Shear Stress Ratio =	0.223 : 1
Section used for this span	W10x45	Section used for this span	W10x45
Ma : Applied	41.399 k-ft	Va : Applied	15.776 k
Mn / Omega : Allowable	136.976 k-ft	Vn/Omega : Allowable	70.70 k
Load Combination	+D+L+H	Load Combination	+D+0.750L+0.750S+0.5250E+H
Location of maximum on span	9.960 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.285 in	Ratio =	884	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.469 in	Ratio =	537	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 21.00 ft	1	0.116	0.086	15.94		15.94	228.75	136.98	1.00	1.00	6.12	106.05	70.70
+D+L+H													
Dsgn. L = 21.00 ft	1	0.302	0.159	41.40		41.40	228.75	136.98	1.00	1.00	11.24	106.05	70.70
+D+Lr+H													
Dsgn. L = 21.00 ft	1	0.116	0.086	15.94		15.94	228.75	136.98	1.00	1.00	6.12	106.05	70.70
+D+S+H													
Dsgn. L = 21.00 ft	1	0.156	0.151	21.39		21.39	228.75	136.98	1.00	1.00	10.71	106.05	70.70
+D+0.750Lr+0.750L+H													
Dsgn. L = 21.00 ft	1	0.256	0.141	35.01		35.01	228.75	136.98	1.00	1.00	9.96	106.05	70.70
+D+0.750L+0.750S+H													
Dsgn. L = 21.00 ft	1	0.282	0.190	38.59		38.59	228.75	136.98	1.00	1.00	13.40	106.05	70.70
+D+0.60W+H													
Dsgn. L = 21.00 ft	1	0.116	0.086	15.94		15.94	228.75	136.98	1.00	1.00	6.12	106.05	70.70
+D+0.70E+H													

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG52

www.dibbleengineers.com

Printed: 23 AUG 2018, 12:42PM

Steel Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB9 - Beam north east

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 21.00 ft +D+0.750Lr+0.750L+0.450W+H	1	0.143	0.131	19.61		19.61	228.75	136.98	1.00	1.00	9.28	106.05	70.70
Dsgn. L = 21.00 ft +D+0.750L+0.750S+0.450W+H	1	0.256	0.141	35.01		35.01	228.75	136.98	1.00	1.00	9.96	106.05	70.70
Dsgn. L = 21.00 ft +D+0.750L+0.750S+0.5250E+H	1	0.282	0.190	38.59		38.59	228.75	136.98	1.00	1.00	13.40	106.05	70.70
Dsgn. L = 21.00 ft +0.60D+0.60W+0.60H	1	0.300	0.223	41.16		41.16	228.75	136.98	1.00	1.00	15.78	106.05	70.70
Dsgn. L = 21.00 ft +0.60D+0.70E+0.60H	1	0.070	0.052	9.56		9.56	228.75	136.98	1.00	1.00	3.67	106.05	70.70
Dsgn. L = 21.00 ft	1	0.098	0.097	13.36		13.36	228.75	136.98	1.00	1.00	6.83	106.05	70.70

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E+H	1	0.4692	10.140		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	15.776	7.491
Overall MINimum	3.669	0.411
+D+H	6.116	2.635
+D+L+H	11.237	7.491
+D+Lr+H	6.116	2.635
+D+S+H	10.711	3.052
+D+0.750Lr+0.750L+H	9.957	6.277
+D+0.750L+0.750S+H	13.403	6.590
+D+0.60W+H	6.116	2.635
+D+0.70E+H	9.280	2.922
+D+0.750Lr+0.750L+0.450W+H	9.957	6.277
+D+0.750L+0.750S+0.450W+H	13.403	6.590
+D+0.750L+0.750S+0.5250E+H	15.776	6.806
+0.60D+0.60W+0.60H	3.669	1.581
+0.60D+0.70E+0.60H	6.833	1.868
D Only	6.116	2.635
Lr Only		
L Only	5.122	4.857
S Only	4.595	0.418
W Only		
E Only	4.520	0.411
H Only		

Project Title:
Engineer:
Project ID:
Project Descr:

SG53

Printed: 16 MAY 2019, 9:37AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB10 - PSL Pickup Roof Above along Grid 9

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

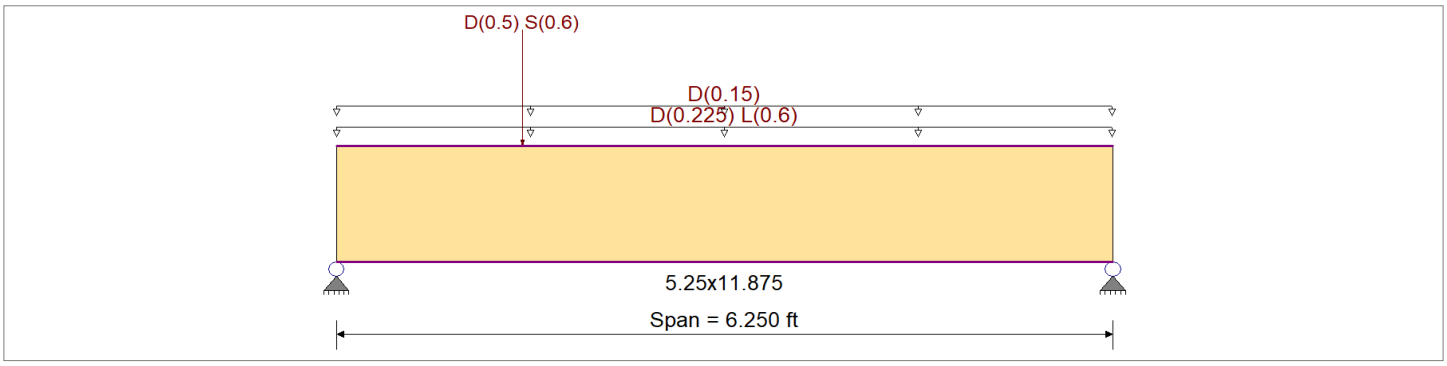
Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,900.0 psi
Fb -	2,900.0 psi
Fc - Prll	2,900.0 psi
Fc - Perp	625.0 psi
Fv	290.0 psi
Ft	2,025.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,016.54 ksi
Density	45.070 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 15.0 ft, (Floor)
Point Load : D = 0.50, S = 0.60 k @ 1.50 ft, (RB6 Beam Rxn)
Uniform Load : D = 0.0150 ksf, Tributary Width = 10.0 ft, (Wall Above)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.176 : 1	Maximum Shear Stress Ratio	=	0.208 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	509.44 psi	fv : Actual	=	60.45 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.011 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.014 in	Ratio =	5303	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.025 in	Ratio =	2947	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.086	0.113	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.32	225.59	2610.00	1.23	29.50	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.176	0.208	1.00	1.000	1.00	1.00	1.00	1.00	1.00	5.24	509.44	2900.00	2.51	60.45	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.062	0.081	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.32	225.59	3625.00	1.23	29.50	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.083	0.121	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.84	276.17	3335.00	1.68	40.47	333.50

Project Title:
Engineer:
Project ID:
Project Descr:

SG54

Printed: 16 MAY 2019, 9:37AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB10 - PSL Pickup Roof Above along Grid 9

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.121	0.145	1.25	1.000	1.00	1.00	1.00	1.00	1.00	4.51	438.33	3625.00	2.19	52.71	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.142	0.183	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.87	473.32	3335.00	2.53	60.94	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.049	0.064	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.32	225.59	4640.00	1.23	29.50	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.049	0.064	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.32	225.59	4640.00	1.23	29.50	464.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.094	0.114	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.51	438.33	4640.00	2.19	52.71	464.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.102	0.131	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.87	473.32	4640.00	2.53	60.94	464.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.102	0.131	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.87	473.32	4640.00	2.53	60.94	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.029	0.038	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.39	135.35	4640.00	0.74	17.70	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.250 ft	1	0.029	0.038	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.39	135.35	4640.00	0.74	17.70	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0254	3.102		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.488	3.228
Overall MINimum	0.456	0.144
+D+H	1.613	1.353
+D+L+H	3.488	3.228
+D+Lr+H	1.613	1.353
+D+S+H	2.069	1.497
+D+0.750Lr+0.750L+H	3.019	2.759
+D+0.750L+0.750S+H	3.361	2.867
+D+0.60W+H	1.613	1.353
+D+0.70E+H	1.613	1.353
+D+0.750Lr+0.750L+0.450W+H	3.019	2.759
+D+0.750L+0.750S+0.450W+H	3.361	2.867
+D+0.750L+0.750S+0.5250E+H	3.361	2.867
+0.60D+0.60W+0.60H	0.968	0.812
+0.60D+0.70E+0.60H	0.968	0.812
D Only	1.613	1.353
Lr Only		
L Only	1.875	1.875
S Only	0.456	0.144
W Only		
E Only		
H Only		

Project Title:
Engineer:
Project ID:
Project Descr:

SG55

Printed: 16 MAY 2019, 12:41PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB11 - PSL Pickup Roof Above along Grid E.5

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

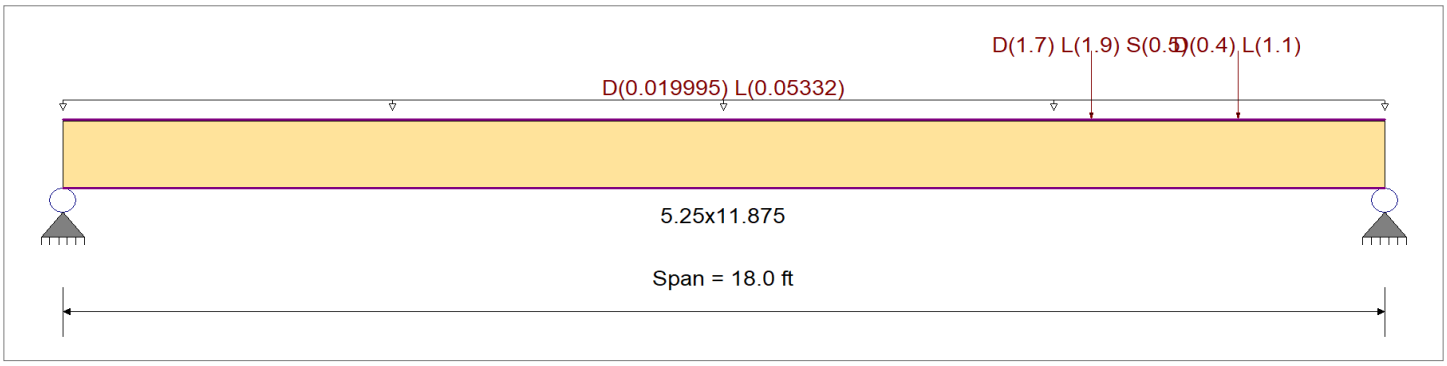
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Fc - Perp	625.0 psi		
Fv	290.0 psi		
Ft	2,025.0 psi	Density	45.070 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft, (Floor)
Point Load : D = 1.70, L = 1.90, S = 0.50 k @ 14.0 ft, (UB10 Beam Rxn)
Point Load : D = 0.40, L = 1.10 k @ 16.0 ft, ((E) Framing Rxn)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.541 : 1	Maximum Shear Stress Ratio	=	0.405 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	1,568.59 psi	fv : Actual	=	117.35 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	13.993 ft	Location of maximum on span	=	17.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.312 in	Ratio =	692	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.550 in	Ratio =	393	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.261	0.184	0.90	1.000	1.00	1.00	1.00	1.00	1.00	7.02	682.30	2610.00	1.99	47.99	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.541	0.405	1.00	1.000	1.00	1.00	1.00	1.00	1.00	16.13	1,568.59	2900.00	4.88	117.35	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.188	0.132	1.25	1.000	1.00	1.00	1.00	1.00	1.00	7.02	682.30	3625.00	1.99	47.99	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.250	0.172	1.15	1.000	1.00	1.00	1.00	1.00	1.00	8.57	833.51	3335.00	2.38	57.34	333.50

Project Title:
Engineer:
Project ID:
Project Descr:

SG56

Printed: 16 MAY 2019, 12:41PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UB11 - PSL Pickup Roof Above along Grid E.5

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.372	0.276	1.25	1.000	1.00	1.00	1.00	1.00	1.00	13.85	1,347.02	3625.00	4.16	100.01	362.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.438	0.321	1.15	1.000	1.00	1.00	1.00	1.00	1.00	15.02	1,460.42	3335.00	4.45	107.03	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.147	0.103	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.02	682.30	4640.00	1.99	47.99	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.147	0.103	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.02	682.30	4640.00	1.99	47.99	464.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.290	0.216	1.60	1.000	1.00	1.00	1.00	1.00	1.00	13.85	1,347.02	4640.00	4.16	100.01	464.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.315	0.231	1.60	1.000	1.00	1.00	1.00	1.00	1.00	15.02	1,460.42	4640.00	4.45	107.03	464.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.315	0.231	1.60	1.000	1.00	1.00	1.00	1.00	1.00	15.02	1,460.42	4640.00	4.45	107.03	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.088	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.21	409.38	4640.00	1.20	28.79	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 18.0 ft	1	0.088	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.21	409.38	4640.00	1.20	28.79	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.5496	9.854		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.802	4.969
Overall MINimum	0.111	0.389
+D+H	0.778	2.033
+D+L+H	1.802	4.969
+D+Lr+H	0.778	2.033
+D+S+H	0.889	2.422
+D+0.750Lr+0.750L+H	1.546	4.235
+D+0.750L+0.750S+H	1.629	4.527
+D+0.60W+H	0.778	2.033
+D+0.70E+H	0.778	2.033
+D+0.750Lr+0.750L+0.450W+H	1.546	4.235
+D+0.750L+0.750S+0.450W+H	1.629	4.527
+D+0.750L+0.750S+0.5250E+H	1.629	4.527
+0.60D+0.60W+0.60H	0.467	1.220
+0.60D+0.70E+0.60H	0.467	1.220
D Only	0.778	2.033
Lr Only		
L Only	1.024	2.935
S Only	0.111	0.389
W Only		
E Only		
H Only		

Project Title:
Engineer:
Project ID:
Project Descr:

SG57

Printed: 16 MAY 2019, 11:28AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH1 - Header north-west

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.250 ft
(Used for non-slender calculations)

Wood Species DF/DF
Wood Grade 24F - V4

Fb +	2,400.0 psi	Fv	265.0 psi
Fb -	1,850.0 psi	Ft	1,100.0 psi
Fc - Prll	1,650.0 psi	Density	31.20 pcf
Fc - Perp	650.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,800.0	1,600.0	1,800.0 ksi
Minimum	950.0	850.0	

Wood Section Name **3.5x9**
Wood Grading/Manuf. Western
Wood Member Type GLB

Exact Width **3.50** in
Exact Depth **9.0** in
Area 31.50 in^2
Ix 212.625 in^4
Iy 32.156 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 9.250 ft, K =

Applied Loads

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

Column self weight included : 63.131 lbs * Dead Load Factor
BENDING LOADS . . .
Wall: Lat. Uniform Load creating Mx-x, D = 0.1445 k/ft
Deck: Lat. Uniform Load creating Mx-x, D = 0.2250, L = 0.450 k/ft
C&C Wind: Lat. Uniform Load creating My-y, W = 0.1830 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9278** : 1
Load Combination +D+L+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 4.594 ft
At maximum location values are . . .
Applied Axial 0.06313 k
Applied Mx 8.764 k-ft
Applied My 0.0 k-ft
Fc : Allowable 1,579.78 psi

PASS Maximum Shear Stress Ratio = **0.6811** : 1
Load Combination +D+L+H
Location of max.above base 0.0 ft
Applied Design Shear 180.485 psi
Allowable Shear 265.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	3.790 k	Bottom along Y-Y	3.790 k
Top along X-X	0.8464 k	Bottom along X-X	0.8464 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.3565 in at 4.656 ft above base
for load combination : +D+L+H
Along X-X 0.5922 in at 4.656 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.963	0.4648	PASS	4.656 ft	0.3412	PASS	0.0 ft
+D+L+H	1.000	0.957	0.9278	PASS	4.594 ft	0.6811	PASS	0.0 ft
+D+Lr+H	1.250	0.943	0.3347	PASS	4.656 ft	0.2457	PASS	0.0 ft
+D+S+H	1.150	0.949	0.3638	PASS	4.656 ft	0.2670	PASS	0.0 ft
+D+0.750Lr+0.750L+H	1.250	0.943	0.6404	PASS	4.594 ft	0.4701	PASS	0.0 ft
+D+0.750L+0.750S+H	1.150	0.949	0.6960	PASS	4.656 ft	0.5109	PASS	0.0 ft
+D+0.60W+H	1.600	0.918	0.5205	PASS	4.594 ft	0.1919	PASS	0.0 ft
+D+0.70E+H	1.600	0.918	0.2615	PASS	4.656 ft	0.1919	PASS	0.0 ft
+D+0.750Lr+0.750L+0.450W+H	1.600	0.918	0.6946	PASS	4.656 ft	0.3672	PASS	0.0 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG58

Printed: 16 MAY 2019, 11:28AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH1 - Header north-west

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+0.750L+0.750S+0.450W+H	1.600	0.918	0.6946	PASS	4.656 ft	0.3672	PASS	0.0 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.918	0.5003	PASS	4.594 ft	0.3672	PASS	0.0 ft
+0.60D+0.60W+0.60H	1.600	0.918	0.4159	PASS	4.594 ft	0.1152	PASS	0.0 ft
+0.60D+0.70E+0.60H	1.600	0.918	0.1569	PASS	4.656 ft	0.1152	PASS	0.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				1.709	1.709	0.063					
+D+L+H				3.790	3.790	0.063					
+D+Lr+H				1.709	1.709	0.063					
+D+S+H				1.709	1.709	0.063					
+D+0.750Lr+0.750L+H				3.270	3.270	0.063					
+D+0.750L+0.750S+H				3.270	3.270	0.063					
+D+0.60W+H	0.508	0.508		1.709	1.709	0.063					
+D+0.70E+H				1.709	1.709	0.063					
+D+0.750Lr+0.750L+0.450W+H	0.381	0.381		3.270	3.270	0.063					
+D+0.750L+0.750S+0.450W+H	0.381	0.381		3.270	3.270	0.063					
+D+0.750L+0.750S+0.5250E+H				3.270	3.270	0.063					
+0.60D+0.60W+0.60H	0.508	0.508		1.025	1.025	0.038					
+0.60D+0.70E+0.60H				1.025	1.025	0.038					
D Only				1.709	1.709	0.063					
Lr Only											
L Only				2.081	2.081						
S Only											
W Only	0.846	0.846									
E Only											
H Only											

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.161 in	4.656 ft
+D+L+H	0.0000 in	0.000 ft	0.357 in	4.656 ft
+D+Lr+H	0.0000 in	0.000 ft	0.161 in	4.656 ft
+D+S+H	0.0000 in	0.000 ft	0.161 in	4.656 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.308 in	4.656 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.308 in	4.656 ft
+D+0.60W+H	0.3553 in	4.656 ft	0.161 in	4.656 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.161 in	4.656 ft
+D+0.750Lr+0.750L+0.450W+H	0.2665 in	4.656 ft	0.308 in	4.656 ft
+D+0.750L+0.750S+0.450W+H	0.2665 in	4.656 ft	0.308 in	4.656 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.308 in	4.656 ft
+0.60D+0.60W+0.60H	0.3553 in	4.656 ft	0.096 in	4.656 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.096 in	4.656 ft
D Only	0.0000 in	0.000 ft	0.161 in	4.656 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.196 in	4.656 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.5922 in	4.656 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG59

Printed: 16 MAY 2019, 11:28AM

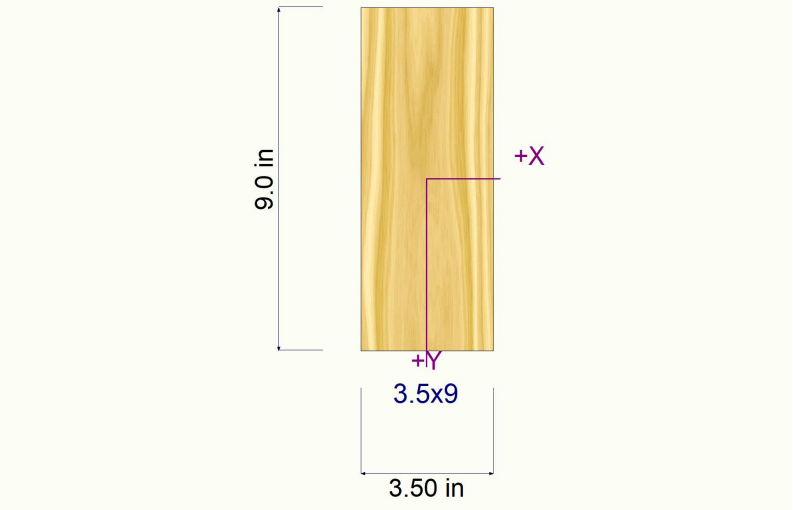
Wood Column

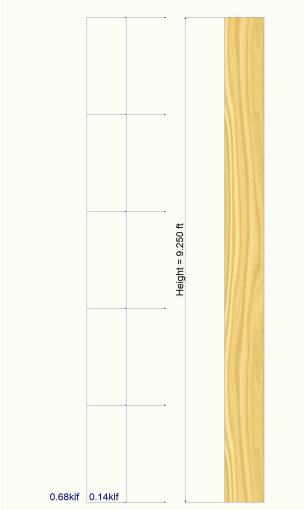
File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

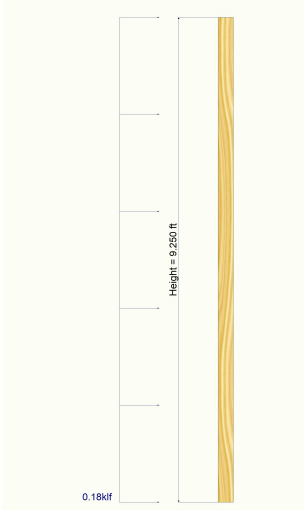
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH1 - Header north-west

Sketches







Project Title:
Engineer:
Project ID:
Project Descr:

SG60

Printed: 16 MAY 2019, 11:28AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH2 - Header north-west

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 14.250 ft
(Used for non-slender calculations)

Wood Species DF/DF
Wood Grade 24F - V4

Fb +	2,400.0 psi	Fv	265.0 psi
Fb -	1,850.0 psi	Ft	1,100.0 psi
Fc - Prll	1,650.0 psi	Density	31.20 pcf
Fc - Perp	650.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,800.0	1,600.0	1,800.0 ksi
Minimum	950.0	850.0	

Wood Section Name **3.5x9**
Wood Grading/Manuf. Western
Wood Member Type GLB

Exact Width **3.50** in
Exact Depth **9.0** in
Area 31.50 in^2
Ix 212.625 in^4
Iy 32.156 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0	
Cf or Cv for Compression	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Factor	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 14.250 ft, K =

Applied Loads

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

Column self weight included : 97.256 lbs * Dead Load Factor
BENDING LOADS . . .
Wall: Lat. Uniform Load creating Mx-x, D = 0.1445 k/ft
Deck: Lat. Uniform Load creating Mx-x, D = 0.020, L = 0.040 k/ft
C&C Wind: Lat. Uniform Load creating My-y, W = 0.1830 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8914** : 1
Load Combination +D+0.60W+H
Governing NDS Forumla np + Mxx + Myy, NDS Eq. 3.9-
Location of max.above base 7.077 ft
At maximum location values are . . .
Applied Axial 0.09726 k
Applied Mx 4.175 k-ft
Applied My 2.787 k-ft
Fc : Allowable 1,787.96 psi

PASS Maximum Shear Stress Ratio = **0.2618** : 1
Load Combination +D+L+H
Location of max.above base 14.250 ft
Applied Design Shear 69.384 psi
Allowable Shear 265.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.457 k	Bottom along Y-Y	1.457 k
Top along X-X	1.304 k	Bottom along X-X	1.304 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.5011 in at 7.173 ft above base
for load combination : +D+L+H
Along X-X 3.336 in at 7.173 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.871	0.4916	PASS	7.077 ft	0.2340	PASS	0.0 ft
+D+L+H	1.000	0.846	0.5501	PASS	7.173 ft	0.2618	PASS	14.250 ft
+D+Lr+H	1.250	0.777	0.3540	PASS	7.077 ft	0.1685	PASS	0.0 ft
+D+S+H	1.150	0.806	0.3848	PASS	7.077 ft	0.1831	PASS	0.0 ft
+D+0.750Lr+0.750L+H	1.250	0.777	0.4185	PASS	7.077 ft	0.1992	PASS	14.250 ft
+D+0.750L+0.750S+H	1.150	0.806	0.4549	PASS	7.077 ft	0.2165	PASS	14.250 ft
+D+0.60W+H	1.600	0.677	0.8914	PASS	7.077 ft	0.1316	PASS	0.0 ft
+D+0.70E+H	1.600	0.677	0.2765	PASS	7.077 ft	0.1316	PASS	0.0 ft
+D+0.750Lr+0.750L+0.450W+H	1.600	0.677	0.7881	PASS	7.077 ft	0.1556	PASS	14.250 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG61

Printed: 16 MAY 2019, 11:28AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH2 - Header north-west

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+0.750L+0.750S+0.450W+H	1.600	0.677	0.7881	PASS	7.077 ft	0.1556	PASS	14.250 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.677	0.3270	PASS	7.077 ft	0.1556	PASS	14.250 ft
+0.60D+0.60W+0.60H	1.600	0.677	0.7807	PASS	7.077 ft	0.08786	PASS	14.250 ft
+0.60D+0.70E+0.60H	1.600	0.677	0.1658	PASS	7.077 ft	0.07898	PASS	0.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				1.172	1.172	0.097					
+D+L+H				1.457	1.457	0.097					
+D+Lr+H				1.172	1.172	0.097					
+D+S+H				1.172	1.172	0.097					
+D+0.750Lr+0.750L+H				1.386	1.386	0.097					
+D+0.750L+0.750S+H				1.386	1.386	0.097					
+D+0.60W+H	0.782	0.782		1.172	1.172	0.097					
+D+0.70E+H				1.172	1.172	0.097					
+D+0.750Lr+0.750L+0.450W+H	0.587	0.587		1.386	1.386	0.097					
+D+0.750L+0.750S+0.450W+H	0.587	0.587		1.386	1.386	0.097					
+D+0.750L+0.750S+0.5250E+H				1.386	1.386	0.097					
+0.60D+0.60W+0.60H	0.782	0.782		0.703	0.703	0.058					
+0.60D+0.70E+0.60H				0.703	0.703	0.058					
D Only				1.172	1.172	0.097					
Lr Only											
L Only				0.285	0.285						
S Only											
W Only	1.304	1.304									
E Only											
H Only											

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.403 in	7.173 ft
+D+L+H	0.0000 in	0.000 ft	0.501 in	7.173 ft
+D+Lr+H	0.0000 in	0.000 ft	0.403 in	7.173 ft
+D+S+H	0.0000 in	0.000 ft	0.403 in	7.173 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.477 in	7.173 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.477 in	7.173 ft
+D+0.60W+H	2.0014 in	7.173 ft	0.403 in	7.173 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.403 in	7.173 ft
+D+0.750Lr+0.750L+0.450W+H	1.5010 in	7.173 ft	0.477 in	7.173 ft
+D+0.750L+0.750S+0.450W+H	1.5010 in	7.173 ft	0.477 in	7.173 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.477 in	7.173 ft
+0.60D+0.60W+0.60H	2.0014 in	7.173 ft	0.242 in	7.173 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.242 in	7.173 ft
D Only	0.0000 in	0.000 ft	0.403 in	7.173 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.098 in	7.173 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	3.3356 in	7.173 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG62

Printed: 16 MAY 2019, 11:28AM

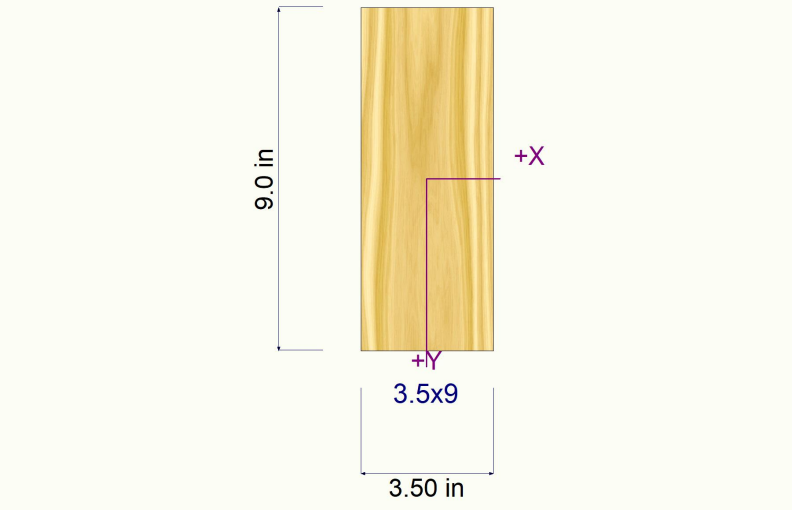
Wood Column

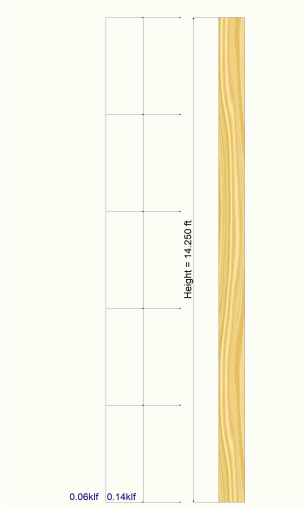
File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

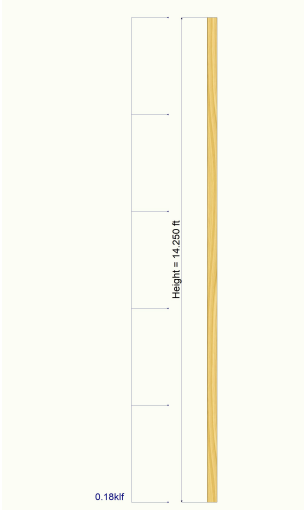
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH2 - Header north-west

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG63

Printed: 23 AUG 2018, 4:37PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH3 - Header west sliding door

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 19.50 ft
(Used for non-slender calculations)

Wood Species DF/DF
Wood Grade 24F - V4

Fb +	2,400.0 psi	Fv	265.0 psi
Fb -	1,850.0 psi	Ft	1,100.0 psi
Fc - Prll	1,650.0 psi	Density	31.20 pcf
Fc - Perp	650.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,800.0	1,600.0	1,800.0 ksi
Minimum	950.0	850.0	

Wood Section Name **3.5x20**
Wood Grading/Manuf. Western
Wood Member Type GLB

Exact Width	**3.50**	in
Exact Depth	**20.0**	in
Area	70.0	in^2
Ix	2,333.33	in^4
Iy	71.458	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	0.9945
Cf or Cv for Compression	0.9945
Cf or Cv for Tension	0.9945
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 19.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 295.750 lbs * Dead Load Factor
BENDING LOADS . . .
Wall: Lat. Uniform Load creating Mx-x, D = 0.1445 k/ft
Floor: Lat. Uniform Load creating Mx-x, D = 0.090, L = 0.240 k/ft
C&C Wind: Lat. Uniform Load creating My-y, W = 0.1830 k/ft
Roof: Lat. Uniform Load from 0.0-->3.250 ft creating Mx-x, D = 0.2940, S = 0.350 k/ft
Roof: Lat. Uniform Load from 17.667-->19.50 ft creating Mx-x, D = 0.2940, S = 0.350 k/ft
RH3 Hdr RXN abv: Lat. Point Load at 3.250 ft creating Mx-x, D = 2.208, S = 2.494 k
RH3 Hdr RXN abv: Lat. Point Load at 17.250 ft creating Mx-x, D = 2.208, S = 2.494 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8342** : 1
Load Combination +D+0.750L+0.750S+0.450W+H
Governing NDS Forumlanp + Mxx + Myy, NDS Eq. 3.9-
Location of max.above base 9.423 ft
At maximum location values are . . .
Applied Axial 0.2958 k
Applied Mx 32.934 k-ft
Applied My 3.910 k-ft
Fc : Allowable 2,454.61 psi

PASS Maximum Shear Stress Ratio = **0.6762** : 1
Load Combination +D+0.750L+0.750S+H
Location of max.above base 0.0 ft
Applied Design Shear 206.069 psi
Allowable Shear 304.750 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	9.452 k	Bottom along Y-Y	9.617 k
Top along X-X	1.784 k	Bottom along X-X	1.784 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.5780 in at 9.685 ft above base
for load combination : +D+0.750L+0.750S+H
Along X-X 5.263 in at 9.815 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
+D+H			5.282	5.201	0.296				
+D+L+H			7.622	7.541	0.296				
+D+Lr+H			5.282	5.201	0.296				
+D+S+H			8.721	8.529	0.296				
+D+0.750Lr+0.750L+H			7.037	6.956	0.296				

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG64

Printed: 23 AUG 2018, 4:37PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH3 - Header west sliding door

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+H				9.617	9.452	0.296					
+D+0.60W+H	1.071	1.071		5.282	5.201	0.296					
+D+0.70E+H				5.282	5.201	0.296					
+D+0.750Lr+0.750L+0.450W+H	0.803	0.803		7.037	6.956	0.296					
+D+0.750L+0.750S+0.450W+H	0.803	0.803		9.617	9.452	0.296					
+D+0.750L+0.750S+0.5250E+H				9.617	9.452	0.296					
+0.60D+0.60W+0.60H	1.071	1.071		3.169	3.120	0.177					
+0.60D+0.70E+0.60H				3.169	3.120	0.177					
D Only				5.282	5.201	0.296					
Lr Only											
L Only				2.340	2.340						
S Only				3.439	3.328						
W Only	1.784	1.784									
E Only											
H Only											

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.320 in	9.685 ft
+D+L+H	0.0000 in	0.000 ft	0.508 in	9.685 ft
+D+Lr+H	0.0000 in	0.000 ft	0.320 in	9.685 ft
+D+S+H	0.0000 in	0.000 ft	0.476 in	9.685 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.461 in	9.685 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.578 in	9.685 ft
+D+0.60W+H	3.1580 in	9.815 ft	0.320 in	9.685 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.320 in	9.685 ft
+D+0.750Lr+0.750L+0.450W+H	2.3685 in	9.815 ft	0.461 in	9.685 ft
+D+0.750L+0.750S+0.450W+H	2.3685 in	9.815 ft	0.578 in	9.685 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.578 in	9.685 ft
+0.60D+0.60W+0.60H	3.1580 in	9.815 ft	0.192 in	9.685 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.192 in	9.685 ft
D Only	0.0000 in	0.000 ft	0.320 in	9.685 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.188 in	9.815 ft
S Only	0.0000 in	0.000 ft	0.156 in	9.554 ft
W Only	5.2634 in	9.815 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG65

www.dibbleengineers.com

Printed: 23 AUG 2018, 4:37PM

Wood Column

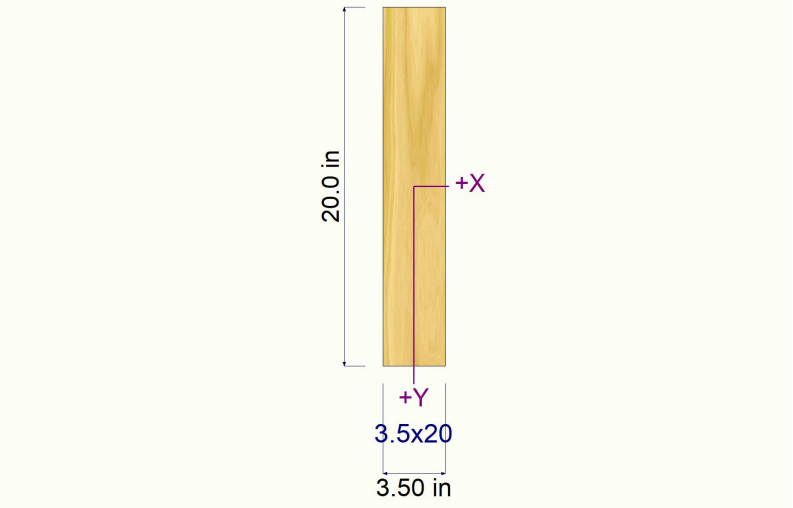
File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

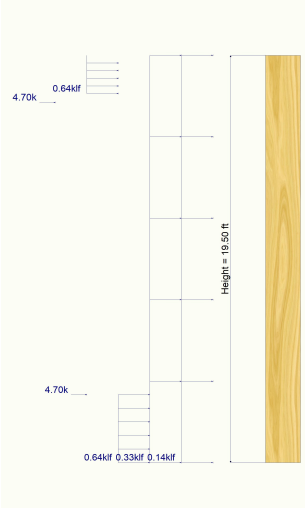
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

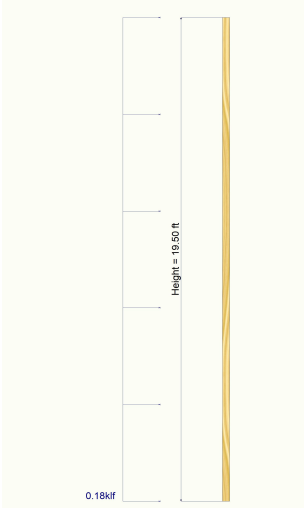
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH3 - Header west sliding door

Sketches







Project Title:
Engineer:
Project ID:
Project Descr:

SG66

Printed: 16 MAY 2019, 11:29AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH5 - Header east, typ gov

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name **3-2x8**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**4.50**	in
Exact Depth	**7.250**	in
Area	32.625	in^2
Ix	142.904	in^4
Iy	55.055	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 9.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 54.708 lbs * Dead Load Factor
BENDING LOADS . . .
Floor: Lat. Uniform Load creating Mx-x, D = 0.010, L = 0.02667 k/ft
C&C Wind: Lat. Uniform Load creating My-y, W = 0.1830 k/ft
Roof: Lat. Uniform Load creating Mx-x, D = 0.030, L = 0.050 k/ft
Wall: Lat. Uniform Load creating Mx-x, D = 0.1445 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7899** : 1
Load Combination +D+L+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 4.470 ft
At maximum location values are . . .
Applied Axial 0.05471 k
Applied Mx 2.644 k-ft
Applied My 0.0 k-ft
Fc : Allowable 1,048.01 psi

PASS Maximum Shear Stress Ratio = **0.3602** : 1
Load Combination +D+L+H
Location of max.above base 9.0 ft
Applied Design Shear 54.035 psi
Allowable Shear 150.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.175 k	Bottom along Y-Y	1.175 k
Top along X-X	0.8235 k	Bottom along X-X	0.8235 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.2098 in at 4.530 ft above base
for load combination : +D+L+H
Along X-X 0.3815 in at 4.530 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.796	0.620	PASS	4.470 ft	0.2828	PASS	9.0 ft
+D+L+H	1.000	0.768	0.7899	PASS	4.470 ft	0.3602	PASS	9.0 ft
+D+Lr+H	1.250	0.698	0.4464	PASS	4.470 ft	0.2036	PASS	9.0 ft
+D+S+H	1.150	0.726	0.4852	PASS	4.470 ft	0.2213	PASS	9.0 ft
+D+0.750Lr+0.750L+H	1.250	0.698	0.5855	PASS	4.470 ft	0.2670	PASS	9.0 ft
+D+0.750L+0.750S+H	1.150	0.726	0.6364	PASS	4.470 ft	0.2903	PASS	9.0 ft
+D+0.60W+H	1.600	0.608	0.6828	PASS	4.470 ft	0.1591	PASS	9.0 ft
+D+0.70E+H	1.600	0.608	0.3488	PASS	4.470 ft	0.1591	PASS	9.0 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG67

Printed: 16 MAY 2019, 11:29AM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH5 - Header east, typ gov

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+0.750Lr+0.750L+0.450W+H	1.600	0.608	0.7080	PASS	4.470 ft	0.2086	PASS	9.0 ft
+D+0.750L+0.750S+0.450W+H	1.600	0.608	0.7080	PASS	4.470 ft	0.2086	PASS	9.0 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.608	0.4574	PASS	4.470 ft	0.2086	PASS	9.0 ft
+0.60D+0.60W+0.60H	1.600	0.608	0.5432	PASS	4.530 ft	0.09543	PASS	9.0 ft
+0.60D+0.70E+0.60H	1.600	0.608	0.2092	PASS	4.530 ft	0.09543	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				0.830	0.830	0.055					
+D+L+H				1.175	1.175	0.055					
+D+Lr+H				0.830	0.830	0.055					
+D+S+H				0.830	0.830	0.055					
+D+0.750Lr+0.750L+H				1.089	1.089	0.055					
+D+0.750L+0.750S+H				1.089	1.089	0.055					
+D+0.60W+H	0.494	0.494		0.830	0.830	0.055					
+D+0.70E+H				0.830	0.830	0.055					
+D+0.750Lr+0.750L+0.450W+H	0.371	0.371		1.089	1.089	0.055					
+D+0.750L+0.750S+0.450W+H	0.371	0.371		1.089	1.089	0.055					
+D+0.750L+0.750S+0.5250E+H				1.089	1.089	0.055					
+0.60D+0.60W+0.60H	0.494	0.494		0.498	0.498	0.033					
+0.60D+0.70E+0.60H				0.498	0.498	0.033					
D Only				0.830	0.830	0.055					
Lr Only											
L Only				0.345	0.345						
S Only											
W Only	0.824	0.824									
E Only											
H Only											

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.148 in	4.530 ft
+D+L+H	0.0000 in	0.000 ft	0.210 in	4.530 ft
+D+Lr+H	0.0000 in	0.000 ft	0.148 in	4.530 ft
+D+S+H	0.0000 in	0.000 ft	0.148 in	4.530 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.194 in	4.530 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.194 in	4.530 ft
+D+0.60W+H	0.2289 in	4.530 ft	0.148 in	4.530 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.148 in	4.530 ft
+D+0.750Lr+0.750L+0.450W+H	0.1717 in	4.530 ft	0.194 in	4.530 ft
+D+0.750L+0.750S+0.450W+H	0.1717 in	4.530 ft	0.194 in	4.530 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.194 in	4.530 ft
+0.60D+0.60W+0.60H	0.2289 in	4.530 ft	0.089 in	4.530 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.089 in	4.530 ft
D Only	0.0000 in	0.000 ft	0.148 in	4.530 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.062 in	4.530 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.3815 in	4.530 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SG68

Printed: 16 MAY 2019, 11:29AM

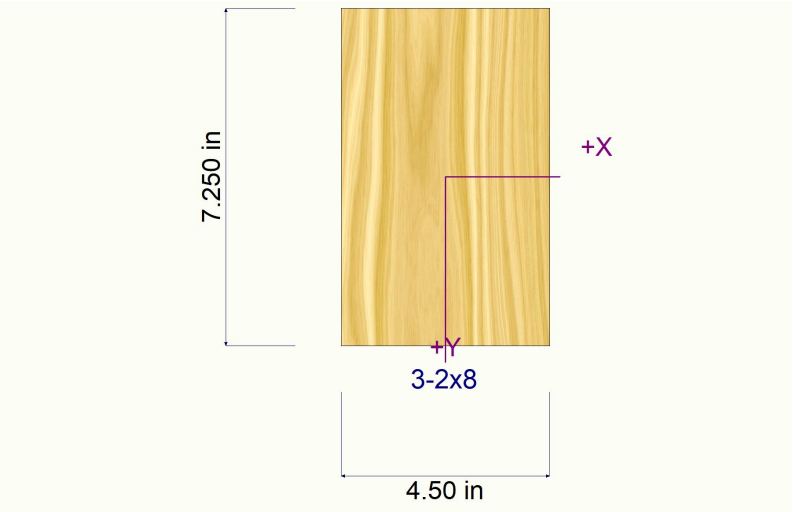
Wood Column

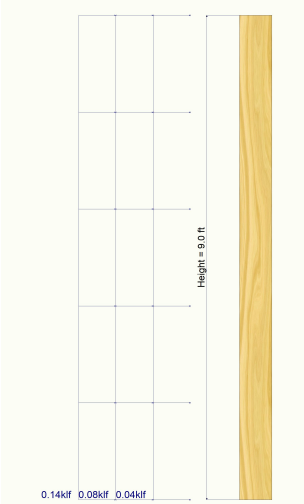
File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

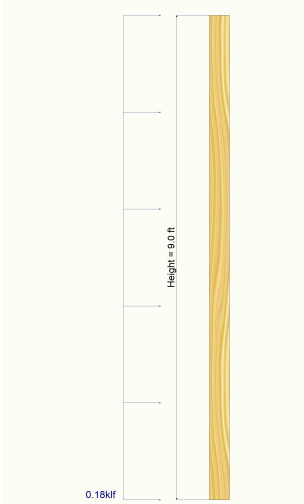
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH5 - Header east, typ gov

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG69

Printed: 23 AUG 2018, 12:47PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH6 - Header east

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

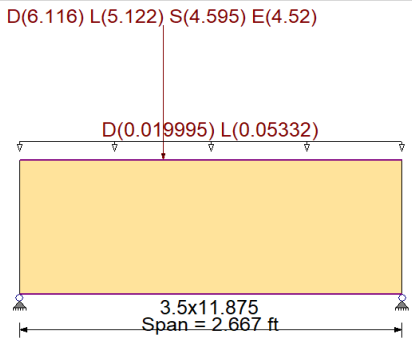
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 1.333 ft, (Floor)
Point Load : D = 6.116, L = 5.122, S = 4.595, E = 4.520 k @ 1.0 ft, (UB9 Beam RXN)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.445 : 1	Maximum Shear Stress Ratio	=	0.995 : 1
Section used for this span		3.5x11.875	Section used for this span		3.5x11.875
fb : Actual	=	1,228.70 psi	fv : Actual	=	303.24 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+0.750L+0.750S+H	Load Combination		+D+0.750L+0.750S+H
Location of maximum on span	=	1.003 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.004 in	Ratio =	8531	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.011 in	Ratio =	2799	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	9.953	6.008
Overall MINimum	2.825	1.695
+D+H	3.861	2.332
+D+L+H	7.134	4.323
+D+Lr+H	3.861	2.332
+D+S+H	6.734	4.055
+D+0.750Lr+0.750L+H	6.316	3.826
+D+0.750L+0.750S+H	8.470	5.118
+D+0.60W+H	3.861	2.332
+D+0.70E+H	5.839	3.518
+D+0.750Lr+0.750L+0.450W+H	6.316	3.826

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG70

www.dibbleengineers.com

Printed: 23 AUG 2018, 12:47PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UH6 - Header east

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W+H	8.470	5.118
+D+0.750L+0.750S+0.5250E+H	9.953	6.008
+0.60D+0.60W+0.60H	2.317	1.399
+0.60D+0.70E+0.60H	4.295	2.585
D Only	3.861	2.332
Lr Only		
L Only	3.273	1.992
S Only	2.872	1.723
W Only		
E Only	2.825	1.695
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG71

Printed: 23 AUG 2018, 4:10PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC1 - Post center w/ SW HD

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.50 ft
(Used for non-slender calculations)

Wood Species Douglas Fir - Larch
Wood Grade No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,600.0	1,600.0	1,600.0	ksi
Minimum	580.0	580.0		

Wood Section Name **6x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width **5.50** in
Exact Depth **5.50** in

Area	30.250	in^2
Ix	76.255	in^4
Iy	76.255	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0	
Cf or Cv for Compression	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Factor	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 62.265 lbs * Dead Load Factor
AXIAL LOADS . . .
UB3 Bm rxn: Axial Load at 9.50 ft, D = 1.197, L = 2.60 k
UB5 Bm rxn: Axial Load at 9.50 ft, D = 0.8990, L = 2.10 k
UB8 Bm rxn: Axial Load at 9.50 ft, D = 2.879, L = 2.030, S = 2.280 k
UTB2 Trnsf Bm rxn w/ SW HD: Axial Load at 9.50 ft, D = 0.1740, L = 0.320, E = 15.183 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7710** : 1
Load Combination +1.102D+0.750L+0.750S+0.5250E+
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 20.714 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 888.12 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.4634D+0.70E+0.60H
Location of max.above base 9.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
	k				k-ft				
+D+H					5.211				
+D+L+H					12.261				
+D+Lr+H					5.211				
+D+S+H					7.491				
+D+0.750Lr+0.750L+H					10.499				
+D+0.750L+0.750S+H					12.209				
+D+0.60W+H					5.211				
+D+0.70E+H					15.839				

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG72

Printed: 23 AUG 2018, 4:10PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC1 - Post center w/ SW HD

Maximum Reactions

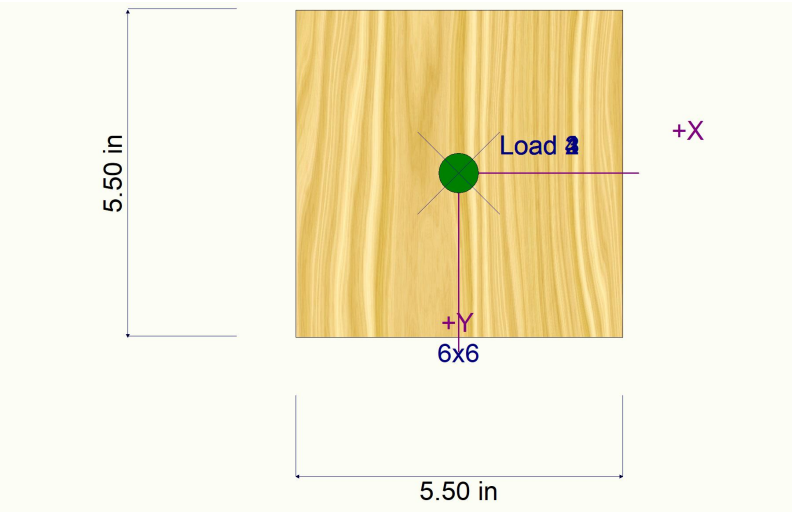
Note: Only non-zero reactions are listed.

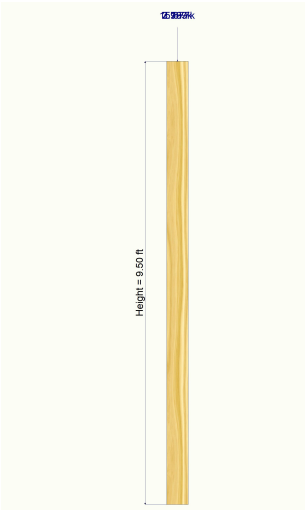
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750Lr+0.750L+0.450W+H						10.499					
+D+0.750L+0.750S+0.450W+H						12.209					
+D+0.750L+0.750S+0.5250E+H						20.180					
+0.60D+0.60W+0.60H						3.127					
+0.60D+0.70E+0.60H						13.755					
D Only						5.211					
Lr Only											
L Only						7.050					
S Only						2.280					
W Only											
E Only						15.183					
H Only											

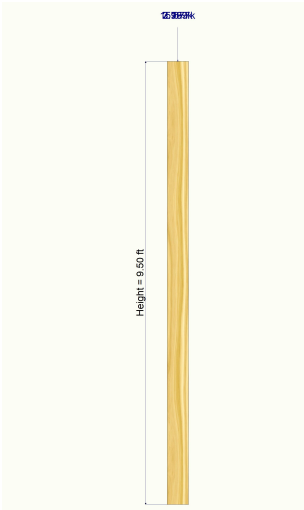
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG73

Printed: 23 AUG 2018, 4:11PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC2 - Post center

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.50 ft
(Used for non-slender calculations)

Wood Species Douglas Fir - Larch
Wood Grade No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,600.0	1,600.0	1,600.0	ksi
Minimum	580.0	580.0		

Wood Section Name **6x8**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**5.50**	in
Exact Depth	**7.50**	in
Area	41.250	in^2
Ix	193.359	in^4
Iy	103.984	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 84.906 lbs * Dead Load Factor
AXIAL LOADS . . .
UB8 Bm rxn: Axial Load at 9.50 ft, D = 10.175, L = 2.302, S = 10.260 k
UBX Bm rxn: Axial Load at 9.50 ft, D = 1.682, L = 4.260 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8071** : 1
Load Combination +D+0.750L+0.750S+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 24.558 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 737.63 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.4634D+0.70E+0.60H
Location of max.above base 9.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .
Bending Compression Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						11.942					
+D+L+H						18.504					
+D+Lr+H						11.942					
+D+S+H						22.202					
+D+0.750Lr+0.750L+H						16.863					
+D+0.750L+0.750S+H						24.558					
+D+0.60W+H						11.942					
+D+0.70E+H						11.942					
+D+0.750Lr+0.750L+0.450W+H						16.863					
+D+0.750L+0.750S+0.450W+H						24.558					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG74

Printed: 23 AUG 2018, 4:11PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC2 - Post center

Maximum Reactions

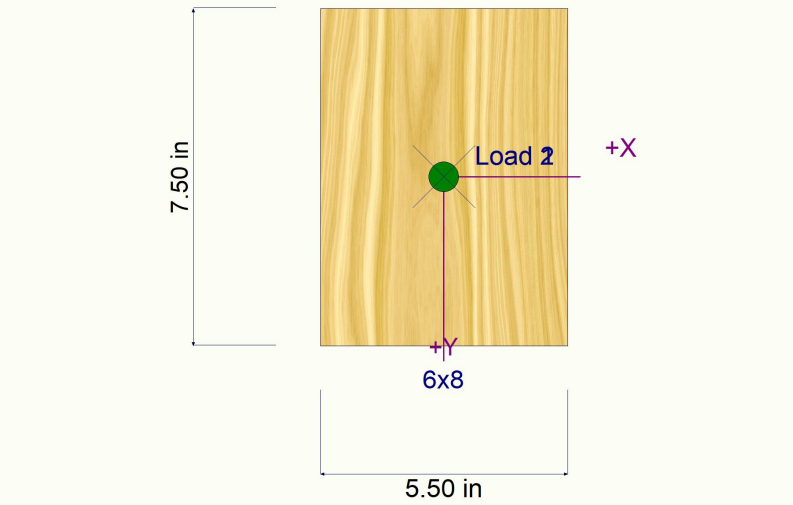
Note: Only non-zero reactions are listed.

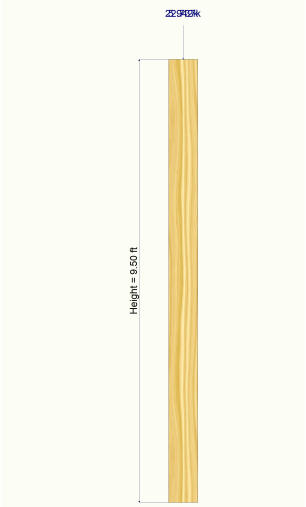
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H						24.558					
+0.60D+0.60W+0.60H						7.165					
+0.60D+0.70E+0.60H						7.165					
D Only						11.942					
Lr Only											
L Only						6.562					
S Only						10.260					
W Only											
E Only											
H Only											

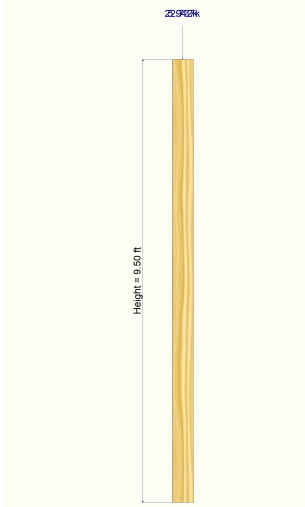
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG75

Printed: 23 AUG 2018, 4:39PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC3 - Post west w/ SW HD

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.50 ft
(Used for non-slender calculations)

Wood Species Hem Fir
Wood Grade No.2

Fb +	850.0 psi	Fv	150.0 psi
Fb -	850.0 psi	Ft	525.0 psi
Fc - Prll	1,300.0 psi	Density	26.830 pcf
Fc - Perp	405.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial	
Basic	1,300.0	1,300.0	1,300.0	ksi
Minimum	470.0	470.0		

Wood Section Name **4x8**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**3.50**	in
Exact Depth	**7.250**	in
Area	25.375	in^2
Ix	111.148	in^4
Iy	25.904	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 44.915 lbs * Dead Load Factor
AXIAL LOADS . . .
UH3 Header rxn: Axial Load at 9.50 ft, D = 5.282, L = 2.340, S = 3.439 k
UTB2 Bm rxn w/ SW HD: Axial Load at 9.50 ft, D = 0.1740, L = 0.320, E = 15.183 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5923** : 1
Load Combination +1.102D+0.750L+0.750S+0.5250E+
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 18.610 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 1,238.29 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination +0.4634D+0.70E+0.60H
Location of max.above base 9.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H						5.501					
+D+L+H						8.161					
+D+Lr+H						5.501					
+D+S+H						8.940					
+D+0.750Lr+0.750L+H						7.496					
+D+0.750L+0.750S+H						10.075					
+D+0.60W+H						5.501					
+D+0.70E+H						16.129					
+D+0.750Lr+0.750L+0.450W+H						7.496					
+D+0.750L+0.750S+0.450W+H						10.075					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG76

Printed: 23 AUG 2018, 4:39PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - roof, upper R1.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : UC3 - Post west w/ SW HD

Maximum Reactions

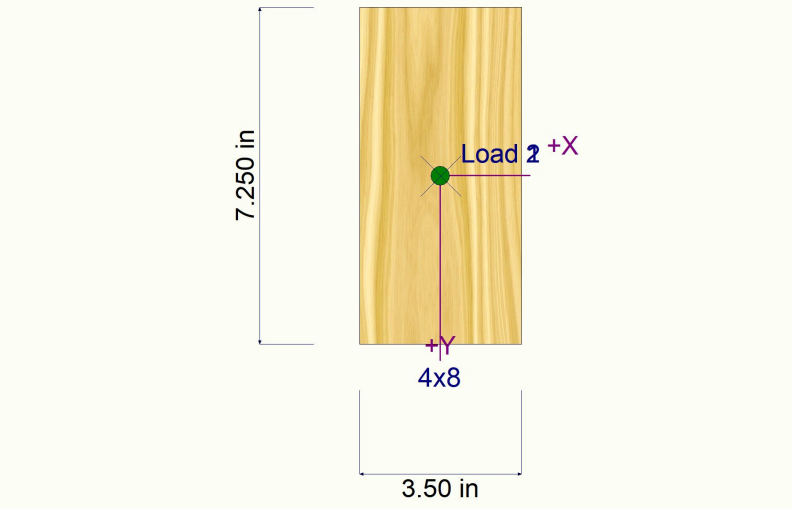
Note: Only non-zero reactions are listed.

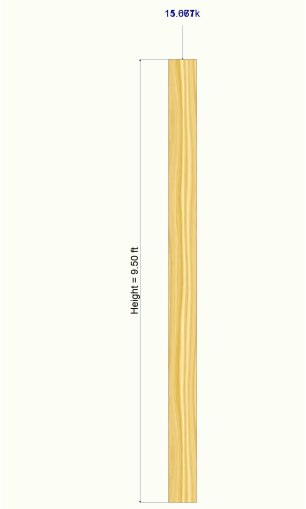
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.5250E+H						18.046					
+0.60D+0.60W+0.60H						3.301					
+0.60D+0.70E+0.60H						13.929					
D Only						5.501					
Lr Only											
L Only						2.660					
S Only						3.439					
W Only											
E Only						15.183					
H Only											

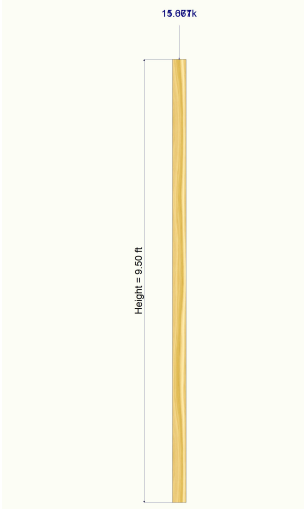
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

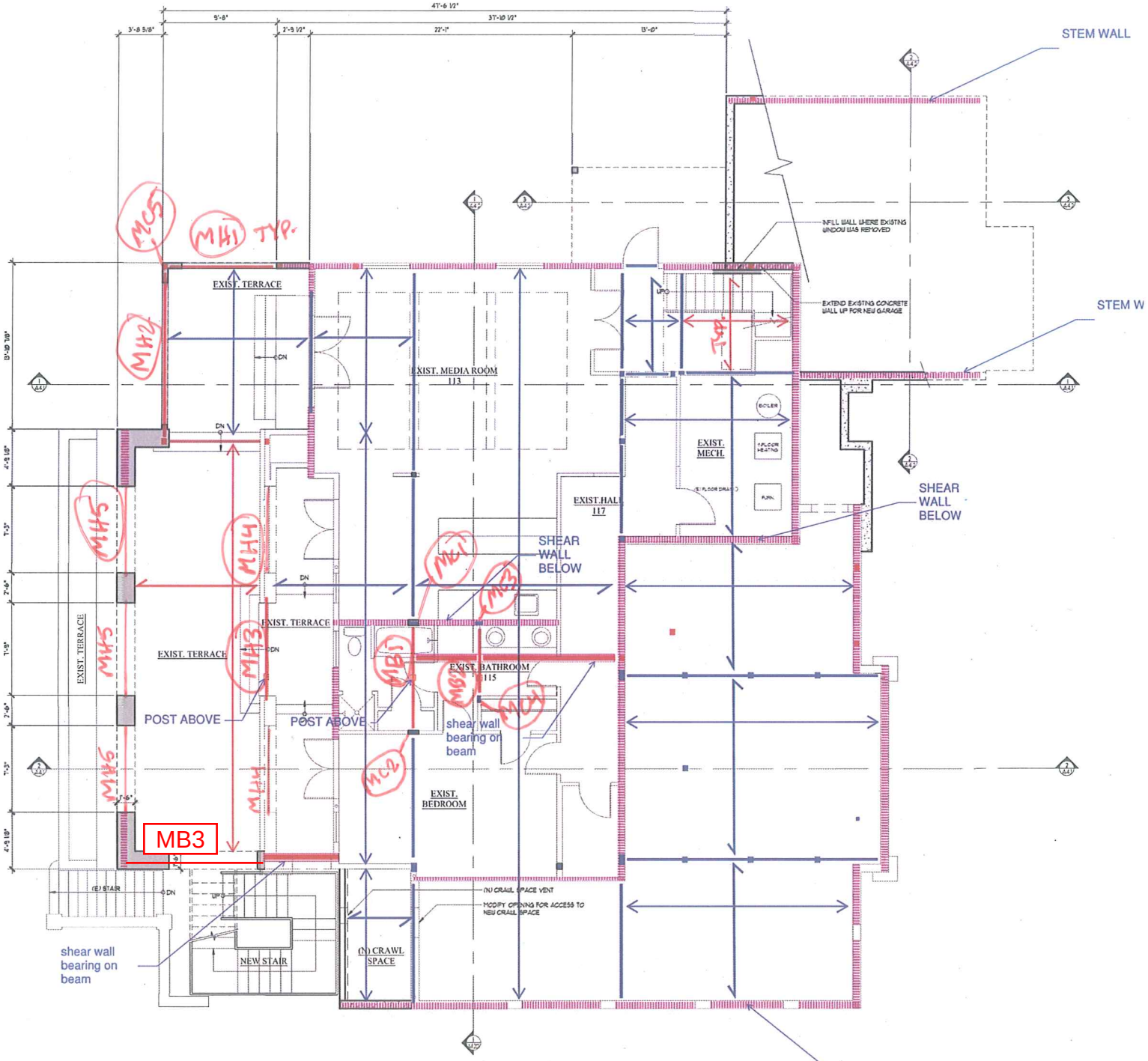
Sketches







GRAVITY KEY PLAN - MAIN FLOOR FRAMING



MAIN FLOOR FRAMING AND SHEAR WALLS BELOW

1 PROPOSED LOWER FLOOR PLAN
SCALE: 1/4" = 1'-0"

0 1 2 4 8 12 FT.

NORTH

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG78

Printed: 23 AUG 2018, 4:15PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : MB1 - Beam center w/ SW HD

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

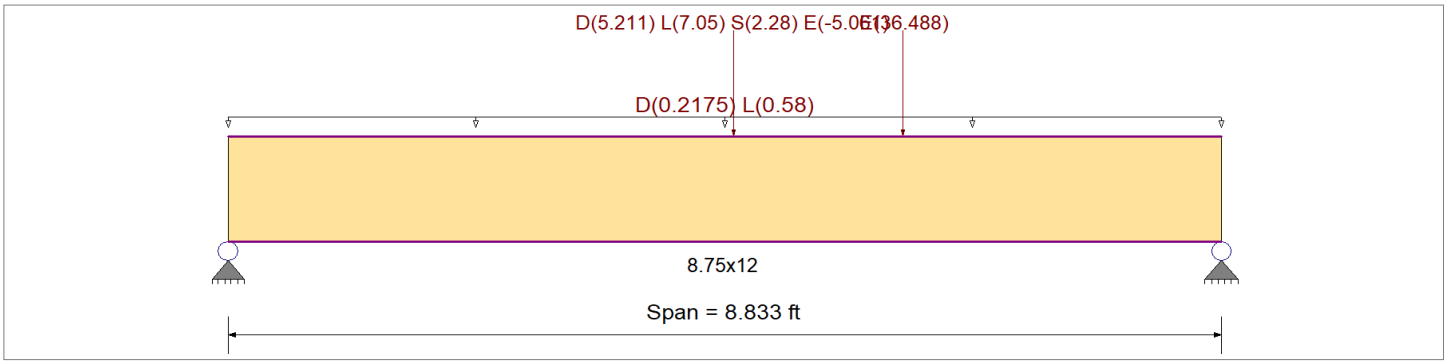
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 14.50 ft, (Floor)
Point Load : D = 5.211, L = 7.050, S = 2.280, E = -5.061 k @ 4.50 ft, (UC1 Post abv w/ SW HD)
Point Load : E = 36.488 k @ 6.0 ft, (SW HD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.854 : 1	Maximum Shear Stress Ratio	=	0.692 : 1
Section used for this span		8.75x12	Section used for this span		8.75x12
fb : Actual	=	3,278.84 psi	fv : Actual	=	293.45 psi
FB : Allowable	=	3,840.00 psi	Fv : Allowable	=	424.00 psi
Load Combination		+1.102D+0.750L+0.750S+0.5250E+H	Load Combination		+1.102D+0.750L+0.750S+0.5250E+H
Location of maximum on span	=	5.867 ft	Location of maximum on span	=	7.834 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.113 in	Ratio =	940	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.185 in	Ratio =	574	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	13.812	22.206
Overall MINimum	9.221	22.206
+D+H	3.617	3.716
+D+L+H	9.638	9.869
+D+Lr+H	3.617	3.716
+D+S+H	4.736	4.877
+D+0.750Lr+0.750L+H	8.133	8.331
+D+0.750L+0.750S+H	8.971	9.202
+D+0.60W+H	3.617	3.716
+D+0.70E+H	10.072	19.260

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG79

www.dibbleengineers.com

Printed: 23 AUG 2018, 4:15PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : MB1 - Beam center w/ SW HD

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr+0.750L+0.450W+H	8.133	8.331
+D+0.750L+0.750S+0.450W+H	8.971	9.202
+D+0.750L+0.750S+0.5250E+H	13.812	20.860
+0.60D+0.60W+0.60H	2.170	2.229
+0.60D+0.70E+0.60H	8.625	17.774
D Only	3.617	3.716
Lr Only		
L Only	6.020	6.153
S Only	1.118	1.162
W Only		
E Only	9.221	22.206
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG80

Printed: 23 AUG 2018, 4:47PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MB2 - Beam center

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

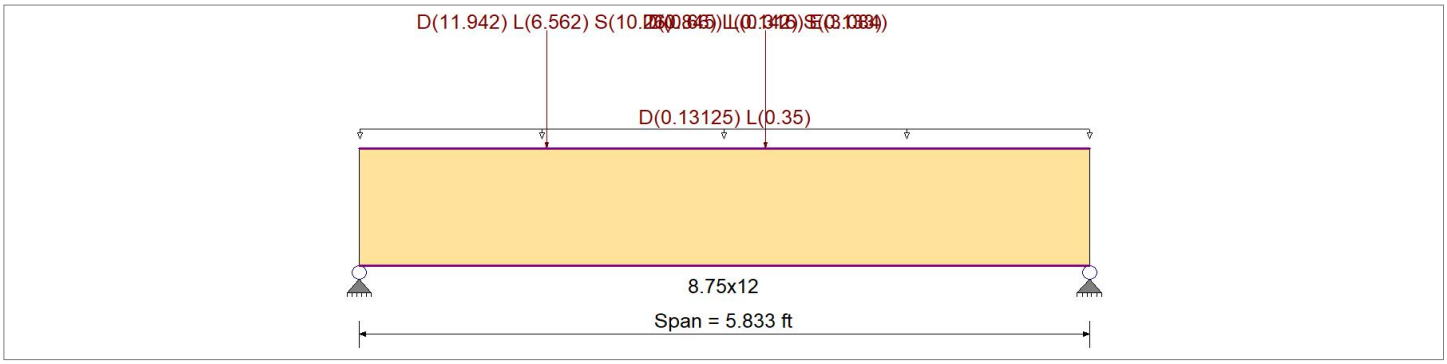
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 8.750 ft, (Floor)
Point Load : D = 11.942, L = 6.562, S = 10.260 k @ 1.50 ft, (UC2 Post abv)
Point Load : D = 0.8450, L = 0.3160, E = 3.084 k @ 3.250 ft, (MTB2 - Transfer Bm rxn w/ SW HD)
Point Load : D = 0.660, L = 0.1420, S = 0.1330 k @ 3.250 ft, (MTB1 - Transfer Bm rxn w/ SW HD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.620** : 1	Maximum Shear Stress Ratio	=	**0.933** : 1
Section used for this span		**8.75x12**	Section used for this span		**8.75x12**
fb : Actual	=	1,711.81 psi	fv : Actual	=	284.47 psi
FB : Allowable	=	2,760.00 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+0.750L+0.750S+H	Load Combination		+D+0.750L+0.750S+H
Location of maximum on span	=	1.512 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	2958	>=480.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0
Max Downward Total Deflection	0.071 in	Ratio =	982	>=360.
Max Upward Total Deflection	0.000 in	Ratio =	0	<360.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	21.038	9.517
Overall MINimum	1.366	1.718
+D+H	9.987	4.358
+D+L+H	16.085	7.322
+D+Lr+H	9.987	4.358
+D+S+H	17.667	7.071
+D+0.750Lr+0.750L+H	14.561	6.581
+D+0.750L+0.750S+H	20.321	8.615

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

Project Title:
Engineer:
Project ID:
Project Descr:

SG81

www.dibbleengineers.com

Printed: 23 AUG 2018, 4:47PM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .

Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MB2 - Beam center

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.60W+H	9.987	4.358
+D+0.70E+H	10.943	5.561
+D+0.750Lr+0.750L+0.450W+H	14.561	6.581
+D+0.750L+0.750S+0.450W+H	20.321	8.615
+D+0.750L+0.750S+0.5250E+H	21.038	9.517
+0.60D+0.60W+0.60H	5.992	2.615
+0.60D+0.70E+0.60H	6.948	3.818
D Only	9.987	4.358
Lr Only		
L Only	6.098	2.963
S Only	7.681	2.712
W Only		
E Only	1.366	1.718
H Only		

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG82

Printed: 23 AUG 2018, 4:45PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MH3 - Header east w/ SW HD

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 7.750 ft
(Used for non-slender calculations)

Wood Species DF/DF
Wood Grade 24F - V4

Fb +	2,400.0 psi	Fv	265.0 psi
Fb -	1,850.0 psi	Ft	1,100.0 psi
Fc - Prll	1,650.0 psi	Density	31.20 pcf
Fc - Perp	650.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,800.0	1,600.0	1,800.0 ksi
Minimum	950.0	850.0	

Wood Section Name **5.5x11.875**
Wood Grading/Manuf. Western
Wood Member Type GLB

Exact Width	**5.50**	in
Exact Depth	**11.875**	in
Area	65.313	in^2
Ix	767.51	in^4
Iy	164.642	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0	
Cf or Cv for Compression	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Factor	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 7.750 ft, K = 1.0

Applied Loads

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

Column self weight included : 109.671 lbs * Dead Load Factor
BENDING LOADS . . .
Floor: Lat. Uniform Load creating Mx-x, D = 0.1760, L = 0.470 k/ft
C&C Wind: Lat. Uniform Load creating My-y, W = 0.1959 k/ft
UC3 Post abv rxn: Lat. Point Load at 1.50 ft creating Mx-x, D = 5.501, L = 2.660, S = 3.439, E = 15.061 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5854** : 1
Load Combination +D+0.750L+0.750S+0.5250E+H
Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 1.508 ft
At maximum location values are . . .
Applied Axial 0.1097 k
Applied Mx 24.211 k-ft
Applied My 0.0 k-ft
Fc : Allowable 2,574.73 psi

PASS Maximum Shear Stress Ratio = **0.8964** : 1
Load Combination +D+0.750L+0.750S+0.5250E+H
Location of max.above base 0.0 ft
Applied Design Shear 380.090 psi
Allowable Shear 424.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	5.528 k	Bottom along Y-Y	16.550 k
Top along X-X	0.7591 k	Bottom along X-X	0.7591 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.1553 in at 3.485 ft above base
for load combination : +D+0.750L+0.750S+0.5250E+H
Along X-X 0.06101 in at 3.901 ft above base
for load combination : W Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H				5.118	1.747	0.110					
+D+L+H				9.085	4.083	0.110					
+D+Lr+H				5.118	1.747	0.110					
+D+S+H				7.892	2.412	0.110					
+D+0.750Lr+0.750L+H				8.093	3.499	0.110					
+D+0.750L+0.750S+H				10.173	3.998	0.110					
+D+0.60W+H	0.455	0.455		5.118	1.747	0.110					
+D+0.70E+H				13.620	3.787	0.110					
+D+0.750Lr+0.750L+0.450W+H	0.342	0.342		8.093	3.499	0.110					

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
t: 435.828.4200

www.dibbleengineers.com

Project Title:
Engineer:
Project ID:
Project Descr:

SG83

Printed: 23 AUG 2018, 4:45PM

Wood Column

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Gravity\Updated Roof Weights\peyree - main, fndt.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.8.15 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MH3 - Header east w/ SW HD

Maximum Reactions

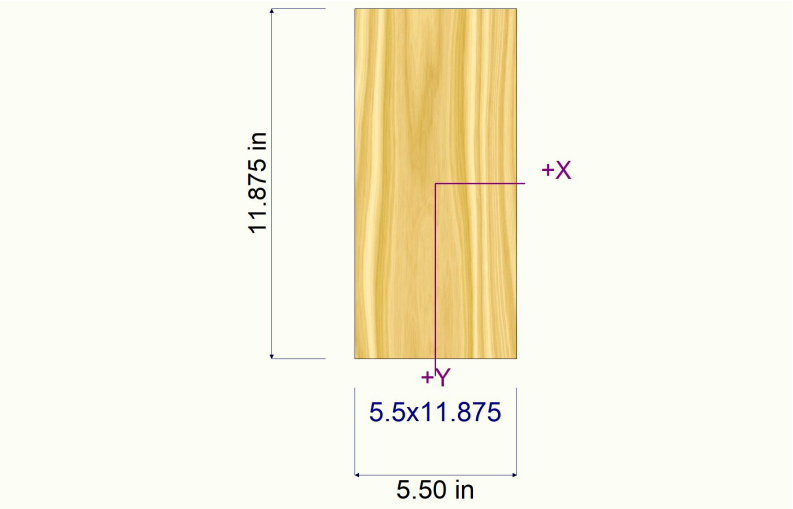
Note: Only non-zero reactions are listed.

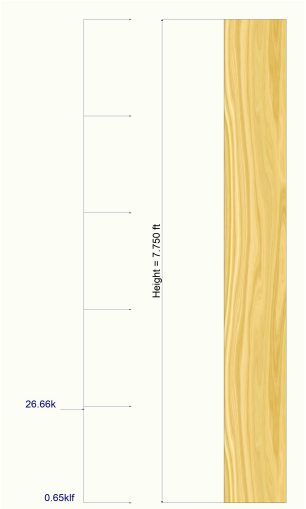
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750L+0.750S+0.450W+H	0.342	0.342		10.173	3.998	0.110					
+D+0.750L+0.750S+0.5250E+H				16.550	5.528	0.110					
+0.60D+0.60W+0.60H	0.455	0.455		3.071	1.048	0.066					
+0.60D+0.70E+0.60H				11.573	3.089	0.066					
D Only				5.118	1.747	0.110					
Lr Only											
L Only				3.966	2.336						
S Only				2.773	0.666						
W Only	0.759	0.759									
E Only				12.146	2.915						
H Only											

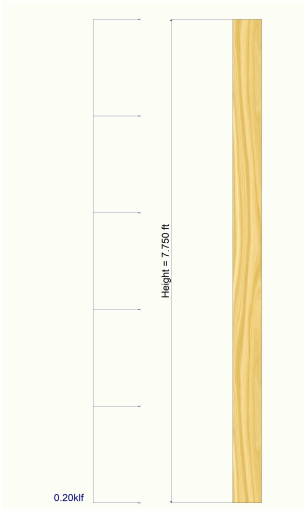
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.048 in	3.485 ft
+D+L+H	0.0000 in	0.000 ft	0.094 in	3.589 ft
+D+Lr+H	0.0000 in	0.000 ft	0.048 in	3.485 ft
+D+S+H	0.0000 in	0.000 ft	0.072 in	3.485 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.083 in	3.589 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.101 in	3.537 ft
+D+0.60W+H	0.0366 in	3.901 ft	0.048 in	3.485 ft
+D+0.70E+H	0.0000 in	0.000 ft	0.121 in	3.433 ft
+D+0.750Lr+0.750L+0.450W+H	0.0275 in	3.901 ft	0.083 in	3.589 ft
+D+0.750L+0.750S+0.450W+H	0.0275 in	3.901 ft	0.101 in	3.537 ft
+D+0.750L+0.750S+0.5250E+H	0.0000 in	0.000 ft	0.155 in	3.485 ft
+0.60D+0.60W+0.60H	0.0366 in	3.901 ft	0.029 in	3.485 ft
+0.60D+0.70E+0.60H	0.0000 in	0.000 ft	0.102 in	3.433 ft
D Only	0.0000 in	0.000 ft	0.048 in	3.485 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.046 in	3.693 ft
S Only	0.0000 in	0.000 ft	0.024 in	3.381 ft
W Only	0.0610 in	3.901 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.104 in	3.381 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







DEI
DIBBLE ENGINEERS INC

LATERAL

Peyree Remodel #17-291

Project Title:
Engineer:
Project ID:
Project Descr:

SL1

Printed: 15 MAY 2019, 9:54AM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

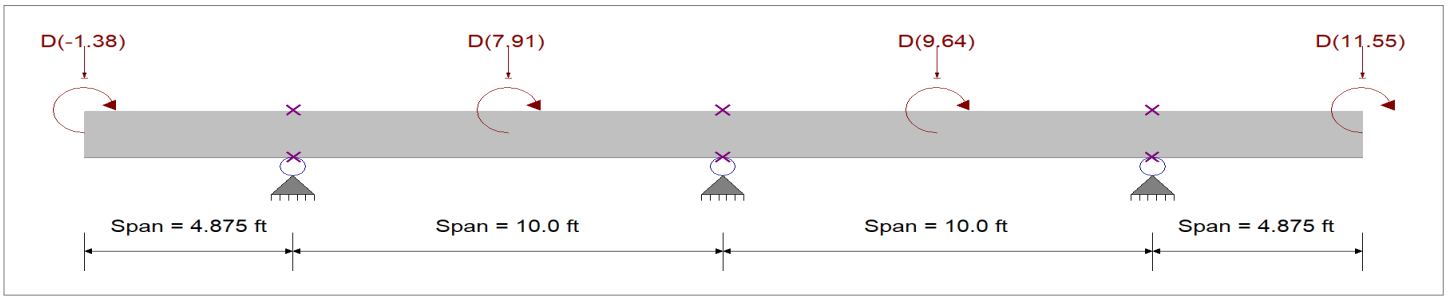
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 18 (input already factored) -- 3 supports

General Beam Properties

Elastic Modulus 1,000.0 ksi

Span #1	Span Length =	4.875 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #2	Span Length =	10.0 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #3	Span Length =	10.0 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #4	Span Length =	4.875 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4



Applied Loads

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

Load(s) for Span Number 1
Point Load : D = -1.380 k @ 0.0 ft

Moment : D = 23.930 k-ft, Loc = 0.0 ft in span

Load(s) for Span Number 2
Point Load : D = 7.910 k @ 5.0 ft

Moment : D = 34.550 k-ft, Loc = 5.0 ft in span

Load(s) for Span Number 3
Point Load : D = 9.640 k @ 5.0 ft

Moment : D = 34.590 k-ft, Loc = 5.0 ft in span

Load(s) for Span Number 4
Point Load : D = 11.550 k @ 4.875 ft

Moment : D = 24.440 k-ft, Loc = 4.875 ft in span

DESIGN SUMMARY

Maximum Bending =	80.746 k-ft	Maximum Shear =	15.961 k
Load Combination	+D+H	Load Combination	+D+H
Span # where maximum occurs	Span # 3	Span # where maximum occurs	Span # 3
Location of maximum on span	10.000 ft	Location of maximum on span	5.102 ft

Maximum Deflection

Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	2.944 in	38
Max Upward Total Deflection	-1.696 in	68

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values (k-ft) Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Shear Values (k) Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 18 (input already factored) -- 3 supports

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values (k-ft) Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Shear Values (k) Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+L+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+Lr+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+S+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.750Lr+0.750L+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.750L+0.750S+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.60W+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.750Lr+0.450W+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.750S+0.450W+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+0.60D+0.60W+0.60H													
Dsgn. L = 4.88 ft	1			18.39	14.36	18.39					1.78		
Dsgn. L = 10.00 ft	2			29.58	-2.36	29.58					6.52		
Dsgn. L = 10.00 ft	3			-0.00	-48.45	48.45					9.58		
Dsgn. L = 4.88 ft	4			-0.00	-48.45	48.45					6.93		
+D+0.70E+0.60H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 4.88 ft	1			30.66	23.93	30.66					2.96		
Dsgn. L = 10.00 ft	2			49.31	-3.93	49.31					10.87		
Dsgn. L = 10.00 ft	3			-0.00	-80.75	80.75					15.96		
Dsgn. L = 4.88 ft	4			-0.00	-80.75	80.75					11.55		
+0.60D+0.70E+H													
Dsgn. L = 4.88 ft	1			18.39	14.36	18.39					1.78		
Dsgn. L = 10.00 ft	2			29.58	-2.36	29.58					6.52		
Dsgn. L = 10.00 ft	3			-0.00	-48.45	48.45					9.58		
Dsgn. L = 4.88 ft	4			-0.00	-48.45	48.45					6.93		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	D Only	-1.6958	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

SL3

Printed: 15 MAY 2019, 9:54AM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 18 (input already factored) -- 3 supports

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	2	0.6000	5.306		0.0000	0.000
	3	0.0000	5.306	D Only	-0.5875	5.714
D Only	4	2.9440	4.875		0.0000	5.714

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum		-4.339	4.548	27.511	
Overall MINimum		-4.339			
+D+H		-4.339	4.548	27.511	
+D+L+H		-4.339	4.548	27.511	
+D+Lr+H		-4.339	4.548	27.511	
+D+S+H		-4.339	4.548	27.511	
+D+0.750Lr+0.750L+H		-4.339	4.548	27.511	
+D+0.750L+0.750S+H		-4.339	4.548	27.511	
+D+0.60W+H		-4.339	4.548	27.511	
+D+0.750Lr+0.450W+H		-4.339	4.548	27.511	
+D+0.750S+0.450W+H		-4.339	4.548	27.511	
+0.60D+0.60W+0.60H		-2.603	2.729	16.506	
+D+0.70E+0.60H		-4.339	4.548	27.511	
+D+0.750L+0.750S+0.5250E+H		-4.339	4.548	27.511	
+0.60D+0.70E+H		-2.603	2.729	16.506	
D Only		-4.339	4.548	27.511	
Lr Only					
L Only					
S Only					
W Only					
E Only					
H Only					

Therefore, (1) Helical Pile and (1) Pin Pile Req'd Each End, (1) Pin Pile Req'd Center

Project Title:
Engineer:
Project ID:
Project Descr:

SL4

Printed: 15 MAY 2019, 9:52AM

General Beam Analysis

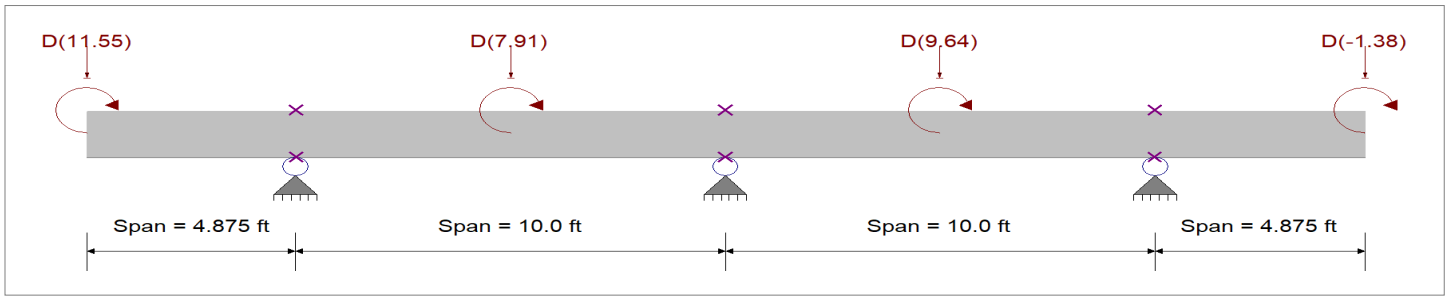
= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 19 (input already factored) -- 3 supports

General Beam Properties

Elastic Modulus	1,000.0 ksi							
Span #1	Span Length =	4.875 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #2	Span Length =	10.0 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #3	Span Length =	10.0 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4
Span #4	Span Length =	4.875 ft	Area =	1.0 in^2	Moment of Inertia =	1,000.0 in^4



Applied Loads

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

Load(s) for Span Number 1
Point Load : D = 11.550 k @ 0.0 ft

Moment : D = -24.440 k-ft, Loc = 0.0 ft in span

Load(s) for Span Number 2
Point Load : D = 7.910 k @ 5.0 ft

Moment : D = -34.590 k-ft, Loc = 5.0 ft in span

Load(s) for Span Number 3
Point Load : D = 9.640 k @ 5.0 ft

Moment : D = -34.550 k-ft, Loc = 5.0 ft in span

Load(s) for Span Number 4
Point Load : D = -1.380 k @ 4.875 ft

Moment : D = -23.930 k-ft, Loc = 4.875 ft in span

DESIGN SUMMARY

Maximum Bending =	80.746 k-ft	Maximum Shear =	15.096 k
Load Combination	+D+H	Load Combination	+D+H
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	4.875 ft	Location of maximum on span	4.875 ft

Maximum Deflection

Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	3.079 in	36
Max Upward Total Deflection	-1.764 in	66

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)							Shear Values (k)		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		

Project Title:
Engineer:
Project ID:
Project Descr:

SL5

Printed: 15 MAY 2019, 9:52AM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 19 (input already factored) -- 3 supports

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
				Summary of Moment Values (k-ft)							Shear Values (k)		
+D+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+L+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+Lr+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+S+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.750Lr+0.750L+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.750L+0.750S+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.60W+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.750Lr+0.450W+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.750S+0.450W+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+0.60D+0.60W+0.60H													
Dsgn. L = 4.88 ft	1			-0.00	-48.45	48.45					9.06		
Dsgn. L = 10.00 ft	2			-0.00	-48.45	48.45					9.06		
Dsgn. L = 10.00 ft	3			32.13	-2.36	32.13					7.04		
Dsgn. L = 4.88 ft	4			18.39	14.36	18.39					0.83		
+D+0.70E+0.60H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 4.88 ft	1			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	2			-0.00	-80.75	80.75					15.10		
Dsgn. L = 10.00 ft	3			53.54	-3.93	53.54					11.73		
Dsgn. L = 4.88 ft	4			30.66	23.93	30.66					1.38		
+0.60D+0.70E+H													
Dsgn. L = 4.88 ft	1			-0.00	-48.45	48.45					9.06		
Dsgn. L = 10.00 ft	2			-0.00	-48.45	48.45					9.06		
Dsgn. L = 10.00 ft	3			32.13	-2.36	32.13					7.04		
Dsgn. L = 4.88 ft	4			18.39	14.36	18.39					0.83		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	3.0792	0.000		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

SL6

Printed: 15 MAY 2019, 9:52AM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 19 (input already factored) -- 3 supports

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	2	0.0000	0.000	D Only	-0.6688	4.694
D Only	3	0.6588	4.898		0.0000	4.694
	4	0.0000	4.898	D Only	-1.7640	4.875

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum		26.646	4.548	-3.474	
Overall MINimum				-3.474	
+D+H		26.646	4.548	-3.474	
+D+L+H		26.646	4.548	-3.474	
+D+Lr+H		26.646	4.548	-3.474	
+D+S+H		26.646	4.548	-3.474	
+D+0.750Lr+0.750L+H		26.646	4.548	-3.474	
+D+0.750L+0.750S+H		26.646	4.548	-3.474	
+D+0.60W+H		26.646	4.548	-3.474	
+D+0.750Lr+0.450W+H		26.646	4.548	-3.474	
+D+0.750S+0.450W+H		26.646	4.548	-3.474	
+0.60D+0.60W+0.60H		15.987	2.729	-2.084	
+D+0.70E+0.60H		26.646	4.548	-3.474	
+D+0.750L+0.750S+0.5250E+H		26.646	4.548	-3.474	
+0.60D+0.70E+H		15.987	2.729	-2.084	
D Only		26.646	4.548	-3.474	
Lr Only					
L Only					
S Only					
W Only					
E Only					
H Only					

Therefore, (1) Helical Pile and (1) Pin Pile Req'd Each End, (1) Pin Pile Req'd Center

Project Title:
Engineer:
Project ID:
Project Descr:

SL7

Checking Beam Reinforcement, LRFD

Printed: 15 MAY 2019, 9:55AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

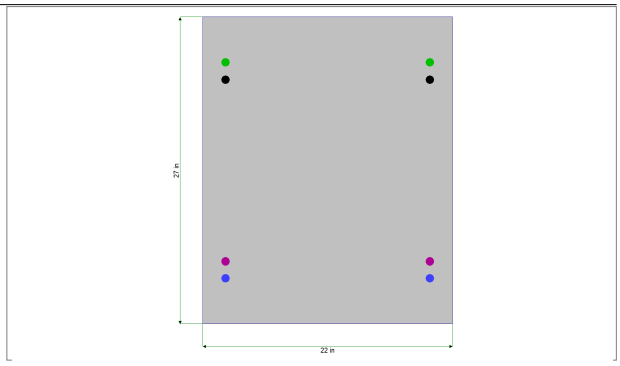
Description : MF 1: LC 56 (input already factored) -- 3 supports

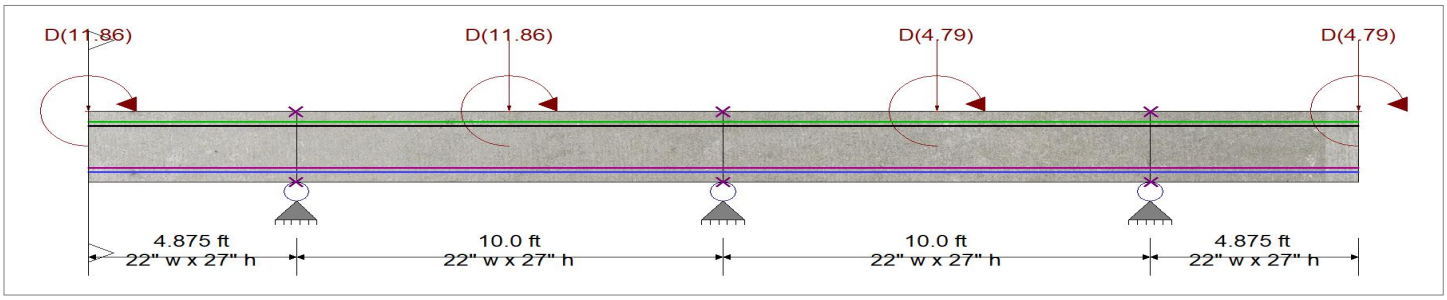
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 27.0 in

Span #1 Reinforcing....
- 2-#6 at 4.0 in from Bottom, from 0.0 to 4.875 ft in this span
- 2-#6 at 5.50 in from Bottom, from 0.0 to 4.875 ft in this span
- 2-#6 at 4.0 in from Top, from 0.0 to 4.875 ft in this span
- 2-#6 at 5.50 in from Top, from 0.0 to 4.875 ft in this span

Span #2 Reinforcing....
- 2-#6 at 4.0 in from Bottom, from 0.0 to 10.0 ft in this span
- 2-#6 at 5.50 in from Bottom, from 0.0 to 10.0 ft in this span
- 2-#6 at 4.0 in from Top, from 0.0 to 10.0 ft in this span
- 2-#6 at 5.50 in from Top, from 0.0 to 10.0 ft in this span

Span #3 Reinforcing....
- 2-#6 at 4.0 in from Bottom, from 0.0 to 10.0 ft in this span
- 2-#6 at 5.50 in from Bottom, from 0.0 to 10.0 ft in this span
- 2-#6 at 4.0 in from Top, from 0.0 to 10.0 ft in this span
- 2-#6 at 5.50 in from Top, from 0.0 to 10.0 ft in this span

Span #4 Reinforcing....
- 2-#6 at 4.0 in from Bottom, from 0.0 to 4.875 ft in this span
- 2-#6 at 5.50 in from Bottom, from 0.0 to 4.875 ft in this span
- 2-#6 at 4.0 in from Top, from 0.0 to 4.875 ft in this span
- 2-#6 at 5.50 in from Top, from 0.0 to 4.875 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads

Load for Span Number 1
- Point Load : D = 11.860 k @ 0.0 ft
- Moment : D = -49.420 k-ft, Location = 0.0 ft from left end of this span

Load for Span Number 2
- Point Load : D = 11.860 k @ 5.0 ft
- Moment : D = -49.420 k-ft, Location = 5.0 ft from left end of this span

Load for Span Number 3
- Point Load : D = 4.790 k @ 5.0 ft

Project Title:
Engineer:
Project ID:
Project Descr:

SL8

Printed: 15 MAY 2019, 9:55AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Applied Loads

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

Moment : D = -49.350 k-ft, Location = 5.0 ft from left end of this span

Load for Span Number 4
Point Load : D = 4.790 k @ 4.875 ft

Moment : D = -49.350 k-ft, Location = 4.875 ft from left end of this span

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.632** : 1

Section used for this span	**Typical Section**	
Mu : Applied	-114.335	k-ft
Mn * Phi : Allowable	180.922	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 2	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Downward Total Deflection	0.045 in	Ratio =	2624	>=180.
Max Upward Total Deflection	-0.013 in	Ratio =	9064	>=180.

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum		40.171	4.557	6.366	
Overall MINimum		24.090	2.760	3.807	
+D+H		40.171	4.557	6.366	
+D+L+H		40.171	4.557	6.366	
+D+Lr+H		40.171	4.557	6.366	
+D+S+H		40.171	4.557	6.366	
+D+0.750Lr+0.750L+H		40.171	4.557	6.366	
+D+0.750L+0.750S+H		40.171	4.557	6.366	
+D+0.60W+H		40.171	4.557	6.366	
+D+0.750Lr+0.450W+H		40.171	4.557	6.366	
+D+0.750S+0.450W+H		40.171	4.557	6.366	
+0.60D+0.60W+0.60H		24.090	2.760	3.807	
+D+0.70E+0.60H		40.171	4.557	6.366	
+D+0.750L+0.750S+0.5250E+H		40.171	4.557	6.366	
+0.60D+0.70E+H		24.090	2.760	3.807	
D Only		40.171	4.557	6.366	
Lr Only					
L Only					
S Only					
W Only					
E Only					
H Only					

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	0.00	23.00	-0.00	0.00	0.00	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	1	0.05	23.00	-11.89	11.89	50.02	0.46	41.00	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	1	0.10	23.00	-11.92	11.92	50.63	0.45	40.98	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	1	0.15	23.00	-11.95	11.95	51.24	0.45	40.97	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	1	0.21	23.00	-11.98	11.98	51.86	0.44	40.96	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	1	0.26	23.00	-12.01	12.01	52.47	0.44	40.94	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.31	23.00	-12.04	12.04	53.09	0.43	40.93	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.36	23.00	-12.07	12.07	53.71	0.43	40.92	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.41	23.00	-12.11	12.11	54.33	0.43	40.90	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.46	23.00	-12.14	12.14	54.95	0.42	40.89	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.51	23.00	-12.17	12.17	55.57	0.42	40.88	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.56	23.00	-12.20	12.20	56.20	0.42	40.87	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.62	23.00	-12.23	12.23	56.83	0.41	40.85	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	1	0.67	23.00	-12.26	12.26	57.46	0.41	40.84	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0

Project Title:
Engineer:
Project ID:
Project Descr:

SL9

Printed: 15 MAY 2019, 9:55AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	0.72	23.00	-12.29	12.29	58.08	0.41	40.83	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	0.77	23.00	-12.32	12.32	58.72	0.40	40.82	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	0.82	23.00	-12.35	12.35	59.35	0.40	40.81	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	0.87	23.00	-12.38	12.38	59.98	0.40	40.80	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	0.92	23.00	-12.41	12.41	60.62	0.39	40.79	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	0.98	23.00	-12.44	12.44	61.26	0.39	40.78	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	1.03	23.00	-12.47	12.47	61.90	0.39	40.77	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	1.08	23.00	-12.50	12.50	62.54	0.38	40.76	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	1	1.13	23.00	-12.54	12.54	63.18	0.38	40.75	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.18	23.00	-12.57	12.57	63.82	0.38	40.74	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.23	23.00	-12.60	12.60	64.47	0.37	40.73	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.28	23.00	-12.63	12.63	65.12	0.37	40.72	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.33	23.00	-12.66	12.66	65.77	0.37	40.71	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.39	23.00	-12.69	12.69	66.42	0.37	40.70	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.44	23.00	-12.72	12.72	67.07	0.36	40.69	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.49	23.00	-12.75	12.75	67.72	0.36	40.68	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.54	23.00	-12.78	12.78	68.38	0.36	40.68	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.59	23.00	-12.81	12.81	69.03	0.36	40.67	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.64	23.00	-12.84	12.84	69.69	0.35	40.66	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.69	23.00	-12.87	12.87	70.35	0.35	40.65	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	1	1.74	23.00	-12.90	12.90	71.01	0.35	40.64	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	1.80	23.00	-12.93	12.93	71.68	0.35	40.63	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	1.85	23.00	-12.96	12.96	72.34	0.34	40.63	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	1.90	23.00	-13.00	13.00	73.01	0.34	40.62	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	1.95	23.00	-13.03	13.03	73.67	0.34	40.61	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.00	23.00	-13.06	13.06	74.34	0.34	40.60	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.05	23.00	-13.09	13.09	75.01	0.33	40.60	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.10	23.00	-13.12	13.12	75.69	0.33	40.59	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.16	23.00	-13.15	13.15	76.36	0.33	40.58	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.21	23.00	-13.18	13.18	77.04	0.33	40.58	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.26	23.00	-13.21	13.21	77.71	0.33	40.57	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.31	23.00	-13.24	13.24	78.39	0.32	40.56	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.36	23.00	-13.27	13.27	79.07	0.32	40.56	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	1	2.41	23.00	-13.30	13.30	79.75	0.32	40.55	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.46	23.00	-13.33	13.33	80.44	0.32	40.54	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.51	23.00	-13.36	13.36	81.12	0.32	40.54	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.57	23.00	-13.39	13.39	81.81	0.31	40.53	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.62	23.00	-13.43	13.43	82.50	0.31	40.52	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.67	23.00	-13.46	13.46	83.19	0.31	40.52	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.72	23.00	-13.49	13.49	83.88	0.31	40.51	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.77	23.00	-13.52	13.52	84.57	0.31	40.50	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.82	23.00	-13.55	13.55	85.27	0.30	40.50	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.87	23.00	-13.58	13.58	85.96	0.30	40.49	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.93	23.00	-13.61	13.61	86.66	0.30	40.49	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	2.98	23.00	-13.64	13.64	87.36	0.30	40.48	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.03	23.00	-13.67	13.67	88.06	0.30	40.48	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.08	23.00	-13.70	13.70	88.76	0.30	40.47	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.13	23.00	-13.73	13.73	89.47	0.29	40.46	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.18	23.00	-13.76	13.76	90.17	0.29	40.46	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.23	23.00	-13.79	13.79	90.88	0.29	40.45	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	1	3.28	23.00	-13.82	13.82	91.59	0.29	40.45	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.34	23.00	-13.86	13.86	92.30	0.29	40.44	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.39	23.00	-13.89	13.89	93.01	0.29	40.44	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.44	23.00	-13.92	13.92	93.72	0.28	40.43	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0

Concrete Beam

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	3.49	23.00	-13.95	13.95	94.44	0.28	40.43	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.54	23.00	-13.98	13.98	95.15	0.28	40.42	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.59	23.00	-14.01	14.01	95.87	0.28	40.42	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.64	23.00	-14.04	14.04	96.59	0.28	40.41	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.69	23.00	-14.07	14.07	97.31	0.28	40.41	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.75	23.00	-14.10	14.10	98.04	0.28	40.40	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.80	23.00	-14.13	14.13	98.76	0.27	40.40	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.85	23.00	-14.16	14.16	99.49	0.27	40.39	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.90	23.00	-14.19	14.19	100.21	0.27	40.39	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	3.95	23.00	-14.22	14.22	100.94	0.27	40.38	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.00	23.00	-14.25	14.25	101.67	0.27	40.38	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.05	23.00	-14.28	14.28	102.40	0.27	40.38	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.11	23.00	-14.32	14.32	103.14	0.27	40.37	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.16	23.00	-14.35	14.35	103.87	0.26	40.37	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.21	23.00	-14.38	14.38	104.61	0.26	40.36	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.26	23.00	-14.41	14.41	105.35	0.26	40.36	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.31	23.00	-14.44	14.44	106.09	0.26	40.35	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.36	23.00	-14.47	14.47	106.83	0.26	40.35	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	1	4.41	23.00	-14.50	14.50	107.57	0.26	40.35	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.46	23.00	-14.53	14.53	108.32	0.26	40.34	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.52	23.00	-14.56	14.56	109.07	0.26	40.34	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.57	23.00	-14.59	14.59	109.81	0.25	40.33	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.62	23.00	-14.62	14.62	110.56	0.25	40.33	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.67	23.00	-14.65	14.65	111.31	0.25	40.33	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.72	23.00	-14.68	14.68	112.07	0.25	40.32	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.77	23.00	-14.71	14.71	112.82	0.25	40.32	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	1	4.82	23.00	-14.75	14.75	113.58	0.25	40.31	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	2	4.88	23.00	25.37	25.37	114.34	0.43	40.90	PhiVc/2 < Vu <=	Min 11.5.6.3	59.9	8.0	8.0
D Only	2	4.98	23.00	25.31	25.31	111.67	0.43	40.93	PhiVc/2 < Vu <=	Min 11.5.6.3	59.9	8.0	8.0
D Only	2	5.09	23.00	25.25	25.25	109.01	0.44	40.96	PhiVc/2 < Vu <=	Min 11.5.6.3	59.9	8.0	8.0
D Only	2	5.19	23.00	25.19	25.19	106.35	0.45	40.99	PhiVc/2 < Vu <=	Min 11.5.6.3	60.0	8.0	8.0
D Only	2	5.30	23.00	25.12	25.12	103.70	0.46	41.03	PhiVc/2 < Vu <=	Min 11.5.6.3	60.0	8.0	8.0
D Only	2	5.40	23.00	25.06	25.06	101.06	0.48	41.06	PhiVc/2 < Vu <=	Min 11.5.6.3	60.0	8.0	8.0
D Only	2	5.51	23.00	25.00	25.00	98.43	0.49	41.10	PhiVc/2 < Vu <=	Min 11.5.6.3	60.1	8.0	8.0
D Only	2	5.61	23.00	24.93	24.93	95.80	0.50	41.14	PhiVc/2 < Vu <=	Min 11.5.6.3	60.1	8.0	8.0
D Only	2	5.72	23.00	24.87	24.87	93.18	0.51	41.18	PhiVc/2 < Vu <=	Min 11.5.6.3	60.2	8.0	8.0
D Only	2	5.82	23.00	24.81	24.81	90.56	0.53	41.23	PhiVc/2 < Vu <=	Min 11.5.6.3	60.2	8.0	8.0
D Only	2	5.93	23.00	24.74	24.74	87.96	0.54	41.27	PhiVc/2 < Vu <=	Min 11.5.6.3	60.2	8.0	8.0
D Only	2	6.03	23.00	24.68	24.68	85.36	0.55	41.32	PhiVc/2 < Vu <=	Min 11.5.6.3	60.3	8.0	8.0
D Only	2	6.14	23.00	24.62	24.62	82.76	0.57	41.37	PhiVc/2 < Vu <=	Min 11.5.6.3	60.3	8.0	8.0
D Only	2	6.24	23.00	24.56	24.56	80.17	0.59	41.43	PhiVc/2 < Vu <=	Min 11.5.6.3	60.4	8.0	8.0
D Only	2	6.35	23.00	24.49	24.49	77.59	0.61	41.49	PhiVc/2 < Vu <=	Min 11.5.6.3	60.5	8.0	8.0
D Only	2	6.45	23.00	24.43	24.43	75.02	0.62	41.55	PhiVc/2 < Vu <=	Min 11.5.6.3	60.5	8.0	8.0
D Only	2	6.56	23.00	24.37	24.37	72.45	0.64	41.62	PhiVc/2 < Vu <=	Min 11.5.6.3	60.6	8.0	8.0
D Only	2	6.66	23.00	24.30	24.30	69.89	0.67	41.69	PhiVc/2 < Vu <=	Min 11.5.6.3	60.7	8.0	8.0
D Only	2	6.77	23.00	24.24	24.24	67.33	0.69	41.77	PhiVc/2 < Vu <=	Min 11.5.6.3	60.7	8.0	8.0
D Only	2	6.88	23.00	24.18	24.18	64.78	0.72	41.85	PhiVc/2 < Vu <=	Min 11.5.6.3	60.8	8.0	8.0
D Only	2	6.98	23.00	24.11	24.11	62.24	0.74	41.94	PhiVc/2 < Vu <=	Min 11.5.6.3	60.9	8.0	8.0
D Only	2	7.09	23.00	24.05	24.05	59.71	0.77	42.04	PhiVc/2 < Vu <=	Min 11.5.6.3	61.0	8.0	8.0
D Only	2	7.19	23.00	23.99	23.99	57.18	0.80	42.15	PhiVc/2 < Vu <=	Min 11.5.6.3	61.1	8.0	8.0
D Only	2	7.30	23.00	23.93	23.93	54.66	0.84	42.26	PhiVc/2 < Vu <=	Min 11.5.6.3	61.2	8.0	8.0
D Only	2	7.40	23.00	23.86	23.86	52.14	0.88	42.39	PhiVc/2 < Vu <=	Min 11.5.6.3	61.4	8.0	8.0
D Only	2	7.51	23.00	23.80	23.80	49.63	0.92	42.53	PhiVc/2 < Vu <=	Min 11.5.6.3	61.5	8.0	8.0
D Only	2	7.61	23.00	23.74	23.74	47.13	0.97	42.68	PhiVc/2 < Vu <=	Min 11.5.6.3	61.7	8.0	8.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	2	7.72	23.00	23.67	23.67	44.64	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	7.82	23.00	23.61	23.61	42.15	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	7.93	23.00	23.55	23.55	39.66	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.03	23.00	23.49	23.49	37.19	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.14	23.00	23.42	23.42	34.72	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.24	23.00	23.36	23.36	32.26	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.35	23.00	23.30	23.30	29.80	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.45	23.00	23.23	23.23	27.35	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.56	23.00	23.17	23.17	24.91	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.66	23.00	23.11	23.11	22.48	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.77	23.00	23.04	23.04	20.05	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.88	23.00	22.98	22.98	17.62	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	8.98	23.00	22.92	22.92	15.21	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.09	23.00	22.86	22.86	12.80	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.19	23.00	22.79	22.79	10.40	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.30	23.00	22.73	22.73	8.00	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.40	23.00	22.67	22.67	5.61	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.51	23.00	22.60	22.60	3.23	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.61	23.00	22.54	22.54	0.85	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.72	23.00	22.48	22.48	1.52	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.82	23.00	22.41	22.41	3.88	1.00	42.79	PhiVc/2 < Vu <=	Min 11.5.6.3	61.8	8.0	8.0
D Only	2	9.93	23.00	10.49	10.49	43.81	0.46	41.01	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	2	10.03	23.00	10.43	10.43	42.71	0.47	41.04	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	2	10.14	23.00	10.37	10.37	41.61	0.48	41.07	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	2	10.24	23.00	10.30	10.30	40.53	0.49	41.10	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	2	10.35	23.00	10.24	10.24	39.44	0.50	41.14	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	2	10.45	23.00	10.18	10.18	38.37	0.51	41.17	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	2	10.56	23.00	10.11	10.11	37.30	0.52	41.21	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	2	10.66	23.00	10.05	10.05	36.24	0.53	41.25	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	2	10.77	23.00	9.99	9.99	35.19	0.54	41.29	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	2	10.88	23.00	9.93	9.93	34.14	0.56	41.33	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	2	10.98	23.00	9.86	9.86	33.10	0.57	41.38	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	2	11.09	23.00	9.80	9.80	32.06	0.59	41.43	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	2	11.19	23.00	9.74	9.74	31.03	0.60	41.48	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	2	11.30	23.00	9.67	9.67	30.01	0.62	41.53	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	2	11.40	23.00	9.61	9.61	29.00	0.64	41.59	Vu < PhiVc/2	lot Reqd 9.6.	41.6	0.0	0.0
D Only	2	11.51	23.00	9.55	9.55	27.99	0.65	41.65	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	2	11.61	23.00	9.48	9.48	26.99	0.67	41.72	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	2	11.72	23.00	9.42	9.42	25.99	0.69	41.79	Vu < PhiVc/2	lot Reqd 9.6.	41.8	0.0	0.0
D Only	2	11.82	23.00	9.36	9.36	25.00	0.72	41.86	Vu < PhiVc/2	lot Reqd 9.6.	41.9	0.0	0.0
D Only	2	11.93	23.00	9.30	9.30	24.02	0.74	41.94	Vu < PhiVc/2	lot Reqd 9.6.	41.9	0.0	0.0
D Only	2	12.03	23.00	9.23	9.23	23.05	0.77	42.03	Vu < PhiVc/2	lot Reqd 9.6.	42.0	0.0	0.0
D Only	2	12.14	23.00	9.17	9.17	22.08	0.80	42.12	Vu < PhiVc/2	lot Reqd 9.6.	42.1	0.0	0.0
D Only	2	12.24	23.00	9.11	9.11	21.12	0.83	42.22	Vu < PhiVc/2	lot Reqd 9.6.	42.2	0.0	0.0
D Only	2	12.35	23.00	9.04	9.04	20.16	0.86	42.33	Vu < PhiVc/2	lot Reqd 9.6.	42.3	0.0	0.0
D Only	2	12.45	23.00	8.98	8.98	19.21	0.90	42.45	Vu < PhiVc/2	lot Reqd 9.6.	42.5	0.0	0.0
D Only	2	12.56	23.00	8.92	8.92	18.27	0.94	42.58	Vu < PhiVc/2	lot Reqd 9.6.	42.6	0.0	0.0
D Only	2	12.66	23.00	8.85	8.85	17.33	0.98	42.72	Vu < PhiVc/2	lot Reqd 9.6.	42.7	0.0	0.0
D Only	2	12.77	23.00	8.79	8.79	16.41	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	12.88	23.00	8.73	8.73	15.48	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	12.98	23.00	8.67	8.67	14.57	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.09	23.00	8.60	8.60	13.66	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.19	23.00	8.54	8.54	12.76	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.30	23.00	8.48	8.48	11.86	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	2	13.40	23.00	8.41	8.41	10.97	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.51	23.00	8.35	8.35	10.09	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.61	23.00	8.29	8.29	9.21	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.72	23.00	8.23	8.23	8.34	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.82	23.00	8.16	8.16	7.48	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	13.93	23.00	8.10	8.10	6.63	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.03	23.00	8.04	8.04	5.78	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.14	23.00	7.97	7.97	4.93	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.24	23.00	7.91	7.91	4.10	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.35	23.00	7.85	7.85	3.27	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.45	23.00	7.78	7.78	2.45	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.56	23.00	7.72	7.72	1.63	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.66	23.00	7.66	7.66	0.82	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	2	14.77	23.00	7.60	7.60	0.02	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	14.88	23.00	12.13	12.13	0.78	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	14.98	23.00	12.07	12.07	2.05	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.09	23.00	12.01	12.01	3.32	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.19	23.00	11.94	11.94	4.58	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.30	23.00	11.88	11.88	5.83	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.40	23.00	11.82	11.82	7.08	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.51	23.00	11.75	11.75	8.32	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.61	23.00	11.69	11.69	9.56	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.72	23.00	11.63	11.63	10.78	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.82	23.00	11.57	11.57	12.00	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	15.93	23.00	11.50	11.50	13.22	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.03	23.00	11.44	11.44	14.43	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.14	23.00	11.38	11.38	15.63	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.24	23.00	11.31	11.31	16.82	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.35	23.00	11.25	11.25	18.01	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.45	23.00	11.19	11.19	19.19	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.56	23.00	11.12	11.12	20.36	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	16.66	23.00	11.06	11.06	21.53	0.98	42.74	Vu < PhiVc/2	lot Reqd 9.6.	42.7	0.0	0.0
D Only	3	16.77	23.00	11.00	11.00	22.69	0.93	42.56	Vu < PhiVc/2	lot Reqd 9.6.	42.6	0.0	0.0
D Only	3	16.88	23.00	10.94	10.94	23.85	0.88	42.39	Vu < PhiVc/2	lot Reqd 9.6.	42.4	0.0	0.0
D Only	3	16.98	23.00	10.87	10.87	24.99	0.83	42.25	Vu < PhiVc/2	lot Reqd 9.6.	42.2	0.0	0.0
D Only	3	17.09	23.00	10.81	10.81	26.14	0.79	42.11	Vu < PhiVc/2	lot Reqd 9.6.	42.1	0.0	0.0
D Only	3	17.19	23.00	10.75	10.75	27.27	0.76	41.99	Vu < PhiVc/2	lot Reqd 9.6.	42.0	0.0	0.0
D Only	3	17.30	23.00	10.68	10.68	28.40	0.72	41.87	Vu < PhiVc/2	lot Reqd 9.6.	41.9	0.0	0.0
D Only	3	17.40	23.00	10.62	10.62	29.52	0.69	41.77	Vu < PhiVc/2	lot Reqd 9.6.	41.8	0.0	0.0
D Only	3	17.51	23.00	10.56	10.56	30.63	0.66	41.67	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	3	17.61	23.00	10.50	10.50	31.74	0.63	41.58	Vu < PhiVc/2	lot Reqd 9.6.	41.6	0.0	0.0
D Only	3	17.72	23.00	10.43	10.43	32.84	0.61	41.50	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	3	17.82	23.00	10.37	10.37	33.94	0.59	41.43	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	3	17.93	23.00	10.31	10.31	35.03	0.56	41.35	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	3	18.03	23.00	10.24	10.24	36.11	0.54	41.29	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	3	18.14	23.00	10.18	10.18	37.18	0.52	41.23	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	3	18.24	23.00	10.12	10.12	38.25	0.51	41.17	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	3	18.35	23.00	10.05	10.05	39.31	0.49	41.11	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	3	18.45	23.00	9.99	9.99	40.37	0.47	41.06	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	3	18.56	23.00	9.93	9.93	41.42	0.46	41.01	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	3	18.66	23.00	9.87	9.87	42.46	0.45	40.96	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	3	18.77	23.00	9.80	9.80	43.49	0.43	40.92	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	3	18.88	23.00	9.74	9.74	44.52	0.42	40.88	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	3	18.98	23.00	9.68	9.68	45.54	0.41	40.84	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0

Concrete Beam

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	3	19.09	23.00	9.61	9.61	46.56	0.40	40.80	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	3	19.19	23.00	9.55	9.55	47.57	0.38	40.76	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	3	19.30	23.00	9.49	9.49	48.57	0.37	40.73	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	3	19.40	23.00	9.43	9.43	49.57	0.36	40.70	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	3	19.51	23.00	9.36	9.36	50.55	0.35	40.66	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	3	19.61	23.00	9.30	9.30	51.54	0.35	40.63	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	3	19.72	23.00	9.24	9.24	52.51	0.34	40.61	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	3	19.82	23.00	9.17	9.17	53.48	0.33	40.58	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	3	19.93	23.00	4.32	4.32	4.84	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.03	23.00	4.26	4.26	5.29	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.14	23.00	4.19	4.19	5.74	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.24	23.00	4.13	4.13	6.18	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.35	23.00	4.07	4.07	6.61	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.45	23.00	4.01	4.01	7.03	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.56	23.00	3.94	3.94	7.45	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0
D Only	3	20.66	23.00	3.88	3.88	7.86	0.95	42.61	Vu < PhiVc/2	lot Reqd 9.6.	42.6	0.0	0.0
D Only	3	20.77	23.00	3.82	3.82	8.27	0.88	42.41	Vu < PhiVc/2	lot Reqd 9.6.	42.4	0.0	0.0
D Only	3	20.88	23.00	3.75	3.75	8.67	0.83	42.23	Vu < PhiVc/2	lot Reqd 9.6.	42.2	0.0	0.0
D Only	3	20.98	23.00	3.69	3.69	9.06	0.78	42.07	Vu < PhiVc/2	lot Reqd 9.6.	42.1	0.0	0.0
D Only	3	21.09	23.00	3.63	3.63	9.44	0.74	41.92	Vu < PhiVc/2	lot Reqd 9.6.	41.9	0.0	0.0
D Only	3	21.19	23.00	3.56	3.56	9.82	0.70	41.79	Vu < PhiVc/2	lot Reqd 9.6.	41.8	0.0	0.0
D Only	3	21.30	23.00	3.50	3.50	10.19	0.66	41.67	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	3	21.40	23.00	3.44	3.44	10.56	0.62	41.55	Vu < PhiVc/2	lot Reqd 9.6.	41.6	0.0	0.0
D Only	3	21.51	23.00	3.38	3.38	10.92	0.59	41.45	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	3	21.61	23.00	3.31	3.31	11.27	0.56	41.35	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	3	21.72	23.00	3.25	3.25	11.61	0.54	41.26	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	3	21.82	23.00	3.19	3.19	11.95	0.51	41.18	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	3	21.93	23.00	3.12	3.12	12.29	0.49	41.10	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	3	22.03	23.00	3.06	3.06	12.61	0.47	41.03	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	3	22.14	23.00	3.00	3.00	12.93	0.44	40.96	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	3	22.24	23.00	2.94	2.94	13.24	0.42	40.90	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	3	22.35	23.00	2.87	2.87	13.55	0.41	40.83	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	3	22.45	23.00	2.81	2.81	13.85	0.39	40.78	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	3	22.56	23.00	2.75	2.75	14.14	0.37	40.72	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	3	22.66	23.00	2.68	2.68	14.43	0.36	40.67	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	3	22.77	23.00	2.62	2.62	14.70	0.34	40.62	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	3	22.88	23.00	2.56	2.56	14.98	0.33	40.57	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	3	22.98	23.00	2.49	2.49	15.24	0.31	40.53	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	3	23.09	23.00	2.43	2.43	15.50	0.30	40.49	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	3	23.19	23.00	2.37	2.37	15.75	0.29	40.44	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	3	23.30	23.00	2.31	2.31	16.00	0.28	40.40	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	3	23.40	23.00	2.24	2.24	16.24	0.26	40.37	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	3	23.51	23.00	2.18	2.18	16.47	0.25	40.33	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	3	23.61	23.00	2.12	2.12	16.70	0.24	40.30	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	3	23.72	23.00	2.05	2.05	16.92	0.23	40.26	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	3	23.82	23.00	1.99	1.99	17.13	0.22	40.23	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	3	23.93	23.00	1.93	1.93	17.34	0.21	40.20	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	3	24.03	23.00	1.86	1.86	17.54	0.20	40.17	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	3	24.14	23.00	1.80	1.80	17.73	0.19	40.14	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	3	24.24	23.00	1.74	1.74	17.92	0.19	40.11	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	3	24.35	23.00	1.68	1.68	18.10	0.18	40.08	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	3	24.45	23.00	1.61	1.61	18.27	0.17	40.05	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	3	24.56	23.00	1.55	1.55	18.44	0.16	40.03	Vu < PhiVc/2	lot Reqd 9.6.	40.0	0.0	0.0
D Only	3	24.66	23.00	1.49	1.49	18.60	0.15	40.00	Vu < PhiVc/2	lot Reqd 9.6.	40.0	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	3	24.77	23.00	1.42	1.42	18.75	0.15	39.97	Vu < PhiVc/2	lot Reqd 9.6.	40.0	0.0	0.0
D Only	4	24.88	23.00	7.71	7.71	18.90	0.78	42.07	Vu < PhiVc/2	lot Reqd 9.6.	42.1	0.0	0.0
D Only	4	24.93	23.00	7.68	7.68	19.29	0.76	42.01	Vu < PhiVc/2	lot Reqd 9.6.	42.0	0.0	0.0
D Only	4	24.98	23.00	7.64	7.64	19.68	0.74	41.95	Vu < PhiVc/2	lot Reqd 9.6.	42.0	0.0	0.0
D Only	4	25.03	23.00	7.61	7.61	20.07	0.73	41.89	Vu < PhiVc/2	lot Reqd 9.6.	41.9	0.0	0.0
D Only	4	25.08	23.00	7.58	7.58	20.46	0.71	41.84	Vu < PhiVc/2	lot Reqd 9.6.	41.8	0.0	0.0
D Only	4	25.13	23.00	7.55	7.55	20.85	0.69	41.78	Vu < PhiVc/2	lot Reqd 9.6.	41.8	0.0	0.0
D Only	4	25.18	23.00	7.52	7.52	21.24	0.68	41.73	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	4	25.23	23.00	7.49	7.49	21.62	0.66	41.68	Vu < PhiVc/2	lot Reqd 9.6.	41.7	0.0	0.0
D Only	4	25.29	23.00	7.46	7.46	22.01	0.65	41.64	Vu < PhiVc/2	lot Reqd 9.6.	41.6	0.0	0.0
D Only	4	25.34	23.00	7.43	7.43	22.39	0.64	41.59	Vu < PhiVc/2	lot Reqd 9.6.	41.6	0.0	0.0
D Only	4	25.39	23.00	7.40	7.40	22.77	0.62	41.55	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	4	25.44	23.00	7.37	7.37	23.15	0.61	41.51	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	4	25.49	23.00	7.34	7.34	23.53	0.60	41.47	Vu < PhiVc/2	lot Reqd 9.6.	41.5	0.0	0.0
D Only	4	25.54	23.00	7.31	7.31	23.90	0.59	41.43	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	4	25.59	23.00	7.28	7.28	24.28	0.57	41.39	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	4	25.64	23.00	7.25	7.25	24.65	0.56	41.35	Vu < PhiVc/2	lot Reqd 9.6.	41.4	0.0	0.0
D Only	4	25.70	23.00	7.21	7.21	25.02	0.55	41.32	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	4	25.75	23.00	7.18	7.18	25.39	0.54	41.28	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	4	25.80	23.00	7.15	7.15	25.76	0.53	41.25	Vu < PhiVc/2	lot Reqd 9.6.	41.3	0.0	0.0
D Only	4	25.85	23.00	7.12	7.12	26.12	0.52	41.22	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	4	25.90	23.00	7.09	7.09	26.49	0.51	41.19	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	4	25.95	23.00	7.06	7.06	26.85	0.50	41.16	Vu < PhiVc/2	lot Reqd 9.6.	41.2	0.0	0.0
D Only	4	26.00	23.00	7.03	7.03	27.21	0.50	41.13	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	4	26.06	23.00	7.00	7.00	27.57	0.49	41.10	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	4	26.11	23.00	6.97	6.97	27.93	0.48	41.07	Vu < PhiVc/2	lot Reqd 9.6.	41.1	0.0	0.0
D Only	4	26.16	23.00	6.94	6.94	28.29	0.47	41.04	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	4	26.21	23.00	6.91	6.91	28.64	0.46	41.02	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	4	26.26	23.00	6.88	6.88	29.00	0.45	40.99	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	4	26.31	23.00	6.85	6.85	29.35	0.45	40.97	Vu < PhiVc/2	lot Reqd 9.6.	41.0	0.0	0.0
D Only	4	26.36	23.00	6.82	6.82	29.70	0.44	40.95	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	4	26.41	23.00	6.79	6.79	30.05	0.43	40.92	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	4	26.47	23.00	6.75	6.75	30.40	0.43	40.90	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	4	26.52	23.00	6.72	6.72	30.74	0.42	40.88	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	4	26.57	23.00	6.69	6.69	31.09	0.41	40.86	Vu < PhiVc/2	lot Reqd 9.6.	40.9	0.0	0.0
D Only	4	26.62	23.00	6.66	6.66	31.43	0.41	40.83	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	4	26.67	23.00	6.63	6.63	31.77	0.40	40.81	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	4	26.72	23.00	6.60	6.60	32.11	0.39	40.79	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	4	26.77	23.00	6.57	6.57	32.45	0.39	40.77	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	4	26.83	23.00	6.54	6.54	32.78	0.38	40.76	Vu < PhiVc/2	lot Reqd 9.6.	40.8	0.0	0.0
D Only	4	26.88	23.00	6.51	6.51	33.12	0.38	40.74	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	4	26.93	23.00	6.48	6.48	33.45	0.37	40.72	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	4	26.98	23.00	6.45	6.45	33.78	0.37	40.70	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	4	27.03	23.00	6.42	6.42	34.11	0.36	40.68	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	4	27.08	23.00	6.39	6.39	34.44	0.36	40.67	Vu < PhiVc/2	lot Reqd 9.6.	40.7	0.0	0.0
D Only	4	27.13	23.00	6.36	6.36	34.77	0.35	40.65	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.18	23.00	6.32	6.32	35.10	0.35	40.63	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.24	23.00	6.29	6.29	35.42	0.34	40.62	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.29	23.00	6.26	6.26	35.74	0.34	40.60	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.34	23.00	6.23	6.23	36.06	0.33	40.59	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.39	23.00	6.20	6.20	36.38	0.33	40.57	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.44	23.00	6.17	6.17	36.70	0.32	40.56	Vu < PhiVc/2	lot Reqd 9.6.	40.6	0.0	0.0
D Only	4	27.49	23.00	6.14	6.14	37.01	0.32	40.54	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.54	23.00	6.11	6.11	37.33	0.31	40.53	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0

Project Title:
Engineer:
Project ID:
Project Descr:

SL15

Printed: 15 MAY 2019, 9:55AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 1: LC 56 (input already factored) -- 3 supports

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	4	27.59	23.00	6.08	6.08	37.64	0.31	40.52	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.65	23.00	6.05	6.05	37.95	0.31	40.50	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.70	23.00	6.02	6.02	38.26	0.30	40.49	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.75	23.00	5.99	5.99	38.57	0.30	40.48	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.80	23.00	5.96	5.96	38.88	0.29	40.46	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.85	23.00	5.93	5.93	39.18	0.29	40.45	Vu < PhiVc/2	lot Reqd 9.6.	40.5	0.0	0.0
D Only	4	27.90	23.00	5.89	5.89	39.48	0.29	40.44	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	27.95	23.00	5.86	5.86	39.79	0.28	40.43	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.01	23.00	5.83	5.83	40.09	0.28	40.41	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.06	23.00	5.80	5.80	40.38	0.28	40.40	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.11	23.00	5.77	5.77	40.68	0.27	40.39	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.16	23.00	5.74	5.74	40.98	0.27	40.38	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.21	23.00	5.71	5.71	41.27	0.27	40.37	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.26	23.00	5.68	5.68	41.56	0.26	40.36	Vu < PhiVc/2	lot Reqd 9.6.	40.4	0.0	0.0
D Only	4	28.31	23.00	5.65	5.65	41.85	0.26	40.35	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.36	23.00	5.62	5.62	42.14	0.26	40.34	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.42	23.00	5.59	5.59	42.43	0.25	40.33	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.47	23.00	5.56	5.56	42.72	0.25	40.32	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.52	23.00	5.53	5.53	43.00	0.25	40.31	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.57	23.00	5.50	5.50	43.28	0.24	40.30	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.62	23.00	5.47	5.47	43.57	0.24	40.29	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.67	23.00	5.43	5.43	43.84	0.24	40.28	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.72	23.00	5.40	5.40	44.12	0.23	40.27	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.78	23.00	5.37	5.37	44.40	0.23	40.26	Vu < PhiVc/2	lot Reqd 9.6.	40.3	0.0	0.0
D Only	4	28.83	23.00	5.34	5.34	44.67	0.23	40.25	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	28.88	23.00	5.31	5.31	44.95	0.23	40.24	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	28.93	23.00	5.28	5.28	45.22	0.22	40.23	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	28.98	23.00	5.25	5.25	45.49	0.22	40.22	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.03	23.00	5.22	5.22	45.76	0.22	40.22	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.08	23.00	5.19	5.19	46.03	0.22	40.21	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.13	23.00	5.16	5.16	46.29	0.21	40.20	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.19	23.00	5.13	5.13	46.55	0.21	40.19	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.24	23.00	5.10	5.10	46.82	0.21	40.18	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.29	23.00	5.07	5.07	47.08	0.21	40.17	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.34	23.00	5.04	5.04	47.34	0.20	40.17	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.39	23.00	5.00	5.00	47.59	0.20	40.16	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.44	23.00	4.97	4.97	47.85	0.20	40.15	Vu < PhiVc/2	lot Reqd 9.6.	40.2	0.0	0.0
D Only	4	29.49	23.00	4.94	4.94	48.11	0.20	40.14	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	4	29.54	23.00	4.91	4.91	48.36	0.19	40.14	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	4	29.60	23.00	4.88	4.88	48.61	0.19	40.13	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	4	29.65	23.00	4.85	4.85	48.86	0.19	40.12	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	4	29.70	23.00	4.82	4.82	49.11	0.19	40.11	Vu < PhiVc/2	lot Reqd 9.6.	40.1	0.0	0.0
D Only	4	29.75	23.00	0.00	0.00	0.00	1.00	42.79	Vu < PhiVc/2	lot Reqd 9.6.	42.8	0.0	0.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	4.875	-113.58	180.92	0.63
Span # 2	2	10.000	-114.34	180.92	0.63
Span # 3	3	10.000	53.48	180.92	0.30
Span # 4	4	4.875	49.11	180.92	0.27
D Only					
Span # 1	1	4.875	-113.58	180.92	0.63
Span # 2	2	10.000	-114.34	180.92	0.63
Span # 3	3	10.000	53.48	180.92	0.30

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 Licensee : DIBBLE ENGINEERS INC.

Description : MF 1: LC 56 (input already factored) -- 3 supports

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	4.875	49.11	180.92	0.27

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
D Only	1	0.0446	0.000	D Only	-0.0010	5.138
D Only	2	0.0004	10.263	D Only	-0.0061	3.947
D Only	3	0.0048	4.474	D Only	-0.0002	10.128
	4	0.0000	4.474	D Only	-0.0129	4.875

Project Title:
Engineer:
Project ID:
Project Descr:

SL17

Printed: 13 MAY 2019, 3:21PM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

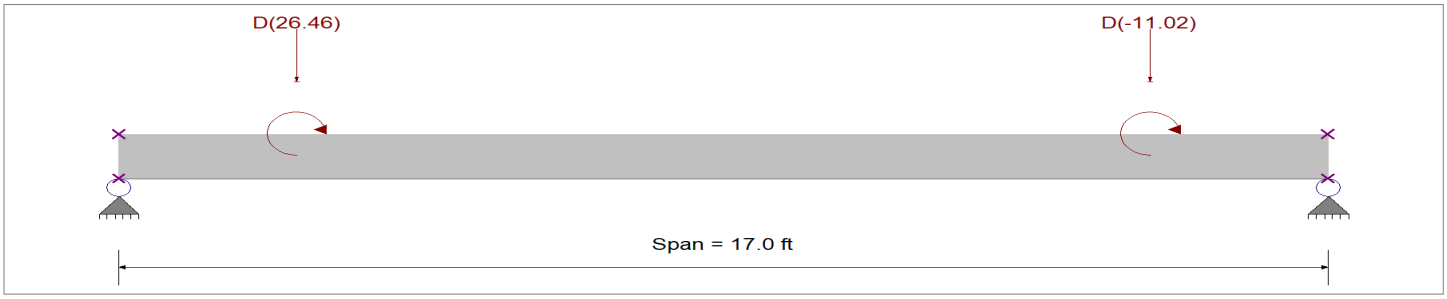
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 19 (input already factored)

General Beam Properties

Elastic Modulus 1,000.0 ksi

Span #1 Span Length = 17.0 ft Area = 1.0 in^2 Moment of Inertia = 1,000.0 in^4



Applied Loads

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

Load(s) for Span Number 1

Point Load : D = 26.460 k @ 2.50 ft

Moment : D = -130.240 k-ft, Loc = 2.50 ft in span

Point Load : D = -11.020 k @ 14.50 ft

Moment : D = -129.30 k-ft, Loc = 14.50 ft in span

DESIGN SUMMARY

Maximum Bending =	89.271 k-ft	Maximum Shear =	36.215 k
Load Combination	+D+H	Load Combination	+D+H
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	2.465 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	1.319 in	154
Max Upward Total Deflection	0.000 in	632060

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)							Shear Values (k)		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+L+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+Lr+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+S+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+0.750Lr+0.750L+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+0.750L+0.750S+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+0.60W+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+0.750Lr+0.450W+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+D+0.750S+0.450W+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+0.60D+0.60W+0.60H													
Dsgn. L = 17.00 ft	1			53.56	-30.73	53.56					21.73		
+D+0.70E+0.60H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		

Project Title:
Engineer:
Project ID:
Project Descr:

SL18

Printed: 13 MAY 2019, 3:21PM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 19 (input already factored)

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values (k-ft) Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Shear Values (k) Va Max	Vnx	Vnx/Omega
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 17.00 ft	1			89.27	-51.21	89.27					36.22		
+0.60D+0.70E+H													
Dsgn. L = 17.00 ft	1			53.56	-30.73	53.56					21.73		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	1.3194	10.965		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	36.215	-20.775
Overall MINimum		-20.775
+D+H	36.215	-20.775
+D+L+H	36.215	-20.775
+D+Lr+H	36.215	-20.775
+D+S+H	36.215	-20.775
+D+0.750Lr+0.750L+H	36.215	-20.775
+D+0.750L+0.750S+H	36.215	-20.775
+D+0.60W+H	36.215	-20.775
+D+0.750Lr+0.450W+H	36.215	-20.775
+D+0.750S+0.450W+H	36.215	-20.775
+0.60D+0.60W+0.60H	21.729	-12.465
+D+0.70E+0.60H	36.215	-20.775
+D+0.750L+0.750S+0.5250E+H	36.215	-20.775
+0.60D+0.70E+H	21.729	-12.465
D Only	36.215	-20.775
Lr Only		
L Only		
S Only		
W Only		
E Only		
H Only		

Therefore, (2) Helical Piles Req'd Each End

Project Title:
Engineer:
Project ID:
Project Descr:

SL19

Printed: 13 MAY 2019, 3:22PM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

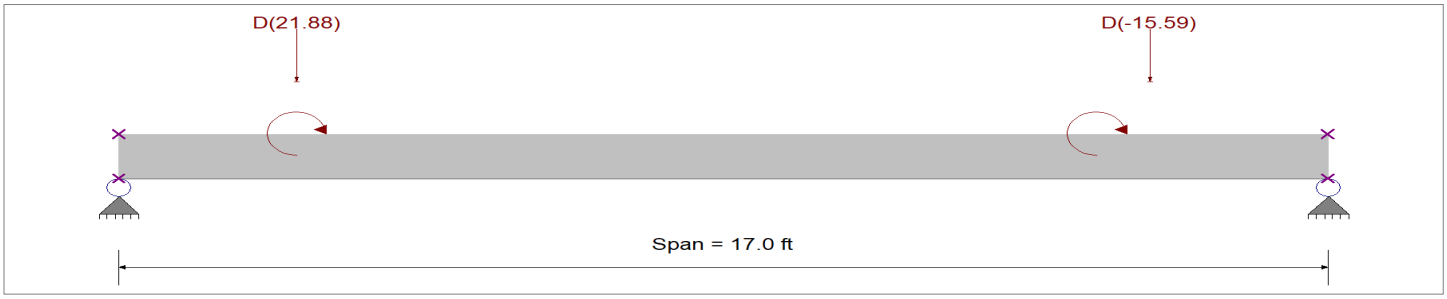
Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 35 (input already factored)

General Beam Properties

Elastic Modulus 1,000.0 ksi

Span #1 Span Length = 17.0 ft Area = 1.0 in^2 Moment of Inertia = 1,000.0 in^4



Applied Loads

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

Load(s) for Span Number 1

Point Load : D = 21.880 k @ 2.50 ft

Moment : D = -129.90 k-ft, Loc = 2.50 ft in span

Moment : D = -129.530 k-ft, Loc = 13.750 ft in span

Point Load : D = -15.590 k @ 14.50 ft

DESIGN SUMMARY

Maximum Bending =	77.969 k-ft	Maximum Shear =	31.630 k
Load Combination	+D+H	Load Combination	+D+H
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	2.465 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.384 in	530
Max Upward Total Deflection	-0.114 in	1791

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)							Shear Values (k)		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+L+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+Lr+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+S+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+0.750Lr+0.750L+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+0.750L+0.750S+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+0.60W+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+0.750Lr+0.450W+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+D+0.750S+0.450W+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+0.60D+0.60W+0.60H													
Dsgn. L = 17.00 ft	1			46.78	-42.28	46.78					18.98		
+D+0.70E+0.60H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		

Project Title:
Engineer:
Project ID:
Project Descr:

SL20

Printed: 13 MAY 2019, 3:22PM

General Beam Analysis

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 35 (input already factored)

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 17.00 ft	1			77.97	-70.47	77.97					31.63		
+0.60D+0.70E+H													
Dsgn. L = 17.00 ft	1			46.78	-42.28	46.78					18.98		

Summary of Moment Values (k-ft) — Shear Values (k)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.3845	11.305	D Only	-0.1139	4.250

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	31.630	-25.340
Overall MINimum		-25.340
+D+H	31.630	-25.340
+D+L+H	31.630	-25.340
+D+Lr+H	31.630	-25.340
+D+S+H	31.630	-25.340
+D+0.750Lr+0.750L+H	31.630	-25.340
+D+0.750L+0.750S+H	31.630	-25.340
+D+0.60W+H	31.630	-25.340
+D+0.750Lr+0.450W+H	31.630	-25.340
+D+0.750S+0.450W+H	31.630	-25.340
+0.60D+0.60W+0.60H	18.978	-15.204
+D+0.70E+0.60H	31.630	-25.340
+D+0.750L+0.750S+0.5250E+H	31.630	-25.340
+0.60D+0.70E+H	18.978	-15.204
D Only	31.630	-25.340
Lr Only		
L Only		
S Only		
W Only		
E Only		
H Only		

Therefore, (2) Helical Piles Req'd Each End

Project Title:
Engineer:
Project ID:
Project Descr:

SL21

Checking Beam Reinforcement, LRFD

Printed: 14 MAY 2019, 11:21AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

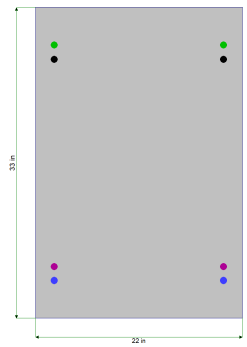
Description : MF 2: LC 56 (input already factored)

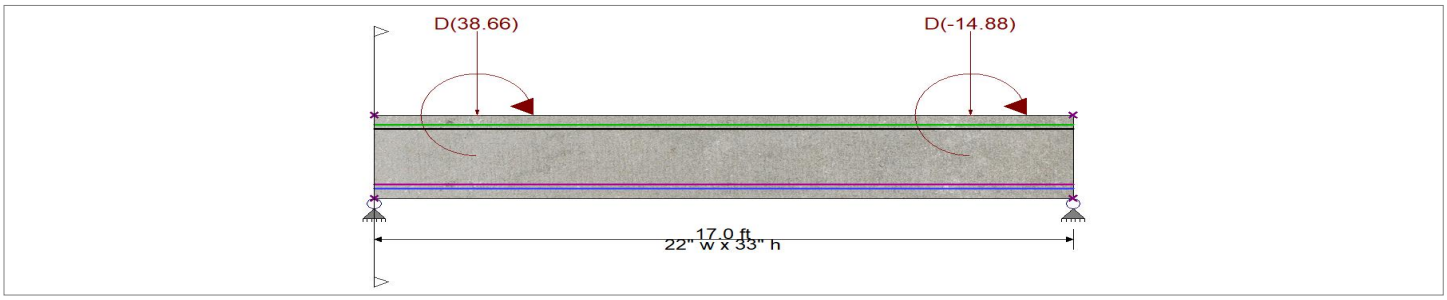
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	443.706 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 33.0 in
Span #1 Reinforcing....
2-#6 at 4.0 in from Bottom, from 0.0 to 17.0 ft in this span
2-#6 at 5.50 in from Bottom, from 0.0 to 17.0 ft in this span
2-#6 at 4.0 in from Top, from 0.0 to 17.0 ft in this span
2-#6 at 5.50 in from Top, from 0.0 to 17.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Point Load : D = 38.660 k @ 2.50 ft

Moment : D = -186.40 k-ft, Location = 2.50 ft from left end of this span

Moment : D = -184.30 k-ft, Location = 14.50 ft from left end of this span

Point Load : D = -14.880 k @ 14.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.615** : 1
Section used for this span **Typical Section**
Mu : Applied 143.434 k-ft
Mn * Phi : Allowable 233.218 k-ft
Location of maximum on span 2.477 ft
Span # where maximum occurs Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Downward Total Deflection	0.016 in	Ratio =	12813	>=180.
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	58.806	-22.598
Overall MINimum	35.284	-13.559
+D+H	58.806	-22.598
+D+L+H	58.806	-22.598

Project Title:
Engineer:
Project ID:
Project Descr:

SL22

Printed: 14 MAY 2019, 11:21AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+Lr+H	58.806	-22.598
+D+S+H	58.806	-22.598
+D+0.750Lr+0.750L+H	58.806	-22.598
+D+0.750L+0.750S+H	58.806	-22.598
+D+0.60W+H	58.806	-22.598
+D+0.750Lr+0.450W+H	58.806	-22.598
+D+0.750S+0.450W+H	58.806	-22.598
+0.60D+0.60W+0.60H	35.284	-13.559
+D+0.70E+0.60H	58.806	-22.598
+D+0.750L+0.750S+0.5250E+H	58.806	-22.598
+0.60D+0.70E+H	35.284	-13.559
D Only	58.806	-22.598
Lr Only		
L Only		
S Only		
W Only		
E Only		
H Only		

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	0.00	29.00	58.81	58.81	0.00	1.00	57.09	PhiVc < Vu	1.720	78.4	9.0	9.0
D Only	1	0.03	29.00	58.78	58.78	1.82	1.00	57.09	PhiVc < Vu	1.698	78.4	9.0	9.0
D Only	1	0.06	29.00	58.76	58.76	3.64	1.00	57.09	PhiVc < Vu	1.675	78.4	9.0	9.0
D Only	1	0.09	29.00	58.74	58.74	5.46	1.00	57.09	PhiVc < Vu	1.652	78.4	9.0	9.0
D Only	1	0.12	29.00	58.72	58.72	7.28	1.00	57.09	PhiVc < Vu	1.630	78.4	9.0	9.0
D Only	1	0.15	29.00	58.69	58.69	9.10	1.00	57.09	PhiVc < Vu	1.607	78.4	9.0	9.0
D Only	1	0.19	29.00	58.67	58.67	10.91	1.00	57.09	PhiVc < Vu	1.584	78.4	9.0	9.0
D Only	1	0.22	29.00	58.65	58.65	12.73	1.00	57.09	PhiVc < Vu	1.562	78.4	9.0	9.0
D Only	1	0.25	29.00	58.63	58.63	14.55	1.00	57.09	PhiVc < Vu	1.539	78.4	9.0	9.0
D Only	1	0.28	29.00	58.60	58.60	16.36	1.00	57.09	PhiVc < Vu	1.516	78.4	9.0	9.0
D Only	1	0.31	29.00	58.58	58.58	18.17	1.00	57.09	PhiVc < Vu	1.494	78.4	9.0	9.0
D Only	1	0.34	29.00	58.56	58.56	19.99	1.00	57.09	PhiVc < Vu	1.471	78.4	9.0	9.0
D Only	1	0.37	29.00	58.53	58.53	21.80	1.00	57.09	PhiVc < Vu	1.449	78.4	9.0	9.0
D Only	1	0.40	29.00	58.51	58.51	23.61	1.00	57.09	PhiVc < Vu	1.426	78.4	9.0	9.0
D Only	1	0.43	29.00	58.49	58.49	25.42	1.00	57.09	PhiVc < Vu	1.403	78.4	9.0	9.0
D Only	1	0.46	29.00	58.47	58.47	27.24	1.00	57.09	PhiVc < Vu	1.381	78.4	9.0	9.0
D Only	1	0.50	29.00	58.44	58.44	29.05	1.00	57.09	PhiVc < Vu	1.358	78.4	9.0	9.0
D Only	1	0.53	29.00	58.42	58.42	30.85	1.00	57.09	PhiVc < Vu	1.335	78.4	9.0	9.0
D Only	1	0.56	29.00	58.40	58.40	32.66	1.00	57.09	PhiVc < Vu	1.313	78.4	9.0	9.0
D Only	1	0.59	29.00	58.38	58.38	34.47	1.00	57.09	PhiVc < Vu	1.290	78.4	9.0	9.0
D Only	1	0.62	29.00	58.35	58.35	36.28	1.00	57.09	PhiVc < Vu	1.267	78.4	9.0	9.0
D Only	1	0.65	29.00	58.33	58.33	38.09	1.00	57.09	PhiVc < Vu	1.245	78.4	9.0	9.0
D Only	1	0.68	29.00	58.31	58.31	39.89	1.00	57.09	PhiVc < Vu	1.222	78.4	9.0	9.0
D Only	1	0.71	29.00	58.29	58.29	41.70	1.00	57.09	PhiVc < Vu	1.20	78.4	9.0	9.0
D Only	1	0.74	29.00	58.26	58.26	43.50	1.00	57.09	PhiVc < Vu	1.177	78.4	9.0	9.0
D Only	1	0.77	29.00	58.24	58.24	45.30	1.00	57.09	PhiVc < Vu	1.154	78.4	9.0	9.0
D Only	1	0.81	29.00	58.22	58.22	47.11	1.00	57.09	PhiVc < Vu	1.132	78.4	9.0	9.0
D Only	1	0.84	29.00	58.20	58.20	48.91	1.00	57.09	PhiVc < Vu	1.109	78.4	9.0	9.0
D Only	1	0.87	29.00	58.17	58.17	50.71	1.00	57.09	PhiVc < Vu	1.086	78.4	9.0	9.0
D Only	1	0.90	29.00	58.15	58.15	52.51	1.00	57.09	PhiVc < Vu	1.064	78.4	9.0	9.0
D Only	1	0.93	29.00	58.13	58.13	54.31	1.00	57.09	PhiVc < Vu	1.041	78.4	9.0	9.0
D Only	1	0.96	29.00	58.10	58.10	56.11	1.00	57.09	PhiVc < Vu	1.018	78.4	9.0	9.0
D Only	1	0.99	29.00	58.08	58.08	57.91	1.00	57.09	PhiVc < Vu	0.9958	78.4	9.0	9.0
D Only	1	1.02	29.00	58.06	58.06	59.71	1.00	57.09	PhiVc < Vu	0.9731	78.4	9.0	9.0
D Only	1	1.05	29.00	58.04	58.04	61.51	1.00	57.09	PhiVc < Vu	0.9505	78.4	9.0	9.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	1.08	29.00	58.01	58.01	63.30	1.00	57.09	PhiVc < Vu	0.9279	78.4	9.0	9.0
D Only	1	1.11	29.00	57.99	57.99	65.10	1.00	57.09	PhiVc < Vu	0.9052	78.4	9.0	9.0
D Only	1	1.15	29.00	57.97	57.97	66.90	1.00	57.09	PhiVc < Vu	0.8826	78.4	9.0	9.0
D Only	1	1.18	29.00	57.95	57.95	68.69	1.00	57.09	PhiVc < Vu	0.860	78.4	9.0	9.0
D Only	1	1.21	29.00	57.92	57.92	70.48	1.00	57.09	PhiVc < Vu	0.8373	78.4	9.0	9.0
D Only	1	1.24	29.00	57.90	57.90	72.28	1.00	57.09	PhiVc < Vu	0.8147	78.4	9.0	9.0
D Only	1	1.27	29.00	57.88	57.88	74.07	1.00	57.09	PhiVc < Vu	0.7921	78.4	9.0	9.0
D Only	1	1.30	29.00	57.86	57.86	75.86	1.00	57.09	PhiVc < Vu	0.7694	78.4	9.0	9.0
D Only	1	1.33	29.00	57.83	57.83	77.65	1.00	57.09	PhiVc < Vu	0.7468	78.4	9.0	9.0
D Only	1	1.36	29.00	57.81	57.81	79.44	1.00	57.09	PhiVc < Vu	0.7241	78.4	9.0	9.0
D Only	1	1.39	29.00	57.79	57.79	81.23	1.00	57.09	PhiVc < Vu	0.7015	78.4	9.0	9.0
D Only	1	1.42	29.00	57.76	57.76	83.02	1.00	57.09	PhiVc < Vu	0.6789	78.4	9.0	9.0
D Only	1	1.46	29.00	57.74	57.74	84.81	1.00	57.09	PhiVc < Vu	0.6562	78.4	9.0	9.0
D Only	1	1.49	29.00	57.72	57.72	86.60	1.00	57.09	PhiVc < Vu	0.6336	78.4	9.0	9.0
D Only	1	1.52	29.00	57.70	57.70	88.39	1.00	57.09	PhiVc < Vu	0.6110	78.4	9.0	9.0
D Only	1	1.55	29.00	57.67	57.67	90.17	1.00	57.09	PhiVc < Vu	0.5883	78.4	9.0	9.0
D Only	1	1.58	29.00	57.65	57.65	91.96	1.00	57.09	PhiVc < Vu	0.5657	78.4	9.0	9.0
D Only	1	1.61	29.00	57.63	57.63	93.74	1.00	57.09	PhiVc < Vu	0.5430	78.4	9.0	9.0
D Only	1	1.64	29.00	57.61	57.61	95.53	1.00	57.09	PhiVc < Vu	0.5204	78.4	9.0	9.0
D Only	1	1.67	29.00	57.58	57.58	97.31	1.00	57.09	PhiVc < Vu	0.4978	78.4	9.0	9.0
D Only	1	1.70	29.00	57.56	57.56	99.09	1.00	57.09	PhiVc < Vu	0.4751	78.4	9.0	9.0
D Only	1	1.73	29.00	57.54	57.54	100.87	1.00	57.09	PhiVc < Vu	0.4525	78.4	9.0	9.0
D Only	1	1.77	29.00	57.52	57.52	102.66	1.00	57.09	PhiVc < Vu	0.4299	78.4	9.0	9.0
D Only	1	1.80	29.00	57.49	57.49	104.44	1.00	57.09	PhiVc < Vu	0.4072	78.4	9.0	9.0
D Only	1	1.83	29.00	57.47	57.47	106.22	1.00	57.09	PhiVc < Vu	0.3846	78.4	9.0	9.0
D Only	1	1.86	29.00	57.45	57.45	108.00	1.00	57.09	PhiVc < Vu	0.3619	78.4	9.0	9.0
D Only	1	1.89	29.00	57.43	57.43	109.77	1.00	57.09	PhiVc < Vu	0.3393	78.4	9.0	9.0
D Only	1	1.92	29.00	57.40	57.40	111.55	1.00	57.09	PhiVc < Vu	0.3167	78.4	9.0	9.0
D Only	1	1.95	29.00	57.38	57.38	113.33	1.00	57.09	PhiVc < Vu	0.2940	78.4	9.0	9.0
D Only	1	1.98	29.00	57.36	57.36	115.11	1.00	57.09	PhiVc < Vu	0.2714	78.4	9.0	9.0
D Only	1	2.01	29.00	57.33	57.33	116.88	1.00	57.09	PhiVc < Vu	0.2488	78.4	9.0	9.0
D Only	1	2.04	29.00	57.31	57.31	118.66	1.00	57.09	PhiVc < Vu	0.2261	78.4	9.0	9.0
D Only	1	2.07	29.00	57.29	57.29	120.43	1.00	57.09	PhiVc < Vu	0.2035	78.4	9.0	9.0
D Only	1	2.11	29.00	57.27	57.27	122.20	1.00	57.09	PhiVc < Vu	0.1809	78.4	9.0	9.0
D Only	1	2.14	29.00	57.24	57.24	123.98	1.00	57.09	PhiVc < Vu	0.1582	78.4	9.0	9.0
D Only	1	2.17	29.00	57.22	57.22	125.75	1.00	57.09	PhiVc < Vu	0.1356	78.4	9.0	9.0
D Only	1	2.20	29.00	57.20	57.20	127.52	1.00	57.09	PhiVc < Vu	0.1129	78.4	9.0	9.0
D Only	1	2.23	29.00	57.18	57.18	129.29	1.00	57.09	PhiVc < Vu	0.09030	78.4	9.0	9.0
D Only	1	2.26	29.00	57.15	57.15	131.06	1.00	57.09	PhiVc < Vu	0.06767	78.4	9.0	9.0
D Only	1	2.29	29.00	57.13	57.13	132.83	1.00	57.09	PhiVc < Vu	0.04503	78.4	9.0	9.0
D Only	1	2.32	29.00	57.11	57.11	134.60	1.00	57.09	PhiVc < Vu	0.02239	78.4	9.0	9.0
D Only	1	2.35	29.00	57.09	57.09	136.37	1.00	57.09	PhiVc/2 < Vu <=	Min 11.5.6.3	81.0	8.0	8.0
D Only	1	2.38	29.00	57.06	57.06	138.14	1.00	57.08	PhiVc/2 < Vu <=	Min 11.5.6.3	81.0	8.0	8.0
D Only	1	2.42	29.00	57.04	57.04	139.90	0.99	57.04	PhiVc < Vu	0.002942	78.3	9.0	9.0
D Only	1	2.45	29.00	57.02	57.02	141.67	0.97	57.00	PhiVc < Vu	0.02211	78.3	9.0	9.0
D Only	1	2.48	29.00	57.00	57.00	143.43	0.96	56.96	PhiVc < Vu	0.04023	78.2	9.0	9.0
D Only	1	2.51	29.00	18.31	18.31	41.52	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.54	29.00	18.29	18.29	40.95	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.57	29.00	18.27	18.27	40.39	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.60	29.00	18.24	18.24	39.82	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.63	29.00	18.22	18.22	39.26	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.66	29.00	18.20	18.20	38.69	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.69	29.00	18.18	18.18	38.13	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.72	29.00	18.15	18.15	37.57	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0

Project Title:
Engineer:
Project ID:
Project Descr:

SL24

Printed: 14 MAY 2019, 11:21AM

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	2.76	29.00	18.13	18.13	37.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.79	29.00	18.11	18.11	36.44	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.82	29.00	18.09	18.09	35.88	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.85	29.00	18.06	18.06	35.32	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.88	29.00	18.04	18.04	34.76	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.91	29.00	18.02	18.02	34.21	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.94	29.00	18.00	18.00	33.65	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	2.97	29.00	17.97	17.97	33.09	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.00	29.00	17.95	17.95	32.54	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.03	29.00	17.93	17.93	31.98	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.07	29.00	17.91	17.91	31.43	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.10	29.00	17.88	17.88	30.87	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.13	29.00	17.86	17.86	30.32	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.16	29.00	17.84	17.84	29.77	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.19	29.00	17.81	17.81	29.21	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.22	29.00	17.79	17.79	28.66	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.25	29.00	17.77	17.77	28.11	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.28	29.00	17.75	17.75	27.56	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.31	29.00	17.72	17.72	27.01	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.34	29.00	17.70	17.70	26.46	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.38	29.00	17.68	17.68	25.92	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.41	29.00	17.66	17.66	25.37	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.44	29.00	17.63	17.63	24.82	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.47	29.00	17.61	17.61	24.28	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.50	29.00	17.59	17.59	23.73	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.53	29.00	17.57	17.57	23.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.56	29.00	17.54	17.54	22.64	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.59	29.00	17.52	17.52	22.10	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.62	29.00	17.50	17.50	21.56	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.65	29.00	17.48	17.48	21.02	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.68	29.00	17.45	17.45	20.48	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.72	29.00	17.43	17.43	19.94	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.75	29.00	17.41	17.41	19.40	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.78	29.00	17.38	17.38	18.86	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.81	29.00	17.36	17.36	18.32	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.84	29.00	17.34	17.34	17.78	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.87	29.00	17.32	17.32	17.25	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.90	29.00	17.29	17.29	16.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.93	29.00	17.27	17.27	16.18	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.96	29.00	17.25	17.25	15.64	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	3.99	29.00	17.23	17.23	15.11	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.03	29.00	17.20	17.20	14.57	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.06	29.00	17.18	17.18	14.04	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.09	29.00	17.16	17.16	13.51	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.12	29.00	17.14	17.14	12.98	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.15	29.00	17.11	17.11	12.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.18	29.00	17.09	17.09	11.92	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.21	29.00	17.07	17.07	11.39	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.24	29.00	17.04	17.04	10.86	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.27	29.00	17.02	17.02	10.34	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.30	29.00	17.00	17.00	9.81	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.34	29.00	16.98	16.98	9.28	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.37	29.00	16.95	16.95	8.76	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.40	29.00	16.93	16.93	8.23	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	4.43	29.00	16.91	16.91	7.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.46	29.00	16.89	16.89	7.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.49	29.00	16.86	16.86	6.66	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.52	29.00	16.84	16.84	6.14	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.55	29.00	16.82	16.82	5.62	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.58	29.00	16.80	16.80	5.10	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.61	29.00	16.77	16.77	4.58	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.64	29.00	16.75	16.75	4.06	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.68	29.00	16.73	16.73	3.54	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.71	29.00	16.71	16.71	3.02	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.74	29.00	16.68	16.68	2.51	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.77	29.00	16.66	16.66	1.99	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.80	29.00	16.64	16.64	1.48	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.83	29.00	16.61	16.61	0.96	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.86	29.00	16.59	16.59	0.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.89	29.00	16.57	16.57	0.07	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.92	29.00	16.55	16.55	0.58	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.95	29.00	16.52	16.52	1.09	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	4.99	29.00	16.50	16.50	1.60	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.02	29.00	16.48	16.48	2.11	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.05	29.00	16.46	16.46	2.62	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.08	29.00	16.43	16.43	3.13	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.11	29.00	16.41	16.41	3.64	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.14	29.00	16.39	16.39	4.15	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.17	29.00	16.37	16.37	4.66	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.20	29.00	16.34	16.34	5.16	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.23	29.00	16.32	16.32	5.67	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.26	29.00	16.30	16.30	6.17	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.30	29.00	16.28	16.28	6.68	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.33	29.00	16.25	16.25	7.18	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.36	29.00	16.23	16.23	7.68	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.39	29.00	16.21	16.21	8.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.42	29.00	16.18	16.18	8.69	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.45	29.00	16.16	16.16	9.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.48	29.00	16.14	16.14	9.69	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.51	29.00	16.12	16.12	10.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.54	29.00	16.09	16.09	10.69	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.57	29.00	16.07	16.07	11.18	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.60	29.00	16.05	16.05	11.68	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.64	29.00	16.03	16.03	12.18	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.67	29.00	16.00	16.00	12.67	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.70	29.00	15.98	15.98	13.17	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.73	29.00	15.96	15.96	13.66	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.76	29.00	15.94	15.94	14.16	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.79	29.00	15.91	15.91	14.65	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.82	29.00	15.89	15.89	15.14	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.85	29.00	15.87	15.87	15.64	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.88	29.00	15.85	15.85	16.13	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.91	29.00	15.82	15.82	16.62	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.95	29.00	15.80	15.80	17.11	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	5.98	29.00	15.78	15.78	17.60	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.01	29.00	15.75	15.75	18.08	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.04	29.00	15.73	15.73	18.57	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.07	29.00	15.71	15.71	19.06	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	6.10	29.00	15.69	15.69	19.54	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.13	29.00	15.66	15.66	20.03	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.16	29.00	15.64	15.64	20.51	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.19	29.00	15.62	15.62	21.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.22	29.00	15.60	15.60	21.48	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.26	29.00	15.57	15.57	21.96	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.29	29.00	15.55	15.55	22.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.32	29.00	15.53	15.53	22.93	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.35	29.00	15.51	15.51	23.41	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.38	29.00	15.48	15.48	23.89	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.41	29.00	15.46	15.46	24.37	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.44	29.00	15.44	15.44	24.84	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.47	29.00	15.42	15.42	25.32	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.50	29.00	15.39	15.39	25.80	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.53	29.00	15.37	15.37	26.28	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.56	29.00	15.35	15.35	26.75	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.60	29.00	15.32	15.32	27.23	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.63	29.00	15.30	15.30	27.70	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.66	29.00	15.28	15.28	28.17	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.69	29.00	15.26	15.26	28.65	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.72	29.00	15.23	15.23	29.12	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.75	29.00	15.21	15.21	29.59	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.78	29.00	15.19	15.19	30.06	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.81	29.00	15.17	15.17	30.53	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.84	29.00	15.14	15.14	31.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.87	29.00	15.12	15.12	31.47	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.91	29.00	15.10	15.10	31.94	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.94	29.00	15.08	15.08	32.40	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	6.97	29.00	15.05	15.05	32.87	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.00	29.00	15.03	15.03	33.34	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.03	29.00	15.01	15.01	33.80	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.06	29.00	14.98	14.98	34.26	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.09	29.00	14.96	14.96	34.73	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.12	29.00	14.94	14.94	35.19	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.15	29.00	14.92	14.92	35.65	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.18	29.00	14.89	14.89	36.12	1.00	57.08	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	7.21	29.00	14.87	14.87	36.58	0.98	57.03	Vu < PhiVc/2	lot Reqd 9.6.	57.0	0.0	0.0
D Only	1	7.25	29.00	14.85	14.85	37.04	0.97	56.98	Vu < PhiVc/2	lot Reqd 9.6.	57.0	0.0	0.0
D Only	1	7.28	29.00	14.83	14.83	37.50	0.96	56.94	Vu < PhiVc/2	lot Reqd 9.6.	56.9	0.0	0.0
D Only	1	7.31	29.00	14.80	14.80	37.95	0.94	56.90	Vu < PhiVc/2	lot Reqd 9.6.	56.9	0.0	0.0
D Only	1	7.34	29.00	14.78	14.78	38.41	0.93	56.85	Vu < PhiVc/2	lot Reqd 9.6.	56.9	0.0	0.0
D Only	1	7.37	29.00	14.76	14.76	38.87	0.92	56.81	Vu < PhiVc/2	lot Reqd 9.6.	56.8	0.0	0.0
D Only	1	7.40	29.00	14.74	14.74	39.33	0.91	56.77	Vu < PhiVc/2	lot Reqd 9.6.	56.8	0.0	0.0
D Only	1	7.43	29.00	14.71	14.71	39.78	0.89	56.74	Vu < PhiVc/2	lot Reqd 9.6.	56.7	0.0	0.0
D Only	1	7.46	29.00	14.69	14.69	40.24	0.88	56.70	Vu < PhiVc/2	lot Reqd 9.6.	56.7	0.0	0.0
D Only	1	7.49	29.00	14.67	14.67	40.69	0.87	56.66	Vu < PhiVc/2	lot Reqd 9.6.	56.7	0.0	0.0
D Only	1	7.52	29.00	14.65	14.65	41.15	0.86	56.62	Vu < PhiVc/2	lot Reqd 9.6.	56.6	0.0	0.0
D Only	1	7.56	29.00	14.62	14.62	41.60	0.85	56.59	Vu < PhiVc/2	lot Reqd 9.6.	56.6	0.0	0.0
D Only	1	7.59	29.00	14.60	14.60	42.05	0.84	56.55	Vu < PhiVc/2	lot Reqd 9.6.	56.6	0.0	0.0
D Only	1	7.62	29.00	14.58	14.58	42.50	0.83	56.52	Vu < PhiVc/2	lot Reqd 9.6.	56.5	0.0	0.0
D Only	1	7.65	29.00	14.55	14.55	42.95	0.82	56.49	Vu < PhiVc/2	lot Reqd 9.6.	56.5	0.0	0.0
D Only	1	7.68	29.00	14.53	14.53	43.41	0.81	56.46	Vu < PhiVc/2	lot Reqd 9.6.	56.5	0.0	0.0
D Only	1	7.71	29.00	14.51	14.51	43.85	0.80	56.42	Vu < PhiVc/2	lot Reqd 9.6.	56.4	0.0	0.0
D Only	1	7.74	29.00	14.49	14.49	44.30	0.79	56.39	Vu < PhiVc/2	lot Reqd 9.6.	56.4	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	7.77	29.00	14.46	14.46	44.75	0.78	56.36	Vu < PhiVc/2	lot Reqd 9.6.	56.4	0.0	0.0
D Only	1	7.80	29.00	14.44	14.44	45.20	0.77	56.33	Vu < PhiVc/2	lot Reqd 9.6.	56.3	0.0	0.0
D Only	1	7.83	29.00	14.42	14.42	45.65	0.76	56.31	Vu < PhiVc/2	lot Reqd 9.6.	56.3	0.0	0.0
D Only	1	7.87	29.00	14.40	14.40	46.09	0.75	56.28	Vu < PhiVc/2	lot Reqd 9.6.	56.3	0.0	0.0
D Only	1	7.90	29.00	14.37	14.37	46.54	0.75	56.25	Vu < PhiVc/2	lot Reqd 9.6.	56.2	0.0	0.0
D Only	1	7.93	29.00	14.35	14.35	46.98	0.74	56.22	Vu < PhiVc/2	lot Reqd 9.6.	56.2	0.0	0.0
D Only	1	7.96	29.00	14.33	14.33	47.43	0.73	56.20	Vu < PhiVc/2	lot Reqd 9.6.	56.2	0.0	0.0
D Only	1	7.99	29.00	14.31	14.31	47.87	0.72	56.17	Vu < PhiVc/2	lot Reqd 9.6.	56.2	0.0	0.0
D Only	1	8.02	29.00	14.28	14.28	48.31	0.71	56.14	Vu < PhiVc/2	lot Reqd 9.6.	56.1	0.0	0.0
D Only	1	8.05	29.00	14.26	14.26	48.75	0.71	56.12	Vu < PhiVc/2	lot Reqd 9.6.	56.1	0.0	0.0
D Only	1	8.08	29.00	14.24	14.24	49.20	0.70	56.09	Vu < PhiVc/2	lot Reqd 9.6.	56.1	0.0	0.0
D Only	1	8.11	29.00	14.22	14.22	49.64	0.69	56.07	Vu < PhiVc/2	lot Reqd 9.6.	56.1	0.0	0.0
D Only	1	8.14	29.00	14.19	14.19	50.08	0.68	56.05	Vu < PhiVc/2	lot Reqd 9.6.	56.0	0.0	0.0
D Only	1	8.17	29.00	14.17	14.17	50.52	0.68	56.02	Vu < PhiVc/2	lot Reqd 9.6.	56.0	0.0	0.0
D Only	1	8.21	29.00	14.15	14.15	50.95	0.67	56.00	Vu < PhiVc/2	lot Reqd 9.6.	56.0	0.0	0.0
D Only	1	8.24	29.00	14.12	14.12	51.39	0.66	55.98	Vu < PhiVc/2	lot Reqd 9.6.	56.0	0.0	0.0
D Only	1	8.27	29.00	14.10	14.10	51.83	0.66	55.96	Vu < PhiVc/2	lot Reqd 9.6.	56.0	0.0	0.0
D Only	1	8.30	29.00	14.08	14.08	52.26	0.65	55.93	Vu < PhiVc/2	lot Reqd 9.6.	55.9	0.0	0.0
D Only	1	8.33	29.00	14.06	14.06	52.70	0.64	55.91	Vu < PhiVc/2	lot Reqd 9.6.	55.9	0.0	0.0
D Only	1	8.36	29.00	14.03	14.03	53.14	0.64	55.89	Vu < PhiVc/2	lot Reqd 9.6.	55.9	0.0	0.0
D Only	1	8.39	29.00	14.01	14.01	53.57	0.63	55.87	Vu < PhiVc/2	lot Reqd 9.6.	55.9	0.0	0.0
D Only	1	8.42	29.00	13.99	13.99	54.00	0.63	55.85	Vu < PhiVc/2	lot Reqd 9.6.	55.9	0.0	0.0
D Only	1	8.45	29.00	13.97	13.97	54.44	0.62	55.83	Vu < PhiVc/2	lot Reqd 9.6.	55.8	0.0	0.0
D Only	1	8.48	29.00	13.94	13.94	54.87	0.61	55.81	Vu < PhiVc/2	lot Reqd 9.6.	55.8	0.0	0.0
D Only	1	8.52	29.00	13.92	13.92	55.30	0.61	55.79	Vu < PhiVc/2	lot Reqd 9.6.	55.8	0.0	0.0
D Only	1	8.55	29.00	13.90	13.90	55.73	0.60	55.77	Vu < PhiVc/2	lot Reqd 9.6.	55.8	0.0	0.0
D Only	1	8.58	29.00	13.88	13.88	56.16	0.60	55.76	Vu < PhiVc/2	lot Reqd 9.6.	55.8	0.0	0.0
D Only	1	8.61	29.00	13.85	13.85	56.59	0.59	55.74	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.64	29.00	13.83	13.83	57.02	0.59	55.72	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.67	29.00	13.81	13.81	57.45	0.58	55.70	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.70	29.00	13.79	13.79	57.87	0.58	55.69	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.73	29.00	13.76	13.76	58.30	0.57	55.67	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.76	29.00	13.74	13.74	58.73	0.57	55.65	Vu < PhiVc/2	lot Reqd 9.6.	55.7	0.0	0.0
D Only	1	8.79	29.00	13.72	13.72	59.15	0.56	55.64	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.83	29.00	13.69	13.69	59.58	0.56	55.62	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.86	29.00	13.67	13.67	60.00	0.55	55.60	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.89	29.00	13.65	13.65	60.42	0.55	55.59	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.92	29.00	13.63	13.63	60.84	0.54	55.57	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.95	29.00	13.60	13.60	61.27	0.54	55.56	Vu < PhiVc/2	lot Reqd 9.6.	55.6	0.0	0.0
D Only	1	8.98	29.00	13.58	13.58	61.69	0.53	55.54	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.01	29.00	13.56	13.56	62.11	0.53	55.53	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.04	29.00	13.54	13.54	62.53	0.52	55.51	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.07	29.00	13.51	13.51	62.95	0.52	55.50	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.10	29.00	13.49	13.49	63.36	0.51	55.48	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.13	29.00	13.47	13.47	63.78	0.51	55.47	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.17	29.00	13.45	13.45	64.20	0.51	55.46	Vu < PhiVc/2	lot Reqd 9.6.	55.5	0.0	0.0
D Only	1	9.20	29.00	13.42	13.42	64.61	0.50	55.44	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.23	29.00	13.40	13.40	65.03	0.50	55.43	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.26	29.00	13.38	13.38	65.44	0.49	55.42	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.29	29.00	13.36	13.36	65.86	0.49	55.40	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.32	29.00	13.33	13.33	66.27	0.49	55.39	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.35	29.00	13.31	13.31	66.68	0.48	55.38	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.38	29.00	13.29	13.29	67.09	0.48	55.37	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0
D Only	1	9.41	29.00	13.26	13.26	67.51	0.47	55.35	Vu < PhiVc/2	lot Reqd 9.6.	55.4	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	9.44	29.00	13.24	13.24	67.92	0.47	55.34	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.48	29.00	13.22	13.22	68.33	0.47	55.33	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.51	29.00	13.20	13.20	68.73	0.46	55.32	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.54	29.00	13.17	13.17	69.14	0.46	55.31	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.57	29.00	13.15	13.15	69.55	0.46	55.29	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.60	29.00	13.13	13.13	69.96	0.45	55.28	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.63	29.00	13.11	13.11	70.36	0.45	55.27	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.66	29.00	13.08	13.08	70.77	0.45	55.26	Vu < PhiVc/2	lot Reqd 9.6.	55.3	0.0	0.0
D Only	1	9.69	29.00	13.06	13.06	71.17	0.44	55.25	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.72	29.00	13.04	13.04	71.58	0.44	55.24	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.75	29.00	13.02	13.02	71.98	0.44	55.23	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.79	29.00	12.99	12.99	72.38	0.43	55.22	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.82	29.00	12.97	12.97	72.79	0.43	55.21	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.85	29.00	12.95	12.95	73.19	0.43	55.20	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.88	29.00	12.93	12.93	73.59	0.42	55.19	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.91	29.00	12.90	12.90	73.99	0.42	55.18	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.94	29.00	12.88	12.88	74.39	0.42	55.17	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	9.97	29.00	12.86	12.86	74.79	0.42	55.16	Vu < PhiVc/2	lot Reqd 9.6.	55.2	0.0	0.0
D Only	1	10.00	29.00	12.83	12.83	75.18	0.41	55.15	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.03	29.00	12.81	12.81	75.58	0.41	55.14	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.06	29.00	12.79	12.79	75.98	0.41	55.13	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.09	29.00	12.77	12.77	76.37	0.40	55.12	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.13	29.00	12.74	12.74	76.77	0.40	55.11	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.16	29.00	12.72	12.72	77.16	0.40	55.10	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.19	29.00	12.70	12.70	77.56	0.40	55.09	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.22	29.00	12.68	12.68	77.95	0.39	55.08	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.25	29.00	12.65	12.65	78.34	0.39	55.07	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.28	29.00	12.63	12.63	78.73	0.39	55.07	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.31	29.00	12.61	12.61	79.12	0.39	55.06	Vu < PhiVc/2	lot Reqd 9.6.	55.1	0.0	0.0
D Only	1	10.34	29.00	12.59	12.59	79.51	0.38	55.05	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.37	29.00	12.56	12.56	79.90	0.38	55.04	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.40	29.00	12.54	12.54	80.29	0.38	55.03	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.44	29.00	12.52	12.52	80.68	0.37	55.02	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.47	29.00	12.49	12.49	81.07	0.37	55.02	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.50	29.00	12.47	12.47	81.45	0.37	55.01	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.53	29.00	12.45	12.45	81.84	0.37	55.00	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.56	29.00	12.43	12.43	82.22	0.37	54.99	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.59	29.00	12.40	12.40	82.61	0.36	54.98	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.62	29.00	12.38	12.38	82.99	0.36	54.98	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.65	29.00	12.36	12.36	83.37	0.36	54.97	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.68	29.00	12.34	12.34	83.76	0.36	54.96	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.71	29.00	12.31	12.31	84.14	0.35	54.95	Vu < PhiVc/2	lot Reqd 9.6.	55.0	0.0	0.0
D Only	1	10.74	29.00	12.29	12.29	84.52	0.35	54.95	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.78	29.00	12.27	12.27	84.90	0.35	54.94	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.81	29.00	12.25	12.25	85.28	0.35	54.93	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.84	29.00	12.22	12.22	85.66	0.34	54.92	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.87	29.00	12.20	12.20	86.04	0.34	54.92	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.90	29.00	12.18	12.18	86.41	0.34	54.91	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.93	29.00	12.16	12.16	86.79	0.34	54.90	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.96	29.00	12.13	12.13	87.17	0.34	54.90	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	10.99	29.00	12.11	12.11	87.54	0.33	54.89	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	11.02	29.00	12.09	12.09	87.92	0.33	54.88	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	11.05	29.00	12.06	12.06	88.29	0.33	54.88	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	11.09	29.00	12.04	12.04	88.66	0.33	54.87	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	11.12	29.00	12.02	12.02	89.04	0.33	54.86	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	11.15	29.00	12.00	12.00	89.41	0.32	54.86	Vu < PhiVc/2	lot Reqd 9.6.	54.9	0.0	0.0
D Only	1	11.18	29.00	11.97	11.97	89.78	0.32	54.85	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.21	29.00	11.95	11.95	90.15	0.32	54.84	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.24	29.00	11.93	11.93	90.52	0.32	54.84	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.27	29.00	11.91	11.91	90.89	0.32	54.83	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.30	29.00	11.88	11.88	91.26	0.31	54.82	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.33	29.00	11.86	11.86	91.62	0.31	54.82	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.36	29.00	11.84	11.84	91.99	0.31	54.81	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.40	29.00	11.82	11.82	92.36	0.31	54.81	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.43	29.00	11.79	11.79	92.72	0.31	54.80	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.46	29.00	11.77	11.77	93.09	0.31	54.79	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.49	29.00	11.75	11.75	93.45	0.30	54.79	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.52	29.00	11.73	11.73	93.82	0.30	54.78	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.55	29.00	11.70	11.70	94.18	0.30	54.78	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.58	29.00	11.68	11.68	94.54	0.30	54.77	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.61	29.00	11.66	11.66	94.90	0.30	54.77	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.64	29.00	11.63	11.63	95.26	0.30	54.76	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.67	29.00	11.61	11.61	95.62	0.29	54.75	Vu < PhiVc/2	lot Reqd 9.6.	54.8	0.0	0.0
D Only	1	11.70	29.00	11.59	11.59	95.98	0.29	54.75	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.74	29.00	11.57	11.57	96.34	0.29	54.74	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.77	29.00	11.54	11.54	96.70	0.29	54.74	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.80	29.00	11.52	11.52	97.05	0.29	54.73	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.83	29.00	11.50	11.50	97.41	0.29	54.73	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.86	29.00	11.48	11.48	97.77	0.28	54.72	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.89	29.00	11.45	11.45	98.12	0.28	54.72	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.92	29.00	11.43	11.43	98.48	0.28	54.71	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.95	29.00	11.41	11.41	98.83	0.28	54.71	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	11.98	29.00	11.39	11.39	99.18	0.28	54.70	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.01	29.00	11.36	11.36	99.54	0.28	54.70	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.05	29.00	11.34	11.34	99.89	0.27	54.69	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.08	29.00	11.32	11.32	100.24	0.27	54.69	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.11	29.00	11.30	11.30	100.59	0.27	54.68	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.14	29.00	11.27	11.27	100.94	0.27	54.68	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.17	29.00	11.25	11.25	101.29	0.27	54.67	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.20	29.00	11.23	11.23	101.63	0.27	54.67	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.23	29.00	11.20	11.20	101.98	0.27	54.66	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.26	29.00	11.18	11.18	102.33	0.26	54.66	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.29	29.00	11.16	11.16	102.67	0.26	54.65	Vu < PhiVc/2	lot Reqd 9.6.	54.7	0.0	0.0
D Only	1	12.32	29.00	11.14	11.14	103.02	0.26	54.65	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.36	29.00	11.11	11.11	103.36	0.26	54.64	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.39	29.00	11.09	11.09	103.71	0.26	54.64	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.42	29.00	11.07	11.07	104.05	0.26	54.63	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.45	29.00	11.05	11.05	104.39	0.26	54.63	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.48	29.00	11.02	11.02	104.73	0.25	54.63	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.51	29.00	11.00	11.00	105.08	0.25	54.62	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.54	29.00	10.98	10.98	105.42	0.25	54.62	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.57	29.00	10.96	10.96	105.76	0.25	54.61	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.60	29.00	10.93	10.93	106.09	0.25	54.61	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.63	29.00	10.91	10.91	106.43	0.25	54.60	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.66	29.00	10.89	10.89	106.77	0.25	54.60	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.70	29.00	10.87	10.87	107.11	0.25	54.60	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.73	29.00	10.84	10.84	107.44	0.24	54.59	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.76	29.00	10.82	10.82	107.78	0.24	54.59	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	12.79	29.00	10.80	10.80	108.11	0.24	54.58	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.82	29.00	10.77	10.77	108.45	0.24	54.58	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.85	29.00	10.75	10.75	108.78	0.24	54.57	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.88	29.00	10.73	10.73	109.11	0.24	54.57	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.91	29.00	10.71	10.71	109.44	0.24	54.57	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.94	29.00	10.68	10.68	109.78	0.24	54.56	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	12.97	29.00	10.66	10.66	110.11	0.23	54.56	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	13.01	29.00	10.64	10.64	110.44	0.23	54.55	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	13.04	29.00	10.62	10.62	110.76	0.23	54.55	Vu < PhiVc/2	lot Reqd 9.6.	54.6	0.0	0.0
D Only	1	13.07	29.00	10.59	10.59	111.09	0.23	54.55	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.10	29.00	10.57	10.57	111.42	0.23	54.54	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.13	29.00	10.55	10.55	111.75	0.23	54.54	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.16	29.00	10.53	10.53	112.07	0.23	54.54	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.19	29.00	10.50	10.50	112.40	0.23	54.53	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.22	29.00	10.48	10.48	112.72	0.22	54.53	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.25	29.00	10.46	10.46	113.05	0.22	54.52	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.28	29.00	10.43	10.43	113.37	0.22	54.52	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.32	29.00	10.41	10.41	113.70	0.22	54.52	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.35	29.00	10.39	10.39	114.02	0.22	54.51	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.38	29.00	10.37	10.37	114.34	0.22	54.51	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.41	29.00	10.34	10.34	114.66	0.22	54.51	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.44	29.00	10.32	10.32	114.98	0.22	54.50	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.47	29.00	10.30	10.30	115.30	0.22	54.50	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.50	29.00	10.28	10.28	115.62	0.21	54.49	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.53	29.00	10.25	10.25	115.93	0.21	54.49	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.56	29.00	10.23	10.23	116.25	0.21	54.49	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.59	29.00	10.21	10.21	116.57	0.21	54.48	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.62	29.00	10.19	10.19	116.88	0.21	54.48	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.66	29.00	10.16	10.16	117.20	0.21	54.48	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.69	29.00	10.14	10.14	117.51	0.21	54.47	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.72	29.00	10.12	10.12	117.83	0.21	54.47	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.75	29.00	10.10	10.10	118.14	0.21	54.47	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.78	29.00	10.07	10.07	118.45	0.21	54.46	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.81	29.00	10.05	10.05	118.76	0.20	54.46	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.84	29.00	10.03	10.03	119.07	0.20	54.46	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.87	29.00	10.00	10.00	119.39	0.20	54.45	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.90	29.00	9.98	9.98	119.69	0.20	54.45	Vu < PhiVc/2	lot Reqd 9.6.	54.5	0.0	0.0
D Only	1	13.93	29.00	9.96	9.96	120.00	0.20	54.45	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	13.97	29.00	9.94	9.94	120.31	0.20	54.44	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.00	29.00	9.91	9.91	120.62	0.20	54.44	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.03	29.00	9.89	9.89	120.93	0.20	54.44	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.06	29.00	9.87	9.87	121.23	0.20	54.44	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.09	29.00	9.85	9.85	121.54	0.20	54.43	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.12	29.00	9.82	9.82	121.84	0.19	54.43	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.15	29.00	9.80	9.80	122.14	0.19	54.43	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.18	29.00	9.78	9.78	122.45	0.19	54.42	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.21	29.00	9.76	9.76	122.75	0.19	54.42	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.24	29.00	9.73	9.73	123.05	0.19	54.42	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.28	29.00	9.71	9.71	123.35	0.19	54.41	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.31	29.00	9.69	9.69	123.65	0.19	54.41	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.34	29.00	9.67	9.67	123.95	0.19	54.41	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.37	29.00	9.64	9.64	124.25	0.19	54.40	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.40	29.00	9.62	9.62	124.55	0.19	54.40	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.43	29.00	9.60	9.60	124.85	0.19	54.40	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	14.46	29.00	9.57	9.57	125.14	0.18	54.40	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.49	29.00	9.55	9.55	125.44	0.18	54.39	Vu < PhiVc/2	lot Reqd 9.6.	54.4	0.0	0.0
D Only	1	14.52	29.00	24.41	24.41	58.22	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.55	29.00	24.39	24.39	57.47	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.58	29.00	24.36	24.36	56.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.62	29.00	24.34	24.34	55.96	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.65	29.00	24.32	24.32	55.21	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.68	29.00	24.30	24.30	54.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.71	29.00	24.27	24.27	53.70	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.74	29.00	24.25	24.25	52.95	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.77	29.00	24.23	24.23	52.20	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.80	29.00	24.21	24.21	51.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.83	29.00	24.18	24.18	50.70	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.86	29.00	24.16	24.16	49.95	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.89	29.00	24.14	24.14	49.21	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.93	29.00	24.12	24.12	48.46	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.96	29.00	24.09	24.09	47.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	14.99	29.00	24.07	24.07	46.97	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.02	29.00	24.05	24.05	46.22	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.05	29.00	24.02	24.02	45.48	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.08	29.00	24.00	24.00	44.73	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.11	29.00	23.98	23.98	43.99	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.14	29.00	23.96	23.96	43.25	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.17	29.00	23.93	23.93	42.51	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.20	29.00	23.91	23.91	41.77	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.23	29.00	23.89	23.89	41.03	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.27	29.00	23.87	23.87	40.29	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.30	29.00	23.84	23.84	39.55	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.33	29.00	23.82	23.82	38.81	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.36	29.00	23.80	23.80	38.07	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.39	29.00	23.78	23.78	37.34	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.42	29.00	23.75	23.75	36.60	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.45	29.00	23.73	23.73	35.86	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.48	29.00	23.71	23.71	35.13	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.51	29.00	23.69	23.69	34.40	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.54	29.00	23.66	23.66	33.66	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.58	29.00	23.64	23.64	32.93	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.61	29.00	23.62	23.62	32.20	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.64	29.00	23.59	23.59	31.47	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.67	29.00	23.57	23.57	30.74	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.70	29.00	23.55	23.55	30.01	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.73	29.00	23.53	23.53	29.28	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.76	29.00	23.50	23.50	28.55	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.79	29.00	23.48	23.48	27.82	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.82	29.00	23.46	23.46	27.10	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.85	29.00	23.44	23.44	26.37	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.89	29.00	23.41	23.41	25.65	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.92	29.00	23.39	23.39	24.92	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.95	29.00	23.37	23.37	24.20	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	15.98	29.00	23.35	23.35	23.47	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.01	29.00	23.32	23.32	22.75	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.04	29.00	23.30	23.30	22.03	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.07	29.00	23.28	23.28	21.31	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.10	29.00	23.25	23.25	20.59	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0

Concrete Beam

= J:\2017 Projects\17-291 Peyree Remodel\Calcs\Lateral\Grade Beams for Simpson Frames MF1 and MF2.ec6 .
Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.30 .

Lic. # : KW-06006102 **Licensee : DIBBLE ENGINEERS INC.**

Description : MF 2: LC 56 (input already factored)

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
D Only	1	16.13	29.00	23.23	23.23	19.87	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.16	29.00	23.21	23.21	19.15	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.19	29.00	23.19	23.19	18.43	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.23	29.00	23.16	23.16	17.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.26	29.00	23.14	23.14	17.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.29	29.00	23.12	23.12	16.28	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.32	29.00	23.10	23.10	15.56	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.35	29.00	23.07	23.07	14.85	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.38	29.00	23.05	23.05	14.14	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.41	29.00	23.03	23.03	13.42	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.44	29.00	23.01	23.01	12.71	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.47	29.00	22.98	22.98	12.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.50	29.00	22.96	22.96	11.29	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.54	29.00	22.94	22.94	10.58	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.57	29.00	22.92	22.92	9.87	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.60	29.00	22.89	22.89	9.16	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.63	29.00	22.87	22.87	8.45	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.66	29.00	22.85	22.85	7.74	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.69	29.00	22.82	22.82	7.03	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.72	29.00	22.80	22.80	6.33	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.75	29.00	22.78	22.78	5.62	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.78	29.00	22.76	22.76	4.92	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.81	29.00	22.73	22.73	4.21	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.85	29.00	22.71	22.71	3.51	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.88	29.00	22.69	22.69	2.80	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.91	29.00	22.67	22.67	2.10	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.94	29.00	22.64	22.64	1.40	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	16.97	29.00	22.62	22.62	0.70	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0
D Only	1	17.00	29.00	22.60	22.60	0.00	1.00	57.09	Vu < PhiVc/2	lot Reqd 9.6.	57.1	0.0	0.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	17.000	143.43	233.22	0.62
D Only					
Span # 1	1	17.000	143.43	233.22	0.62

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
D Only	1	0.0159	10.172	D Only	-0.0000	17.000

Checking Required 4" Pin Piles for Lateral Resistance, MF1

V =	11100	lbs, ASD

Solving for Resistance

Grade Beam Dims		
Width =	20	in
Depth =	27	in
Length =	33	ft
ρ_{conc} =	150	pcf
W_{GB} =	18563	lbs
μ =	0.5	Ult.
$R_{friction}$ =	9281	lbs

Passive Resistance		
Area =	3.75	sf
Passive Soil Pressure	350	pcf, Ult.
$R_{passive}$	1313	lbs
$R_{resist\ total}$	10594	lbs, Service: (1.0D + 1.0H)
F.S.	1.5	
R_{allow}	7063	lbs

Solving for Required 4" Pin Piles

Force Left to Resolve	4038	lbs
4" Pin Allow. Lat'l Capacity	1000	lbs
Pins Each Direction Req'd	**5**	
Total Pins Req'd	**10**	

Checking Required 4" Pin Piles for Lateral Resistance, MF2

V =	24610	lbs, ASD

Solving for Resistance

Grade Beam Dims		
Width =	48	in
Depth =	33	in
Length =	20	ft
ρ_{conc} =	150	pcf
W_{GB} =	33000	lbs
μ =	0.5	Ult.
$R_{friction}$ =	16500	lbs

Passive Resistance		
Area =	11.00	sf
Passive Soil Pressure	350	pcf, Ult.
$R_{passive}$	3850	lbs
$R_{resist\ total}$	20350	lbs, Service: (1.0D + 1.0H)
F.S.	1.5	
R_{allow}	13567	lbs

Solving for Required 4" Pin Piles

Force Left to Resolve	11043	lbs
4" Pin Allow. Lat'l Capacity	1000	lbs
Pins Each Direction Req'd	**12**	
Total Pins Req'd	**24**	

DEI
DIBBLE ENGINEERS INC

APPENDIX

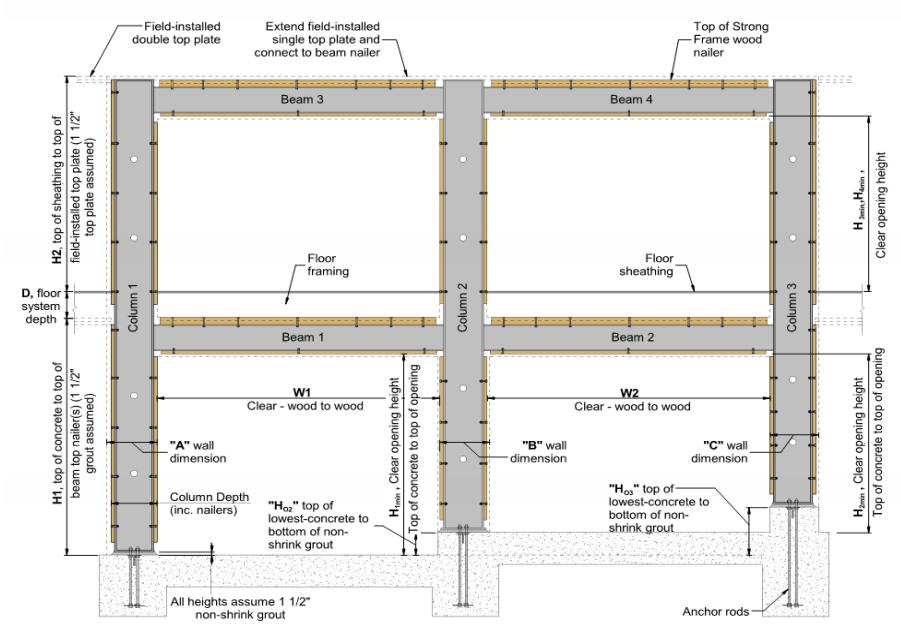
Peyree Remodel #17-291

SIMPSON STRONG-TIE® COMPANY, INC.
The World's "No Equal" Structural Connector Company
5956 W. Las Positas Blvd. ● Pleasanton, California 94588
Telephone: (800) 999-5099 ● Fax: (925) 847-15977

SIMPSON STRONG-FRAME®

SPECIAL MOMENT FRAME

DESIGN CALCULATION PACKAGE



Frame Model: Custom 2 Story 3 Bay Frame

Frame ID: MF-1 (Grid 1)
Project Location: 6059 77th Ave SE, Mercer Island, WA 98040
Engineer of Record: Dibble Engineers, Inc.

Column1 Size:	W10X68	**Link 1 Size:**	MF4-2.25	**W1 =**	103.4 in
Column2 Size:	W10X88	**Link 2 Size:**	MF4-2.25	**W2 =**	109.1 in
Column3 Size:	W10X88	**Link 3 Size:**	MF4-2.25	**W3=**	103.4 in
Column4 Size:	W10X68	**Link 4 Size:**	MF4-2.25	**H1 =**	178.8 in
Beam 1,2,3 Size:	W12X45	**Link 5 Size:**	MF4-2.25	**H2 =**	229.5 in
Beam 4,5,6 Size:	W12X45	**Link 6 Size:**	MF4-2.25	**HO1 =**	0.0 in
				HO2 =	0.0 in
ckage Contents:				**HO2 =**	0.0 in
				D =	0.75

Design Summary
SAP2000 Model Input
SAP2000 beam and column design
Moment Connection Design
Column Base Plate Design
Nailer Attachment Design

PHILLIP HUI
STATE OF WASHINGTON
55823
REGISTERED
PROFESSIONAL ENGINEER
COA#3125
05/17/2019

Design Date: 5/3/2019
Design by: B.Tran
Job No.: ES-191628

Note: These calculations are only applicable to frames designed and manufactured by Simpson Strong-Tie Company Inc. Any other use of these calculations, or portion thereof, for purposes other than specifying frames manufactured by Simpson Strong-Tie is expressly prohibited.

Note: Signed & Sealed Calculations will be provided to the Building Department as a deferred submittal upon request. Signed & Sealed Calculations will not be issued until completion of a frame order from Simpson Strong-Tie Company Inc. Stamped but Unsigned calculations are provided for the plancheck review process. Stamped and Signed Calculations are required to be on file with the building department for final permitting.

Note: Simpson Strong-Tie® Strong Frame® and the Yield-Link™ structural fuse are protected under US patent number 8,001,734 B2 and must be supplied or licensed through Simpson Strong-Tie.

SIMPSON STRONG-TIE® COMPANY, INC.
The World's "No Equal" Structural Connector Company
5956 W. Las Positas Blvd. ● Pleasanton, California 94588
Telephone: (800) 999-5099 ● Fax: (925) 847-1597

Simpson Strong-Tie
Special Moment Frame Design Summary

Frame Model:	**Custom 2 Story 3 Bay Frame**	

Design Codes/References Used:

2015 International Building Code
AISC Steel Construction Manual - 14th Edition
Simpson Strong Tie Design Procedure for SST SMF

Material Design Properties:

Beam/Column Steel:	**ASTM A992**	**F_y = 50 ksi**
Plate Steel:	**ASTM A572 Grade 50**	**F_y = 50 ksi**
		(Link, Baseplate, cap plate, stiffener and shear plate)

High Strength Bolts:

Link-Stem-to beam:	**ASTM F2280 Twisted off bolts (ASTM A449 Equivalent), Pre-tensioned**
Link-flange-to column:	**ASTM A325 (Snug-tight)**
Shear plate-to-column:	**ASTM A325 (Snug-tight)**
Anchor Bolts:	**ASTM F1554 Gr 36, ASTM A36, or ASTM A449 (See Base Plate design)**
Weld Electrodes:	**E70XX**

Notes:

(1) Simpson Strong-Tie Strong Frame Moment Frames are designed using Load and Resistance Factored design (LRFD) methodology for determining frame drift and strength limits. Allowable Stress Design (ASD) shear and drift are determined as V(ASD) = 0.7 X V(LRFD) and Drift(ASD) = 0.7 X Drift(LRFD) for seismic load combinations and V(ASD) = V(LRFD) / 1.6 for wind load combinations.

(2) The following calculations are provided for justifying that Simpson Strong-Tie Strong Frame Moment Frame component capacities meet or exceed the design forces and criteria provided & determined by the Designer. Simpson Strong-Tie has not confirmed and is not responsible for any of the design, engineering, calculations, or derivation of the structural forces or drifts related to the building and the designed elements. Moment frames and other lateral resisting elements placed in the same lateral resisting shear line shall have applied lateral loads proportioned based on relative stiffness.

(3) This Simpson Strong-Tie Strong Frame Moment Frame is part of the overall lateral force resisting system of the structure. Design of the building's lateral force resisting system, including the load path to transfer lateral forces from the structure to the ground, is the responsibility of the designer. The designer must specify the required components of the complete load transfer path including diaphragms, shear transfer, chords and collectors and foundations.

(4) Footing dimensions provided are the minimums required for concrete anchorage requirements only. The designer must determine required footing size and reinforcing for other design limits, such as foundation shear and bending, soil bearing shear transfer, and frame stability / overturning. Designer shall detail actual footing / grade beam size and reinforcing.

(5) Holes in base plates are over-sized for erection tolerance. Designer must evaluate effects of over-sized holes and provide plate washer with standard-size holes welded to base plate or request base plates with standard-size holes where required.

(6) Refer to Strong Frame installation detail sheets for allowable field modifications and additions. Welding shall be in accordance with AWS D1.1 and AWS D1.8 (as applicable for seismic). Welds shall be specified by the designer. Provide welding special inspection as required by the local building department.

COMMENTS:

1. Project Information:

Job Name:	6059 77th Ave SE	Date:	5/3/2019
Project Address:	6059 77th Ave SE, Mercer Island, WA 98040	Engineer:	Dibble Engineers, Inc.
Frame ID:	MF-1 (Grid 1)	Phone:	425.828.4200 x237
SST Enginer:	**B.Tran**	E-mail:	Justin@DibbleEngineers.com

2. Frame Geometry:

Design Criteria:

Frame ID: **Custom 2 Story 3 Bay Frame** | Custom Sizes

	ft	in	
Clear Opening Width(W1)=	8 ft	7.38 in	
Clear Opening Width(W2)=	9 ft	1.13 in	
Clear Opening Width(W3)=	8 ft	7.38 in	
Outside Width(W4)=	30 ft	10.28 in	
Top Plate Height(H1)=	14 ft	2.75 in	
Bottom Nailer Height(H1_min)=	0 ft	0.00 in	OK
Floor Depth (D)=	0 ft	0.75 in	
Top Plate Height(H2)=	19 ft	1.50 in	
Bottom Nailer Height(H2_min)=	0 ft	0.00 in	OK
Foundation offset (HO1)=	0 ft	0.00 in	
Foundation offset (HO2)=	0 ft	0.00 in	
Foundation offset (HO3)=	0 ft	0.00 in	
Foundation offset (HO4)=	0 ft	0.00 in	
A=	24.00 in	OK	
B=	24.00 in	OK	
C=	24.00 in	OK	
Base Fixity:	Fixed		
Slab Height:	8.00 in		
Min. Grade Beam Depth:	27.00 in		

Column 1=	W10X68
Column 2=	W10X88
Column 3=	W10X88
Column 4=	W10X68
Beam 1, 2, 3 =	W12X45
Link1 ID=	MF4-2.25
Link2 ID=	MF4-2.25
Link3 ID=	MF4-2.25
Beam 4, 5, 6=	W12X45
Link4 ID=	MF4-2.25
Link5 ID=	MF4-2.25
Link6 ID=	MF4-2.25
Shear tab 1	Heavy
Shear tab 2	Heavy
Shear tab 3	Heavy
Shear tab 4	Heavy
Shear tab 5	Heavy
Shear tab 6	Heavy

Design Code=	2015 IBC
S_{DS}=	0.976
R_load=	6.5
R_frame=	6.5
Cd=	4
Omega=	3
I=	1
Rho=	1
Seismic Drift Limit=	0.02 Hx
Wind Drift Limit:	hx/ 50

Beam Deflection Limits:

Live Load=	L/ 480
Dead + Live Load=	L/ 360
Snow/Wind Load=	L/ 360

Beam Deflection Limits (Simply Supported):

SS_beam Limit=	L/ 360

3. Loading:

3.1.1 Uniform Loads: (Negative value for uplift), 1st Floor 1st Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	188 plf	0.00	9.75	Y	W_{LR1}=	0 plf	0.00	0.00	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	375 plf	0.00	9.75	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	0 plf	0.00	0.00	Y
					Snow Factor=	0.2			

3.1.2 Uniform Loads: (Negative value for uplift), 1st Floor 2nd Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	188 plf	0.00	10.24	Y	W_{LR1}=	0 plf	0.00	0.00	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	375 plf	0.00	10.24	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	0 plf	0.00	0.00	Y
					Snow Factor=	0.2			

3.1.3 Uniform Loads: (Negative value for uplift), 1st Floor 3rd Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	188 plf	0.00	9.75	Y	W_{LR1}=	0 plf	0.00	0.00	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	375 plf	0.00	9.75	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	0 plf	0.00	0.00	Y
					Snow Factor=	0.2			

3.1.4 Uniform Loads: (Negative value for uplift), 2nd Floor/Roof 1st Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	161 plf	0.00	9.75	Y	W_{LR1}=	153 plf	0.00	9.75	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	0 plf	0.00	0.00	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	192 plf	0.00	9.75	Y
					Snow Factor=	0.2			

3.1.5 Uniform Loads: (Negative value for uplift), 2nd Floor/Roof 2nd Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	161 plf	0.00	10.24	Y	W_{LR1}=	153 plf	0.00	10.24	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	0 plf	0.00	0.00	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	192 plf	0.00	10.24	Y
					Snow Factor=	0.2			

3.1.6 Uniform Loads: (Negative value for uplift), 2nd Floor/Roof 3rd Bay Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	161 plf	0.00	9.75	Y	W_{LR1}=	153 plf	0.00	9.75	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	0 plf	0.00	0.00	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	192 plf	0.00	9.75	Y
					Snow Factor=	0.2			

3.2: Point Loads:

Lateral F_{EQ1}= **1180** lbs, ASD Seismic Load FWIND1= **0** lbs, ASD Wind Load
(1st Flr 1st bay Bm) F_{EQ1}= 1686 lbs, LRFD Seismic Load F_{WIND1}= 0 lbs, LRFD Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 1st Floor Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P1_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	N
$P1_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	N
$P1_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P1_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P1_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P1_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P1_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P1_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

Lateral F_{EQ1}= **1240** lbs, ASD Seismic Load FWIND1= **0** lbs, ASD Wind Load
(1st Flr 2nd bay Bm) F_{EQ1}= 1771 lbs, LRFD Seismic Load F_{WIND1}= 0 lbs, LRFD Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 1st Floor Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P1_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	N
$P1_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	N
$P1_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P1_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P1_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P1_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P1_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P1_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

Lateral F_{EQ1}= **1180** lbs, ASD Seismic Load FWIND1= **0** lbs, ASD Wind Load
(1st Flr 3rd bay Bm) F_{EQ1}= 1686 lbs, LRFD Seismic Load F_{WIND1}= 0 lbs, LRFD Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 1st Floor Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P1_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	N
$P1_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	N
$P1_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P1_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P1_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P1_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P1_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P1_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

Lateral F_{EQ2}= **2458** lbs, ASD Seismic Load FWIND2= **0** lbs, ASD Wind Load
(2nd Flr 1st bay Bm) F_{EQ2}= 3512 lbs, LRFD Seismic Load F_{WIND2}= 0 lbs, LRFD Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 2nd Floor/Roof Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P2_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	N
$P2_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	N
$P2_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P2_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P2_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P2_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P2_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P2_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

Lateral	F_{EQ2}=	**2583**	lbs, ASD	Seismic Load	FWIND2=	**0**	lbs, ASD	Wind Load
(2nd Flr 2nd bay Bm	F_{EQ2}=	3691	lbs, LRFD	Seismic Load	F_{WIND2}=	0	lbs, LRFD	Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 2nd Floor/Roof Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P2_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	**N**
$P2_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	**N**
$P2_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	**N**
$P2_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	**N**
$P2_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	**N**
$P2_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	**N**
$P2_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	**N**
$P2_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	**N**

Lateral	F_{EQ2}=	**2458**	lbs, ASD	Seismic Load	FWIND2=	**0**	lbs, ASD	Wind Load
(2nd Flr 3rd bay Bm	F_{EQ2}=	3512	lbs, LRFD	Seismic Load	F_{WIND2}=	0	lbs, LRFD	Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 2nd Floor/Roof Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P2_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	**N**
$P2_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	**N**
$P2_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	**N**
$P2_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	**N**
$P2_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	**N**
$P2_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	**N**
$P2_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	**N**
$P2_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	**N**

2-STORY X 1-BAY SMF DESIGN SUMMARY:

		DCR Limit	Check
FRAME SEISMIC DRIFT (Hx1) DCR	0.341	1.03	OK
FRAME SEISMIC DRIFT (Hx2) DCR	0.549	1.03	OK
FRAME WIND DRIFT (Hx1) DCR	0.000	1.03	OK
FRAME WIND DRIFT (Hx2) DCR	0.000	1,03	OK

BEAM DELECTION CHECK:

1st Level Beam1	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.029	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.023	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.031	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.001	1.00	OK
1st Level Beam2	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.034	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.024	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.033	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.002	1.00	OK
1st Level Beam3	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.029	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.022	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.030	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.001	1.00	OK
2nd Level Beam4	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.019	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.003	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.019	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.011	1.00	OK
2nd Level Beam5	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.003	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.006	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.001	1.00	OK
2nd Level Beam6	STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.019	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.003	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.018	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.011	1.00	OK

LINK CHECK:

1st Level Beam1	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.454	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.632	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.341	1.03	OK
1st Level Beam2	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.417	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.000	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.341	1.03	OK
1st Level Beam3	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.454	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.750	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.341	1.03	OK

2nd Level Beam4	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.357	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.632	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.549	1.03	OK
2nd Level Beam5	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.316	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.000	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.549	1.03	OK
2nd Level Beam6	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.357	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.715	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.727	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.750	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.600	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.517	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.540	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.687	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.343	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.549	1.03	OK

COLUMN CHECK:

1st Level Column 1	COLUMN MAX DCR	0.352	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.315	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.665	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.594	1.00	OK
1st Level Column 2	COLUMN MAX DCR	0.359	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.470	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.995	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.651	1.00	OK
1st Level Column 3	COLUMN MAX DCR	0.359	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.470	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.995	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.460	1.00	OK
1st Level Column 4	COLUMN MAX DCR	0.352	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.315	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.665	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.594	1.00	OK
2nd Level Column 1	COLUMN MAX DCR	0.225	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.313	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.665	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.594	1.00	OK
	18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.513	1.00	OK
2nd Level Column 2	COLUMN MAX DCR	0.278	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.471	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.995	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.651	1.00	OK
	18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.561	1.00	OK

2nd Level Column 3	COLUMN MAX DCR	0.278	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.467	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.995	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.462	1.00	OK
	18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.399	1.00	OK
2nd Level Column 4	COLUMN MAX DCR	0.273	1.00	OK
	COLUMN FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	COLUMN WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.313	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.665	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.594	1.00	OK
	18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.513	1.00	OK

BEAM CHECK:

1st Level Beam 1	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.032	1.00	OK
	PR FRAME BEAM MAX DCR=	0.427	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.371	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.647	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.576	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.065	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.058	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.233	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.360	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.133	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.048	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.043	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.337	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.238	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK
1st Level Beam 2	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.036	1.00	OK
	PR FRAME BEAM MAX DCR=	0.427	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.371	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.647	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.576	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.065	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.058	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.233	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.360	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.133	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.048	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.043	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.337	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.238	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK

1st Level Beam 3	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.032	1.00	OK
	PR FRAME BEAM MAX DCR=	0.427	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.371	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.647	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.576	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.065	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.058	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.233	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.360	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.133	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.048	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.043	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.337	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.238	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.287	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK
2nd Level Beam 4	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.020	1.00	OK
	PR FRAME BEAM MAX DCR=	0.427	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.359	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.626	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.558	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.107	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.095	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.225	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.349	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.131	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.079	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.071	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.326	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.230	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.288	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.180	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK
2nd Level Beam 5	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.023	1.00	OK
	PR FRAME BEAM MAX DCR=	0.427	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.359	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.626	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.558	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.107	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.095	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.225	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.349	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.131	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.079	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.071	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.326	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.230	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.288	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.180	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK

2nd Level Beam 6	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.020	1.00	OK
	PR FRAME BEAM MAX DCR=	0.231	1.00	OK
	BEAM FLANGE WIDTH-TO-THICKNESS CHECK	OK	-	OK
	BEAM WEB WIDTH-TO-THICKNESS CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.266	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.359	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.626	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.558	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.107	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.095	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.225	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.349	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.119	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.131	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.079	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.071	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.326	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.230	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.288	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.180	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK

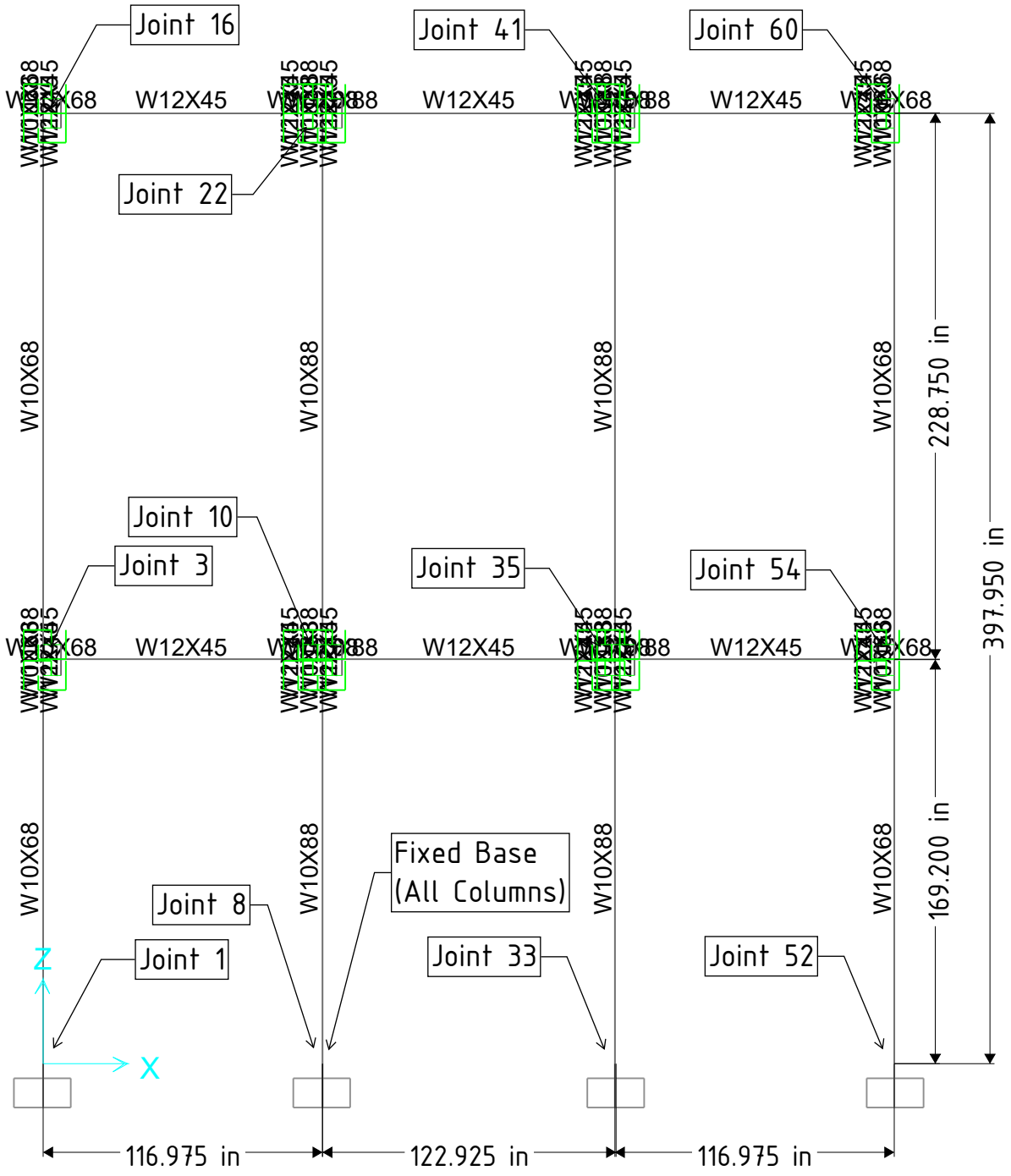
FIXED BASE COLUMN WIDTH-TO-THICKNESS CHECK

Column 1 bf/2*tf_columns=	6.58	7.22	OK
Column 1 h/tw_columns=	16.70	56.86	OK
Column 2 bf/2*tf_columns=	5.18	7.22	OK
Column 2 h/tw_columns=	13.00	58.22	OK
Column 3 bf/2*tf_columns=	5.18	7.22	OK
Column 3 h/tw_columns=	13.00	58.22	OK
Column 4 bf/2*tf_columns=	6.58	7.22	OK
Column 4 h/tw_columns=	16.70	57.36	OK

Code= 2015 IBC
Sds= 0.976
Ω= 3 User input (2.5 or 3)
f_2= 0.2 Snow load factor
ρ= 1

Combo	Load Combinations	Load Combination Multipliers									
		D	L	Lr	S	R	W	E	Ev	Ni	
1	D	1	0	0	0	0	0	0	0	0	Load Cases
2	L	0	1	0	0	0	0	0	0	0	
3	Lr	0	0	1	0	0	0	0	0	0	
4	S	0	0	0	1	0	0	0	0	0	
5	R	0	0	0	0	1	0	0	0	0	
6	W	0	0	0	0	0	1	0	0	0	
7	E	0	0	0	0	0	0	1	1	0	
8	D	1	0	0	0	0	0	0	0	1	ASD
9	D+ L	1	1	0	0	0	0	0	0	1	
10	D + Lr	1	0	1	0	0	0	0	0	1	
11	D + S	1	0	0	1	0	0	0	0	1	
12	D + R	1	0	0	0	1	0	0	0	1	
13	D + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0	0	0	1	
14	D + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0	0	0	1	
15	D + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0	0	0	1	
16	D + 0.6 W	1	0	0	0	0	0.6	0	0	0	
17	D - 0.6 W	1	0	0	0	0	-0.6	0	0	0	
18	$(1.0 + 0.14\ S_{DS})$ D + 0.7 E *ρ	1.13664	0	0	0	0	0	0.7	0.7000	0	
19	$(1.0 + 0.14\ S_{DS})$ D - 0.7 E * ρ	1.13664	0	0	0	0	0	-0.7	-0.700	0	
20	D + 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0.45	0	0	0	
21	D - 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	-0.45	0	0	0	
22	D + 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0.45	0	0	0	
23	D - 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	-0.45	0	0	0	
24	D + 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0.45	0	0	0	
25	D - 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	-0.45	0	0	0	
26	$(1.0 + 0.105\ S_{DS})$ D + 0.525 E*ρ+ 0.75 L + 0.75 Lr	1.10248	0.75	0.75	0	0	0	0.525	0.525	0	
27	$(1.0 + 0.105\ S_{DS})$ D - 0.525 E*ρ+ 0.75 L + 0.75 Lr	1.10248	0.75	0.75	0	0	0	-0.525	-0.525	0	
28	$(1.0 + 0.105\ S_{DS})$ D + 0.525 E*ρ + 0.75 L + 0.75 S	1.10248	0.75	0	0.75	0	0	0.5250	0.525	0	
29	$(1.0 + 0.105\ S_{DS})$ D - 0.525 E*ρ + 0.75 L + 0.75 S	1.10248	0.75	0	0.75	0	0	-0.5250	-0.525	0	
30	$(1.0 + 0.105\ S_{DS})$ D + 0.525 E*ρ + 0.75 L + 0.75 R	1.10248	0.75	0	0	0.75	0	0.5250	0.525	0	
31	$(1.0 + 0.105\ S_{DS})$ D - 0.525 E*ρ + 0.75 L + 0.75 R	1.10248	0.75	0	0	0.75	0	-0.5250	-0.525	0	
32	0.6 D + 0.6 W	0.6	0	0	0	0	0.6	0	0	0	
33	0.6 D - 0.6 W	0.6	0	0	0	0	-0.6	0	0	0	
34	$(0.6 - 0.14\ S_{DS})$ D + 0.7 E*ρ	0.46336	0	0	0	0	0	0.7	0.7000	0	
35	$(0.6 - 0.14\ S_{DS})$ D - 0.7 E*ρ	0.46336	0	0	0	0	0	-0.7	-0.700	0	
36	1.4 D	1.4	0	0	0	0	0	0	0	1	LRFD
37	1.2 D + 1.6 L + 0.5 Lr	1.2	1.6	0.5	0	0	0	0	0	1	
38	1.2 D + 1.6 L + 0.5 S	1.2	1.6	0	0.5	0	0	0	0	1	
39	1.2 D + 1.6 L + 0.5 R	1.2	1.6	0	0	0.5	0	0	0	1	
40	1.2 D + 1.6 Lr + 0.5 L	1.2	0.5	1.6	0	0	0	0	0	1	
41	1.2 D + 1.6 Lr + 0.5 W	1.2	0	1.6	0	0	0.5	0	0	0	
42	1.2 D + 1.6 Lr - 0.5 W	1.2	0	1.6	0	0	-0.5	0	0	0	
43	1.2 D + 1.6 S + 0.5 L	1.2	0.5	0	1.6	0	0	0	0	1	
44	1.2 D + 1.6 S + 0.5 W	1.2	0	0	1.6	0	0.5	0	0	0	
45	1.2 D + 1.6 S - 0.5 W	1.2	0	0	1.6	0	-0.5	0	0	0	
46	1.2 D + 1.6 R + 0.5 L	1.2	0.5	0	0	1.6	0	0	0	1	
47	1.2 D + 1.6 R + 0.5 W	1.2	0	0	0	1.6	0.5	0	0	0	
48	1.2 D + 1.6 R - 0.5 W	1.2	0	0	0	1.6	-0.5	0	0	0	
49	1.2 D + 1 W + 0.5 L + 0.5 Lr	1.2	0.5	0.5	0	0	1	0	0	0	
50	1.2 D - 1 W + 0.5 L + 0.5 Lr	1.2	0.5	0.5	0	0	-1	0	0	0	
51	1.2 D + 1 W + 0.5 L + 0.5 S	1.2	0.5	0	0.5	0	1	0	0	0	
52	1.2 D - 1 W + 0.5 L + 0.5 S	1.2	0.5	0	0.5	0	-1	0	0	0	
53	1.2 D + 1 W + 0.5 L + 0.5 R	1.2	0.5	0	0	0.5	1	0	0	0	
54	1.2 D - 1 W + 0.5 L + 0.5 R	1.2	0.5	0	0	0.5	-1	0	0	0	
55	$(1.2 + 0.2\ S_{DS})$D + E*ρ + 0.5 L + f2*S	1.3952	0.5	0	0.2	0	0	1	1.0000	0	
56	$(1.2 + 0.2\ S_{DS})$D - E*ρ + 0.5 L + f2* S	1.3952	0.5	0	0.2	0	0	-1	-1.000	0	
57	0.9 D + 1 W	0.9	0	0	0	0	1	0	0	0	
58	0.9 D - 1 W	0.9	0	0	0	0	-1	0	0	0	
59	$(0.9 - 0.2\ S_{DS})$ D + E*ρ	0.7048	0	0	0	0	0	1	1.0000	0	
60	$(0.9 - 0.2\ S_{DS})$ D - E*ρ	0.7048	0	0	0	0	0	-1	-1.000	0	
61	$(1.2 + 0.2\ S_{DS})$D + ΩE + 0.5L + f2*S	1.3952	0.5	0	0.2	0	0	3	3	0	
62	$(1.2 + 0.2\ S_{DS})$D -ΩE + 0.5L + f2*S	1.3952	0.5	0	0.2	0	0	-3	-3	0	
63	$(0.9 - 0.2\ S_{DS})$ D + ΩE	0.7048	0	0	0	0	0	3	3	0	
64	$(0.9 - 0.2\ S_{DS})$ D - ΩE	0.7048	0	0	0	0	0	-3	-3	0	

Frame Input


Joint 16
Joint 41
Joint 60
W12X45
W12X45
W12X45
Joint 22
W10X68
W10X88
W10X88
W10X68
228.750 in
397.950 in
Joint 10
Joint 3
Joint 35
Joint 54
W12X45
W12X45
W12X45
W10X68
W10X88
W10X88
W10X68
169.200 in
Fixed Base
(All Columns)
Joint 8
Joint 1
Joint 33
Joint 52
Z
X
116.975 in
122.925 in
116.975 in

Joint Displacement (in) (1st Floor)											
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Seismic Horizontal	Wind Horizontal
3	LCM1	0.000	0.000	-0.001	10	LCM1	0.000	0.000	-0.002		
3	LCM2	0.000	0.000	-0.001	10	LCM2	0.000	0.000	-0.001		
3	LCM3	0.000	0.000	0.000	10	LCM3	0.000	0.000	0.000		
3	LCM4	0.000	0.000	0.000	10	LCM4	0.000	0.000	0.000		
3	LCM5	0.000	0.000	0.000	10	LCM5	0.000	0.000	0.000		
3	LCM6	0.000	0.000	0.000	10	LCM6	0.000	0.000	0.000		
3	LCM7	0.305	0.000	0.003	10	LCM7	0.305	0.000	0.000		
3	LCM8	0.029	0.000	-0.001	10	LCM8	0.029	0.000	-0.002		
3	LCM9	0.029	0.000	-0.001	10	LCM9	0.029	0.000	-0.002		
3	LCM10	0.029	0.000	-0.001	10	LCM10	0.029	0.000	-0.002		
3	LCM11	0.029	0.000	-0.001	10	LCM11	0.029	0.000	-0.002		
3	LCM12	0.029	0.000	-0.001	10	LCM12	0.029	0.000	-0.002		
3	LCM13	0.029	0.000	-0.002	10	LCM13	0.029	0.000	-0.003		
3	LCM14	0.029	0.000	-0.002	10	LCM14	0.029	0.000	-0.003		
3	LCM15	0.029	0.000	-0.001	10	LCM15	0.029	0.000	-0.002		
3	LCM16	0.000	0.000	-0.001	10	LCM16	0.000	0.000	-0.002		
3	LCM17	0.000	0.000	-0.001	10	LCM17	0.000	0.000	-0.002		
3	LCM18	0.213	0.000	0.001	10	LCM18	0.213	0.000	-0.002		
3	LCM19	-0.213	0.000	-0.003	10	LCM19	-0.213	0.000	-0.002		
3	LCM20	0.000	0.000	-0.002	10	LCM20	0.000	0.000	-0.003		0.000
3	LCM21	0.000	0.000	-0.002	10	LCM21	0.000	0.000	-0.003		0.000
3	LCM22	0.000	0.000	-0.002	10	LCM22	0.000	0.000	-0.003		0.000
3	LCM23	0.000	0.000	-0.002	10	LCM23	0.000	0.000	-0.003		0.000
3	LCM24	0.000	0.000	-0.002	10	LCM24	0.000	0.000	-0.002		0.000
3	LCM25	0.000	0.000	-0.002	10	LCM25	0.000	0.000	-0.002		0.000
3	LCM26	0.160	0.000	0.000	10	LCM26	0.160	0.000	-0.003		
3	LCM27	-0.160	0.000	-0.003	10	LCM27	-0.160	0.000	-0.003		
3	LCM28	0.160	0.000	0.000	10	LCM28	0.160	0.000	-0.003		
3	LCM29	-0.160	0.000	-0.003	10	LCM29	-0.160	0.000	-0.003		
3	LCM30	0.160	0.000	0.000	10	LCM30	0.160	0.000	-0.003		
3	LCM31	-0.160	0.000	-0.003	10	LCM31	-0.160	0.000	-0.002		
3	LCM32	0.000	0.000	-0.001	10	LCM32	0.000	0.000	-0.001		
3	LCM33	0.000	0.000	-0.001	10	LCM33	0.000	0.000	-0.001		
3	LCM34	0.213	0.000	0.001	10	LCM34	0.213	0.000	-0.001		
3	LCM35	-0.213	0.000	-0.002	10	LCM35	-0.213	0.000	-0.001		
3	LCM36	0.029	0.000	-0.001	10	LCM36	0.029	0.000	-0.002		
3	LCM37	0.029	0.000	-0.002	10	LCM37	0.029	0.000	-0.003		
3	LCM38	0.029	0.000	-0.002	10	LCM38	0.029	0.000	-0.004		
3	LCM39	0.029	0.000	-0.002	10	LCM39	0.029	0.000	-0.003		
3	LCM40	0.029	0.000	-0.002	10	LCM40	0.029	0.000	-0.003		
3	LCM41	0.000	0.000	-0.002	10	LCM41	0.000	0.000	-0.002		
3	LCM42	0.000	0.000	-0.002	10	LCM42	0.000	0.000	-0.002		
3	LCM43	0.028	0.000	-0.002	10	LCM43	0.029	0.000	-0.003		
3	LCM44	0.000	0.000	-0.002	10	LCM44	0.000	0.000	-0.003		
3	LCM45	0.000	0.000	-0.002	10	LCM45	0.000	0.000	-0.003		
3	LCM46	0.029	0.000	-0.001	10	LCM46	0.029	0.000	-0.002		
3	LCM47	0.000	0.000	-0.001	10	LCM47	0.000	0.000	-0.002		
3	LCM48	0.000	0.000	-0.001	10	LCM48	0.000	0.000	-0.002		
3	LCM49	0.000	0.000	-0.002	10	LCM49	0.000	0.000	-0.003		
3	LCM50	0.000	0.000	-0.002	10	LCM50	0.000	0.000	-0.003		
3	LCM51	0.000	0.000	-0.002	10	LCM51	0.000	0.000	-0.003		
3	LCM52	0.000	0.000	-0.002	10	LCM52	0.000	0.000	-0.003		
3	LCM53	0.000	0.000	-0.002	10	LCM53	0.000	0.000	-0.002		
3	LCM54	0.000	0.000	-0.002	10	LCM54	0.000	0.000	-0.002		
3	LCM55	0.305	0.000	0.001	10	LCM55	0.305	0.000	-0.003	0.305	
3	LCM56	-0.305	0.000	-0.005	10	LCM56	-0.305	0.000	-0.002	-0.305	
3	LCM57	0.000	0.000	-0.001	10	LCM57	0.000	0.000	-0.001		
3	LCM58	0.000	0.000	-0.001	10	LCM58	0.000	0.000	-0.001		
3	LCM59	0.305	0.000	0.002	10	LCM59	0.305	0.000	-0.001	0.305	
3	LCM60	-0.305	0.000	-0.004	10	LCM60	-0.305	0.000	-0.001	-0.305	
3	LCM61	0.914	0.000	0.006	10	LCM61	0.914	0.000	-0.004		
3	LCM62	-0.914	0.000	-0.010	10	LCM62	-0.914	0.000	-0.002		
3	LCM63	0.914	0.000	0.007	10	LCM63	0.914	0.000	-0.002		
3	LCM64	-0.914	0.000	-0.009	10	LCM64	-0.914	0.000	0.000		

Joint Displacement (in) (1st Floor)									
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical
35	LCM1	0.000	0.000	-0.002	54	LCM1	0.000	0.000	-0.001
35	LCM2	0.000	0.000	-0.001	54	LCM2	0.000	0.000	-0.001
35	LCM3	0.000	0.000	0.000	54	LCM3	0.000	0.000	0.000
35	LCM4	0.000	0.000	0.000	54	LCM4	0.000	0.000	0.000
35	LCM5	0.000	0.000	0.000	54	LCM5	0.000	0.000	0.000
35	LCM6	0.000	0.000	0.000	54	LCM6	0.000	0.000	0.000
35	LCM7	0.305	0.000	0.000	54	LCM7	0.305	0.000	-0.003
35	LCM8	0.029	0.000	-0.002	54	LCM8	0.029	0.000	-0.001
35	LCM9	0.029	0.000	-0.002	54	LCM9	0.029	0.000	-0.002
35	LCM10	0.029	0.000	-0.002	54	LCM10	0.029	0.000	-0.002
35	LCM11	0.029	0.000	-0.002	54	LCM11	0.029	0.000	-0.002
35	LCM12	0.029	0.000	-0.002	54	LCM12	0.029	0.000	-0.001
35	LCM13	0.029	0.000	-0.002	54	LCM13	0.029	0.000	-0.002
35	LCM14	0.029	0.000	-0.003	54	LCM14	0.029	0.000	-0.002
35	LCM15	0.029	0.000	-0.002	54	LCM15	0.029	0.000	-0.002
35	LCM16	0.000	0.000	-0.002	54	LCM16	0.000	0.000	-0.001
35	LCM17	0.000	0.000	-0.002	54	LCM17	0.000	0.000	-0.001
35	LCM18	0.213	0.000	-0.002	54	LCM18	0.213	0.000	-0.003
35	LCM19	-0.213	0.000	-0.002	54	LCM19	-0.213	0.000	0.001
35	LCM20	0.000	0.000	-0.003	54	LCM20	0.000	0.000	-0.002
35	LCM21	0.000	0.000	-0.003	54	LCM21	0.000	0.000	-0.002
35	LCM22	0.000	0.000	-0.003	54	LCM22	0.000	0.000	-0.002
35	LCM23	0.000	0.000	-0.003	54	LCM23	0.000	0.000	-0.002
35	LCM24	0.000	0.000	-0.002	54	LCM24	0.000	0.000	-0.002
35	LCM25	0.000	0.000	-0.002	54	LCM25	0.000	0.000	-0.002
35	LCM26	0.160	0.000	-0.003	54	LCM26	0.160	0.000	-0.003
35	LCM27	-0.160	0.000	-0.003	54	LCM27	-0.160	0.000	0.000
35	LCM28	0.160	0.000	-0.003	54	LCM28	0.160	0.000	-0.003
35	LCM29	-0.160	0.000	-0.003	54	LCM29	-0.160	0.000	0.000
35	LCM30	0.160	0.000	-0.002	54	LCM30	0.160	0.000	-0.003
35	LCM31	-0.160	0.000	-0.003	54	LCM31	-0.160	0.000	0.000
35	LCM32	0.000	0.000	-0.001	54	LCM32	0.000	0.000	-0.001
35	LCM33	0.000	0.000	-0.001	54	LCM33	0.000	0.000	-0.001
35	LCM34	0.213	0.000	-0.001	54	LCM34	0.213	0.000	-0.002
35	LCM35	-0.213	0.000	-0.001	54	LCM35	-0.213	0.000	0.001
35	LCM36	0.029	0.000	-0.002	54	LCM36	0.029	0.000	-0.002
35	LCM37	0.029	0.000	-0.003	54	LCM37	0.029	0.000	-0.003
35	LCM38	0.029	0.000	-0.003	54	LCM38	0.029	0.000	-0.003
35	LCM39	0.029	0.000	-0.003	54	LCM39	0.029	0.000	-0.002
35	LCM40	0.029	0.000	-0.003	54	LCM40	0.029	0.000	-0.002
35	LCM41	0.000	0.000	-0.002	54	LCM41	0.000	0.000	-0.002
35	LCM42	0.000	0.000	-0.002	54	LCM42	0.000	0.000	-0.002
35	LCM43	0.029	0.000	-0.003	54	LCM43	0.029	0.000	-0.002
35	LCM44	0.000	0.000	-0.003	54	LCM44	0.000	0.000	-0.002
35	LCM45	0.000	0.000	-0.003	54	LCM45	0.000	0.000	-0.002
35	LCM46	0.029	0.000	-0.002	54	LCM46	0.029	0.000	-0.002
35	LCM47	0.000	0.000	-0.002	54	LCM47	0.000	0.000	-0.001
35	LCM48	0.000	0.000	-0.002	54	LCM48	0.000	0.000	-0.001
35	LCM49	0.000	0.000	-0.003	54	LCM49	0.000	0.000	-0.002
35	LCM50	0.000	0.000	-0.003	54	LCM50	0.000	0.000	-0.002
35	LCM51	0.000	0.000	-0.003	54	LCM51	0.000	0.000	-0.002
35	LCM52	0.000	0.000	-0.003	54	LCM52	0.000	0.000	-0.002
35	LCM53	0.000	0.000	-0.002	54	LCM53	0.000	0.000	-0.002
35	LCM54	0.000	0.000	-0.002	54	LCM54	0.000	0.000	-0.002
35	LCM55	0.305	0.000	-0.002	54	LCM55	0.305	0.000	-0.005
35	LCM56	-0.305	0.000	-0.003	54	LCM56	-0.305	0.000	0.001
35	LCM57	0.000	0.000	-0.001	54	LCM57	0.000	0.000	-0.001
35	LCM58	0.000	0.000	-0.001	54	LCM58	0.000	0.000	-0.001
35	LCM59	0.305	0.000	-0.001	54	LCM59	0.305	0.000	-0.004
35	LCM60	-0.305	0.000	-0.001	54	LCM60	-0.305	0.000	0.002
35	LCM61	0.914	0.000	-0.002	54	LCM61	0.914	0.000	-0.010
35	LCM62	-0.914	0.000	-0.004	54	LCM62	-0.914	0.000	0.006
35	LCM63	0.914	0.000	0.000	54	LCM63	0.914	0.000	-0.009
35	LCM64	-0.914	0.000	-0.002	54	LCM64	-0.914	0.000	0.007

Joint Displacement (in) (2nd Floor)											
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Seismic Horizontal	Wind Horizontal
16	LCM1	0.000	0.000	-0.002	22	LCM1	0.000	0.000	-0.003		
16	LCM2	0.000	0.000	-0.001	22	LCM2	0.000	0.000	-0.001		
16	LCM3	0.000	0.000	0.000	22	LCM3	0.000	0.000	-0.001		
16	LCM4	0.000	0.000	-0.001	22	LCM4	0.000	0.000	-0.001		
16	LCM5	0.000	0.000	0.000	22	LCM5	0.000	0.000	0.000		
16	LCM6	0.000	0.000	0.000	22	LCM6	0.000	0.000	0.000		
16	LCM7	0.933	0.000	0.004	22	LCM7	0.933	0.000	0.000		
16	LCM8	0.073	0.000	-0.002	22	LCM8	0.073	0.000	-0.003		
16	LCM9	0.073	0.000	-0.002	22	LCM9	0.073	0.000	-0.003		
16	LCM10	0.073	0.000	-0.002	22	LCM10	0.073	0.000	-0.003		
16	LCM11	0.073	0.000	-0.002	22	LCM11	0.073	0.000	-0.004		
16	LCM12	0.073	0.000	-0.002	22	LCM12	0.073	0.000	-0.003		
16	LCM13	0.073	0.000	-0.002	22	LCM13	0.073	0.000	-0.004		
16	LCM14	0.073	0.000	-0.002	22	LCM14	0.073	0.000	-0.004		
16	LCM15	0.073	0.000	-0.002	22	LCM15	0.073	0.000	-0.003		
16	LCM16	0.000	0.000	-0.002	22	LCM16	0.000	0.000	-0.003		
16	LCM17	0.000	0.000	-0.002	22	LCM17	0.000	0.000	-0.003		
16	LCM18	0.653	0.000	0.001	22	LCM18	0.653	0.000	-0.003		
16	LCM19	-0.653	0.000	-0.005	22	LCM19	-0.653	0.000	-0.003		
16	LCM20	0.000	0.000	-0.003	22	LCM20	0.000	0.000	-0.004		0.000
16	LCM21	0.000	0.000	-0.003	22	LCM21	0.000	0.000	-0.004		0.000
16	LCM22	0.000	0.000	-0.003	22	LCM22	0.000	0.000	-0.004		0.000
16	LCM23	0.000	0.000	-0.003	22	LCM23	0.000	0.000	-0.004		0.000
16	LCM24	0.000	0.000	-0.002	22	LCM24	0.000	0.000	-0.003		0.000
16	LCM25	0.000	0.000	-0.002	22	LCM25	0.000	0.000	-0.003		0.000
16	LCM26	0.490	0.000	-0.001	22	LCM26	0.490	0.000	-0.004		
16	LCM27	-0.490	0.000	-0.005	22	LCM27	-0.490	0.000	-0.004		
16	LCM28	0.490	0.000	-0.001	22	LCM28	0.490	0.000	-0.004		
16	LCM29	-0.490	0.000	-0.005	22	LCM29	-0.490	0.000	-0.004		
16	LCM30	0.490	0.000	0.000	22	LCM30	0.490	0.000	-0.004		
16	LCM31	-0.490	0.000	-0.005	22	LCM31	-0.490	0.000	-0.003		
16	LCM32	0.000	0.000	-0.001	22	LCM32	0.000	0.000	-0.002		
16	LCM33	0.000	0.000	-0.001	22	LCM33	0.000	0.000	-0.002		
16	LCM34	0.653	0.000	0.002	22	LCM34	0.653	0.000	-0.001		
16	LCM35	-0.653	0.000	-0.004	22	LCM35	-0.653	0.000	-0.001		
16	LCM36	0.073	0.000	-0.002	22	LCM36	0.073	0.000	-0.004		
16	LCM37	0.073	0.000	-0.003	22	LCM37	0.073	0.000	-0.005		
16	LCM38	0.073	0.000	-0.003	22	LCM38	0.073	0.000	-0.005		
16	LCM39	0.073	0.000	-0.003	22	LCM39	0.073	0.000	-0.004		
16	LCM40	0.073	0.000	-0.003	22	LCM40	0.073	0.000	-0.005		
16	LCM41	0.000	0.000	-0.003	22	LCM41	0.000	0.000	-0.004		
16	LCM42	0.000	0.000	-0.003	22	LCM42	0.000	0.000	-0.004		
16	LCM43	0.073	0.000	-0.003	22	LCM43	0.073	0.000	-0.005		
16	LCM44	0.000	0.000	-0.003	22	LCM44	0.000	0.000	-0.005		
16	LCM45	0.000	0.000	-0.003	22	LCM45	0.000	0.000	-0.005		
16	LCM46	0.073	0.000	-0.002	22	LCM46	0.073	0.000	-0.004		
16	LCM47	0.000	0.000	-0.002	22	LCM47	0.000	0.000	-0.003		
16	LCM48	0.000	0.000	-0.002	22	LCM48	0.000	0.000	-0.003		
16	LCM49	0.000	0.000	-0.003	22	LCM49	0.000	0.000	-0.004		
16	LCM50	0.000	0.000	-0.003	22	LCM50	0.000	0.000	-0.004		
16	LCM51	0.000	0.000	-0.003	22	LCM51	0.000	0.000	-0.004		
16	LCM52	0.000	0.000	-0.003	22	LCM52	0.000	0.000	-0.004		
16	LCM53	0.000	0.000	-0.002	22	LCM53	0.000	0.000	-0.003		
16	LCM54	0.000	0.000	-0.002	22	LCM54	0.000	0.000	-0.003		
16	LCM55	0.933	0.000	0.001	22	LCM55	0.933	0.000	-0.005	0.933	
16	LCM56	-0.933	0.000	-0.007	22	LCM56	-0.933	0.000	-0.004	-0.933	
16	LCM57	0.000	0.000	-0.002	22	LCM57	0.000	0.000	-0.002		
16	LCM58	0.000	0.000	-0.002	22	LCM58	0.000	0.000	-0.002		
16	LCM59	0.933	0.000	0.003	22	LCM59	0.933	0.000	-0.002	0.933	
16	LCM60	-0.933	0.000	-0.006	22	LCM60	-0.933	0.000	-0.001	-0.933	
16	LCM61	2.798	0.000	0.010	22	LCM61	2.798	0.000	-0.006		
16	LCM62	-2.798	0.000	-0.016	22	LCM62	-2.798	0.000	-0.003		
16	LCM63	2.798	0.000	0.011	22	LCM63	2.798	0.000	-0.003		
16	LCM64	-2.798	0.000	-0.014	22	LCM64	-2.798	0.000	0.000		

Joint Displacement (in) (2nd Floor)									
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical
41	LCM1	0.000	0.000	-0.003	60	LCM1	0.000	0.000	-0.002
41	LCM2	0.000	0.000	-0.001	60	LCM2	0.000	0.000	-0.001
41	LCM3	0.000	0.000	-0.001	60	LCM3	0.000	0.000	0.000
41	LCM4	0.000	0.000	-0.001	60	LCM4	0.000	0.000	-0.001
41	LCM5	0.000	0.000	0.000	60	LCM5	0.000	0.000	0.000
41	LCM6	0.000	0.000	0.000	60	LCM6	0.000	0.000	0.000
41	LCM7	0.933	0.000	0.000	60	LCM7	0.933	0.000	-0.004
41	LCM8	0.073	0.000	-0.003	60	LCM8	0.073	0.000	-0.002
41	LCM9	0.073	0.000	-0.003	60	LCM9	0.073	0.000	-0.003
41	LCM10	0.073	0.000	-0.003	60	LCM10	0.073	0.000	-0.003
41	LCM11	0.073	0.000	-0.004	60	LCM11	0.073	0.000	-0.003
41	LCM12	0.073	0.000	-0.003	60	LCM12	0.073	0.000	-0.002
41	LCM13	0.073	0.000	-0.004	60	LCM13	0.073	0.000	-0.003
41	LCM14	0.073	0.000	-0.004	60	LCM14	0.073	0.000	-0.003
41	LCM15	0.073	0.000	-0.003	60	LCM15	0.073	0.000	-0.003
41	LCM16	0.000	0.000	-0.003	60	LCM16	0.000	0.000	-0.002
41	LCM17	0.000	0.000	-0.003	60	LCM17	0.000	0.000	-0.002
41	LCM18	0.653	0.000	-0.003	60	LCM18	0.653	0.000	-0.005
41	LCM19	-0.653	0.000	-0.003	60	LCM19	-0.653	0.000	0.001
41	LCM20	0.000	0.000	-0.004	60	LCM20	0.000	0.000	-0.003
41	LCM21	0.000	0.000	-0.004	60	LCM21	0.000	0.000	-0.003
41	LCM22	0.000	0.000	-0.004	60	LCM22	0.000	0.000	-0.003
41	LCM23	0.000	0.000	-0.004	60	LCM23	0.000	0.000	-0.003
41	LCM24	0.000	0.000	-0.003	60	LCM24	0.000	0.000	-0.002
41	LCM25	0.000	0.000	-0.003	60	LCM25	0.000	0.000	-0.002
41	LCM26	0.490	0.000	-0.004	60	LCM26	0.490	0.000	-0.005
41	LCM27	-0.490	0.000	-0.004	60	LCM27	-0.490	0.000	-0.001
41	LCM28	0.490	0.000	-0.004	60	LCM28	0.490	0.000	-0.005
41	LCM29	-0.490	0.000	-0.004	60	LCM29	-0.490	0.000	-0.001
41	LCM30	0.490	0.000	-0.003	60	LCM30	0.490	0.000	-0.005
41	LCM31	-0.490	0.000	-0.004	60	LCM31	-0.490	0.000	0.000
41	LCM32	0.000	0.000	-0.002	60	LCM32	0.000	0.000	-0.001
41	LCM33	0.000	0.000	-0.002	60	LCM33	0.000	0.000	-0.001
41	LCM34	0.653	0.000	-0.001	60	LCM34	0.653	0.000	-0.004
41	LCM35	-0.653	0.000	-0.001	60	LCM35	-0.653	0.000	0.002
41	LCM36	0.073	0.000	-0.004	60	LCM36	0.073	0.000	-0.003
41	LCM37	0.073	0.000	-0.005	60	LCM37	0.073	0.000	-0.004
41	LCM38	0.073	0.000	-0.005	60	LCM38	0.073	0.000	-0.004
41	LCM39	0.073	0.000	-0.004	60	LCM39	0.073	0.000	-0.003
41	LCM40	0.073	0.000	-0.005	60	LCM40	0.073	0.000	-0.004
41	LCM41	0.000	0.000	-0.004	60	LCM41	0.000	0.000	-0.003
41	LCM42	0.000	0.000	-0.004	60	LCM42	0.000	0.000	-0.003
41	LCM43	0.073	0.000	-0.005	60	LCM43	0.073	0.000	-0.004
41	LCM44	0.000	0.000	-0.005	60	LCM44	0.000	0.000	-0.003
41	LCM45	0.000	0.000	-0.005	60	LCM45	0.000	0.000	-0.003
41	LCM46	0.073	0.000	-0.003	60	LCM46	0.073	0.000	-0.003
41	LCM47	0.000	0.000	-0.003	60	LCM47	0.000	0.000	-0.002
41	LCM48	0.000	0.000	-0.003	60	LCM48	0.000	0.000	-0.002
41	LCM49	0.000	0.000	-0.004	60	LCM49	0.000	0.000	-0.003
41	LCM50	0.000	0.000	-0.004	60	LCM50	0.000	0.000	-0.003
41	LCM51	0.000	0.000	-0.004	60	LCM51	0.000	0.000	-0.003
41	LCM52	0.000	0.000	-0.004	60	LCM52	0.000	0.000	-0.003
41	LCM53	0.000	0.000	-0.003	60	LCM53	0.000	0.000	-0.002
41	LCM54	0.000	0.000	-0.003	60	LCM54	0.000	0.000	-0.002
41	LCM55	0.933	0.000	-0.004	60	LCM55	0.933	0.000	-0.007
41	LCM56	-0.933	0.000	-0.005	60	LCM56	-0.933	0.000	0.001
41	LCM57	0.000	0.000	-0.002	60	LCM57	0.000	0.000	-0.002
41	LCM58	0.000	0.000	-0.002	60	LCM58	0.000	0.000	-0.002
41	LCM59	0.933	0.000	-0.001	60	LCM59	0.933	0.000	-0.006
41	LCM60	-0.933	0.000	-0.002	60	LCM60	-0.933	0.000	0.003
41	LCM61	2.798	0.000	-0.003	60	LCM61	2.798	0.000	-0.016
41	LCM62	-2.798	0.000	-0.006	60	LCM62	-2.798	0.000	0.010
41	LCM63	2.798	0.000	0.000	60	LCM63	2.798	0.000	-0.014
41	LCM64	-2.798	0.000	-0.003	60	LCM64	-2.798	0.000	0.011

STEP 1: Beam and Column Sizes

Note: Slightly Modified from Design Procedure, uses PR connection instead of FR connection

Beam1,2,3 Size:	W12X45		**Beam4,5,6 Size:**	W12X45	
d=	12.1	in	d=	12.1	in
bbf=	8.05	in	bbf=	8.05	in
tbf=	0.575	in	tbf=	0.575	in
tbw=	0.335	in	tbw=	0.335	in
Fy_bm=	50	ksi	Fy_bm=	50	ksi
Fu_bm=	65	ksi	Fu_bm=	65	ksi

Column 1 Size:	W10X68		**Column 2 Size:**	W10X88	
dc=	10.4	in	dc=	10.8	in
bcf=	10.1	in	bcf=	10.3	in
tcf=	0.77	in	tcf=	0.99	in
tcw=	0.47	in	tcw=	0.605	in
Agc=	19.9	in^2	Agc=	26	in^2
Fy_col=	50	ksi	Fy_col=	50	ksi
Fu_col=	65	ksi	Fu_col=	65	ksi

Column 3 Size:	W10X88		**Column 4 Size:**	W10X68	
dc=	10.8	in	dc=	10.4	in
bcf=	10.3	in	bcf=	10.1	in
tcf=	0.99	in	tcf=	0.77	in
tcw=	0.605	in	tcw=	0.47	in
Agc=	26	in^2	Agc=	19.9	in^2
Fy_col=	50	ksi	Fy_col=	50	ksi
Fu_col=	65	ksi	Fu_col=	65	ksi

Link1 Size:	MF4-2.25
Link2 Size:	MF4-2.25
Link3 Size:	MF4-2.25
Link4 Size:	MF4-2.25
Link5 Size:	MF4-2.25
Link6 Size:	MF4-2.25

Seismic Drift Check:

Hcc1_aver.=	178.75	in	Hcc2=	228.75	in
R_frame=	6.5		R_frame=	6.5	
Cd=	4		Cd=	4	
Allowable Drift=	0.02	Hx	Allowable Drift=	0.02	Hx
I=	1		I=	1	
Δ_{allow}=	0.894	in	Δ_{allow}=	1.144	in
Δs_PR=	**0.305**	in	Δs_PR=	**0.628**	in
DCR_SeismicDrift=	0.341	OK	DCR_SeismicDrift=	0.549	OK

Wind Drift Check:

Allowable Drift= Hx/	50		Allowable Drift= Hx/	50	
Allowable Drift=	3.575	in	Allowable Drift=	4.575	in
Δ_PR1=	0.000	in	Δ_PR2=	0.000	in
DCR_WindDrift=	0.000	OK	DCR_WindDrift=	0.000	OK

SAP2000 Axial Spring Parameters:

Axial Force vs. link displacement relationship

Link 1 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

Link 2 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

Link 3 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

Link4 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

Link5 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

Link6 Points	Force kips	Displ. in
1	-87.750	-0.441
2	-87.750	-0.252
3	-61.875	-0.019
4	0.000	0.000
5	61.875	0.019
6	87.750	0.252
7	87.750	0.441

STEP 2: Check Beam with Simply Supported Connections in SAP2000

Lcc1=	116.975	in	*Column center line to center line*
a=	2.75	in	*Distance from column face to center hole of beam*
Lh1=	100.875	in	=Lcc1 - dc1/2 - dc2/2 - 2*a
Lcc2=	122.925	in	*Column center line to center line*
a=	2.75	in	*Distance from column face to center hole of beam*
Lh2=	106.625	in	=Lcc2 - dc2/2 - dc3/2 - 2*a
Lcc3=	116.975	in	*Column center line to center line*
a=	2.75	in	*Distance from column face to center hole of beam*
Lh3=	100.875	in	=Lcc2 - dc2/2 - dc3/2 - 2*a

Beam 1 (1st bay)

Delta_ss1=	0.008	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.280	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.029	OK	*=Delta_ss/ Delta_allow*

Beam 1 (2nd bay)

Delta_ss1=	0.010	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.296	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.034	OK	*=Delta_ss/ Delta_allow*

Beam 1 (3rd bay)

Delta_ss1=	0.008	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.280	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.029	OK	*=Delta_ss/ Delta_allow*

Beam 2 (1st bay)

Delta_ss2=	0.005	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.280	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.019	OK	*=Delta_ss/ Delta_allow*

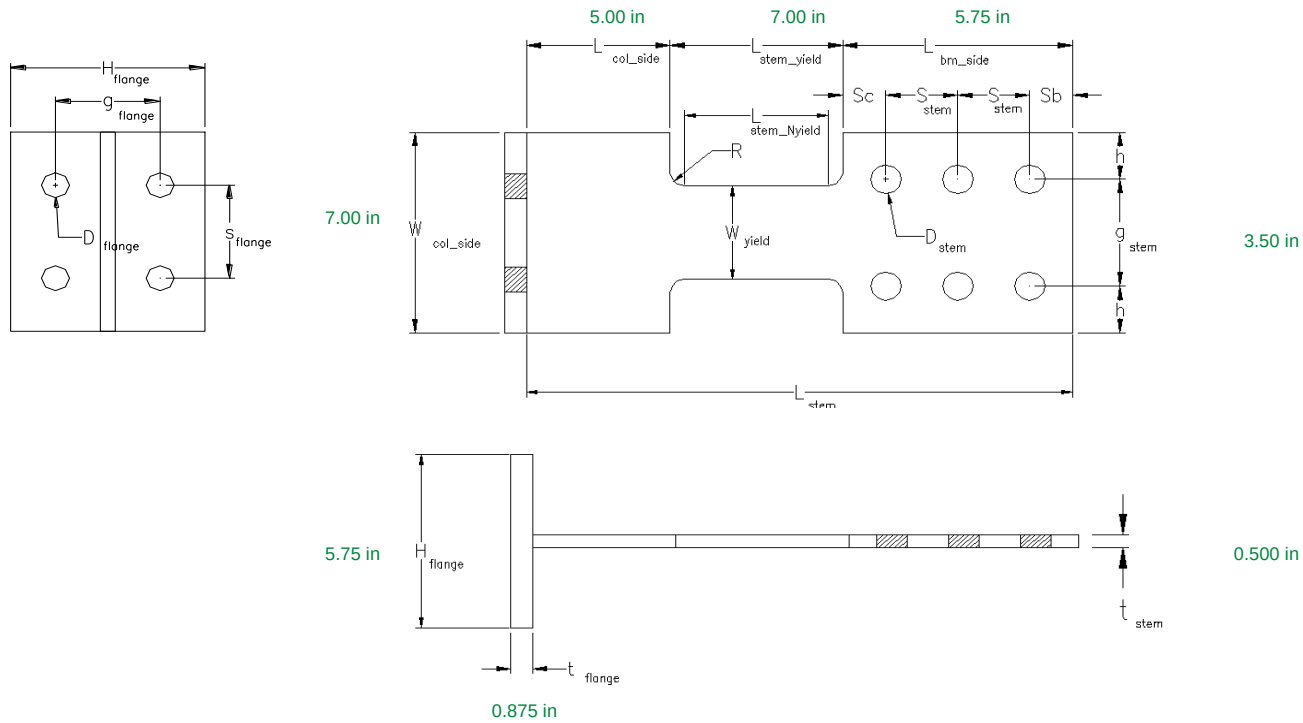
Beam 2 (2nd bay)

Delta_ss2=	0.000	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.296	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.000	OK	*=Delta_ss/ Delta_allow*

Beam 3 (3rd bay)

Delta_ss2=	0.005	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.280	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.019	OK	*=Delta_ss/ Delta_allow*

SMF Link Design/Check (For Beam 1)



LINK1 ID MF4-2.25

Beam1= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2 =t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.313	in			

LINK MATERIAL:

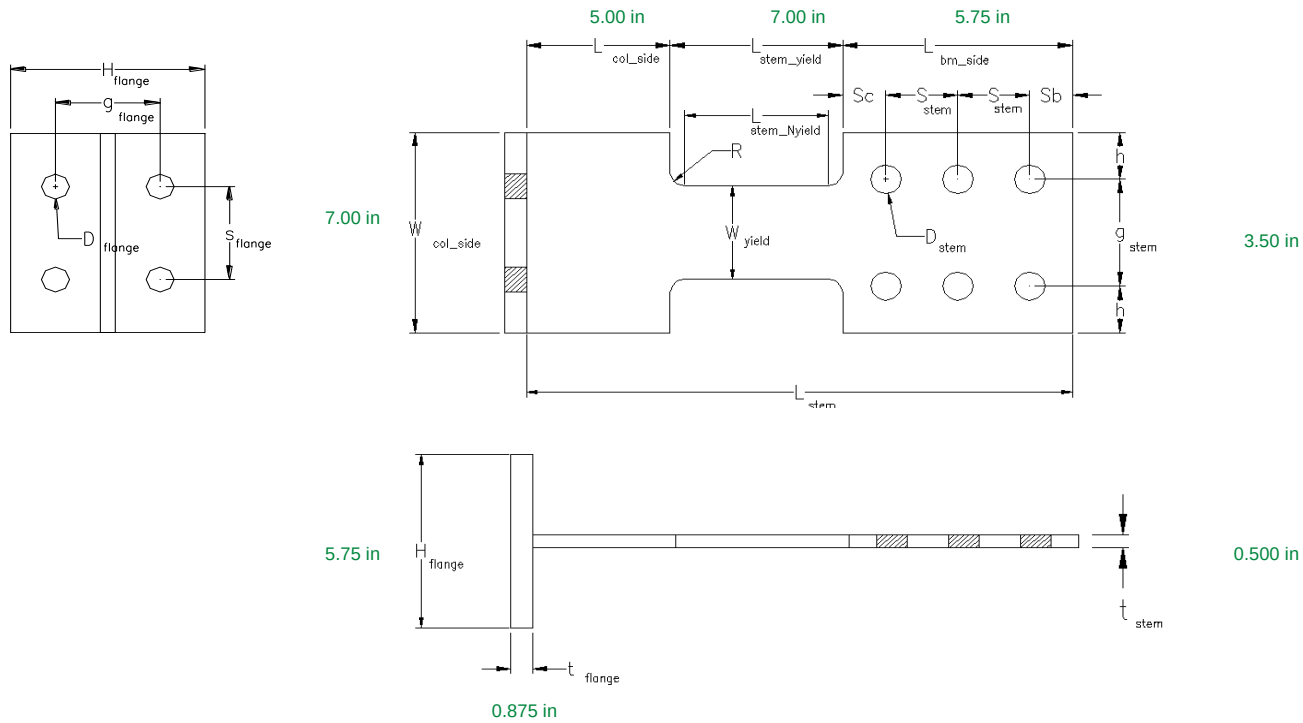
Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 2)



LINK2 ID MF4-2.25

Beam2= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2 =t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.313	in			

LINK MATERIAL:

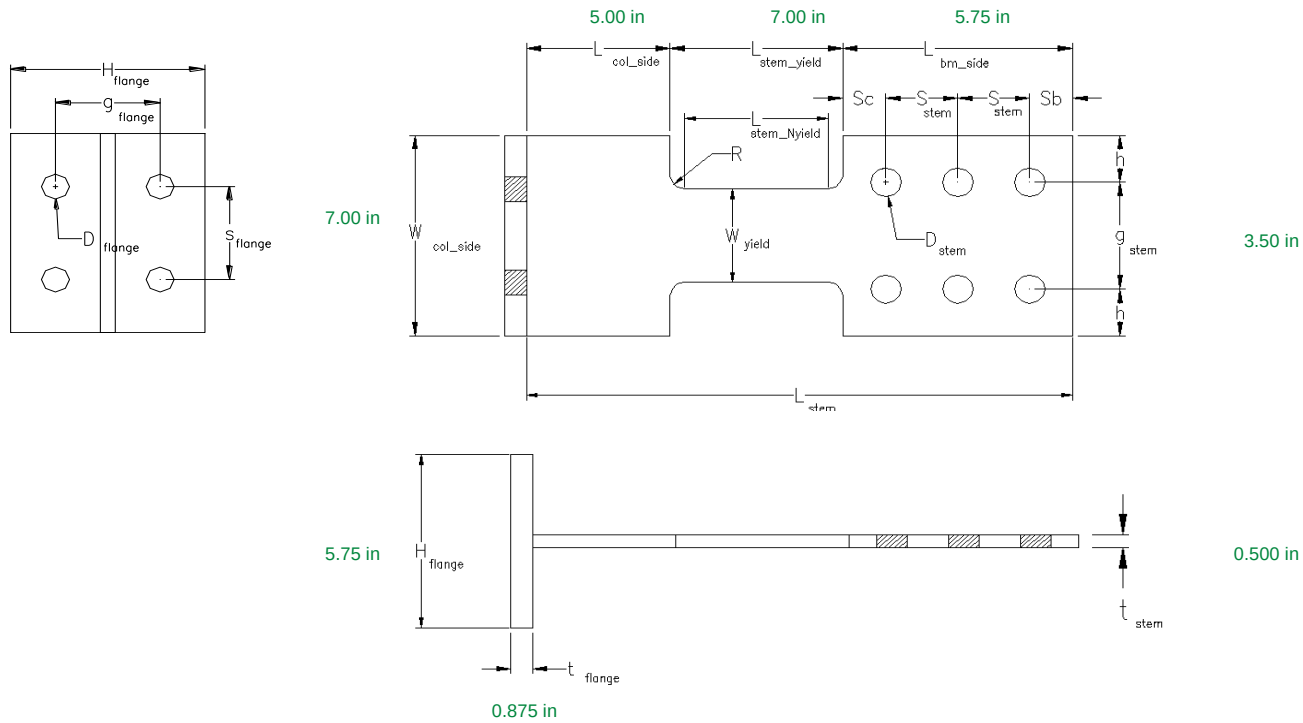
Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 3)



LINK3 ID MF4-2.25

Beam3= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in	
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in	
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in	
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in	
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2	=t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along WIdth (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.313	in			

LINK MATERIAL:

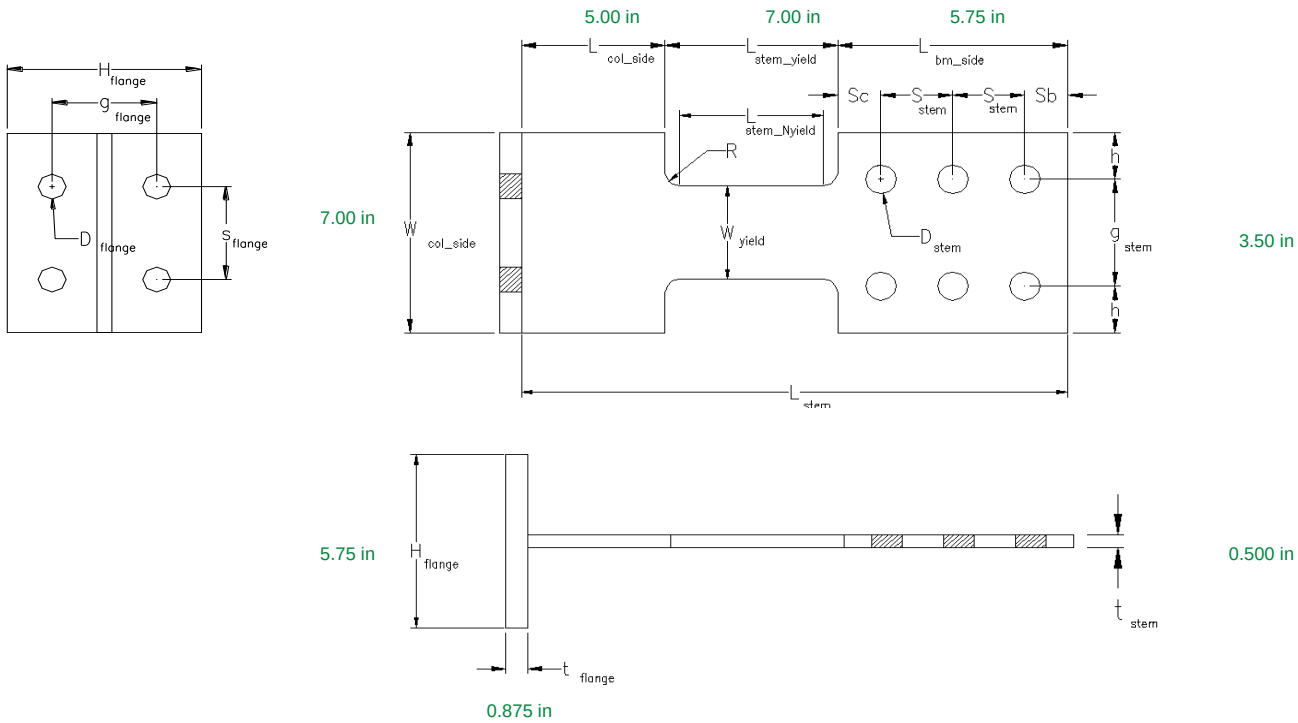
Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 4)



LINK4 ID MF4-2.25

Beam4= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in	
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in	
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in	
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in	
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2	=t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.3125	in			

LINK MATERIAL:

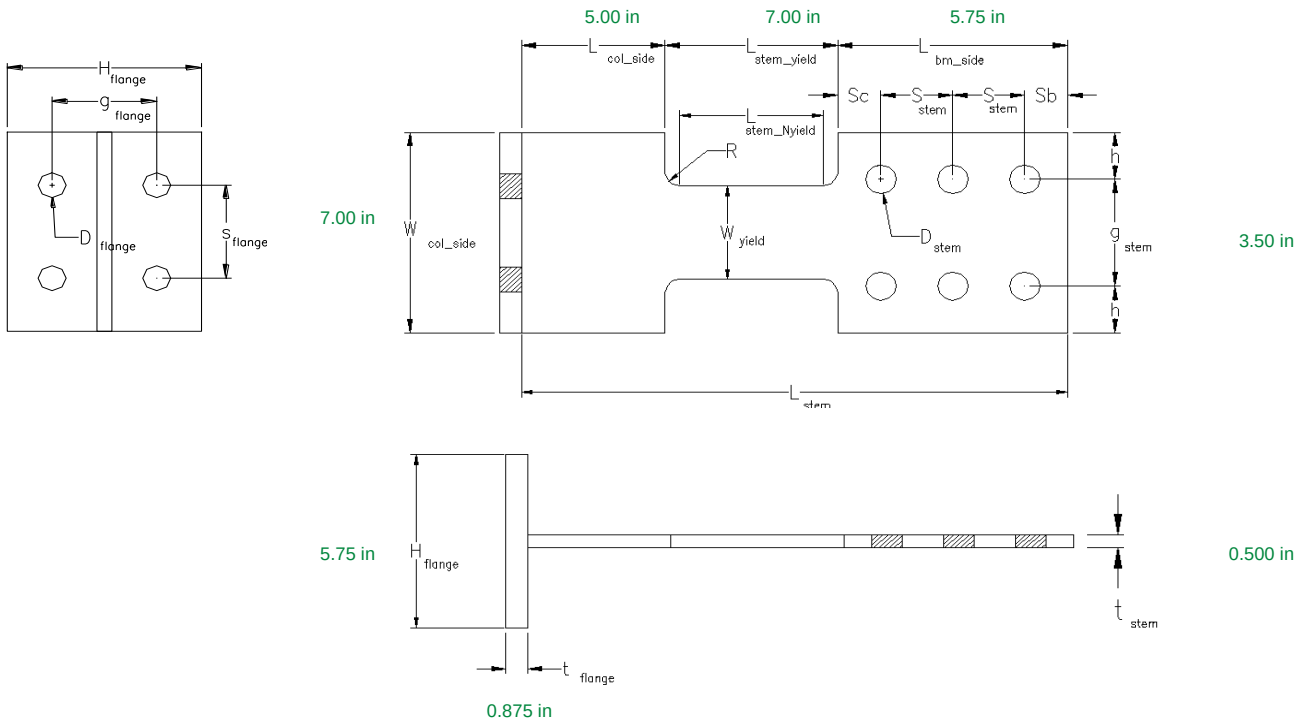
Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 5)



LINK5 ID MF4-2.25

Beam5= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2 =t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.3125	in			

LINK MATERIAL:

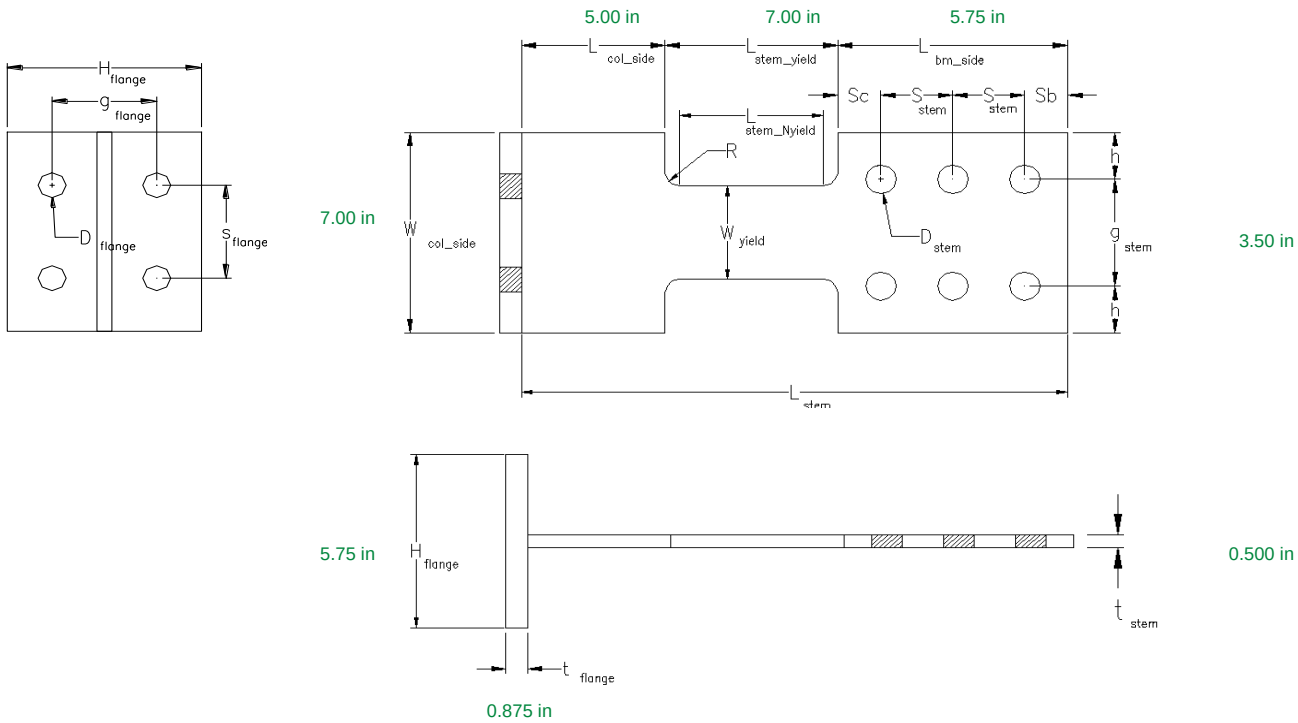
Fy_link =	50	ksi	Fillet Weld Size, (t_weld_stemFlange) =	**0.750**	in
Fu_link =	65	ksi	FEXX =	70	ksi
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1	

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 6)



LINK6 ID MF4-2.25

Beam6= W12X45 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	7.00	in
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	2.250	in
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	7.00	in
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.125	in^2 =t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	7.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	4.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.3125	in			

LINK MATERIAL:

Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

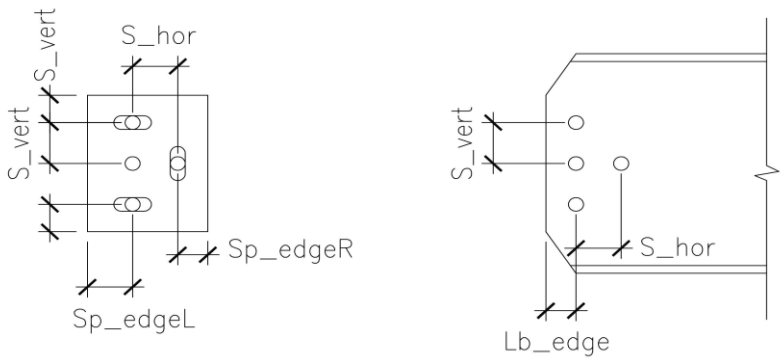
A_link=	1.125	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	61.875	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.238	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	0.866	kips	*=0.7*Bracing Force (LRFD) at Top of column*

STEP 15: DESIGN BEAM WEB TO COLUMN FLANGE CONNECTION (BEAM 1, 2, 3)

15.1a: SHEAR PLATE BOLT SIZE

Beam Section (bmSize) =	W12X45		*Value previously defined*
Beam Depth (db)=	12.10	in	*Value previously defined*
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Fy_bm=	50	ksi	
Fu_bm =	65	ksi	
Axial Load (Pu_SST)=	3.21	kips	*Max axial force per LCM61-64 from SAP_BeamReactions, SAP output*
Vertical Load (Vu_bm)=	23.95	kips	*Value previously defined from Step 12*
Vu_bolt=	8.1	kips	*= Sqrt ((Pu_SST/ n_Hbolt_SST)^2 + (Vu_bm/ n_Vbolt_SST)^2)*
ϕbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	A325-X		*[A325-X default]*
Bolt Dia (db_sp)=	7/8	in	
No of vertical Bolts (n_Vbolts_SST)=	3		
No of horizontal Bolts (n_Hbolts_SST)=	2		
Fnv_A325N=	68	ksi	*= if (Bolt Type = "A490-X", 84, 68)*
Anb_sp=	0.60	in^2	*= Pi* (db_sp/2)^2*
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*
ϕRn_stBolt_Shear=	30.7	kips	*= Rn_stBolt_shear* ϕbolt*
DCR_shearTabl_Cbolt=	0.266	OK	*= Vu_bolt/ ϕRn_stBolt_shear*
			OK if DCR_shearTabl_Cbolt <= 1.05

15.2: SHEAR PLATE GEOMETRY



a=	2.75	in	*Value previously defined*
tsp=	0.375	in	*[0.375 default]*
db =	12.1	in	*Value previously defined*
h_flange =	5.75	in	*Value previously defined*
t_stem =	0.5	in	*Value previously defined*
h_clear =	0.25	in	*minimum clearance between link flange and shear tab*
Bolt Spacing (S_min)=	2.33	in	*= 2.6667* max (db_sp)*
Bolt Spacing (Svert)=	2.19	in	OK *looked up value*
Bolt Spacing (Shorz)=	2.19	in	OK *looked up value*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lv_sp=	1	in	Need to Check J3.10 and J4, see Section 15.5 below
h_sp max =	6.35	in	
Plate Depth (h_sp)=	6.375	in	OK *= Lv_sp* 2 + Svert* 2*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lh_sp=	1.75	in	OK
Plate Width (W_shearTab) =	6.69	in	*= Lh_sp + a + Shorz*
Lslot_min=	1.30625	in	*= db_sp + 1/8 + 0.14* Svert*
LslotH = LslotV =	1.50	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	**OK**		*OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)*

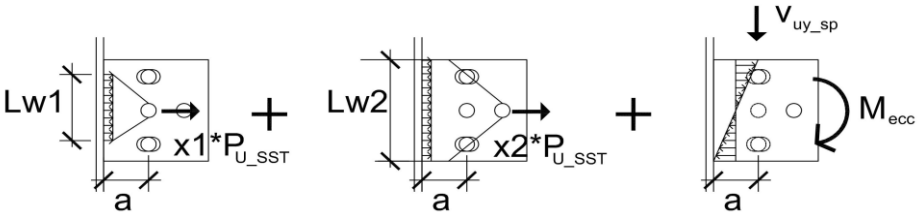
15.3a: SHEAR PLATE YIELDING (VERTICAL)

ϕyield=	0.9		
dHole_sp=	1.00	in	*= db_sp + 1/8*
Asp_Agv=	2.391	in^2	*= tsp* h_sp*
ϕVy_sp=	64.55	kips	*= ϕyield* 0.6* Asp_Agv* Fy_sp*
DCR_spYield=	0.371	OK	*= Vu_bm/ ϕVy_sp*
			OK if DCR_spYield <= DCR_allowed

15.3b: SHEAR PLATE RUPTURE (VERTICAL)

ϕrupture=	0.75		
Asp_nv=	1.27	in^2	*= hsp* tsp - dHole_sp* n_Vbolts_SST* tsp*
ϕVrupture_sp=	37.02	kips	*= ϕrupture* 0.6* Fu_shearTab* Asp_nv*
DCR_spRupture=	0.647	OK	*= Vu_bm/ ϕVrupture_sp*

15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT



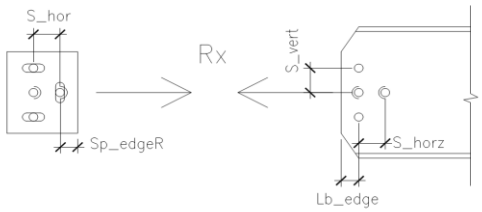
ϕb=	0.90		
a=	2.75	in	*Value previously defined*
Vuy=	23.95	kips	*= Vu_bm*
Mecc=	65.86	kips*in	*= Vuy* a*
x1 =	0.50		*assume 1/2 of axial is resisted by each bolt*
x2 =	0.50		*=1-x1*
θWhitmore=	30	deg	
Lwhitmore1 =	4.05	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore1 =	1.52	in^2	*= Lwhitmore1* tsp*
Lwhitmore2 =	6.38	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore2 =	2.39	in^2	*= Lwhitmore2* tsp*
Ssp=	2.54	in^3	*= tsp* hsp^2/ 6*
Isp=	8.10	in^4	*= tsp* hsp^3/ 12*
fb1=	25.93	ksi	*= Mecc/ Ssp*
yb=	2.03	in	*= Lwhitmore1/ 2*
fb2=	16.47	ksi	*= Mecc* yb/ Isp*
fa2=	1.73	ksi	*= x1*Pu_SST/ Awhitmore1 + x2*Pu_SST/ Awhitmore2*
ftot2=	18.20	ksi	*= fa2 + fb2*
fmax_sp=	25.93	ksi	*= max (fb1, ftot2)*
ϕb*Fy_sp=	45.00	ksi	*= ϕb* Fy_shearTab*
DCR_sp=	0.576	OK	*= fmax_sp/ (ϕb* Fy_shearTab), OK if DCR_sp <= DCR_Allowed*

15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD

tsp=	0.375	in	*value previously defined*
tw_sp_min=	0.2344	in	*= 5/8*tsp Fillet size to develop plate, Per AISC Steel Manual 14th edition*
tw_sp=	**0.2500**	in	*Each Side [0.25 default] , page 10-102*
DCR_spWeld=	0.938	OK	*= tw_sp_min/ tw_sp, OK if DCR_spWeld <= DCR_Allowed*

15.5: BEAM WEB AND SHEAR TAB BEARING

CASE 1: HORIZONTAL REACTIONS



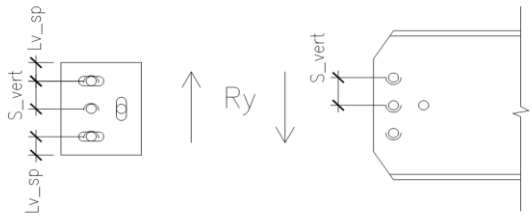
BEAM WEB:

ϕbolt=	0.75		
tbw=	0.335	in	*value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lc_bmWeb1 =	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
Lc_bmWeb2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_bmWeb=	2.531	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
ϕRn_beamWeb1=	49.61	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb*
ϕRn_beamWeb2=	68.59	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb *n_Hbolt*
ϕRn_beamWeb=	49.61	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.065	OK	*= Pu_SST/ ϕRn_beamWeb, OK if DCR_bmWebX <= DCR_Allowed*

SHEAR PLATE:

tsp=	0.375	in	*value previously defined*
Sp_edgeR =	1.75	in	*Edge distance for shear tab bolt hole*
Lc_sp1 =	1.281	in	*= Sp_edgeR - (db_sp + 1/16)* 0.5*
Lc_sp2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_sp=	2.531	in	*= Lc_sp1 + Lc_sp2*
ϕRn_sp1=	55.53	kips	*= ϕbolt* 1.2* Lc_sp* tbw* Fu_bmWeb*
ϕRn_sp2=	76.78	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕRn_sp=	55.53	kips	*= min (ϕRn_sp1, ϕRn_sp2)*
DCR_spX=	0.058	OK	*= Pu_SST/ ϕRn_sp, OK if DCR_spX <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



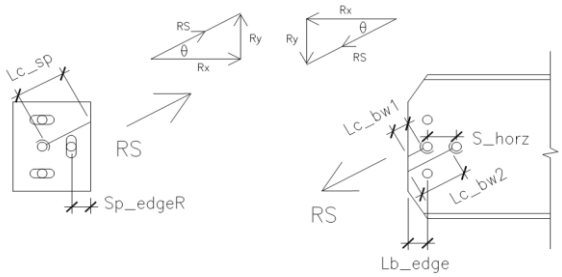
BEAM WEB:

Lc1=	**1.250**	in	*= Svert - (db_sp + 1/16)*
Lc2=	1.250	in	*= Lc1*
Lc3=	3.288	in	*= (db - 2Svert - 2tbf)*0.5*
ϕ*Rn_bmWebY1=	113.42	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc3)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY2=	102.89	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY=	102.89	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	0.233	OK	*= Vu_bm/ (ϕ* Rn_bmWebY), OK if DCR_bmWebY <= DCR_Allowed*

SHEAR PLATE:

Lc4=	0.531	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
ϕ*Rn_spY1=	66.50	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc4)* tsp* Fu_shearTab*
ϕ*Rn_spY2=	115.17	kips	*= ϕbolt* 2.4* db_sp* n_bolts_SST* tsp* Fu_shearTab*
ϕ*Rn_spY=	66.50	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	0.360	OK	*= Vu_bm/ (ϕ* Rn_spY), OK if DCR_spY <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	1.61	kips	*= Pu_SST/ n_Hbolts_SST*
Py=	7.98	kips	*= Vu_bm/ n_Vbolts_SST*
Pr=	8.14	kips	*= Sqrt (Px^2 + Py^2)*
θ=	1.372	radians	*= min(Atan (Py/ Px),1.571))*

BEAM WEB:

Method 1: T and V circular interaction

DCR_bmWebθBearing1 =	0.058		*= DCR_bmWebX^2 + DCR_bmWebY^2*

Method 2: Bearing in a diagonal line

Lvg_web1 =	8.879	in	*= Lb_edge/ (cos θ)*
Lvg_web2 =	19.977	in	*= (Lb_edge + Shorz)/ (cos θ)*
Lc_web1 =	8.379	in	*= Lvg_web1 - dHole_sp/2*
Lc_web2 =	19.477	in	*= Lvg_web2 - dHole_sp/2*
La_web=	12.100	in	*= min(d, Lc_web1, Lc_web2)*
ϕ*Rn_bmWebθ1=	237.130	kips	*= ϕbolt* 1.2* La_web* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ2=	68.591	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕ*Rn_bmWebθ=	68.59	kips	*= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)*
DCR_bmWebθBearing2 =	0.119		*= Pr/ ϕ*Rn_bmWeb θ*
			Ok if DCR_bmWebq <= DCR_Allowed
DCR_bmWebθBearing =	0.119	OK	

SHEAR PLATE:

Method 1: T and V circular interaction

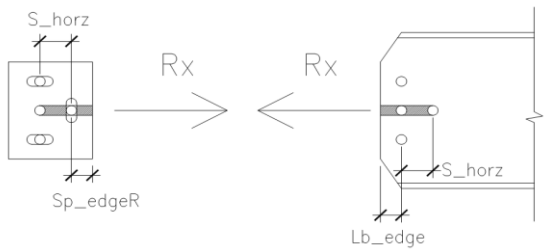
DCR_spθBearing1 =	0.133		= $DCR\_spX^2 + DCR\_spY^2$

Method 2: Bearing in a diagonal line

Lvg_sp=	8.879	in	= Sp_edgeR/ cos(θ)
La_sp=	3.188	in	= min(h_sp/2, Lvg_sp - dHole_sp/2)
ϕ*Rn_spθ1=	69.926	kips	= ϕbolt* 1.2* La_sp* tsp* Fu_shearTab
ϕ*Rn_spθ2=	76.781	kips	= ϕbolt* 2.4* db_sp* tsp* Fu_shearTab*n_boltH
ϕ*Rn_spθ=	69.93	kips	= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)
DCR_spθbearing2=	0.116		= Pr/ (ϕ* Rn_sp θ)
DCR_spθbearing=	0.133	OK	= max(DCR_bmWeb θ, DCR_sp θbearing)
			OK if DCR_sp θbearing <= DCR_Allowed

15.5b: BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK

CASE 1: HORIZONTAL REACTIONS



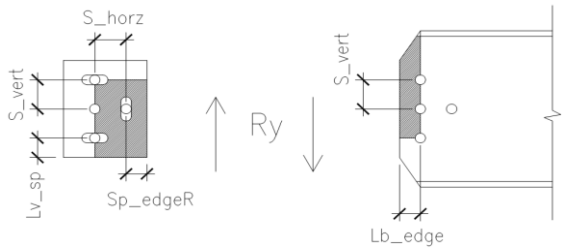
BEAM WEB:

ϕblockshear=	0.75		
Ubs=	1		
Lb_edge=	1.750	in	value previously defined
Lc_bmWeb =	2.531	in	value previously defined
Lh_bmWeb=	7.875	in	= 2* (Lb_edge+Shorz)
Agv_bmWebHorz=	2.973	in^2	= Lh_bmWeb*tbw + Lb_edge* tbw
Ant_bmWebHorz=	0.000	in^2	= 0
Anv_bmWebHorz=	2.973	in^2	= Agv_bmWebHorz
Rn_bmWebHorz1=	115.95	kips	= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz2=	89.19	kips	= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz=	66.90	kips	= ϕblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)
DCR_bmWebHorz_BS=	0.048	OK	= Pu_SST / Rn_bmWebHorz, OK if DCR_bmWebHorz <= DCR_Allowed

SHEAR PLATE:

Lsp_edge=	1.750	in	value previously defined
Lc_sp =	2.531	in	value previously defined
Lh_sp=	7.875	in	= 2* (Lsp_edge + Shorz)
Agv_spHorz=	3.328	in^2	= Lh_sp* tsp + dHole_sp*tsp
Ant_spHorz=	0.000	in^2	= 0
Anv_spHorz=	3.328	in^2	= Agv_bmWebHorz
Rn_spHorz1=	129.80	kips	= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz
Rn_spbHorz2=	99.84	kips	= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz
ϕRn_spHorz=	74.88	kips	= ϕblocksher* min (Rn_spHorz1, Rn_spbHorz2)
DCR_spHorz_BS=	0.043	OK	= Pu_SST/ Rn_spHorz, OK if DCR_spHorz <= DCR_Allowed

CASE 2: VERTICAL REACTIONS



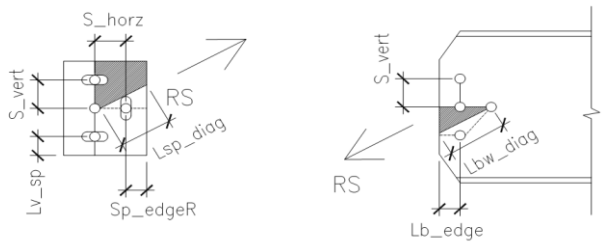
BEAM WEB:

Sbm_web=	2.19	in	= Svert
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	7.663	in	= (d- 2Svert - 2tbf)*0.5 + 2Svert
Agv_bmWeb1=	2.567	in^2	= tbw* h_bmWeb
Anv_bmWeb1=	1.729	in^2	= tbw* (h_bmWeb - 2.5* dHole_sp)
Ant_bmWeb1=	0.419	in^2	= (Lb_edge - dHole_sp/2)* tbw
Rn_bmWebVert1=	94.667	kips	= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
Rn_bmWebVert2=	104.227	kips	= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
ϕRn_bmWebVert=	71.00	kips	= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)
DCR_bmWebVert_BS=	0.337	OK	= Vu_bm/ ϕRn_bmWebVert, OK if DCR_bmWebVert_BS <= DCR_Allowed

SHEAR PLATE:

Ssp=	2.19	in	= sbm_web
Lv_sp=	1.000	in	value previously defined
hsp=	6.375	in	value previously defined
Lslot=	1.500	in	value previously defined
Agv_sp1=	2.391	in^2	= hsp* tsp
Anv_sp1=	1.453	in^2	= tsp* (hsp - 2.5* dHole_sp)
Ant_sp1=	1.195	in^2	= (Lsp_edge - LslotH/2+Shorz)* tsp
Rn_spVert1=	134.367	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	149.414	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ϕRn_spVert=	100.78	kips	= ϕblocksher* min (Rn_spVert1, Rn_spVert2)
DCR_spVert_BS=	0.238	OK	= Vu_bm/ ϕRn_spVert, OK if DCR_spVert_BS <= DCR_Allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: N and V circular interaction

DCR_bmWebθ_BS1 =	0.116		= DCR_bmWebHorz_BS^2 + DCR_bmWebVert_BS^2

Method 2: Failure along hole edges

θ=	1.372	radians	value previously defined
Lb_edge=	1.750	in	value previously defined
Lbw_diag=	4.84	in	min(sqrt(Svert^2+Shorz^2) + Lb_edge, (Lb_edge + Shorz)/cos θ)
Lvg_web=	8.78	in	= Lbw_diag + Lb_edge + Shorz
Lvn_web=	5.78	in	= IF(Lbw_diag = (Lb_edge + Shorz)/cosq, Lvg_web - 2dHole_sp, Lvg_web - 3*dHole_sp)
Ltg_web=	0.00	in	
Ltn_web=	0.00	in	
Agv_web=	2.94	in^2	= Lvg_web* tbw
Anv_web=	1.94	in^2	= Lvn_web* tbw
Agt_web=	0.00	in^2	= Ltg_web* tbw
Ant_web=	0.00	in^2	= Ltn_web* tbw
Rn_bmWeb_BS1=	75.53	kips	= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
Rn_bmWeb_BS2=	88.25	kips	= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
ϕRn_bmWeb_BS=	56.65	kips	= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)
DCR_bmWebθ_BS2=	0.287		= 2*Pr/ fRn_bmWeb_BS
DCR_bmWebθ_BS=	0.287	OK	= max(DCR_bmWeb θ_BS1, DCR_bmWeb θ_BS2) <= DCR_Allowed

SHEAR PLATE:

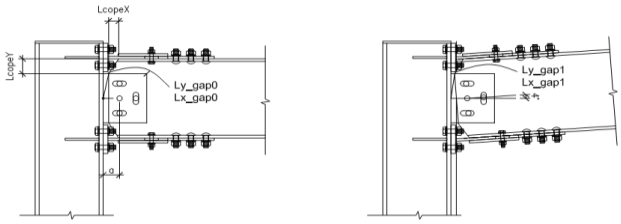
Method 1: N and V circular interaction

DCR_spθ_BS1 =	0.058		= DCR_spVert_BS^2 + DCR_spHorz_BS^2

Method 2: Failure along hole edges

Lsp_diag =	3.94	in	= min (Shorz + Sp_edgeR, (Shorz + Sp_edgeR)/cos θ)
Lsp_edge=	1.750	in	value previously defined
Lvg_sp=	7.13	in	= Lsp_diag + Svert + Lv_sp
Lvn_sp=	4.13	in	= IF(Lsp_diag = Shorz + Sp_edgeR, Lvg_sp - 3dHole_sp, Lvg_sp - 1.5dHole_sp)
Ltg_sp=	0.00	in	
Ltn_sp=	0.00	in	
Agv_sp=	2.672	in	= Lvg_sp* tsp
Anv_sp=	1.547	in	= Lvn_sp* tsp
Agt_sp=	0.000	in	= Ltg_sp* tsp
Ant_sp=	0.000	in	= Ltn_sp* tsp
Rn_sp_BS1=	60.33	kips	= 0.6* Fu_shearTab* Anv_sp + Ubs* Fu_shearTab* Ant_sp
Rn_sp_BS2=	80.16	kips	= 0.6* Fy_shearTab* Agv_sp + Ubs* Fu_shearTab* Ant_sp
ϕRn_sp_BS=	45.25	kips	= ϕshearblock* min (Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.180		= Pr/ ϕRn_sp_BS
DCR_spq_BS=	0.180	OK	= max(DCR_sp θ_BS1, DCR_sp θ_BS2)<= DCR_Allowed

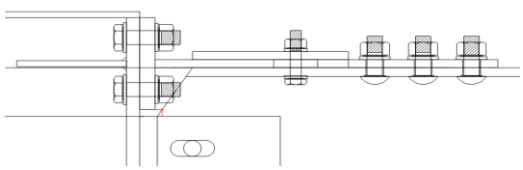
15.6: DETAIL BEAM FLANGE AND WEB COPE DISTANCE



UNDEFORMED DEFORMED TO 7 RADIANS

θrotation=	0.070	radians	
θrotation=	4.011	degrees	
Ly_gap_min =	3/16	in	
Lx_gap_min =	1/8	in	
Lb_edge=	1.75	in	*value previously defined*
t_flange=	0.875	in	*Link flange thickness*
h_flange=	5.750	in	*value previously defined*
d=	12.10	in	*value previously defined*
t_stem=	0.50		*value previously defined*
a=	2.75	in	*value previously defined*
hsp=	6.38	in	*value previously defined*
Ly_gap0 =	0.24	in	OK *= (d + t_steam - h_flange - hsp)/2*
			OK if Ly_gap0 => Ly_gap_min
Lx_gap0 =	0.125	in	OK *= (a + Lb_edge - t_flange)/2*
			OK if Lx_gap0 => Lx_gap_min
LcopeX=	1.75	in	*= Lb_edge*
			Align end of beam flange cut with beam web bolt centerline
LcopeY=	2.8750	in	*= (d - hsp)/2*
			Try to align vertical web cope with top of Shear Plate
LslotV_ck =	1.31	in	OK *= Shorz* θrotation*2 + dHole_sp*
			OK if LslotV => LslotV_ck

Xgap between Beam web (diag cut) and edge of end plate



β =	58.67	degrees	*= atan(LcopeY/ LcopeX)*
Ly_gap0 =	0.24	in	*value previously defined*
Lx_gapβ =	0.27	in	

Check if Lx gap clear >= 0 after 4 degrees of roation

$$RM = \begin{pmatrix} \cos(\theta_{rotation}) & -\sin(\theta_{rotation}) \\ \sin(\theta_{rotation}) & \cos(\theta_{rotation}) \end{pmatrix}$$

$$RM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Rotational matrix*

$$SM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Stretch matrix*

$$X = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *= RM.SM Deformation Gradiant*

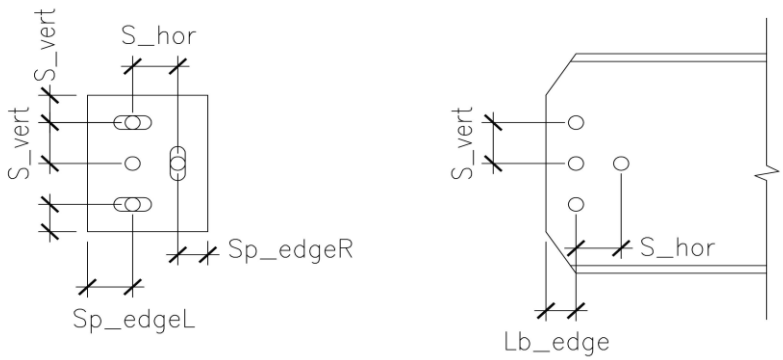
X_0 =	-1.480	in	*= a - Lx_gap β - 1*	with respect to center bolt
Y_0 =	3.425	in	*= hsp/2 + Ly_gap0*	with respect to center bolt
X_1 =	-1.717	in		with respect to center bolt
Y_1 =	3.315	in		with respect to center bolt
Lx_gap1 =	-0.237	in	*=X_1-X_0*	
Ly_gap1 =	-0.110	in	*=Y_1-Y_0*	
Lx_gap_clear =	0.086	in	OK *OK if Lx_gap_clear > 0*	
			*= Lx_gap β - abs(Lx_gap1) + tan(90° - β - θ)*abs(Ly_gap1)*	

STEP 15: DESIGN BEAM WEB TO COLUMN FLANGE CONNECTION (BEAM 4, 5, 6)

15.1a: SHEAR PLATE BOLT SIZE

Beam Section (bmSize) =	W12X45		*Value previously defined*
Beam Depth (db)=	12.10	in	*Value previously defined*
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Fy_bm=	50	ksi	
Fu_bm =	65	ksi	
Axial Load (Pu_SST)=	5.30	kips	*Max axial force per LCM61-64 from SAP_BeamReactions, SAP output*
Vertical Load (Vu_bm)=	23.18	kips	*Value previously defined from Step 12*
Vu_bolt=	8.2	kips	*= Sqrt ((Pu_SST/ n_Hbolt_SST)^2 + (Vu_bm/ n_Vbolt_SST)^2)*
ϕbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	A325-X		*[A325-X default]*
Bolt Dia (db_sp)=	7/8	in	
No of vertical Bolts (n_Vbolts_SST)=	3		
No of horizontal Bolts (n_Hbolts_SST)=	2		
Fnv_A325N=	68	ksi	*= if (Bolt Type = "A490-X", 84, 68)*
Anb_sp=	0.60	in^2	*= Pi* (db_sp/2)^2*
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*
ϕRn_stBolt_Shear=	30.7	kips	*= Rn_stBolt_shear* ϕbolt*
DCR_shearTabl_Cbolt=	0.266	OK	*= Vu_bolt/ ϕRn_stBolt_shear*
			OK if DCR_shearTabl_Cbolt <= 1.05

15.2: SHEAR PLATE GEOMETRY



a=	2.75	in	*Value previously defined*
tsp=	0.375	in	*[0.375 default]*
db =	12.1	in	*Value previously defined*
h_flange =	5.75	in	*Value previously defined*
t_stem =	0.50	in	*Value previously defined*
h_clear =	0.25	in	*minimum clearance between link flange and shear tab*
Bolt Spacing (S_min)=	2.33	in	*= 2.6667* max (db_sp)*
Bolt Spacing (Svert)=	2.19	in	OK *looked up value*
Bolt Spacing (Shorz)=	2.19	in	OK *looked up value*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lv_sp=	1	in	Need to Check J3.10 and J4, see Section 15.5 below
h_sp max =	6.35	in	
Plate Depth (h_sp)=	6.375	in	OK *= Lv_sp* 2 + Svert* 2*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lh_sp=	1.75	in	OK
Plate Width (W_shearTab) =	6.69	in	*= Lh_sp + a + Shorz*
Lslot_min=	1.30625	in	*= db_sp + 1/8 + 0.14* Svert*
LslotH = LslotV =	1.50	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	**OK**		*OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)*

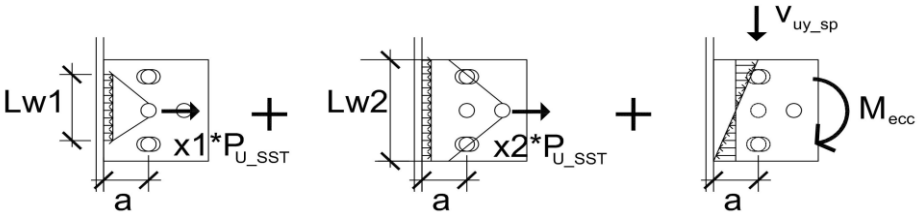
15.3a: SHEAR PLATE YIELDING (VERTICAL)

ϕyield=	0.9		
dHole_sp=	1.00	in	*= db_sp + 1/8*
Asp_Agv=	2.391	in^2	*= tsp* h_sp*
ϕVy_sp=	64.55	kips	*= ϕyield* 0.6* Asp_Agv* Fy_sp*
DCR_spYield=	0.359	OK	*= Vu_bm/ ϕVy_sp*
			OK if DCR_spYield <= DCR_allowed

15.3b: SHEAR PLATE RUPTURE (VERTICAL)

ϕrupture=	0.75		
Asp_nv=	1.27	in^2	*= hsp* tsp - dHole_sp* n_Vbolts_SST* tsp*
ϕVrupture_sp=	37.02	kips	*= ϕrupture* 0.6* Fu_shearTab* Asp_nv*
DCR_spRupture=	0.626	OK	*= Vu_bm/ ϕVrupture_sp*

15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT



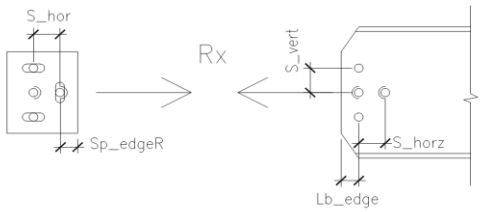
ϕb=	0.90		
a=	2.75	in	*Value previously defined*
Vuy=	23.18	kips	*= Vu_bm*
Mecc=	63.73	kips*in	*= Vuy* a*
x1 =	0.50		*assume 1/2 of axial is resisted by each bolt*
x2 =	0.50		*=1-x1*
θWhitmore=	30	deg	
Lwhitmore1 =	4.05	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore1 =	1.52	in^2	*= Lwhitmore1* tsp*
Lwhitmore2 =	6.38	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore2 =	2.39	in^2	*= Lwhitmore2* tsp*
Ssp=	2.54	in^3	*= tsp* hsp^2/ 6*
Isp=	8.10	in^4	*= tsp* hsp^3/ 12*
fb1=	25.09	ksi	*= Mecc/ Ssp*
yb=	2.03	in	*= Lwhitmore1/ 2*
fb2=	15.94	ksi	*= Mecc* yb/ Isp*
fa2=	2.85	ksi	*= x1*Pu_SST/ Awhitmore1 + x2*Pu_SST/ Awhitmore2*
ftot2=	18.79	ksi	*= fa2 + fb2*
fmax_sp=	25.09	ksi	*= max (fb1, ftot2)*
ϕb*Fy_sp=	45.00	ksi	*= ϕb* Fy_shearTab*
DCR_sp=	0.558	OK	*= fmax_sp/ (ϕb* Fy_shearTab), OK if DCR_sp <= DCR_Allowed*

15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD

tsp=	0.375	in	*value previously defined*
tw_sp_min=	0.2344	in	*= 5/8*tsp Fillet size to develop plate, Per AISC Steel Manual 14th edition*
tw_sp=	**0.2500**	in	*Each Side [0.25 default] , page 10-102*
DCR_spWeld=	0.938	OK	*= tw_sp_min/ tw_sp, OK if DCR_spWeld <= DCR_Allowed*

15.5: BEAM WEB AND SHEAR TAB BEARING

CASE 1: HORIZONTAL REACTIONS



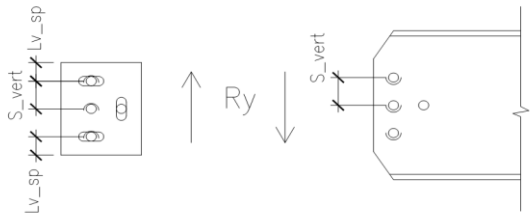
BEAM WEB:

ϕbolt=	0.75		
tbw=	0.335	in	*value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lc_bmWeb1 =	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
Lc_bmWeb2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_bmWeb=	2.531	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
ϕRn_beamWeb1=	49.61	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb*
ϕRn_beamWeb2=	68.59	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb *n_Hbolt*
ϕRn_beamWeb=	49.61	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.107	OK	*= Pu_SST/ ϕRn_beamWeb, OK if DCR_bmWebX <= DCR_Allowed*

SHEAR PLATE:

tsp=	0.375	in	*value previously defined*
Sp_edgeR =	1.75	in	*Edge distance for shear tab bolt hole*
Lc_sp1 =	1.281	in	*= Sp_edgeR - (db_sp + 1/16)* 0.5*
Lc_sp2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_sp=	2.531	in	*= Lc_sp1 + Lc_sp2*
ϕRn_sp1=	55.53	kips	*= ϕbolt* 1.2* Lc_sp* tbw* Fu_bmWeb*
ϕRn_sp2=	76.78	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕRn_sp=	55.53	kips	*= min (ϕRn_sp1, ϕRn_sp2)*
DCR_spX=	0.095	OK	*= Pu_SST/ ϕRn_sp, OK if DCR_spX <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



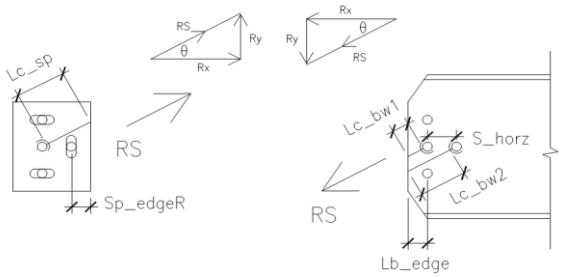
BEAM WEB:

Lc1=	**1.250**	in	*= Svert - (db_sp + 1/16)*
Lc2=	1.250	in	*= Lc1*
Lc3=	3.288	in	*= (db - 2Svert - 2tbf)*0.5*
ϕ*Rn_bmWebY1=	113.42	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc3)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY2=	102.89	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY=	102.89	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	0.225	OK	*= Vu_bm/ (ϕ* Rn_bmWebY), OK if DCR_bmWebY <= DCR_Allowed*

SHEAR PLATE:

Lc4=	0.531	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
ϕ*Rn_spY1=	66.50	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc4)* tsp* Fu_shearTab*
ϕ*Rn_spY2=	115.17	kips	*= ϕbolt* 2.4* db_sp* n_bolts_SST* tsp* Fu_shearTab*
ϕ*Rn_spY=	66.50	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	0.349	OK	*= Vu_bm/ (ϕ* Rn_spY), OK if DCR_spY <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	2.65	kips	*= Pu_SST/ n_Hbolts_SST*
Py=	7.73	kips	*= Vu_bm/ n_Vbolts_SST*
Pr=	8.17	kips	*= Sqrt (Px^2 + Py^2)*
θ=	1.241	radians	*= min(Atan (Py/ Px),1.571))*

BEAM WEB:

Method 1: T and V circular interaction

DCR_bmWebθBearing1 =	0.062		*= DCR_bmWebX^2 + DCR_bmWebY^2*

Method 2: Bearing in a diagonal line

Lvg_web1 =	5.397	in	*= Lb_edge/ (cos θ)*
Lvg_web2 =	12.143	in	*= (Lb_edge + Shorz)/ (cos θ)*
Lc_web1 =	4.897	in	*= Lvg_web1 - dHole_sp/2*
Lc_web2 =	11.643	in	*= Lvg_web2 - dHole_sp/2*
La_web=	12.100	in	*= min(d, Lc_web1, Lc_web2)*
ϕ*Rn_bmWebθ1=	237.130	kips	*= ϕbolt* 1.2* La_web* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ2=	68.591	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕ*Rn_bmWebθ=	68.59	kips	*= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)*
DCR_bmWebθBearing2 =	0.119		*= Pr/ ϕ*Rn_bmWeb θ*
			Ok if DCR_bmWebq <= DCR_Allowed
DCR_bmWebθBearing =	0.119	OK	

SHEAR PLATE:

Method 1: T and V circular interaction

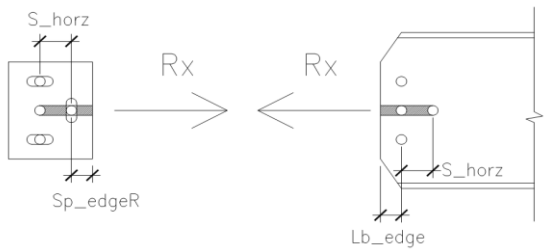
DCR_spθBearing1 =	0.131		= $DCR\_spX^2 + DCR\_spY^2$

Method 2: Bearing in a diagonal line

Lvg_sp=	5.397	in	= Sp_edgeR/ cos(θ)
La_sp=	3.188	in	= min(h_sp/2, Lvg_sp - dHole_sp/2)
ϕ*Rn_spθ1=	69.926	kips	= ϕbolt* 1.2* La_sp* tsp* Fu_shearTab
ϕ*Rn_spθ2=	76.781	kips	= ϕbolt* 2.4* db_sp* tsp* Fu_shearTab*n_boltH
ϕ*Rn_spθ=	69.93	kips	= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)
DCR_spθbearing2=	0.117		= Pr/ (ϕ* Rn_sp θ)
DCR_spθbearing=	0.131	OK	= max(DCR_bmWeb θ, DCR_sp θbearing)
			OK if DCR_sp θbearing <= DCR_Allowed

15.5b: BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK

CASE 1: HORIZONTAL REACTIONS



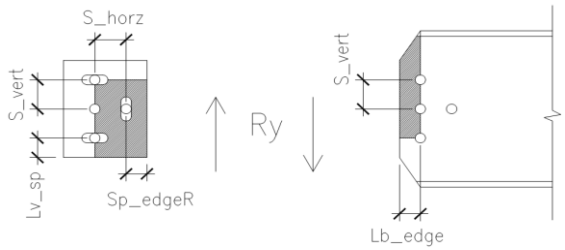
BEAM WEB:

ϕblockshear=	0.75		
Ubs=	1		
Lb_edge=	1.750	in	value previously defined
Lc_bmWeb =	2.531	in	value previously defined
Lh_bmWeb=	7.875	in	= 2* (Lb_edge+Shorz)
Agv_bmWebHorz=	2.973	in^2	= Lh_bmWeb*tbw + Lb_edge* tbw
Ant_bmWebHorz=	0.000	in^2	= 0
Anv_bmWebHorz=	2.973	in^2	= Agv_bmWebHorz
Rn_bmWebHorz1=	115.95	kips	= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz2=	89.19	kips	= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz=	66.90	kips	= ϕblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)
DCR_bmWebHorz_BS=	0.079	OK	= Pu_SST / Rn_bmWebHorz, OK if DCR_bmWebHorz <= DCR_Allowed

SHEAR PLATE:

Lsp_edge=	1.750	in	value previously defined
Lc_sp =	2.531	in	value previously defined
Lh_sp=	7.875	in	= 2* (Lsp_edge + Shorz)
Agv_spHorz=	3.328	in^2	= Lh_sp* tsp + dHole_sp*tsp
Ant_spHorz=	0.000	in^2	= 0
Anv_spHorz=	3.328	in^2	= Agv_bmWebHorz
Rn_spHorz1=	129.80	kips	= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz
Rn_spbHorz2=	99.84	kips	= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz
ϕRn_spHorz=	74.88	kips	= ϕblocksher* min (Rn_spHorz1, Rn_spbHorz2)
DCR_spHorz_BS=	0.071	OK	= Pu_SST/ Rn_spHorz, OK if DCR_spHorz <= DCR_Allowed

CASE 2: VERTICAL REACTIONS



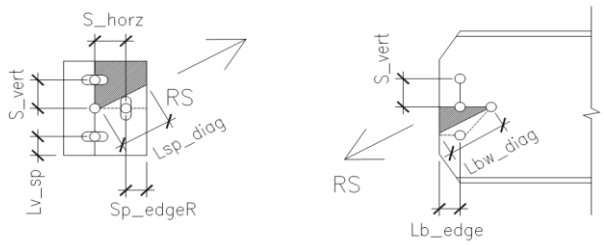
BEAM WEB:

Sbm_web=	2.19	in	= Svert
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	7.663	in	= (d- 2Svert - 2tbf)*0.5 + 2Svert
Agv_bmWeb1=	2.567	in^2	= tbw* h_bmWeb
Anv_bmWeb1=	1.729	in^2	= tbw* (h_bmWeb - 2.5* dHole_sp)
Ant_bmWeb1=	0.419	in^2	= (Lb_edge - dHole_sp/2)* tbw
Rn_bmWebVert1=	94.667	kips	= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
Rn_bmWebVert2=	104.227	kips	= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
ϕRn_bmWebVert=	71.00	kips	= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)
DCR_bmWebVert_BS=	0.326	OK	= Vu_bm/ ϕRn_bmWebVert, OK if DCR_bmWebVert_BS <= DCR_Allowed

SHEAR PLATE:

Ssp=	2.19	in	= sbm_web
Lv_sp=	1.000	in	value previously defined
hsp=	6.375	in	value previously defined
Lslot=	1.500	in	value previously defined
Agv_sp1=	2.391	in^2	= hsp* tsp
Anv_sp1=	1.453	in^2	= tsp* (hsp - 2.5* dHole_sp)
Ant_sp1=	1.195	in^2	= (Lsp_edge - LslotH/2+Shorz)* tsp
Rn_spVert1=	134.367	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	149.414	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ϕRn_spVert=	100.78	kips	= ϕblocksher* min (Rn_spVert1, Rn_spVert2)
DCR_spVert_BS=	0.230	OK	= Vu_bm/ ϕRn_spVert, OK if DCR_spVert_BS <= DCR_Allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: N and V circular interaction

DCR_bmWebθ_BS1 =	0.113		= DCR_bmWebHorz_BS^2 + DCR_bmWebVert_BS^2

Method 2: Failure along hole edges

θ=	1.241	radians	value previously defined
Lb_edge=	1.750	in	value previously defined
Lbw_diag=	4.84	in	min(sqrt(Svert^2+Shorz^2) + Lb_edge, (Lb_edge + Shorz)/cos θ)
Lvg_web=	8.78	in	= Lbw_diag + Lb_edge + Shorz
Lvn_web=	5.78	in	= IF(Lbw_diag = (Lb_edge + Shorz)/cosq, Lvg_web - 2dHole_sp, Lvg_web - 3*dHole_sp)
Ltg_web=	0.00	in	
Ltn_web=	0.00	in	
Agv_web=	2.94	in^2	= Lvg_web* tbw
Anv_web=	1.94	in^2	= Lvn_web* tbw
Agt_web=	0.00	in^2	= Ltg_web* tbw
Ant_web=	0.00	in^2	= Ltn_web* tbw
Rn_bmWeb_BS1=	75.53	kips	= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
Rn_bmWeb_BS2=	88.25	kips	= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
ϕRn_bmWeb_BS=	56.65	kips	= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)
DCR_bmWebθ_BS2=	0.288		= 2*Pr/ fRn_bmWeb_BS
DCR_bmWebθ_BS=	0.288	OK	= max(DCR_bmWeb θ_BS1, DCR_bmWeb θ_BS2) <= DCR_Allowed

SHEAR PLATE:

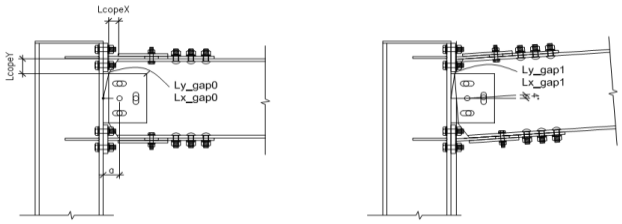
Method 1: N and V circular interaction

DCR_spθ_BS1 =	0.058		= DCR_spVert_BS^2 + DCR_spHorz_BS^2

Method 2: Failure along hole edges

Lsp_diag =	3.94	in	= min (Shorz + Sp_edgeR, (Shorz + Sp_edgeR)/cos θ)
Lsp_edge=	1.750	in	value previously defined
Lvg_sp=	7.13	in	= Lsp_diag + Svert + Lv_sp
Lvn_sp=	4.13	in	= IF(Lsp_diag = Shorz + Sp_edgeR, Lvg_sp - 3dHole_sp, Lvg_sp - 1.5dHole_sp)
Ltg_sp=	0.00	in	
Ltn_sp=	0.00	in	
Agv_sp=	2.672	in	= Lvg_sp* tsp
Anv_sp=	1.547	in	= Lvn_sp* tsp
Agt_sp=	0.000	in	= Ltg_sp* tsp
Ant_sp=	0.000	in	= Ltn_sp* tsp
Rn_sp_BS1=	60.33	kips	= 0.6* Fu_shearTab* Anv_sp + Ubs* Fu_shearTab* Ant_sp
Rn_sp_BS2=	80.16	kips	= 0.6* Fy_shearTab* Agv_sp + Ubs* Fu_shearTab* Ant_sp
ϕRn_sp_BS=	45.25	kips	= ϕshearblock* min (Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.180		= Pr/ ϕRn_sp_BS
DCR_spq_BS=	0.180	OK	= max(DCR_sp θ_BS1, DCR_sp θ_BS2)<= DCR_Allowed

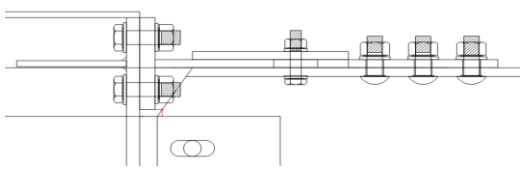
15.6: DETAIL BEAM FLANGE AND WEB COPE DISTANCE



UNDEFORMED DEFORMED TO 7 RADIANS

θrotation=	0.070	radians	
θrotation=	4.011	degrees	
Ly_gap_min =	3/16	in	
Lx_gap_min =	1/8	in	
Lb_edge=	1.75	in	*value previously defined*
t_flange=	0.875	in	*Link flange thickness*
h_flange=	5.750	in	*value previously defined*
d=	12.10	in	*value previously defined*
t_stem=	0.50		*value previously defined*
a=	2.75	in	*value previously defined*
hsp=	6.38	in	*value previously defined*
Ly_gap0 =	0.24	in	OK *= (d + t_steam - h_flange - hsp)/2*
			OK if Ly_gap0 => Ly_gap_min
Lx_gap0 =	0.125	in	OK *= (a + Lb_edge - t_flange)/2*
			OK if Lx_gap0 => Lx_gap_min
LcopeX=	1.75	in	*= Lb_edge*
			Align end of beam flange cut with beam web bolt centerline
LcopeY=	2.8750	in	*= (d - hsp)/2*
			Try to align vertical web cope with top of Shear Plate
LslotV_ck =	1.31	in	OK *= Shorz* θrotation*2 + dHole_sp*
			OK if LslotV => LslotV_ck

Xgap between Beam web (diag cut) and edge of end plate



β =	58.67	degrees	*= atan(LcopeY/ LcopeX)*
Ly_gap0 =	0.24	in	*value previously defined*
Lx_gapβ =	0.27	in	

Check if Lx gap clear >= 0 after 4 degrees of roation

$$RM = \begin{pmatrix} \cos(\theta_{rotation}) & -\sin(\theta_{rotation}) \\ \sin(\theta_{rotation}) & \cos(\theta_{rotation}) \end{pmatrix}$$

$$RM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Rotational matrix*

$$SM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Stretch matrix*

$$X = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *= RM.SM Deformation Gradiant*

X_0 =	-1.480	in	*= a - Lx_gap β - 1*	with respect to center bolt
Y_0 =	3.425	in	*= hsp/2 + Ly_gap0*	with respect to center bolt
X_1 =	-1.717	in		with respect to center bolt
Y_1 =	3.315	in		with respect to center bolt
Lx_gap1 =	-0.237	in	*=X_1-X_0*	
Ly_gap1 =	-0.110	in	*=Y_1-Y_0*	
Lx_gap_clear =	0.086	in	OK *OK if Lx_gap_clear > 0*	
			*= Lx_gap β - abs(Lx_gap1) + tan(90° - β - θ)*abs(Ly_gap1)*	

Base Reaction (kips, in)												
Load Combos	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)
1	1	LCM1	0.05	0.00	4.47	2.70	8	LCM1	0.00	0.00	7.73	0.21
2	1	LCM2	0.10	0.00	1.78	5.55	8	LCM2	0.00	0.00	3.79	0.13
3	1	LCM3	-0.01	0.00	0.71	-0.61	8	LCM3	0.00	0.00	1.57	0.09
4	1	LCM4	-0.01	0.00	0.89	-0.76	8	LCM4	0.00	0.00	1.96	0.12
5	1	LCM5	0.00	0.00	0.00	0.00	8	LCM5	0.00	0.00	0.00	0.00
6	1	LCM6	0.00	0.00	0.00	0.00	8	LCM6	0.00	0.00	0.00	0.00
7	1	LCM7	-3.13	0.00	-9.23	-414.63	8	LCM7	-4.79	0.00	1.21	-592.60
8	1	LCM8	-0.30	0.00	3.77	-38.88	8	LCM8	-0.51	0.00	7.82	-58.86
9	1	LCM9	-0.19	0.00	5.55	-33.33	8	LCM9	-0.51	0.00	11.61	-58.72
10	1	LCM10	-0.31	0.00	4.48	-39.48	8	LCM10	-0.51	0.00	9.39	-58.76
11	1	LCM11	-0.31	0.00	4.66	-39.64	8	LCM11	-0.51	0.00	9.78	-58.74
12	1	LCM12	-0.30	0.00	3.77	-38.88	8	LCM12	-0.51	0.00	7.82	-58.86
13	1	LCM13	-0.23	0.00	5.64	-35.17	8	LCM13	-0.51	0.00	11.84	-58.69
14	1	LCM14	-0.23	0.00	5.77	-35.29	8	LCM14	-0.51	0.00	12.13	-58.67
15	1	LCM15	-0.22	0.00	5.10	-34.72	8	LCM15	-0.51	0.00	10.66	-58.76
16	1	LCM16	0.05	0.00	4.47	2.70	8	LCM16	0.00	0.00	7.73	0.21
17	1	LCM17	0.05	0.00	4.47	2.70	8	LCM17	0.00	0.00	7.73	0.21
18	1	LCM18	-2.14	0.00	-1.38	-287.17	8	LCM18	-3.35	0.00	9.63	-414.58
19	1	LCM19	2.25	0.00	11.55	293.31	8	LCM19	3.36	0.00	7.94	415.06
20	1	LCM20	0.12	0.00	6.34	6.41	8	LCM20	0.01	0.00	11.75	0.38
21	1	LCM21	0.12	0.00	6.34	6.41	8	LCM21	0.01	0.00	11.75	0.38
22	1	LCM22	0.12	0.00	6.47	6.30	8	LCM22	0.01	0.00	12.04	0.40
23	1	LCM23	0.12	0.00	6.47	6.30	8	LCM23	0.01	0.00	12.04	0.40
24	1	LCM24	0.13	0.00	5.81	6.86	8	LCM24	0.01	0.00	10.57	0.31
25	1	LCM25	0.13	0.00	5.81	6.86	8	LCM25	0.01	0.00	10.57	0.31
26	1	LCM26	-1.52	0.00	1.95	-210.99	8	LCM26	-2.51	0.00	13.18	-310.71
27	1	LCM27	1.77	0.00	11.64	224.37	8	LCM27	2.52	0.00	11.90	311.52
28	1	LCM28	-1.53	0.00	2.08	-211.11	8	LCM28	-2.51	0.00	13.47	-310.69
29	1	LCM29	1.77	0.00	11.78	224.25	8	LCM29	2.52	0.00	12.20	311.54
30	1	LCM30	-1.51	0.00	1.42	-210.54	8	LCM30	-2.51	0.00	12.00	-310.78
31	1	LCM31	1.78	0.00	11.11	224.82	8	LCM31	2.52	0.00	10.73	311.45
32	1	LCM32	0.03	0.00	2.68	1.62	8	LCM32	0.00	0.00	4.64	0.13
33	1	LCM33	0.03	0.00	2.68	1.62	8	LCM33	0.00	0.00	4.64	0.13
34	1	LCM34	-2.17	0.00	-4.39	-288.99	8	LCM34	-3.35	0.00	4.43	-414.72
35	1	LCM35	2.22	0.00	8.53	291.49	8	LCM35	3.36	0.00	2.73	414.92
36	1	LCM36	-0.28	0.00	5.56	-37.80	8	LCM36	-0.51	0.00	10.91	-58.77
37	1	LCM37	-0.13	0.00	7.86	-29.76	8	LCM37	-0.51	0.00	16.22	-58.55
38	1	LCM38	-0.13	0.00	7.95	-29.84	8	LCM38	-0.51	0.00	16.41	-58.54
39	1	LCM39	-0.12	0.00	7.51	-29.46	8	LCM39	-0.51	0.00	15.43	-58.60
40	1	LCM40	-0.25	0.00	6.69	-36.53	8	LCM40	-0.51	0.00	13.77	-58.60
41	1	LCM41	0.04	0.00	6.50	2.27	8	LCM41	0.01	0.00	11.78	0.41
42	1	LCM42	0.04	0.00	6.50	2.27	8	LCM42	0.01	0.00	11.78	0.41
43	1	LCM43	-0.26	0.00	6.98	-36.78	8	LCM43	-0.51	0.00	14.39	-58.56
44	1	LCM44	0.04	0.00	6.79	2.03	8	LCM44	0.01	0.00	12.41	0.44
45	1	LCM45	0.04	0.00	6.79	2.03	8	LCM45	0.01	0.00	12.41	0.44
46	1	LCM46	-0.24	0.00	5.55	-35.56	8	LCM46	-0.51	0.00	11.26	-58.75
47	1	LCM47	0.06	0.00	5.37	3.24	8	LCM47	0.00	0.00	9.27	0.26
48	1	LCM48	0.06	0.00	5.37	3.24	8	LCM48	0.00	0.00	9.27	0.26
49	1	LCM49	0.10	0.00	6.61	5.71	8	LCM49	0.01	0.00	11.95	0.37
50	1	LCM50	0.10	0.00	6.61	5.71	8	LCM50	0.01	0.00	11.95	0.37
51	1	LCM51	0.10	0.00	6.70	5.64	8	LCM51	0.01	0.00	12.15	0.38
52	1	LCM52	0.10	0.00	6.70	5.64	8	LCM52	0.01	0.00	12.15	0.38
53	1	LCM53	0.11	0.00	6.26	6.02	8	LCM53	0.01	0.00	11.17	0.32
54	1	LCM54	0.11	0.00	6.26	6.02	8	LCM54	0.01	0.00	11.17	0.32
55	1	LCM55	-3.02	0.00	-1.92	-408.24	8	LCM55	-4.79	0.00	14.29	-592.21
56	1	LCM56	3.25	0.00	16.54	421.02	8	LCM56	4.80	0.00	11.86	592.99
57	1	LCM57	0.04	0.00	4.03	2.43	8	LCM57	0.00	0.00	6.96	0.19
58	1	LCM58	0.04	0.00	4.03	2.43	8	LCM58	0.00	0.00	6.96	0.19
59	1	LCM59	-3.10	0.00	-6.08	-412.72	8	LCM59	-4.79	0.00	6.66	-592.45
60	1	LCM60	3.17	0.00	12.38	416.53	8	LCM60	4.80	0.00	4.23	592.75
61	1	LCM61	-9.29	0.00	-20.39	-1237.50	8	LCM61	-14.38	0.00	16.71	-1777.41
62	1	LCM62	9.52	0.00	35.00	1250.28	8	LCM62	14.39	0.00	9.43	1778.19
63	1	LCM63	-9.37	0.00	-24.54	-1241.98	8	LCM63	-14.38	0.00	9.09	-1777.65
64	1	LCM64	9.44	0.00	30.85	1245.79	8	LCM64	14.39	0.00	1.81	1777.95

Note: Negative (-) values for axial load indicate tension

Base Reaction (kips, in)											
Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)
33	LCM1	0.00	0.00	7.73	-0.21	52	LCM1	-0.05	0.00	4.47	-2.70
33	LCM2	0.00	0.00	3.80	-0.13	52	LCM2	-0.10	0.00	1.78	-5.55
33	LCM3	0.00	0.00	1.57	-0.09	52	LCM3	0.01	0.00	0.71	0.61
33	LCM4	0.00	0.00	1.96	-0.12	52	LCM4	0.01	0.00	0.89	0.76
33	LCM5	0.00	0.00	0.00	0.00	52	LCM5	0.00	0.00	0.00	0.00
33	LCM6	0.00	0.00	0.00	0.00	52	LCM6	0.00	0.00	0.00	0.00
33	LCM7	-4.79	0.00	-1.21	-592.60	52	LCM7	-3.13	0.00	9.23	-414.63
33	LCM8	-0.52	0.00	7.64	-59.28	52	LCM8	-0.39	0.00	5.17	-44.28
33	LCM9	-0.52	0.00	11.44	-59.42	52	LCM9	-0.50	0.00	6.95	-49.83
33	LCM10	-0.52	0.00	9.21	-59.38	52	LCM10	-0.38	0.00	5.89	-43.68
33	LCM11	-0.52	0.00	9.60	-59.40	52	LCM11	-0.38	0.00	6.06	-43.53
33	LCM12	-0.52	0.00	7.64	-59.28	52	LCM12	-0.39	0.00	5.17	-44.28
33	LCM13	-0.52	0.00	11.67	-59.45	52	LCM13	-0.46	0.00	7.04	-47.99
33	LCM14	-0.52	0.00	11.96	-59.47	52	LCM14	-0.46	0.00	7.17	-47.88
33	LCM15	-0.52	0.00	10.49	-59.38	52	LCM15	-0.47	0.00	6.51	-48.45
33	LCM16	0.00	0.00	7.73	-0.21	52	LCM16	-0.05	0.00	4.47	-2.70
33	LCM17	0.00	0.00	7.73	-0.21	52	LCM17	-0.05	0.00	4.47	-2.70
33	LCM18	-3.36	0.00	7.94	-415.06	52	LCM18	-2.25	0.00	11.55	-293.31
33	LCM19	3.35	0.00	9.64	414.58	52	LCM19	2.14	0.00	-1.38	287.17
33	LCM20	-0.01	0.00	11.76	-0.38	52	LCM20	-0.12	0.00	6.34	-6.41
33	LCM21	-0.01	0.00	11.76	-0.38	52	LCM21	-0.12	0.00	6.34	-6.41
33	LCM22	-0.01	0.00	12.05	-0.40	52	LCM22	-0.12	0.00	6.47	-6.30
33	LCM23	-0.01	0.00	12.05	-0.40	52	LCM23	-0.12	0.00	6.47	-6.30
33	LCM24	-0.01	0.00	10.58	-0.31	52	LCM24	-0.13	0.00	5.81	-6.86
33	LCM25	-0.01	0.00	10.58	-0.31	52	LCM25	-0.13	0.00	5.81	-6.86
33	LCM26	-2.52	0.00	11.92	-311.52	52	LCM26	-1.77	0.00	11.64	-224.37
33	LCM27	2.51	0.00	13.19	310.71	52	LCM27	1.52	0.00	1.95	210.99
33	LCM28	-2.52	0.00	12.21	-311.54	52	LCM28	-1.77	0.00	11.78	-224.25
33	LCM29	2.51	0.00	13.48	310.69	52	LCM29	1.53	0.00	2.08	211.11
33	LCM30	-2.52	0.00	10.74	-311.45	52	LCM30	-1.78	0.00	11.11	-224.82
33	LCM31	2.51	0.00	12.01	310.78	52	LCM31	1.51	0.00	1.42	210.54
33	LCM32	0.00	0.00	4.64	-0.13	52	LCM32	-0.03	0.00	2.68	-1.62
33	LCM33	0.00	0.00	4.64	-0.13	52	LCM33	-0.03	0.00	2.68	-1.62
33	LCM34	-3.36	0.00	2.73	-414.92	52	LCM34	-2.22	0.00	8.53	-291.49
33	LCM35	3.35	0.00	4.43	414.72	52	LCM35	2.17	0.00	-4.39	288.99
33	LCM36	-0.52	0.00	10.74	-59.37	52	LCM36	-0.41	0.00	6.96	-45.37
33	LCM37	-0.53	0.00	16.05	-59.59	52	LCM37	-0.56	0.00	9.27	-53.40
33	LCM38	-0.53	0.00	16.25	-59.60	52	LCM38	-0.56	0.00	9.36	-53.32
33	LCM39	-0.53	0.00	15.27	-59.54	52	LCM39	-0.57	0.00	8.91	-53.70
33	LCM40	-0.53	0.00	13.60	-59.54	52	LCM40	-0.44	0.00	8.10	-46.63
33	LCM41	-0.01	0.00	11.79	-0.41	52	LCM41	-0.04	0.00	6.50	-2.27
33	LCM42	-0.01	0.00	11.79	-0.41	52	LCM42	-0.04	0.00	6.50	-2.27
33	LCM43	-0.53	0.00	14.23	-59.58	52	LCM43	-0.43	0.00	8.38	-46.39
33	LCM44	-0.01	0.00	12.42	-0.44	52	LCM44	-0.04	0.00	6.79	-2.03
33	LCM45	-0.01	0.00	12.42	-0.44	52	LCM45	-0.04	0.00	6.79	-2.03
33	LCM46	-0.52	0.00	11.09	-59.39	52	LCM46	-0.46	0.00	6.96	-47.60
33	LCM47	0.00	0.00	9.28	-0.26	52	LCM47	-0.06	0.00	5.37	-3.24
33	LCM48	0.00	0.00	9.28	-0.26	52	LCM48	-0.06	0.00	5.37	-3.24
33	LCM49	-0.01	0.00	11.97	-0.37	52	LCM49	-0.10	0.00	6.61	-5.71
33	LCM50	-0.01	0.00	11.97	-0.37	52	LCM50	-0.10	0.00	6.61	-5.71
33	LCM51	-0.01	0.00	12.16	-0.38	52	LCM51	-0.10	0.00	6.70	-5.64
33	LCM52	-0.01	0.00	12.16	-0.38	52	LCM52	-0.10	0.00	6.70	-5.64
33	LCM53	-0.01	0.00	11.18	-0.32	52	LCM53	-0.11	0.00	6.26	-6.02
33	LCM54	-0.01	0.00	11.18	-0.32	52	LCM54	-0.11	0.00	6.26	-6.02
33	LCM55	-4.80	0.00	11.87	-592.99	52	LCM55	-3.25	0.00	16.54	-421.02
33	LCM56	4.79	0.00	14.30	592.21	52	LCM56	3.02	0.00	-1.92	408.24
33	LCM57	0.00	0.00	6.96	-0.19	52	LCM57	-0.04	0.00	4.03	-2.43
33	LCM58	0.00	0.00	6.96	-0.19	52	LCM58	-0.04	0.00	4.03	-2.43
33	LCM59	-4.80	0.00	4.24	-592.75	52	LCM59	-3.17	0.00	12.38	-416.53
33	LCM60	4.79	0.00	6.67	592.45	52	LCM60	3.10	0.00	-6.08	412.72
33	LCM61	-14.39	0.00	9.44	-1778.19	52	LCM61	-9.52	0.00	35.00	-1250.28
33	LCM62	14.38	0.00	16.72	1777.41	52	LCM62	9.29	0.00	-20.39	1237.50
33	LCM63	-14.39	0.00	1.81	-1777.95	52	LCM63	-9.44	0.00	30.85	-1245.79
33	LCM64	14.38	0.00	9.09	1777.65	52	LCM64	9.37	0.00	-24.54	1241.98

Note: Negative (-) values for axial load indicate tension

SMF BASE PLATE CONNECTION DESIGN (COLUMN 1)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W10X68

Base Plate =	**10.125" x 11.5" x 0.5" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 62 **Design Code:** IBC 2015

Left Column:

V=	9.519	kips	
P=	35.000	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	10.125	in	Base plate width
N =	11.500	in	Base plate length
t =	0.500	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	10.40	in	Column depth
t_w =	0.47	in	Column web thickness
h_{web} =	8.86	in	Column web depth
b_f =	10.10	in	Column flange width
t_f =	0.77	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.75	in	Anchor bolt spacing
g =	3.75	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	116.4375	in^2	Area of base plate
A_2 =	116.4375	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	247	kips	Nominal strength for bearing
ϕP_p =	161	kips	Axial design strength for bearing
DCR_conc=	**0.218**	OK	

Plate Yielding:

m =	0.81	in	Base plate cantilever dimension
n =	1.02	in	Base plate cantilever dimension
X =	1.000		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.56	in	Base plate cantilever dimension
l =	2.56	in	Critical base plate cantilever dimension
ϕP_n =	100	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.351**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.47	in	Column web thickness
w_{min} =	3/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.77	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	7.86	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	9.60	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	9.519	kips
t_weld_web=	0.25	in
ϕRn_web=	87.54	kips
DCR_V=	**0.11**	OK

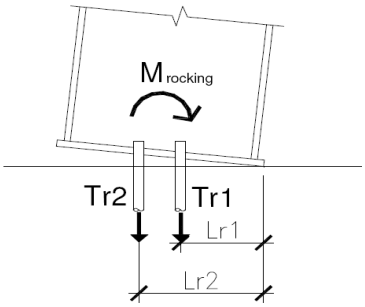
Tension Only

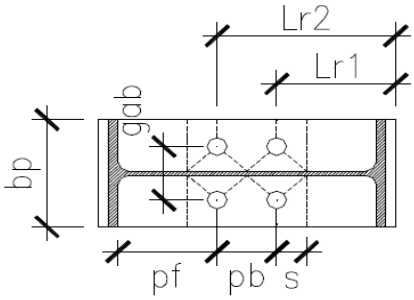
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	133.64	kips
ϕRn=	221.18	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	9.52	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	258.60	kips	
DCR_V+T=	**0.037**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	3.875	in	
Lr2=	7.625	in	
Tr1=	17.50	kips	Assume equal to 1/2 of P
Tr2=	17.50	kips	Assume equal to 1/2 of P
Mrocking=	201.3	kip-in	Rocking moment about Bp tip
treq=	0.331	in	Plate thickness required for tension
tbp=	0.500	in	
DCR_bp=	**0.662**	OK	

bp=	10.125	in	Y=	56.67	in
pb=	3.75	in	gamma_r=	1.25	
gab=	3.75	in	ϕb=	0.9	
pf=	2.555	in			
s_tmp=	3.08	in			
s=	3.080939386	in			

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.035	0.056	0.003	0.264
37	1.2 D + 1.6 L + 0.5 Lr	0.049	0.079	0.001	0.314
38	1.2 D + 1.6 L + 0.5 S	0.049	0.080	0.001	0.315
39	1.2 D + 1.6 L + 0.5 R	0.047	0.075	0.001	0.307
40	1.2 D + 1.6 Lr + 0.5 L	0.042	0.067	0.003	0.289
41	1.2 D + 1.6 Lr + 0.5 W	0.040	0.065	0.000	0.285
42	1.2 D + 1.6 Lr - 0.5 W	0.040	0.065	0.000	0.285
43	1.2 D + 1.6 S + 0.5 L	0.043	0.070	0.003	0.295
44	1.2 D + 1.6 S + 0.5 W	0.042	0.068	0.000	0.291
45	1.2 D + 1.6 S - 0.5 W	0.042	0.068	0.000	0.291
46	1.2 D + 1.6 R + 0.5 L	0.035	0.056	0.003	0.264
47	1.2 D + 1.6 R + 0.5 W	0.033	0.054	0.001	0.259
48	1.2 D + 1.6 R - 0.5 W	0.033	0.054	0.001	0.259
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.041	0.066	0.001	0.288
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.041	0.066	0.001	0.288
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.042	0.067	0.001	0.290
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.042	0.067	0.001	0.290
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.039	0.063	0.001	0.280
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.039	0.063	0.001	0.280
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.000	0.000	0.034	0.155
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.103	0.166	0.037	0.455
57	0.9 D + 1.0 W	0.025	0.040	0.001	0.224
58	0.9 D - 1.0 W	0.025	0.040	0.001	0.224
59	(0.9 - 0.2 SDS) D + E	0.000	0.000	0.035	0.276
60	(0.9 - 0.2 SDS) D - E	0.077	0.124	0.036	0.394
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.000	0.000	0.106	0.505
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.218	0.351	0.109	0.662
63	(0.9 - 0.2 SDS) D + Omega*E	0.000	0.000	0.111	0.554
64	(0.9 - 0.2 SDS) D - Omega*E	0.192	0.309	0.108	0.621
	Max=	0.218	0.351	0.111	0.662
	Check=	OK	OK	OK	OK

SMF BASE PLATE CONNECTION DESIGN (COLUMN 2)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W10X88

Base Plate =	**10.375" x 11.875" x 0.5" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 61 **Design Code:** IBC 2015

Left Column:

V=	-14.377	kips	
P=	16.713	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	10.375	in	Base plate width
N =	11.875	in	Base plate length
t =	0.500	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	10.80	in	Column depth
t_w =	0.61	in	Column web thickness
h_{web} =	8.82	in	Column web depth
b_f =	10.30	in	Column flange width
t_f =	0.99	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.75	in	Anchor bolt spacing
g =	3.75	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	123.20313	in^2	Area of base plate
A_2 =	123.20313	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	262	kips	Nominal strength for bearing
ϕP_p =	170	kips	Axial design strength for bearing
DCR_conc=	**0.098**	OK	

Plate Yielding:

m =	0.81	in	Base plate cantilever dimension
n =	1.07	in	Base plate cantilever dimension
X =	0.999		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.64	in	Base plate cantilever dimension
l =	2.64	in	Critical base plate cantilever dimension
ϕP_n =	100	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.168**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.61	in	Column web thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.99	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	7.82	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	9.80	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	14.377	kips
t_weld_web=	0.25	in
ϕRn_web=	87.09	kips
DCR_V=	**0.17**	OK

Tension Only

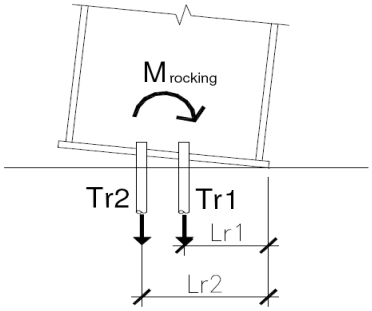
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	136.43	kips
ϕRn=	223.52	kips
DCR_T=	**0.000**	OK

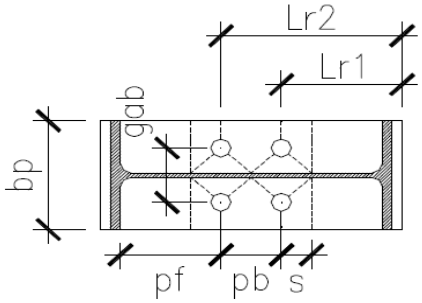
Shear+Tension

Vu_srss=	14.38	kips
θ_srss=	0.00	degrees
$\phi R_{n\text{-}\theta}$=	1.39	kip/in
ϕR_n=	359.95	kips
DCR_V+T=	**0.040**	OK

Weld design strength (per 1/16")

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	4.0625	in	
Lr2=	7.8125	in	
Tr1=	8.36	kips	Assume equal to 1/2 of P
Tr2=	8.36	kips	Assume equal to 1/2 of P
Mrocking=	99.2	kip-in	Rocking moment about Bp tip
treq=	0.228	in	Plate thickness required for tension
tbp=	0.500	in	
DCR_bp=	**0.455**	OK	

bp=	10.375	in
pb=	3.75	in
gab=	3.75	in
pf=	2.535	in
s_tmp=	3.12	in
s=	3.118743737	in

Y=	59.00	in
gamma_r=	1.25	
ϕb=	0.9	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.064	0.109	0.006	0.368
37	1.2 D + 1.6 L + 0.5 Lr	0.095	0.163	0.006	0.449
38	1.2 D + 1.6 L + 0.5 S	0.096	0.165	0.006	0.451
39	1.2 D + 1.6 L + 0.5 R	0.091	0.155	0.006	0.438
40	1.2 D + 1.6 Lr + 0.5 L	0.081	0.138	0.006	0.413
41	1.2 D + 1.6 Lr + 0.5 W	0.069	0.118	0.000	0.382
42	1.2 D + 1.6 Lr - 0.5 W	0.069	0.118	0.000	0.382
43	1.2 D + 1.6 S + 0.5 L	0.085	0.144	0.006	0.423
44	1.2 D + 1.6 S + 0.5 W	0.073	0.124	0.000	0.392
45	1.2 D + 1.6 S - 0.5 W	0.073	0.124	0.000	0.392
46	1.2 D + 1.6 R + 0.5 L	0.066	0.113	0.006	0.374
47	1.2 D + 1.6 R + 0.5 W	0.055	0.093	0.000	0.339
48	1.2 D + 1.6 R - 0.5 W	0.055	0.093	0.000	0.339
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.070	0.120	0.000	0.385
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.070	0.120	0.000	0.385
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.071	0.122	0.000	0.388
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.071	0.122	0.000	0.388
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.066	0.112	0.000	0.372
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.066	0.112	0.000	0.372
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.084	0.143	0.055	0.421
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.070	0.119	0.055	0.384
57	0.9 D + 1.0 W	0.041	0.070	0.000	0.294
58	0.9 D - 1.0 W	0.041	0.070	0.000	0.294
59	(0.9 - 0.2 SDS) D + E	0.039	0.067	0.055	0.288
60	(0.9 - 0.2 SDS) D - E	0.025	0.042	0.055	0.229
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.098	0.168	0.165	0.455
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.055	0.095	0.165	0.342
63	(0.9 - 0.2 SDS) D + Omega*E	0.053	0.091	0.165	0.336
64	(0.9 - 0.2 SDS) D - Omega*E	0.011	0.018	0.165	0.150
	Max=	0.098	0.168	0.165	0.455
	Check=	OK	OK	OK	OK

SMF BASE PLATE CONNECTION DESIGN (COLUMN 3)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W10X88

Base Plate =	**10.375" x 11.875" x 0.5" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 62 **Design Code:** IBC 2015

Left Column:

V=	14.377	kips	
P=	16.724	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	10.375	in	Base plate width
N =	11.875	in	Base plate length
t =	0.500	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	10.80	in	Column depth
t_w =	0.61	in	Column web thickness
h_{web} =	8.82	in	Column web depth
b_f =	10.30	in	Column flange width
t_f =	0.99	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.75	in	Anchor bolt spacing
g =	3.75	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	123.20313	in^2	Area of base plate
A_2 =	123.20313	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	262	kips	Nominal strength for bearing
ϕP_p =	170	kips	Axial design strength for bearing
DCR_conc=	**0.098**	OK	

Plate Yielding:

m =	0.81	in	Base plate cantilever dimension
n =	1.07	in	Base plate cantilever dimension
X =	0.999		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.64	in	Base plate cantilever dimension
l =	2.64	in	Critical base plate cantilever dimension
ϕP_n =	100	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.168**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.61	in	Column web thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.99	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	7.82	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	9.80	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	14.377	kips
t_weld_web=	0.25	in
ϕRn_web=	87.09	kips
DCR_V=	**0.17**	OK

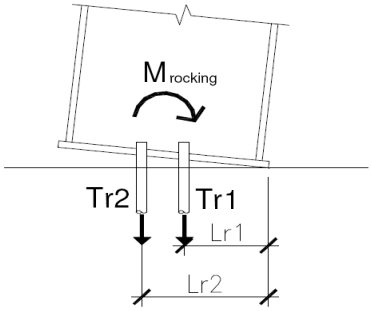
Tension Only

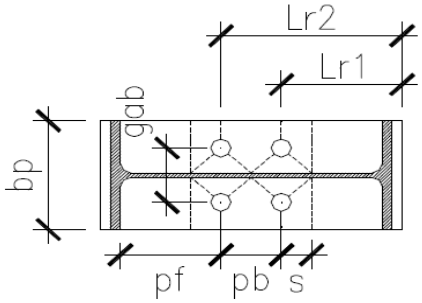
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	136.43	kips
ϕRn=	223.52	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	14.38	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	359.95	kips	
DCR_V+T=	**0.040**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	4.0625	in	
Lr2=	7.8125	in	
Tr1=	8.36	kips	Assume equal to 1/2 of P
Tr2=	8.36	kips	Assume equal to 1/2 of P
Mrocking=	99.3	kip-in	Rocking moment about Bp tip
treq=	0.228	in	Plate thickness required for tension
tbp=	0.500	in	
DCR_bp=	**0.456**	OK	

bp=	10.375	in
pb=	3.75	in
gab=	3.75	in
pf=	2.535	in
s_tmp=	3.12	in
s=	3.118743737	in

Y=	59.00	in
gamma_r=	1.25	
ϕb=	0.9	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.063	0.108	0.006	0.365
37	1.2 D + 1.6 L + 0.5 Lr	0.094	0.161	0.006	0.446
38	1.2 D + 1.6 L + 0.5 S	0.095	0.163	0.006	0.449
39	1.2 D + 1.6 L + 0.5 R	0.090	0.153	0.006	0.435
40	1.2 D + 1.6 Lr + 0.5 L	0.080	0.136	0.006	0.411
41	1.2 D + 1.6 Lr + 0.5 W	0.069	0.118	0.000	0.383
42	1.2 D + 1.6 Lr - 0.5 W	0.069	0.118	0.000	0.383
43	1.2 D + 1.6 S + 0.5 L	0.084	0.143	0.006	0.420
44	1.2 D + 1.6 S + 0.5 W	0.073	0.125	0.000	0.393
45	1.2 D + 1.6 S - 0.5 W	0.073	0.125	0.000	0.393
46	1.2 D + 1.6 R + 0.5 L	0.065	0.111	0.006	0.371
47	1.2 D + 1.6 R + 0.5 W	0.055	0.093	0.000	0.339
48	1.2 D + 1.6 R - 0.5 W	0.055	0.093	0.000	0.339
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.070	0.120	0.000	0.385
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.070	0.120	0.000	0.385
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.071	0.122	0.000	0.389
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.071	0.122	0.000	0.389
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.066	0.112	0.000	0.373
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.066	0.112	0.000	0.373
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.070	0.119	0.055	0.384
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.084	0.143	0.055	0.421
57	0.9 D + 1.0 W	0.041	0.070	0.000	0.294
58	0.9 D - 1.0 W	0.041	0.070	0.000	0.294
59	(0.9 - 0.2 SDS) D + E	0.025	0.043	0.055	0.229
60	(0.9 - 0.2 SDS) D - E	0.039	0.067	0.055	0.288
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.055	0.095	0.165	0.342
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.098	0.168	0.165	0.456
63	(0.9 - 0.2 SDS) D + Omega*E	0.011	0.018	0.165	0.150
64	(0.9 - 0.2 SDS) D - Omega*E	0.053	0.091	0.165	0.336
	Max=	0.098	0.168	0.165	0.456
	Check=	OK	OK	OK	OK

SMF BASE PLATE CONNECTION DESIGN (COLUMN 4)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W10X68

Base Plate =	**10.125" x 11.5" x 0.5" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 61 **Design Code:** IBC 2015

Left Column:

V=	-9.519	kips	
P=	35.000	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	10.125	in	Base plate width
N =	11.500	in	Base plate length
t =	0.500	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	10.40	in	Column depth
t_w =	0.47	in	Column web thickness
h_{web} =	8.86	in	Column web depth
b_f =	10.10	in	Column flange width
t_f =	0.77	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.75	in	Anchor bolt spacing
g =	3.75	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	116.4375	in^2	Area of base plate
A_2 =	116.4375	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	247	kips	Nominal strength for bearing
ϕP_p =	161	kips	Axial design strength for bearing
DCR_conc=	**0.218**	OK	

Plate Yielding:

m =	0.81	in	Base plate cantilever dimension
n =	1.02	in	Base plate cantilever dimension
X =	1.000		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.56	in	Base plate cantilever dimension
l =	2.56	in	Critical base plate cantilever dimension
ϕP_n =	100	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.351**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.47	in	Column web thickness
w_{min} =	3/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.77	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	7.86	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	9.60	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	9.519	kips
t_weld_web=	0.25	in
ϕRn_web=	87.54	kips
DCR_V=	**0.11**	OK

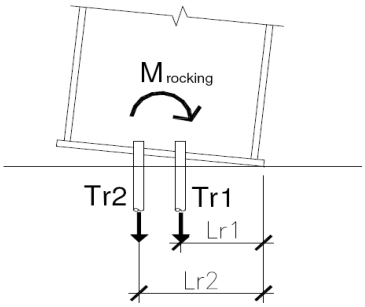
Tension Only

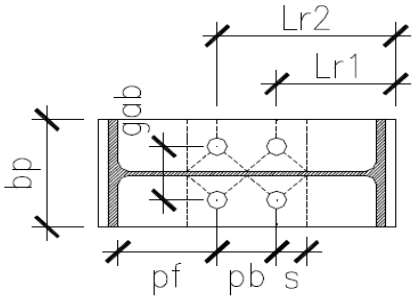
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	133.64	kips
ϕRn=	221.18	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	9.52	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	258.60	kips	
DCR_V+T=	**0.037**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	3.875	in	
Lr2=	7.625	in	
Tr1=	17.50	kips	Assume equal to 1/2 of P
Tr2=	17.50	kips	Assume equal to 1/2 of P
Mrocking=	201.3	kip-in	Rocking moment about Bp tip
treq=	0.331	in	Plate thickness required for tension
tbp=	0.500	in	
DCR_bp=	**0.662**	OK	

bp=	10.125	in
pb=	3.75	in
gab=	3.75	in
pf=	2.555	in
s_tmp=	3.08	in
s=	3.080939386	in

Y=	56.67	in
gamma_r=	1.25	
ϕb=	0.9	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.043	0.070	0.005	0.295
37	1.2 D + 1.6 L + 0.5 Lr	0.058	0.093	0.006	0.341
38	1.2 D + 1.6 L + 0.5 S	0.058	0.094	0.006	0.342
39	1.2 D + 1.6 L + 0.5 R	0.055	0.089	0.006	0.334
40	1.2 D + 1.6 Lr + 0.5 L	0.050	0.081	0.005	0.318
41	1.2 D + 1.6 Lr + 0.5 W	0.040	0.065	0.000	0.285
42	1.2 D + 1.6 Lr - 0.5 W	0.040	0.065	0.000	0.285
43	1.2 D + 1.6 S + 0.5 L	0.052	0.084	0.005	0.324
44	1.2 D + 1.6 S + 0.5 W	0.042	0.068	0.000	0.291
45	1.2 D + 1.6 S - 0.5 W	0.042	0.068	0.000	0.291
46	1.2 D + 1.6 R + 0.5 L	0.043	0.070	0.005	0.295
47	1.2 D + 1.6 R + 0.5 W	0.033	0.054	0.001	0.259
48	1.2 D + 1.6 R - 0.5 W	0.033	0.054	0.001	0.259
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.041	0.066	0.001	0.288
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.041	0.066	0.001	0.288
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.042	0.067	0.001	0.290
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.042	0.067	0.001	0.290
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.039	0.063	0.001	0.280
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.039	0.063	0.001	0.280
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.103	0.166	0.037	0.455
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.000	0.000	0.034	0.155
57	0.9 D + 1.0 W	0.025	0.040	0.001	0.224
58	0.9 D - 1.0 W	0.025	0.040	0.001	0.224
59	(0.9 - 0.2 SDS) D + E	0.077	0.124	0.036	0.394
60	(0.9 - 0.2 SDS) D - E	0.000	0.000	0.035	0.276
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.218	0.351	0.109	0.662
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.000	0.000	0.106	0.505
63	(0.9 - 0.2 SDS) D + Omega*E	0.192	0.309	0.108	0.621
64	(0.9 - 0.2 SDS) D - Omega*E	0.000	0.000	0.111	0.554
	Max=	0.218	0.351	0.111	0.662
	Check=	OK	OK	OK	OK

SMF COLUMN FIXED BASE DESIGN (COLUMN 1& 4)

Ref: AISC Seismic Design Manual, AISC Steel Design Manual, AISC 360-10

Column Fixed base Summary

Column = W10X68

Bearing Plate =	**8.8125" x 4.75" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Transfer Reinforcement =	**(2) 0.75 in. x 36 in. bars**
Bearing Plate to Column Flange Weld Size =	**0.5625 in**
Bearing Plate to Column Web Weld Size =	**0.5625 in**
Longitudinal Reinforcement Requirement =	**0.8 in^2**

1. DESIGN DATA

Design Forces:

LC =	64	
V_{u1} =	9.437	kips
P_u =	30.846	kips

Bearing Plate

Grade =	ASTM A572 Gr 50		*Face plate material grade*
F_{yP} =	50	ksi	*Face plate yield stress*
B_P =	4.750	in	*Bearing plate width*
L_P =	8.813	in	*Bearing plate length*
t_P =	**0.625**	in	*Flange splice plate thickness*

Reinforcement Material:

Grade =	ASTM A615 Gr 60		*Reinforcement material grade*
F_{yR} =	60	ksi	*Reinforcement yield stress*
n_{bar} =	2	bars	*Number of bars in each region*
d_{bar} =	**3/4**	in.	*bar's diameter for transfer reinf.*
L_{bar} =	36	in	*default = 36 inch per ACI 318*

Concrete Material:

f'_c =	**2,500**	psi	*Concrete compressive stress*
b_w =	22.000	in	*Grade beam width*

Column:

Grade =	ASTM A572 Gr 50		
F_{yC} =	50	ksi	*Column yield stress*
F_u =	65	ksi	*Column ultimate stress*
H =	**178.75**	in	*Story Height*
L_e =	21.5	in.	*Column minimum embedment length*
d =	10.40	in	*Column depth*
t_w =	0.47	in	*Column web thickness*
b_f =	10.10	in	*Column flange width*
t_f =	0.77	in	*Exterior column flange thickness*
Z_x =	85.30	in^3	*Plastic section modulus X-axis*
A =	19.90	in^2	*Cross section area of column.*

Include Anchorage Moment:

No

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{min} =	0.5004	in	5/8*tsp Fillet size per AISC Steel Manual 14th edition and to develop Fy*Ast per AISC 341-16.
w_{size} =	9/16	in	*Fillet weld size on each size*

2. AXIAL DESIGN

2.1 Concrete Bearing:

A_1 =	83.71875	in^2	*Area of base plate*
A_2 =	83.71875	in^2	*Area of concrete support concentric with base plate*
Φ_{brg} =	0.65		*Resistance factor for concrete bearing*
$\sqrt{A_2/A_1}$ =	1.0		*=√A2/A1*
$f_{p(max)}$ =	4.25	ksi	*=0.85*f'c/ (1000*min(√A2/A1,2.0))*
P_p =	356	kips	*=A2*$f_{p(max)}$*
ϕP_p =	231	kips	*=Φbrg*Pp*
DCR_conc=	**0.133**	OK	*=P_{u1} / ϕP_P, OK if DCR_conc <= 1.03*

2.2 Plate Yeilding

N =	10.40		=d
B =	10.10		=bf
λ =	1.0		Conservative per AISC Manual Part 14
m =	0.26		=(N - 0.95 * d) / 2
n =	1.01		=(B - 0.8 * bf) / 2
λ_n' =	2.56		=(λ * sqrt(d * bf)) / 4
t_{minP1} =	0.293	in.	= λ n' * sqrt(2 * Pu /(0.9* F_{yP} * B *N))
t_{minP2} =	0.353	in.	=0.75*tw
DCR_Pyield =	**0.564**	OK	=max(t_{minP1}, t_{minP2})/t_P, OK if DCR_Pyield <= 1.03

3. SHEAR DESIGN

3.1 Column Shear Strength

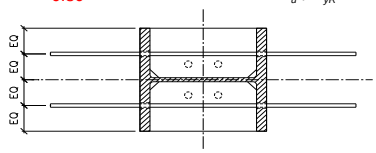
M_{pc} =	4265.0	kip-ft	=Fy * Zx (Nominal plastic flexural strength of the column)
V_{u2} =	47.72	kips	=2*M_{pc} / H
V_u =	47.72	kips	=max(V_{u1}, V_{u2})
β_1 =	0.85		from ACI 318 Section 10.2.7.3
g =	178.75		g=H
V_{n1} =	49.02	kips	=1.54*sqrt(f'c)*(bw/bf)$^{0.66}$ * β1*bf*(Le-2")*(0.58-0.22* β1)/(0.88+g/2/(Le-2"))
P =	45.46	kips	= ϕ_{brg}*(tf + 2)*bf*f'c
M_2 =	236.41	kip.in	= P*d/2
V_{n2} =	11.00	kips	= M_2/Le
V_n =	49.02	kips	= If (Include Anchorage Moment = "Yes", V_{n1} + V_{n2}, V_{n1})
DCR_ColV =	**0.974**	OK	=V_u/V_n, OK if DCR_Col <= 1.03

3.2 Required transfer reinforcement:

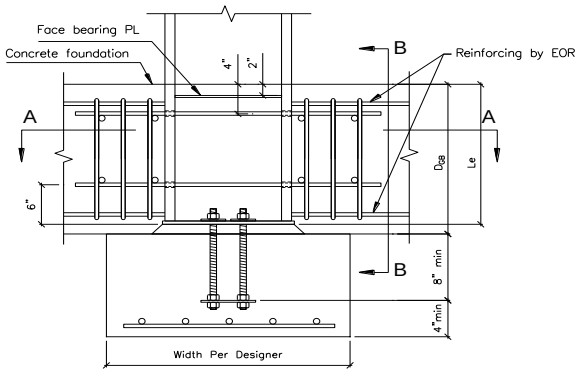
F_{ysr} =	60.00	ksi	value defined previously
A_{tbmin} =	0.246	in.^{2}	=0.03*f'c*(Le-2")*bf/ Fysr
A_{tbmax} =	33.525	in.^{2}	=0.08*(Le-2")*bw-As
A_{tb} =	0.884	in.^{2}	=n_{bar} * π * d_{bar}^2 / 4
DCR_Bar =	**0.279**	OK	=A_{tb}/ min(A_{tbmin}, A_{tbmax}), OK if DCR_Bar <= 1.03

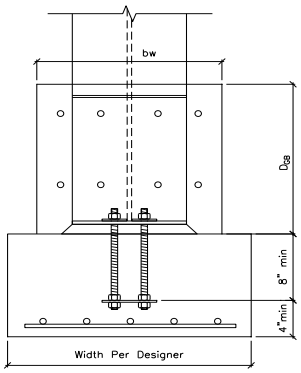
3.2 Longitudinal Foundation Reinforcement Requirement (EOR to provide calculations):

A_s = 0.80 in.^{2} =V_u / F_{yR}



SECTION A-A





SECTION B-B

4. DESIGN SUMMARY:

Axial DCR = 0.564 Shear DCR = 0.974

Check = OK Check = OK

b_w = 22.000 in. (Min.)
L_e = 21.500 in. (Min.)
D_{GB} = 24.000 in. (Min.)

		Axial Check			Shear Check			
Combo	**Load Combinations**	**P_{u1} (kips)**	**DCR**	Check	**V_{u1} (kips)**	**V_u (kips)**	**DCR**	Check
36	1.4 D	6.96	0.56	OK	0.41	47.72	0.97	OK
37	1.2 D + 1.6 L + 0.5 Lr	9.27	0.56	OK	0.56	47.72	0.97	OK
38	1.2 D + 1.6 L + 0.5 S	9.36	0.56	OK	0.56	47.72	0.97	OK
39	1.2 D + 1.6 L + 0.5 R	8.91	0.56	OK	0.57	47.72	0.97	OK
40	1.2 D + 1.6 Lr + 0.5 L	8.10	0.56	OK	0.44	47.72	0.97	OK
41	1.2 D + 1.6 Lr + 0.8 W	6.50	0.56	OK	0.04	47.72	0.97	OK
42	1.2 D + 1.6 Lr - 0.8 W	6.50	0.56	OK	0.04	47.72	0.97	OK
43	1.2 D + 1.6 S + 0.5 L	8.38	0.56	OK	0.43	47.72	0.97	OK
44	1.2 D + 1.6 S + 0.8 W	6.79	0.56	OK	0.04	47.72	0.97	OK
45	1.2 D + 1.6 S - 0.8 W	6.79	0.56	OK	0.04	47.72	0.97	OK
46	1.2 D + 1.6 R + 0.5 L	6.96	0.56	OK	0.46	47.72	0.97	OK
47	1.2 D + 1.6 R + 0.8 W	5.37	0.56	OK	0.06	47.72	0.97	OK
48	1.2 D + 1.6 R - 0.8 W	5.37	0.56	OK	0.06	47.72	0.97	OK
49	1.2 D + 1.6 W + 0.5 L + 0.5 Lr	6.61	0.56	OK	0.10	47.72	0.97	OK
50	1.2 D - 1.6 W + 0.5 L + 0.5 Lr	6.61	0.56	OK	0.10	47.72	0.97	OK
51	1.2 D + 1.6 W + 0.5 L + 0.5 S	6.70	0.56	OK	0.10	47.72	0.97	OK
52	1.2 D - 1.6 W + 0.5 L + 0.5 S	6.70	0.56	OK	0.10	47.72	0.97	OK
53	1.2 D + 1.6 W + 0.5 L + 0.5 R	6.26	0.56	OK	0.11	47.72	0.97	OK
54	1.2 D - 1.6 W + 0.5 L + 0.5 R	6.26	0.56	OK	0.11	47.72	0.97	OK
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2*S	16.54	0.56	OK	3.25	47.72	0.97	OK
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	16.54	0.56	OK	3.25	47.72	0.97	OK
57	0.9 D + 1.6 W	4.03	0.56	OK	0.04	47.72	0.97	OK
58	0.9 D - 1.6 W	4.03	0.56	OK	0.04	47.72	0.97	OK
59	(0.9 - 0.2 SDS) D + E	12.38	0.56	OK	3.17	47.72	0.97	OK
60	(0.9 - 0.2 SDS) D - E	12.38	0.56	OK	3.17	47.72	0.97	OK
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	35.00	0.56	OK	9.52	47.72	0.97	OK
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	35.00	0.56	OK	9.52	47.72	0.97	OK
63	(0.9 - 0.2 SDS) D + Omega*E	30.85	0.56	OK	9.44	47.72	0.97	OK
64	(0.9 - 0.2 SDS) D - Omega*E	30.85	0.56	OK	9.44	47.72	0.97	OK

SMF COLUMN FIXED BASE DESIGN (COLUMN 2& 3)

Ref: AISC Seismic Design Manual, AISC Steel Design Manual, AISC 360-10

Column Fixed base Summary

Column = W10X88

Bearing Plate =	**8.8125" x 4.75" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Transfer Reinforcement =	**(2) 0.75 in. x 36 in. bars**
Bearing Plate to Column Flange Weld Size =	**0.5625 in**
Bearing Plate to Column Web Weld Size =	**0.5625 in**
Longitudinal Reinforcement Requirement =	**1.05 in^2**

1. DESIGN DATA

Design Forces:

LC =	64	
V_{u1} =	14.386	kips
P_u =	9.093	kips

Bearing Plate

Grade =	ASTM A572 Gr 50		*Face plate material grade*
F_{yP} =	50	ksi	*Face plate yield stress*
B_P =	4.750	in	*Bearing plate width*
L_P =	8.813	in	*Bearing plate length*
t_P =	**0.625**	in	*Flange splice plate thickness*

Reinforcement Material:

Grade =	ASTM A615 Gr 60		*Reinforcement material grade*
F_{yR} =	60	ksi	*Reinforcement yield stress*
n_{bar} =	2	bars	*Number of bars in each region*
d_{bar} =	**3/4**	in.	*bar's diameter for transfer reinf.*
L_{bar} =	36	in	*default = 36 inch per ACI 318*

Concrete Material:

f'_c =	**2,500**	psi	*Concrete compressive stress*
b_w =	22.000	in	*Grade beam width*

Column:

Grade =	ASTM A572 Gr 50		
F_{yC} =	50	ksi	*Column yield stress*
F_u =	65	ksi	*Column ultimate stress*
H =	**178.75**	in	*Story Height*
L_e =	24.5	in.	*Column minimum embedment length*
d =	10.80	in	*Column depth*
t_w =	0.61	in	*Column web thickness*
b_f =	10.30	in	*Column flange width*
t_f =	0.99	in	*Exterior column flange thickness*
Z_x =	113.00	in^3	*Plastic section modulus X-axis*
A =	26.00	in^2	*Cross section area of column.*

Include Anchorage Moment:

No

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{min} =	0.5038	in	5/8*tsp Fillet size per AISC Steel Manual 14th edition and to develop Fy*Ast per AISC 341-16.
w_{size} =	9/16	in	*Fillet weld size on each size*

2. AXIAL DESIGN

2.1 Concrete Bearing:

A_1 =	83.71875	in^2	*Area of base plate*
A_2 =	83.71875	in^2	*Area of concrete support concentric with base plate*
Φ_{brg} =	0.65		*Resistance factor for concrete bearing*
$\sqrt{A_2/A_1}$ =	1.0		*=√A2/A1*
$f_{p(max)}$ =	4.25	ksi	*=0.85*f'c/ (1000*min(√A2/A1,2.0))*
P_p =	356	kips	*=A2*$f_{p(max)}$*
ϕP_p =	231	kips	*=Φbrg*Pp*
DCR_conc=	**0.039**	OK	*=P_{u1} / ϕP_P, OK if DCR_conc <= 1.03*

2.2 Plate Yeilding

N =	10.80		=d
B =	10.30		=bf
λ =	1.0		Conservative per AISC Manual Part 14
m =	0.27		=(N - 0.95 * d) / 2
n =	1.03		=(B - 0.8 * bf) / 2
λ_n' =	2.64		=(λ * sqrt(d * bf)) / 4
t_{minP1} =	0.159	in.	= λ n' * sqrt(2 * Pu /(0.9* F_{yP} * B *N))
t_{minP2} =	0.454	in.	=0.75*tw
DCR_Pyield =	**0.726**	OK	=max(t_{minP1}, t_{minP2})/t_P, OK if DCR_Pyield <= 1.03

3. SHEAR DESIGN

3.1 Column Shear Strength

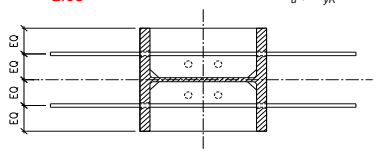
M_{pc} =	5650.0	kip-ft	=Fy * Zx (Nominal plastic flexural strength of the column)
V_{u2} =	63.22	kips	=2*M_{pc} / H
V_u =	63.22	kips	=max(V_{u1}, V_{u2})
β_1 =	0.85		from ACI 318 Section 10.2.7.3
g =	178.75		g=H
V_{n1} =	64.11	kips	=1.54*sqrt(f'c)*(bw/bf)$^{0.66}$ * β1*bf*(Le-2")*(0.58-0.22* β1)/(0.88+g/2/(Le-2"))
P =	50.05	kips	= ϕ_{brg}*(tf + 2)*bf*f'c
M_2 =	270.24	kip.in	= P*d/2
V_{n2} =	11.03	kips	= M_2/Le
V_n =	64.11	kips	= If (Include Anchorage Moment = "Yes", V_{n1} + V_{n2}, V_{n1})
DCR_ColV =	**0.986**	OK	=V_u/V_n, OK if DCR_Col <= 1.03

3.2 Required transfer reinforcement:

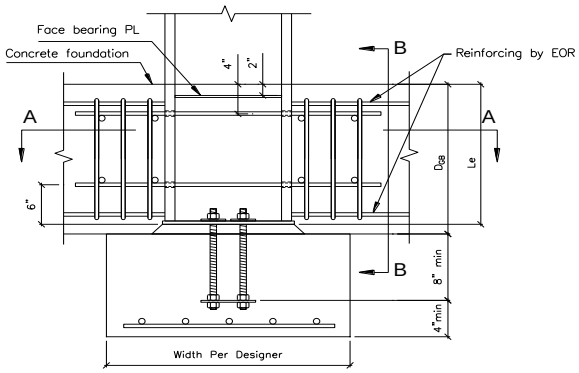
F_{ysr} =	60.00	ksi	value defined previously
A_{tbmin} =	0.290	in.^{2}	=0.03*f'c*(Le-2")*bf/ Fysr
A_{tbmax} =	38.546	in.^{2}	=0.08*(Le-2")*bw-As
A_{tb} =	0.884	in.^{2}	=n_{bar} * π * d_{bar}^2 / 4
DCR_Bar =	**0.328**	OK	=A_{tb}/ min(A_{tbmin}, A_{tbmax}), OK if DCR_Bar <= 1.03

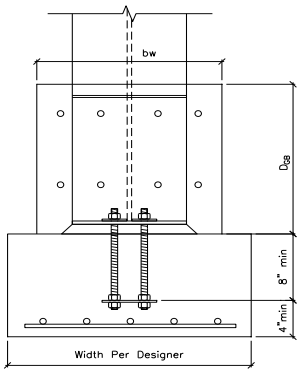
3.2 Longitudinal Foundation Reinforcement Requirement (EOR to provide calculations):

A_s = 1.05 in.^{2} =V_u / F_{yR}



SECTION A-A





SECTION B-B

4. DESIGN SUMMARY:

Axial DCR = 0.726 Shear DCR = 0.986

Check = OK Check = OK

b_w = 22.000 in. (Min.)
L_e = 24.500 in. (Min.)
D_{GB} = 27.000 in. (Min.)

		Axial Check			Shear Check			
Combo	Load Combinations	P_{u1} (kips)	DCR	Check	V_{u1} (kips)	V_u (kips)	DCR	Check
36	1.4 D	10.91	0.73	OK	0.52	63.22	0.99	OK
37	1.2 D + 1.6 L + 0.5 Lr	16.22	0.73	OK	0.53	63.22	0.99	OK
38	1.2 D + 1.6 L + 0.5 S	16.41	0.73	OK	0.53	63.22	0.99	OK
39	1.2 D + 1.6 L + 0.5 R	15.43	0.73	OK	0.53	63.22	0.99	OK
40	1.2 D + 1.6 Lr + 0.5 L	13.77	0.73	OK	0.53	63.22	0.99	OK
41	1.2 D + 1.6 Lr + 0.8 W	11.79	0.73	OK	0.01	63.22	0.99	OK
42	1.2 D + 1.6 Lr - 0.8 W	11.79	0.73	OK	0.01	63.22	0.99	OK
43	1.2 D + 1.6 S + 0.5 L	14.39	0.73	OK	0.53	63.22	0.99	OK
44	1.2 D + 1.6 S + 0.8 W	12.42	0.73	OK	0.01	63.22	0.99	OK
45	1.2 D + 1.6 S - 0.8 W	12.42	0.73	OK	0.01	63.22	0.99	OK
46	1.2 D + 1.6 R + 0.5 L	11.26	0.73	OK	0.52	63.22	0.99	OK
47	1.2 D + 1.6 R + 0.8 W	9.28	0.73	OK	0.00	63.22	0.99	OK
48	1.2 D + 1.6 R - 0.8 W	9.28	0.73	OK	0.00	63.22	0.99	OK
49	1.2 D + 1.6 W + 0.5 L + 0.5 Lr	11.97	0.73	OK	0.01	63.22	0.99	OK
50	1.2 D - 1.6 W + 0.5 L + 0.5 Lr	11.97	0.73	OK	0.01	63.22	0.99	OK
51	1.2 D + 1.6 W + 0.5 L + 0.5 S	12.16	0.73	OK	0.01	63.22	0.99	OK
52	1.2 D - 1.6 W + 0.5 L + 0.5 S	12.16	0.73	OK	0.01	63.22	0.99	OK
53	1.2 D + 1.6 W + 0.5 L + 0.5 R	11.18	0.73	OK	0.01	63.22	0.99	OK
54	1.2 D - 1.6 W + 0.5 L + 0.5 R	11.18	0.73	OK	0.01	63.22	0.99	OK
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2*S	14.29	0.73	OK	4.80	63.22	0.99	OK
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	14.30	0.73	OK	4.80	63.22	0.99	OK
57	0.9 D + 1.6 W	6.96	0.73	OK	0.00	63.22	0.99	OK
58	0.9 D - 1.6 W	6.96	0.73	OK	0.00	63.22	0.99	OK
59	(0.9 - 0.2 SDS) D + E	6.66	0.73	OK	4.80	63.22	0.99	OK
60	(0.9 - 0.2 SDS) D - E	6.67	0.73	OK	4.80	63.22	0.99	OK
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	16.71	0.73	OK	14.39	63.22	0.99	OK
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	16.72	0.73	OK	14.39	63.22	0.99	OK
63	(0.9 - 0.2 SDS) D + Omega*E	9.09	0.73	OK	14.39	63.22	0.99	OK
64	(0.9 - 0.2 SDS) D - Omega*E	9.09	0.73	OK	14.39	63.22	0.99	OK

Simpson Strong-Tie
5956 West Las Positas Boulevard
Pleasanton, CA 94588

SMF BEAM TOP WOOD NAILER ATTACHMENT DESIGN

BOLT LATERAL DESIGN VALUES

Wood Data		Steel Data		Bolt Data	
Species:	**DFL**	**Steel Gauge:**	**0.5**	**Bolt Size:**	**3/4**
Main Member	**Single 4X**	**F_u:**	**65000**	**Angle to Grain**	**0**
Countersink	**3/4"**				

ASTM A572 Gr 50 Steel

Bolt Design Information

D	F_{yb}	l_m	R_t	K_θ	R_e	R_d		
						I_m, I_s	II	III_m, III_s, IV
0.75	45000	2.750	5.500	1.00	0.0627	4	3.6	3.2

Steel Design Information

F_u	t_{steel}
65000	0.5

Wood Design Information

t_{wood}	SG	$F_{em(\theta)}$	F_{es}
2.75	0.5	5600	89375

Design Calculations

k_1	K_2	K_3	Z (I_m)	Z (I_s):	Z (II):	Z (III_m):	Z (III_s):	Z (IV):
0.180	0.604	6.667	2888	8379	1679	1938	2121	2210

	ALLOWABLE LOADS				
	100%	***115%***	***125%***	***133%***	***160%***
Single Shear:	**1679**	**1930**	**2098**	**2233**	**2686**

ASD Bolt Z_{allow} = **2686 lbs** **Per NDS: 3/4" Diameter Bolts, Single Shear, C_D = 1.60**
1.2 x ASD Bolt Z_{allow} = **3223 lbs** **1.2 Increase for Overstrength Forces per ASCE 7-10 12.4.3.3**

BEAM 1:
Seismic Lateral Load = 1,180 lbs
Wind Lateral Load = 0 lbs
Ωo = **3**
Vreq = 7,375 lbs
n_bolts_req'd = 2.29
n_bolt_min = **4** 3/4" dia. A307 Bolts

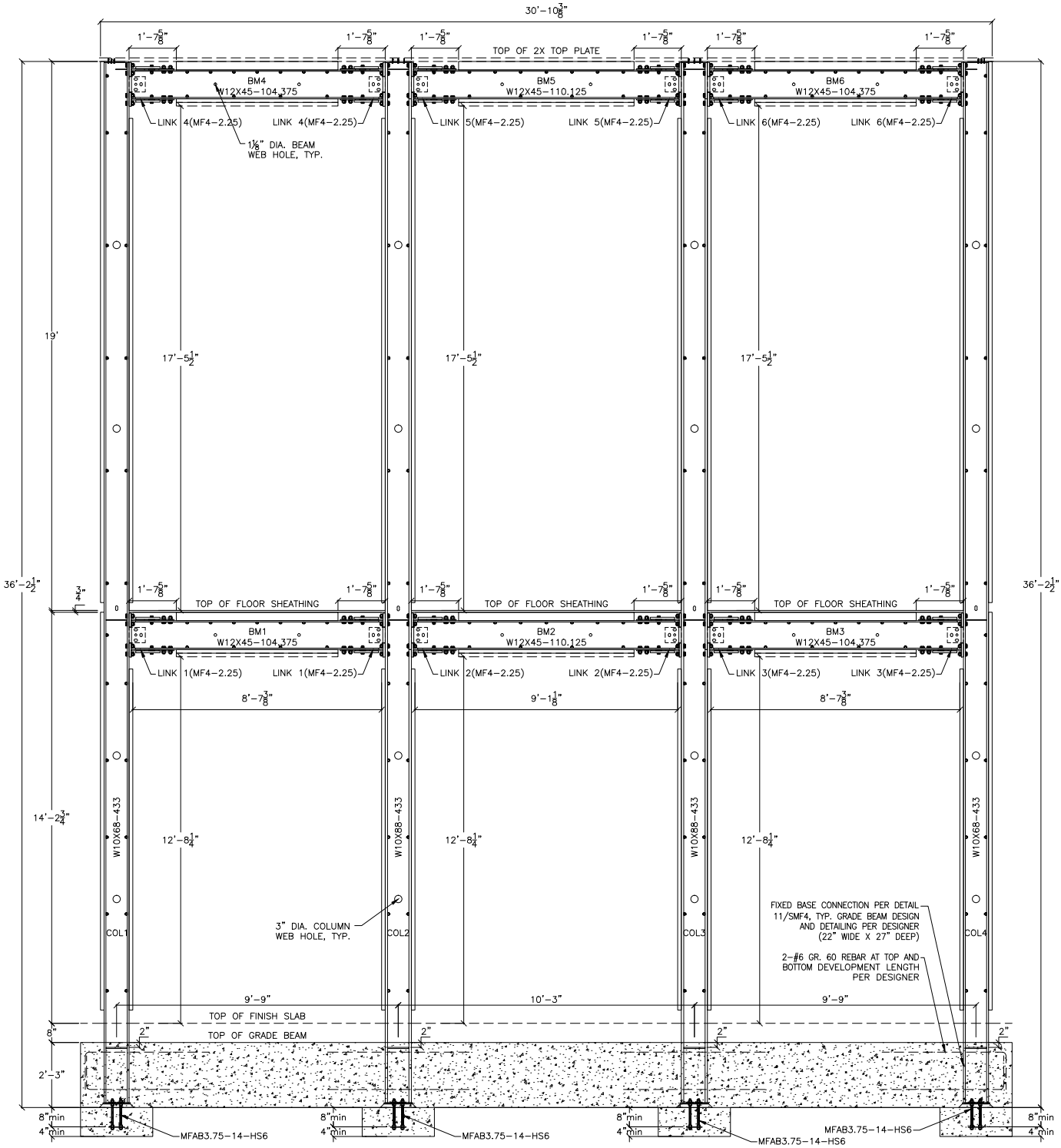
BEAM 2:
Seismic Lateral Load = 1,240 lbs
Wind Lateral Load = 0 lbs
Ωo = **3**
Vreq = 7,750 lbs
n_bolts_req'd = 2.40
n_bolt_min = **4** 3/4" dia. A307 Bolts

BEAM 3:
Seismic Lateral Load = 1,180 lbs
Wind Lateral Load = 0 lbs
Ωo = **3**
Vreq = 7,375 lbs
n_bolts_req'd = 2.29
n_bolt_min = **4** 3/4" dia. A307 Bolts

BEAM 4:
Seismic Lateral Load = 2,458 lbs
Wind Lateral Load = 0 lbs
Link = MF4-2.25 in
Lbm = 104.38 in
Lnailer = 56.125 in
S_bolt = 28.0625
Check = **OK**

BEAM 5:
Seismic Lateral Load = 2,583 lbs
Wind Lateral Load = 0 lbs
Link = MF4-2.25
Lbm = 110.13 in
Lnailer = 61.875 in
S_bolt = 30.9375 in
Check = **OK**

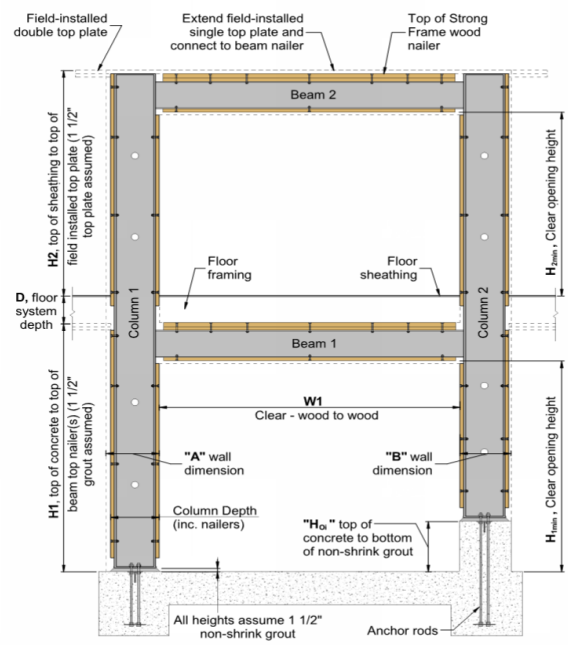
BEAM 6:
Seismic Lateral Load = 2,458 lbs
Wind Lateral Load = 0 lbs
Link = MF4-2.25
Lbm = 104.38 in
Lnailer = 56.125 in
S_bolt = 28.0625 in
Check = **OK**



Note: Footing sizes shown is based on anchorage design only. Actual footing/grade beam size and reinforcing must be determined by Designer based on project specific geometry and allowable soil pressures.

SIMPSON STRONG-TIE® COMPANY, INC.
The World's "No Equal" Structural Connector Company
5956 W. Las Positas Blvd. ● Pleasanton, California 94588
Telephone: (800) 999-5099 ● Fax: (925) 847-15977

SIMPSON STRONG-FRAME®
SPECIAL MOMENT FRAME
DESIGN CALCULATION PACKAGE



Frame Model: Custom 2-story x 1-bay SMF Frame (SMF-2X)

Frame ID: MF-2 (Grid 3)
Project Location: 6059 77th Ave SE, Mercer Island, WA 98040
Engineer of Record: Dibble Engineers, Inc.

Column1 Size:	W12X106	**Column2 Size:**	W12X106		
Beam 1 Size:	W12X65	**Beam 2 Size:**	W12X65		
Link 1 Size:	MF4-4	**Link 2 Size:**	MF4-4		
H1 =	163.8 in	**H2 =**	127.5 in	**HO1 =**	0.0 in
W1 =	115.6 in	**D =**	0.75	**HO2 =**	0.0 in

Package Contents:

- Design Summary
- SAP2000 Model Input
- SAP2000 beam and column design
- Moment Connection Design
- Column Base Plate Design
- Nailer Attachment Design

Design Date: 5/3/2019
Design by: B.Tran
Job No.: ES-191628

PHILLIP HUI
STATE OF WASHINGTON
55823
REGISTERED PROFESSIONAL ENGINEER
COA#3125
05/17/2019

Note: These calculations are only applicable to frames designed and manufactured by Simpson Strong-Tie Company Inc. Any other use of these calculations, or portion thereof, for purposes other than specifying frames manufactured by Simpson Strong-Tie is expressly prohibited.

Note: Signed & Sealed Calculations will be provided to the Building Department as a deferred submittal upon request. Signed & Sealed Calculations will not be issued until completion of a frame order from Simpson Strong-Tie Company Inc. Stamped but Unsigned calculations are provided for the plancheck review process. Stamped and Signed Calculations are required to be on file with the building department for final permitting.

Note: Simpson Strong-Tie® Strong Frame® and the Yield-Link™ structural fuse are protected under US patent number 8,001,734 B2 and must be supplied or licensed through Simpson Strong-Tie.

SIMPSON STRONG-TIE® COMPANY, INC.
The World's "No Equal" Structural Connector Company
5956 W. Las Positas Blvd. ● Pleasanton, California 94588
Telephone: (800) 999-5099 ● Fax: (925) 847-1597

Simpson Strong-Tie
Special Moment Frame Design Summary

Frame Model: Custom 2-story x 1-bay SMF Frame (SMF-2X)

Design Codes/References Used:

2015 International Building Code
AISC Steel Construction Manual - 14th Edition
Simpson Strong Tie Design Procedure for SST SMF

Material Design Properties:

Beam/Column Steel:	**ASTM A992**	**F_y = 50 ksi**
Plate Steel:	**ASTM A572 Grade 50**	**F_y = 50 ksi**
		(Link, Baseplate, cap plate, stiffener and shear plate)

High Strength Bolts:

Link-Stem-to beam:	**ASTM F2280 Twisted off bolts (ASTM A449 Equivalent), Pre-tensioned**
Link-flange-to column:	**ASTM A325 (Snug-tight)**
Shear plate-to-column:	**ASTM A325 (Snug-tight)**
Anchor Bolts:	**ASTM F1554 Gr 36, ASTM A36, or ASTM A449 (See Base Plate design)**
Weld Electrodes:	**E70XX**

Notes:

(1) Simpson Strong-Tie Strong Frame Moment Frames are designed using Load and Resistance Factored design (LRFD) methodology for determining frame drift and strength limits. Allowable Stress Design (ASD) shear and drift are determined as V(ASD) = 0.7 X V(LRFD) and Drift(ASD) = 0.7 X Drift(LRFD) for seismic load combinations and V(ASD) = V(LRFD) / 1.6 for wind load combinations.

(2) The following calculations are provided for justifying that Simpson Strong-Tie Strong Frame Moment Frame component capacities meet or exceed the design forces and criteria provided & determined by the Designer. Simpson Strong-Tie has not confirmed and is not responsible for any of the design, engineering, calculations, or derivation of the structural forces or drifts related to the building and the designed elements. Moment frames and other lateral resisting elements placed in the same lateral resisting shear line shall have applied lateral loads proportioned based on relative stiffness.

(3) This Simpson Strong-Tie Strong Frame Moment Frame is part of the overall lateral force resisting system of the structure. Design of the building's lateral force resisting system, including the load path to transfer lateral forces from the structure to the ground, is the responsibility of the designer. The designer must specify the required components of the complete load transfer path including diaphragms, shear transfer, chords and collectors and foundations.

(4) Footing dimensions provided are the minimums required for concrete anchorage requirements only. The designer must determine required footing size and reinforcing for other design limits, such as foundation shear and bending, soil bearing shear transfer, and frame stability / overturning. Designer shall detail actual footing / grade beam size and reinforcing.

(5) Holes in base plates are over-sized for erection tolerance. Designer must evaluate effects of over-sized holes and provide plate washer with standard-size holes welded to base plate or request base plates with standard-size holes where required.

(6) Refer to Strong Frame installation detail sheets for allowable field modifications and additions. Welding shall be in accordance with AWS D1.1 and AWS D1.8 (as applicable for seismic). Welds shall be specified by the designer. Provide welding special inspection as required by the local building department.

COMMENTS:

1. Project Information:

Job Name:	6059 77th Ave SE	Date:	5/3/2019
Project Address:	6059 77th Ave SE, Mercer Island, WA 98040	Engineer:	Dibble Engineers, Inc.
Frame ID:	MF-2 (Grid 3)	Phone:	425.828.4200 x237
SST Enginer:	**B.Tran**	E-mail:	Justin@DibbleEngineers.com

2. Frame Geometry:

Design Criteria:

Frame ID: SMF-2X12z12z12z-116.625x171.75x330-(MF4- | Custom Sizes

	ft	in	
Clear Opening Width(W1)=	9 ft	7.63 in	
Outside Width(W2)=	12 ft	3.38 in	
Top Plate Height(H1)=	13 ft	7.75 in	
Bottom Nailer Height(H1_min)=	0 ft	0.00 in	OK
Floor Depth (D)=	0 ft	0.75 in	
Top Plate Height(H2)=	10 ft	7.50 in	
Bottom Nailer Height(H2_min)=	0 ft	0.00 in	OK
Foundation offset (HO1)=	0 ft	0.00 in	
Foundation offset (HO2)=	0 ft	0.00 in	
A=	24.00 in	OK	
B=	24.00 in	OK	

Column=	W12X106
Beam2=	W12X65
Link2 ID=	MF4-4
Beam1=	W12X65
Link1 ID=	MF4-4
Shear tab 1	Heavy
Shear tab 2	Heavy

Design Code=	2015 IBC
S_{DS}=	0.976
R_load=	6.5
R_frame=	6.5
Cd=	4
Omega=	3
I=	1
Rho=	1
Seismic Drift Limit=	0.02 Hx
Wind Drift Limit:	hx/ 50

Beam Deflection Limits:

Live Load=	L/ 480
Dead + Live Load=	L/ 360
Snow/Wind Load=	L/ 360

Beam Deflection Limits (Simply Supported):

SS_beam Limit=	L/ 360

Base Fixity:	Fixed
Slab Height:	8.00 in
Min. Grade Beam Depth:	33.00 in

3. Loading:

3.1 Uniform Loads: (Negative value for uplift), 1st Floor Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	282 plf	0.00	10.96	Y	W_{LR1}=	0 plf	0.00	0.00	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	628 plf	0.00	10.96	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	0 plf	0.00	0.00	Y
					Snow Factor=	0.2			

3.2 Uniform Loads: (Negative value for uplift), 2nd Floor/Roof Beam

		Left End (ft)	RightEnd (ft)	To Rcc?			Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	95 plf	0.00	10.96	Y	W_{LR1}=	0 plf	0.00	0.00	Y
W_{DL2}=	0 plf	0.00	0.00	Y	W_{LR2}=	0 plf	0.00	0.00	Y
W_{DL3}=	0 plf	0.00	0.00	Y	W_{LL1}=	253 plf	0.00	10.96	Y
Rain Load=	0 plf	0.00	0.00	Y	W_{LL2}=	0 plf	0.00	0.00	Y
Wind Uplift=	0 plf	0.00	0.00	Y	Snow Load=	0 plf	0.00	0.00	Y
					Snow Factor=	0.2			

3.3: Point Loads:

Lateral (1st Flr Bm)

F_{EQ1}=	12200	lbs, ASD	Seismic Load	F_{WIND1}=	0	lbs, ASD	Wind Load
F_{EQ1}=	17429	lbs, LRFD	Seismic Load	F_{WIND1}=	0	lbs, LRFD	Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 1st Floor Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P1_1$ (lbs)=	0	0	0	0	0	0	0	X_1=	0.00 ft	N
$P1_2$ (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	N
$P1_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P1_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P1_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P1_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P1_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P1_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

Lateral (2nd Flr Bm)

F_{EQ2}=	12410	lbs, ASD	Seismic Load	F_{WIND2}=	0	lbs, ASD	Wind Load
F_{EQ2}=	17729	lbs, LRFD	Seismic Load	F_{WIND2}=	0	lbs, LRFD	Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate), 2nd Floor/Roof Beam***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
$P2_1$ (lbs)=	1200	0	0	0	0	0	0	X_1=	0.00 ft	N
$P2_2$ (lbs)=	1200	0	0	0	0	0	0	X_2=	10.96 ft	Y
$P2_3$ (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	N
$P2_4$ (lbs)=	0	0	0	0	0	0	0	X_4=	0.00 ft	N
$P2_5$ (lbs)=	0	0	0	0	0	0	0	X_5=	0.00 ft	N
$P2_6$ (lbs)=	0	0	0	0	0	0	0	X_6=	0.00 ft	N
$P2_7$ (lbs)=	0	0	0	0	0	0	0	X_7=	0.00 ft	N
$P2_8$ (lbs)=	0	0	0	0	0	0	0	X_8=	0.00 ft	N

2-STORY X 1-BAY SMF DESIGN SUMMARY:

		DCR Limit	Check
FRAME SEISMIC DRIFT (Hx1) DCR	0.797	1.03	OK
FRAME SEISMIC DRIFT (Hx2) DCR	0.952	1.03	OK
FRAME WIND DRIFT (Hx1) DCR	0.000	1.03	OK
FRAME WIND DRIFT (Hx2) DCR	0.000	1,03	OK

BEAM DELECTION CHECK:

1st Level Beam1	STEP 2: BEAM1 WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.043	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.035	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.043	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.000	1.00	OK
2nd Level Beam2	STEP 2: BEAM1 WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.018	1.00	OK
	PR FRAME: LIVE LOAD DEFLECTION DCR=	0.017	1.00	OK
	PR FRAME: LIVE + DEAD LOAD DEFLECTION DCR=	0.024	1.00	OK
	PR FRAME: WIND DEFLECTION DCR=	0.000	1.00	OK
	PR FRAME: SNOW DEFLECTION DCR=	0.000	1.00	OK

LINK CHECK:

1st Level Beam1	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.786	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.848	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.862	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.712	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.889	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.746	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.961	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.857	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.508	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.797	1.00	OK
2nd Level Beam2	STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.612	1.00	OK
	STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
	STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.672	1.00	OK
	8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.848	1.00	OK
	8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.862	1.00	OK
	8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.712	1.00	OK
	8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.889	1.00	OK
	8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.746	1.00	OK
	9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.961	1.00	OK
	9.2: DETERMINE LINK FLANGE THICKNESS	0.857	1.00	OK
	9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.508	1.00	OK
	11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.952	1.00	OK

COLUMN CHECK:

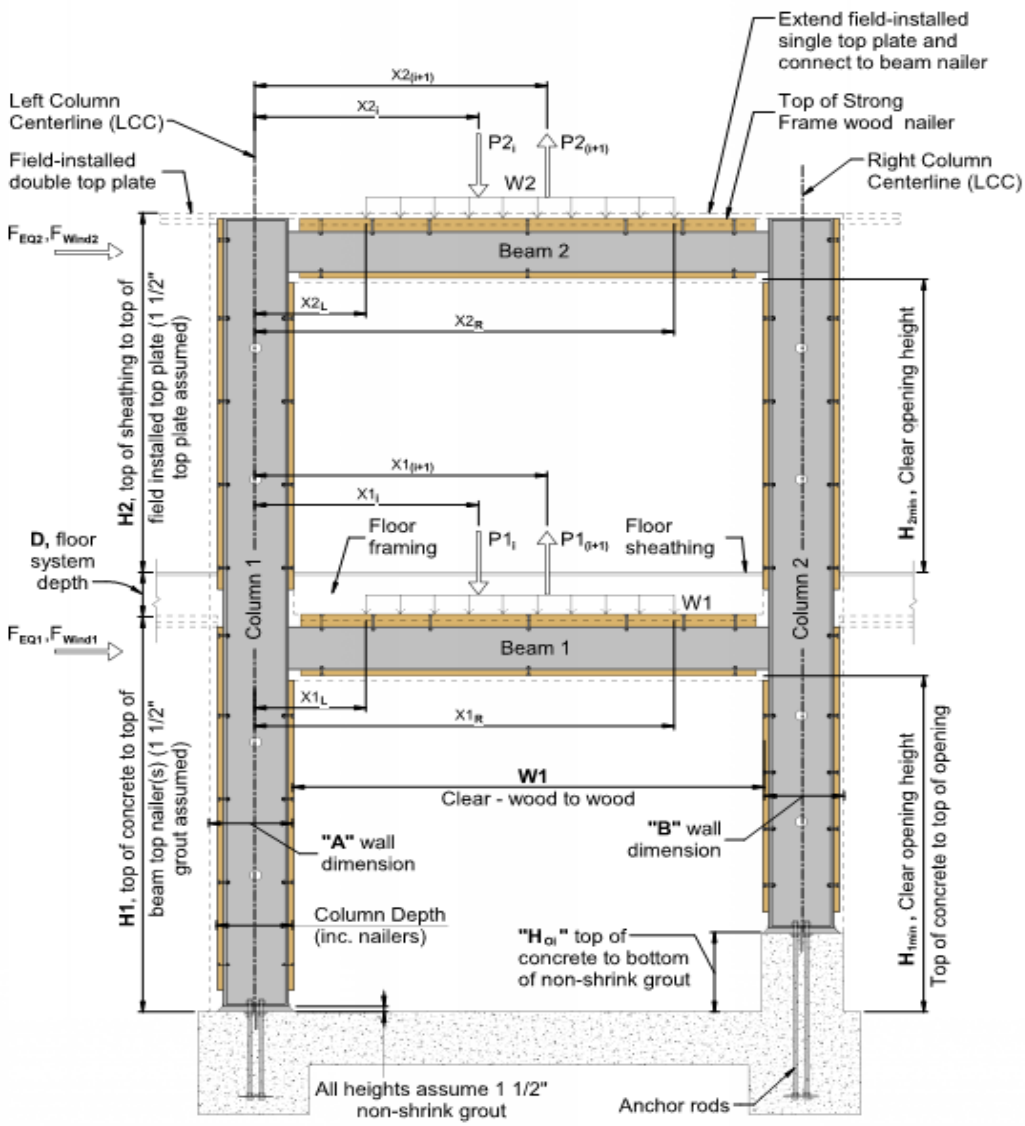
1st Level	LEFT COLUMN MAX DCR	0.945	1.00	OK
	RIGHT COLUMN MAX DCR	0.945	1.00	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.300	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.734	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.580	1.00	OK
2st Level	LEFT COLUMN MAX DCR	0.349	1.00	OK
	RIGHT COLUMN MAX DCR	0.349	1.00	OK
	STEP 14: STRONG COLUMN WEAK LINK CHECK	0.298	1.00	OK
	STEP 16: COLUMN PANEL ZONE CHECK	0.734	1.04	OK
	18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
	18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
	18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.000	1.00	OK
	18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.000	1.00	OK
	18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.580	1.00	OK
	18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.495	1.03	OK

BEAM CHECK:

1st Level Beam 1	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.047	1.00	OK
	PR FRAME BEAM MAX DCR=	0.618	1.00	OK
	BEAM FLANGE CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.557	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.444	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.774	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.689	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	1.000	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.394	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.308	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.319	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.431	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.257	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.280	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.292	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.228	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.464	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.284	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.518	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.518	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK
2nd Level Beam 2	SIMPLY SUPPORTED BEAM MAX STRENGTH DCR=	0.020	1.00	OK
	PR FRAME BEAM MAX DCR=	0.482	1.00	OK
	BEAM FLANGE CHECK	OK	-	OK
	15.1a: SHEAR PLATE BOLT SIZE	0.548	1.00	OK
	SHEAR PLATE GEOMETRY CHECK	OK	-	OK
	15.3a: SHEAR PLATE YIELDING	0.562	1.00	OK
	15.3b: SHEAR PLATE RUPTURE	0.979	1.00	OK
	15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.872	1.00	OK
	15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
	15.5: BEAM WEB BEARING X-DIR.	0.405	1.00	OK
	15.5: SHEAR TAB BEARING X-DIR.	0.421	1.00	OK
	15.5: BEAM WEB BEARING Y-DIR.	0.303	1.00	OK
	15.5: SHEAR TAB BEARING Y-DIR.	0.545	1.00	OK
	15.5: BEAM WEB BEARING X+Y	0.256	1.00	OK
	15.5: SHEAR TAB BEARING X+Y	0.475	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.300	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.312	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.440	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.360	1.00	OK
	15.5b: BEAM WEB BLOCKSHEAR X+Y	0.510	1.00	OK
	15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.372	1.00	OK
	15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK

FIXED BASE COLUMN WIDTH-TO-THICKNESS CHECK

bf/2*tf_columns=	6.17	7.22	OK
h/tw_columns=	15.9	55.40	OK

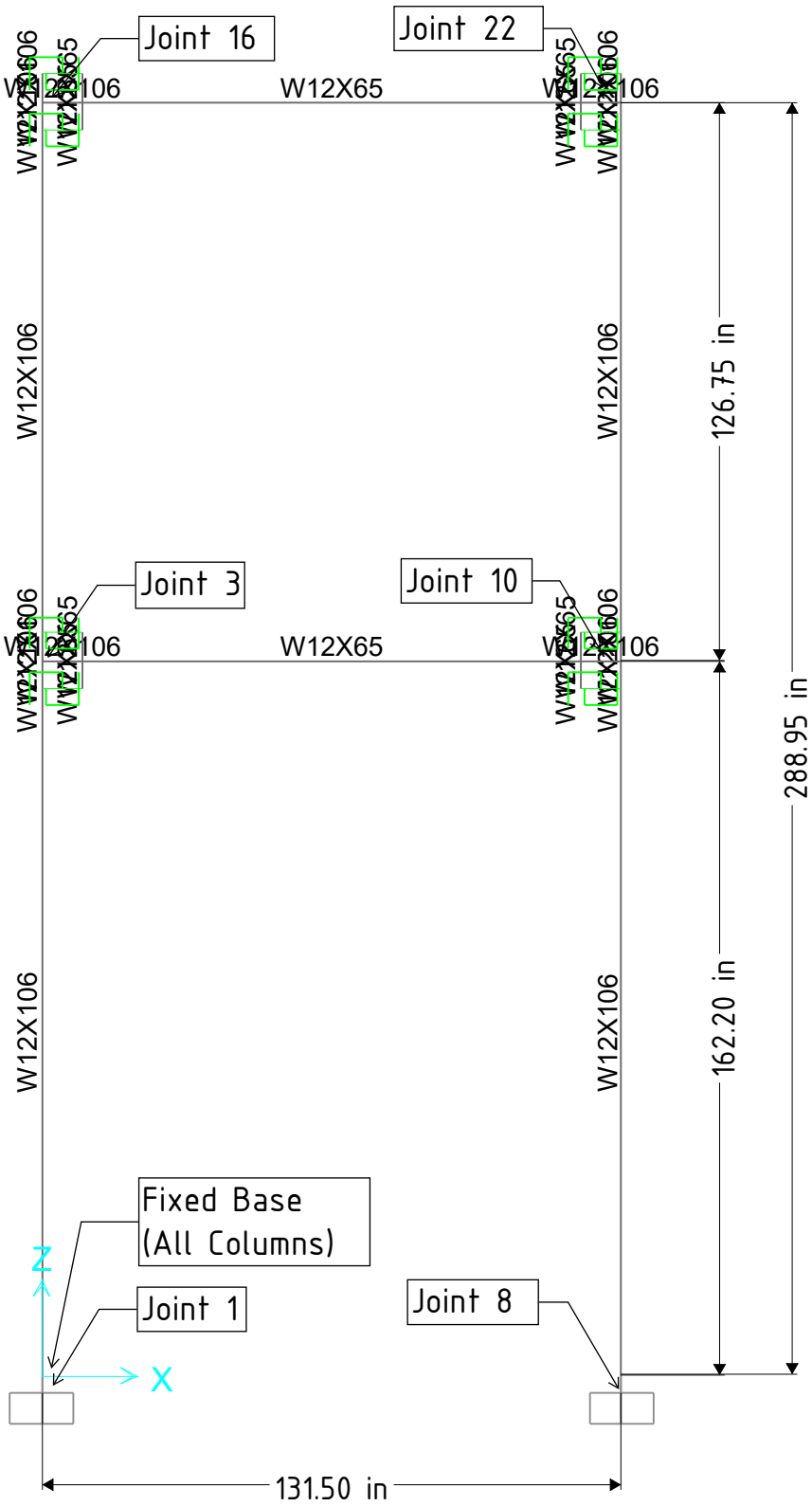


Note: Beam to column connection not shown for clarity. Figure is for illustration purposes only.
Final locations and quantities of nailer attachments and beam or column web holes/ openings may vary.

Code= 2015 IBC 2015 IBC
Sds= 0.976
Ω= 3 User input (2.5 or 3)
f_2= 0.2 Snow load factor
ρ= 1

Combo	Load Combinations	Load Combination Multipliers									
		D	L	Lr	S	R	W	E	Ev	Ni	
1	D	1	0	0	0	0	0	0	0	0	Load Cases
2	L	0	1	0	0	0	0	0	0	0	
3	Lr	0	0	1	0	0	0	0	0	0	
4	S	0	0	0	1	0	0	0	0	0	
5	R	0	0	0	0	1	0	0	0	0	
6	W	0	0	0	0	0	1	0	0	0	
7	E	0	0	0	0	0	0	1	1	0	
8	D	1	0	0	0	0	0	0	0	1	ASD
9	D+ L	1	1	0	0	0	0	0	0	1	
10	D + Lr	1	0	1	0	0	0	0	0	1	
11	D + S	1	0	0	1	0	0	0	0	1	
12	D + R	1	0	0	0	1	0	0	0	1	
13	D + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0	0	0	1	
14	D + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0	0	0	1	
15	D + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0	0	0	1	
16	D + 0.6 W	1	0	0	0	0	0.6	0	0	0	
17	D - 0.6 W	1	0	0	0	0	-0.6	0	0	0	
18	(1.0 + 0.14 S_{DS}) D + 0.7 E *ρ	1.13664	0	0	0	0	0	0.7	0.7000	0	
19	(1.0 + 0.14 S_{DS}) D - 0.7 E * ρ	1.13664	0	0	0	0	0	-0.7	-0.700	0	
20	D + 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0.45	0	0	0	
21	D - 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	-0.45	0	0	0	
22	D + 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0.45	0	0	0	
23	D - 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	-0.45	0	0	0	
24	D + 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0.45	0	0	0	
25	D - 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	-0.45	0	0	0	
26	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ+ 0.75 L + 0.75 Lr	1.10248	0.75	0.75	0	0	0	0.525	0.525	0	
27	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ+ 0.75 L + 0.75 Lr	1.10248	0.75	0.75	0	0	0	-0.525	-0.525	0	
28	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ + 0.75 L + 0.75 S	1.10248	0.75	0	0.75	0	0	0.5250	0.525	0	
29	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ + 0.75 L + 0.75 S	1.10248	0.75	0	0.75	0	0	-0.5250	-0.525	0	
30	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ + 0.75 L + 0.75 R	1.10248	0.75	0	0	0.75	0	0.5250	0.525	0	
31	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ + 0.75 L + 0.75 R	1.10248	0.75	0	0	0.75	0	-0.5250	-0.525	0	
32	0.6 D + 0.6 W	0.6	0	0	0	0	0.6	0	0	0	
33	0.6 D - 0.6 W	0.6	0	0	0	0	-0.6	0	0	0	
34	(0.6 - 0.14 S_{DS}) D + 0.7 E*ρ	0.46336	0	0	0	0	0	0.7	0.7000	0	
35	(0.6 - 0.14 S_{DS}) D - 0.7 E*ρ	0.46336	0	0	0	0	0	-0.7	-0.700	0	
36	1.4 D	1.4	0	0	0	0	0	0	0	1	LRFD
37	1.2 D + 1.6 L + 0.5 Lr	1.2	1.6	0.5	0	0	0	0	0	1	
38	1.2 D + 1.6 L + 0.5 S	1.2	1.6	0	0.5	0	0	0	0	1	
39	1.2 D + 1.6 L + 0.5 R	1.2	1.6	0	0	0.5	0	0	0	1	
40	1.2 D + 1.6 Lr + 0.5 L	1.2	0.5	1.6	0	0	0	0	0	1	
41	1.2 D + 1.6 Lr + 0.5 W	1.2	0	1.6	0	0	0.5	0	0	0	
42	1.2 D + 1.6 Lr - 0.5 W	1.2	0	1.6	0	0	-0.5	0	0	0	
43	1.2 D + 1.6 S + 0.5 L	1.2	0.5	0	1.6	0	0	0	0	1	
44	1.2 D + 1.6 S + 0.5 W	1.2	0	0	1.6	0	0.5	0	0	0	
45	1.2 D + 1.6 S - 0.5 W	1.2	0	0	1.6	0	-0.5	0	0	0	
46	1.2 D + 1.6 R + 0.5 L	1.2	0.5	0	0	1.6	0	0	0	1	
47	1.2 D + 1.6 R + 0.5 W	1.2	0	0	0	1.6	0.5	0	0	0	
48	1.2 D + 1.6 R - 0.5 W	1.2	0	0	0	1.6	-0.5	0	0	0	
49	1.2 D + 1 W + 0.5 L + 0.5 Lr	1.2	0.5	0.5	0	0	1	0	0	0	
50	1.2 D - 1 W + 0.5 L + 0.5 Lr	1.2	0.5	0.5	0	0	-1	0	0	0	
51	1.2 D + 1 W + 0.5 L + 0.5 S	1.2	0.5	0	0.5	0	1	0	0	0	
52	1.2 D - 1 W + 0.5 L + 0.5 S	1.2	0.5	0	0.5	0	-1	0	0	0	
53	1.2 D + 1 W + 0.5 L + 0.5 R	1.2	0.5	0	0	0.5	1	0	0	0	
54	1.2 D - 1 W + 0.5 L + 0.5 R	1.2	0.5	0	0	0.5	-1	0	0	0	
55	(1.2 + 0.2 S_{DS})D + E*ρ + 0.5 L + f2*S	1.3952	0.5	0	0.2	0	0	1	1.0000	0	
56	(1.2 + 0.2 S_{DS})D - E*ρ + 0.5 L + f2* S	1.3952	0.5	0	0.2	0	0	-1	-1.000	0	
57	0.9 D + 1 W	0.9	0	0	0	0	1	0	0	0	
58	0.9 D - 1 W	0.9	0	0	0	0	-1	0	0	0	
59	(0.9 - 0.2 S_{DS}) D + E*ρ	0.7048	0	0	0	0	0	1	1.0000	0	
60	(0.9 - 0.2 S_{DS}) D - E*ρ	0.7048	0	0	0	0	0	-1	-1.000	0	
61	(1.2 + 0.2 S_{DS})D + ΩE + 0.5L + f2*S	1.3952	0.5	0	0.2	0	0	3	3	0	
62	(1.2 + 0.2 S_{DS})D -ΩE + 0.5L + f2*S	1.3952	0.5	0	0.2	0	0	-3	-3	0	
63	(0.9 - 0.2 S_{DS}) D + ΩE	0.7048	0	0	0	0	0	3	3	0	
64	(0.9 - 0.2 S_{DS}) D - ΩE	0.7048	0	0	0	0	0	-3	-3	0	

Frame Input


Joint 16
Joint 22
W12X65
W12X106
W12X106
126.75 in
Joint 3
Joint 10
W12X65
W12X106
W12X106
288.95 in
162.20 in
Fixed Base
(All Columns)
Joint 1
Joint 8
Z
X
131.50 in

Joint Displacement (in) (1st Floor)											
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Seismic Horizontal	Wind Horizontal
3	LCM1	0.000	0.000	-0.001	10	LCM1	0.000	0.000	-0.001		
3	LCM2	0.000	0.000	-0.001	10	LCM2	0.000	0.000	-0.001		
3	LCM3	0.000	0.000	0.000	10	LCM3	0.000	0.000	0.000		
3	LCM4	0.000	0.000	0.000	10	LCM4	0.000	0.000	0.000		
3	LCM5	0.000	0.000	0.000	10	LCM5	0.000	0.000	0.000		
3	LCM6	0.000	0.000	0.000	10	LCM6	0.000	0.000	0.000		
3	LCM7	0.646	0.000	0.005	10	LCM7	0.646	0.000	-0.005		
3	LCM8	0.019	0.000	-0.001	10	LCM8	0.019	0.000	-0.001		
3	LCM9	0.019	0.000	-0.002	10	LCM9	0.019	0.000	-0.002		
3	LCM10	0.019	0.000	-0.001	10	LCM10	0.019	0.000	-0.001		
3	LCM11	0.019	0.000	-0.001	10	LCM11	0.019	0.000	-0.001		
3	LCM12	0.019	0.000	-0.001	10	LCM12	0.019	0.000	-0.001		
3	LCM13	0.019	0.000	-0.002	10	LCM13	0.019	0.000	-0.002		
3	LCM14	0.019	0.000	-0.002	10	LCM14	0.019	0.000	-0.002		
3	LCM15	0.019	0.000	-0.002	10	LCM15	0.019	0.000	-0.002		
3	LCM16	0.000	0.000	-0.001	10	LCM16	0.000	0.000	-0.001		
3	LCM17	0.000	0.000	-0.001	10	LCM17	0.000	0.000	-0.001		
3	LCM18	0.452	0.000	0.002	10	LCM18	0.452	0.000	-0.005		
3	LCM19	-0.452	0.000	-0.005	10	LCM19	-0.452	0.000	0.002		
3	LCM20	0.000	0.000	-0.002	10	LCM20	0.000	0.000	-0.002		0.000
3	LCM21	0.000	0.000	-0.002	10	LCM21	0.000	0.000	-0.002		0.000
3	LCM22	0.000	0.000	-0.002	10	LCM22	0.000	0.000	-0.002		0.000
3	LCM23	0.000	0.000	-0.002	10	LCM23	0.000	0.000	-0.002		0.000
3	LCM24	0.000	0.000	-0.002	10	LCM24	0.000	0.000	-0.002		0.000
3	LCM25	0.000	0.000	-0.002	10	LCM25	0.000	0.000	-0.002		0.000
3	LCM26	0.339	0.000	0.001	10	LCM26	0.339	0.000	-0.004		
3	LCM27	-0.339	0.000	-0.004	10	LCM27	-0.339	0.000	0.001		
3	LCM28	0.339	0.000	0.001	10	LCM28	0.339	0.000	-0.004		
3	LCM29	-0.339	0.000	-0.004	10	LCM29	-0.339	0.000	0.001		
3	LCM30	0.339	0.000	0.001	10	LCM30	0.339	0.000	-0.004		
3	LCM31	-0.339	0.000	-0.004	10	LCM31	-0.339	0.000	0.001		
3	LCM32	0.000	0.000	-0.001	10	LCM32	0.000	0.000	-0.001		
3	LCM33	0.000	0.000	-0.001	10	LCM33	0.000	0.000	-0.001		
3	LCM34	0.452	0.000	0.003	10	LCM34	0.452	0.000	-0.004		
3	LCM35	-0.452	0.000	-0.004	10	LCM35	-0.452	0.000	0.003		
3	LCM36	0.019	0.000	-0.001	10	LCM36	0.019	0.000	-0.002		
3	LCM37	0.019	0.000	-0.003	10	LCM37	0.019	0.000	-0.003		
3	LCM38	0.019	0.000	-0.003	10	LCM38	0.019	0.000	-0.003		
3	LCM39	0.019	0.000	-0.003	10	LCM39	0.019	0.000	-0.003		
3	LCM40	0.019	0.000	-0.002	10	LCM40	0.019	0.000	-0.002		
3	LCM41	0.000	0.000	-0.001	10	LCM41	0.000	0.000	-0.001		
3	LCM42	0.000	0.000	-0.001	10	LCM42	0.000	0.000	-0.001		
3	LCM43	0.019	0.000	-0.002	10	LCM43	0.019	0.000	-0.002		
3	LCM44	0.000	0.000	-0.001	10	LCM44	0.000	0.000	-0.001		
3	LCM45	0.000	0.000	-0.001	10	LCM45	0.000	0.000	-0.001		
3	LCM46	0.019	0.000	-0.002	10	LCM46	0.019	0.000	-0.002		
3	LCM47	0.000	0.000	-0.001	10	LCM47	0.000	0.000	-0.001		
3	LCM48	0.000	0.000	-0.001	10	LCM48	0.000	0.000	-0.001		
3	LCM49	0.000	0.000	-0.002	10	LCM49	0.000	0.000	-0.002		
3	LCM50	0.000	0.000	-0.002	10	LCM50	0.000	0.000	-0.002		
3	LCM51	0.000	0.000	-0.002	10	LCM51	0.000	0.000	-0.002		
3	LCM52	0.000	0.000	-0.002	10	LCM52	0.000	0.000	-0.002		
3	LCM53	0.000	0.000	-0.002	10	LCM53	0.000	0.000	-0.002		
3	LCM54	0.000	0.000	-0.002	10	LCM54	0.000	0.000	-0.002		
3	LCM55	0.646	0.000	0.003	10	LCM55	0.646	0.000	-0.007	0.646	
3	LCM56	-0.646	0.000	-0.007	10	LCM56	-0.646	0.000	0.003	-0.646	
3	LCM57	0.000	0.000	-0.001	10	LCM57	0.000	0.000	-0.001		
3	LCM58	0.000	0.000	-0.001	10	LCM58	0.000	0.000	-0.001		
3	LCM59	0.646	0.000	0.004	10	LCM59	0.646	0.000	-0.006	0.646	
3	LCM60	-0.646	0.000	-0.006	10	LCM60	-0.646	0.000	0.004	-0.646	
3	LCM61	1.938	0.000	0.012	10	LCM61	1.938	0.000	-0.016		
3	LCM62	-1.938	0.000	-0.016	10	LCM62	-1.938	0.000	0.012		
3	LCM63	1.938	0.000	0.014	10	LCM63	1.938	0.000	-0.015		
3	LCM64	-1.938	0.000	-0.015	10	LCM64	-1.938	0.000	0.014		

Joint Displacement (in) (1st Floor)											
Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Joint	Combo	U1 (X) Horizontal	U2 (Y)	U3 (Z) Vertical	Seismic Horizontal	Wind Horizontal
16	LCM1	0.000	0.000	-0.001	22	LCM1	0.000	0.000	-0.001		
16	LCM2	0.000	0.000	-0.001	22	LCM2	0.000	0.000	-0.001		
16	LCM3	0.000	0.000	0.000	22	LCM3	0.000	0.000	0.000		
16	LCM4	0.000	0.000	0.000	22	LCM4	0.000	0.000	0.000		
16	LCM5	0.000	0.000	0.000	22	LCM5	0.000	0.000	0.000		
16	LCM6	0.000	0.000	0.000	22	LCM6	0.000	0.000	0.000		
16	LCM7	1.250	0.000	0.006	22	LCM7	1.250	0.000	-0.006		
16	LCM8	0.037	0.000	-0.001	22	LCM8	0.037	0.000	-0.002		
16	LCM9	0.037	0.000	-0.002	22	LCM9	0.037	0.000	-0.003		
16	LCM10	0.037	0.000	-0.001	22	LCM10	0.037	0.000	-0.002		
16	LCM11	0.037	0.000	-0.001	22	LCM11	0.037	0.000	-0.002		
16	LCM12	0.037	0.000	-0.001	22	LCM12	0.037	0.000	-0.002		
16	LCM13	0.037	0.000	-0.002	22	LCM13	0.037	0.000	-0.002		
16	LCM14	0.037	0.000	-0.002	22	LCM14	0.037	0.000	-0.002		
16	LCM15	0.037	0.000	-0.002	22	LCM15	0.037	0.000	-0.002		
16	LCM16	0.000	0.000	-0.001	22	LCM16	0.000	0.000	-0.001		
16	LCM17	0.000	0.000	-0.001	22	LCM17	0.000	0.000	-0.001		
16	LCM18	0.875	0.000	0.003	22	LCM18	0.875	0.000	-0.006		
16	LCM19	-0.875	0.000	-0.006	22	LCM19	-0.875	0.000	0.003		
16	LCM20	0.000	0.000	-0.002	22	LCM20	0.000	0.000	-0.002		0.000
16	LCM21	0.000	0.000	-0.002	22	LCM21	0.000	0.000	-0.002		0.000
16	LCM22	0.000	0.000	-0.002	22	LCM22	0.000	0.000	-0.002		0.000
16	LCM23	0.000	0.000	-0.002	22	LCM23	0.000	0.000	-0.002		0.000
16	LCM24	0.000	0.000	-0.002	22	LCM24	0.000	0.000	-0.002		0.000
16	LCM25	0.000	0.000	-0.002	22	LCM25	0.000	0.000	-0.002		0.000
16	LCM26	0.656	0.000	0.001	22	LCM26	0.656	0.000	-0.006		
16	LCM27	-0.656	0.000	-0.006	22	LCM27	-0.656	0.000	0.001		
16	LCM28	0.656	0.000	0.001	22	LCM28	0.656	0.000	-0.006		
16	LCM29	-0.656	0.000	-0.006	22	LCM29	-0.656	0.000	0.001		
16	LCM30	0.656	0.000	0.001	22	LCM30	0.656	0.000	-0.006		
16	LCM31	-0.656	0.000	-0.006	22	LCM31	-0.656	0.000	0.001		
16	LCM32	0.000	0.000	-0.001	22	LCM32	0.000	0.000	-0.001		
16	LCM33	0.000	0.000	-0.001	22	LCM33	0.000	0.000	-0.001		
16	LCM34	0.875	0.000	0.004	22	LCM34	0.875	0.000	-0.005		
16	LCM35	-0.875	0.000	-0.005	22	LCM35	-0.875	0.000	0.004		
16	LCM36	0.037	0.000	-0.002	22	LCM36	0.037	0.000	-0.002		
16	LCM37	0.037	0.000	-0.003	22	LCM37	0.037	0.000	-0.004		
16	LCM38	0.037	0.000	-0.003	22	LCM38	0.037	0.000	-0.004		
16	LCM39	0.037	0.000	-0.003	22	LCM39	0.037	0.000	-0.004		
16	LCM40	0.037	0.000	-0.002	22	LCM40	0.037	0.000	-0.002		
16	LCM41	0.000	0.000	-0.002	22	LCM41	0.000	0.000	-0.002		
16	LCM42	0.000	0.000	-0.002	22	LCM42	0.000	0.000	-0.002		
16	LCM43	0.037	0.000	-0.002	22	LCM43	0.037	0.000	-0.002		
16	LCM44	0.000	0.000	-0.002	22	LCM44	0.000	0.000	-0.002		
16	LCM45	0.000	0.000	-0.002	22	LCM45	0.000	0.000	-0.002		
16	LCM46	0.037	0.000	-0.002	22	LCM46	0.037	0.000	-0.002		
16	LCM47	0.000	0.000	-0.002	22	LCM47	0.000	0.000	-0.002		
16	LCM48	0.000	0.000	-0.002	22	LCM48	0.000	0.000	-0.002		
16	LCM49	0.000	0.000	-0.002	22	LCM49	0.000	0.000	-0.002		
16	LCM50	0.000	0.000	-0.002	22	LCM50	0.000	0.000	-0.002		
16	LCM51	0.000	0.000	-0.002	22	LCM51	0.000	0.000	-0.002		
16	LCM52	0.000	0.000	-0.002	22	LCM52	0.000	0.000	-0.002		
16	LCM53	0.000	0.000	-0.002	22	LCM53	0.000	0.000	-0.002		
16	LCM54	0.000	0.000	-0.002	22	LCM54	0.000	0.000	-0.002		
16	LCM55	1.250	0.000	0.004	22	LCM55	1.249	0.000	-0.009	1.250	
16	LCM56	-1.249	0.000	-0.009	22	LCM56	-1.250	0.000	0.004	-1.250	
16	LCM57	0.000	0.000	-0.001	22	LCM57	0.000	0.000	-0.001		
16	LCM58	0.000	0.000	-0.001	22	LCM58	0.000	0.000	-0.001		
16	LCM59	1.250	0.000	0.005	22	LCM59	1.249	0.000	-0.007	1.250	
16	LCM60	-1.249	0.000	-0.007	22	LCM60	-1.250	0.000	0.005	-1.250	
16	LCM61	3.749	0.000	0.017	22	LCM61	3.748	0.000	-0.022		
16	LCM62	-3.748	0.000	-0.022	22	LCM62	-3.749	0.000	0.017		
16	LCM63	3.749	0.000	0.018	22	LCM63	3.749	0.000	-0.020		
16	LCM64	-3.749	0.000	-0.020	22	LCM64	-3.749	0.000	0.018		

STEP 1: Beam and Column Sizes

Note: Slightly Modified from Design Procedure, uses PR connection instead of FR connection

Beam1 Size:	W12X65		**Beam2 Size:**	W12X65	
d=	12.1	in	d=	12.1	in
bbf=	12	in	bbf=	12	in
tbf=	0.605	in	tbf=	0.605	in
tbw=	0.39	in	tbw=	0.39	in
Fy_bm=	50	ksi	Fy_bm=	50	ksi
Fu_bm=	65	ksi	Fu_bm=	65	ksi
Column Size:	W12X106		**Link1 Size:**	MF4-4	
dc=	12.9	in			
bcf=	12.2	in	**Link2 Size:**	MF4-4	
tcf=	0.99	in			
tcw=	0.61	in			
Agc=	31.2	in^2			
Fy_col=	50	ksi			
Fu_col=	65	ksi			

Seismic Drift Check:

Hcc1_aver.=	162.2	in	Hcc2=	126.75	in
R_frame=	6.5		R_frame=	6.5	
Cd=	4		Cd=	4	
Allowable Drift=	0.02	Hx	Allowable Drift=	0.02	Hx
I=	1		I=	1	
Δ_{allow}=	0.811	in	Δ_{allow}=	0.634	in
Δs_PR=	**0.646**	in	Δs_PR=	**0.603**	in
DCR_SeismicDrift=	0.797	OK	DCR_SeismicDrift=	0.952	OK

Wind Drift Check:

Allowable Drift= Hx/	50		Allowable Drift= Hx/	50	
Allowable Drift=	3.244	in	Allowable Drift=	2.535	in
Δ_PR1=	0.000	in	Δ_PR2=	0.000	in
DCR_WindDrift=	0.000	OK	DCR_WindDrift=	0.000	OK

SAP2000 Axial Spring Parameters:

Axial Force vs. link displacement relationship

Link 1 Points	**Force** kips	**Displ.** in
1	-156.000	-0.441
2	-156.000	-0.252
3	-110.000	-0.023
4	0.000	0.000
5	110.000	0.023
6	156.000	0.252
7	156.000	0.441

Link2 Points	**Force** kips	**Displ.** in
1	-156.000	-0.441
2	-156.000	-0.252
3	-110.000	-0.023
4	0.000	0.000
5	110.000	0.023
6	156.000	0.252
7	156.000	0.441

STEP 2: Check Beam with Simply Supported Connections in SAP2000

Lcc=	131.525	in	*Column center line to center line*
a=	2.75	in	*Distance from column face to center hole of beam*
Lh=	113.125	in	=Lcc - dc - 2*a

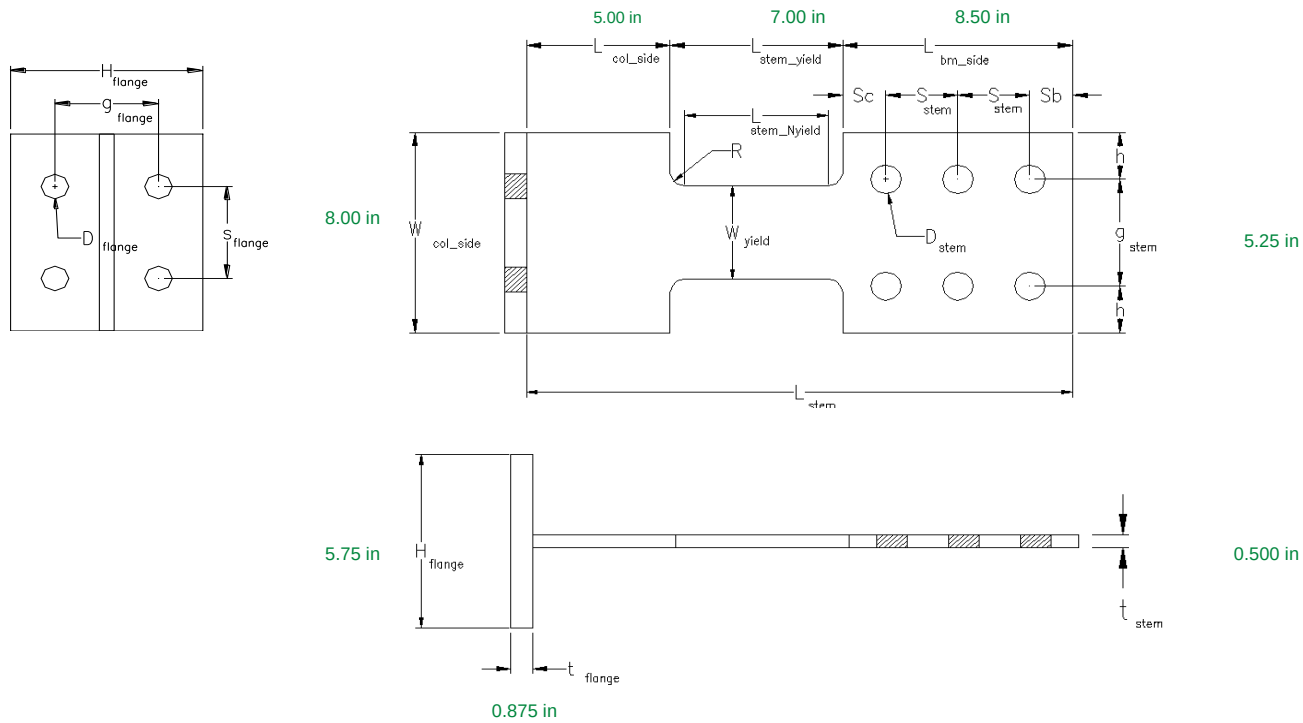
Beam 1

Delta_ss1=	0.01	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.314	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.043	OK	*=Delta_ss/ Delta_allow*

Beam 2

Delta_ss2=	0.01	in	Absolute max beam deflection for LCM 36-40
Delta_allow=	0.314	in	
DCR_Delta_ss=	0.018	OK	

SMF Link Design/Check (For Beam 1)



LINK1 ID MF4-4

Beam1= W12X65 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	8.00	in
NY Length BeamSide (Lbm_side)=	8.50	in	Central Neck Yield Width (w_stemYield)=	4.000	in
L_stem=	20.50	in	NY Width BeamSide (Wbm_side)=	8.00	in
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	2.000	in^2 =t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	6.00		Gauge Along Width (bolt_g_stem)=	5.25	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.750	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	8.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	5.25	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.563	in			

LINK MATERIAL:

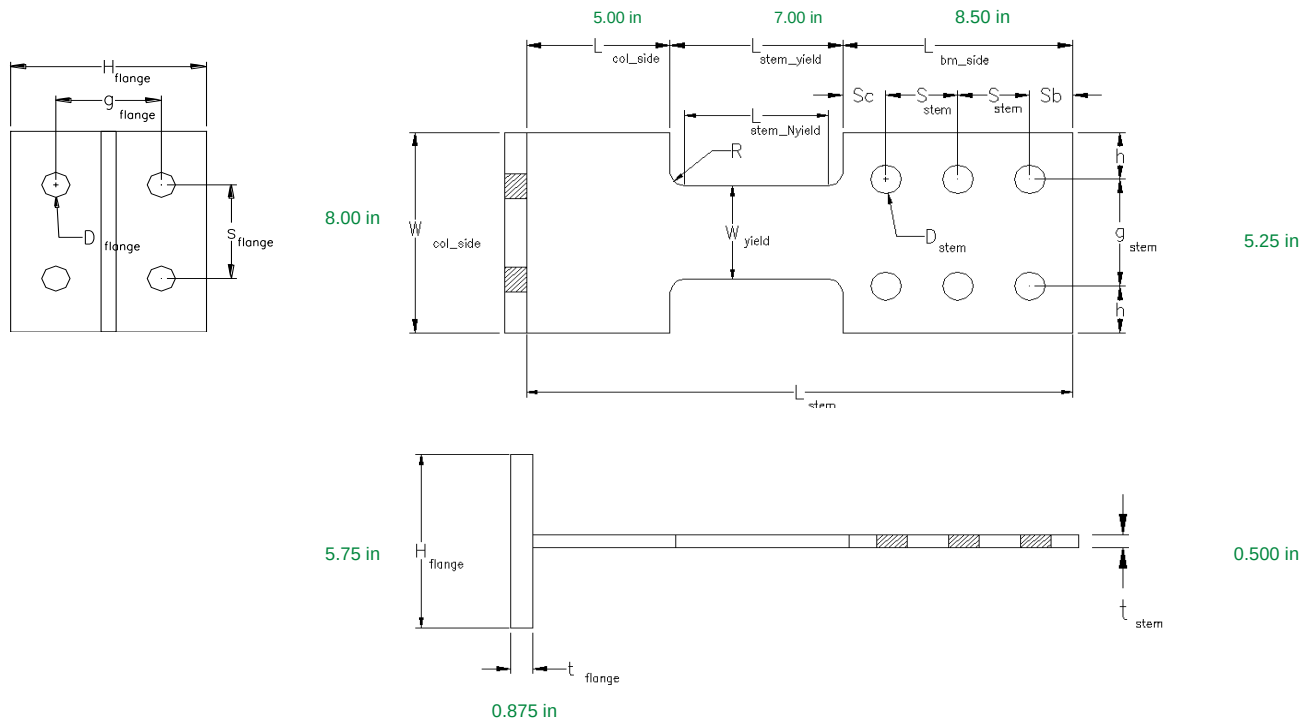
Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	2.000	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	110	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	2.200	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	1.540	kips	*=0.7*Bracing Force (LRFD) at Top of column*

SMF Link Design/Check (For Beam 2)



LINK2 ID MF4-4

Beam2= W12X65 db= 12.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in	
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	8.00	in	
NY Length BeamSide (Lbm_side)=	8.50	in	Central Neck Yield Width (w_stemYield)=	4.000	in	
L_stem=	20.50	in	NY Width BeamSide (Wbm_side)=	8.00	in	
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	2.000	in^2	=t_stem * w_stemYield

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	6.00		Gauge Along Width (bolt_g_stem)=	5.25	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt Length =	2.75	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	8.00	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Length (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Width (horizontal) (bolt_s_flange)=	5.25	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt Length =	3.5625	in			

LINK MATERIAL:

Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Step 13.1: Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

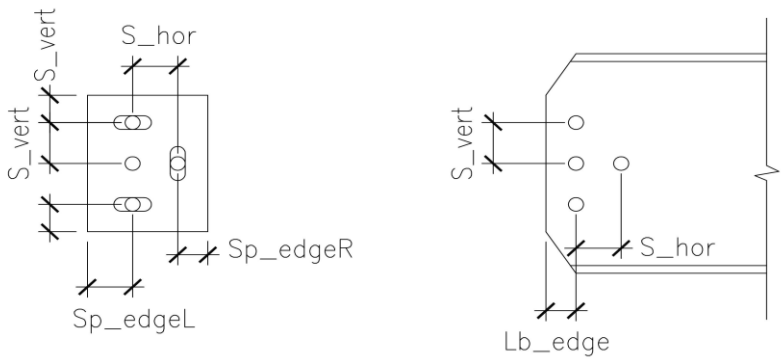
A_link=	2.000	in^2	*Link Yielding Area*
Fy_link=	50.000	ksi	*Link Yield stress*
Ry=	1.100		*Ratio of expected yield to minimum yield*
Pye_link=	110	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	2.200	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	1.540	kips	*=0.7*Bracing Force (LRFD) at Top of column*

STEP 15: DESIGN BEAM WEB TO COLUMN FLANGE CONNECTION

15.1a: SHEAR PLATE BOLT SIZE

Beam Section (bmSize) =	W12X65		*Value previously defined*
Beam Depth (db)=	12.10	in	*Value previously defined*
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Fy_bm=	50	ksi	
Fu_bm =	65	ksi	
Axial Load (Pu_SST)=	22.77	kips	*Max axial force per LCM61-64 from SAP_BeamReactions, SAP output*
Vertical Load (Vu_bm)=	38.19	kips	*Value previously defined from Step 12*
Vu_bolt=	17.1	kips	*= Sqrt ((Pu_SST/ n_Hbolt_SST)^2 + (Vu_bm/ n_Vbolt_SST)^2)*
ϕbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	A325-X		*[A325-X default]*
Bolt Dia (db_sp)=	7/8	in	
No of vertical Bolts (n_Vbolts_SST)=	3		
No of horizontal Bolts (n_Hbolts_SST)=	2		
Fnv_A325N=	68	ksi	*= if (Bolt Type = "A490-X", 84, 68)*
Anb_sp=	0.60	in^2	*= Pi* (db_sp/2)^2*
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*
ϕRn_stBolt_Shear=	30.7	kips	*= Rn_stBolt_shear* ϕbolt*
DCR_shearTabl_Cbolt=	0.557	OK	*= Vu_bolt/ ϕRn_stBolt_shear*
			OK if DCR_shearTabl_Cbolt <= 1.05

15.2: SHEAR PLATE GEOMETRY



a=	2.75	in	*Value previously defined*
tsp=	0.5	in	*[0.375 default]*
db =	12.1	in	*Value previously defined*
h_flange =	5.75	in	*Value previously defined*
t_stem =	0.5	in	*Value previously defined*
h_clear =	0.25	in	*minimum clearance between link flange and shear tab*
Bolt Spacing (S_min)=	2.33	in	*= 2.6667* max (db_sp)*
Bolt Spacing (Svert)=	2.19	in	OK *looked up value*
Bolt Spacing (Shorz)=	2.19	in	OK *looked up value*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lv_sp=	1	in	Need to Check J3.10 and J4, see Section 15.5 below
h_sp max =	6.35	in	
Plate Depth (h_sp)=	6.375	in	OK *= Lv_sp* 2 + Svert* 2*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lh_sp=	1.75	in	OK
Plate Width (W_shearTab) =	6.69	in	*= Lh_sp + a + Shorz*
Lslot_min=	1.30625	in	*= db_sp + 1/8 + 0.14* Svert*
LslotH = LslotV =	1.50	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	**OK**		*OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)*

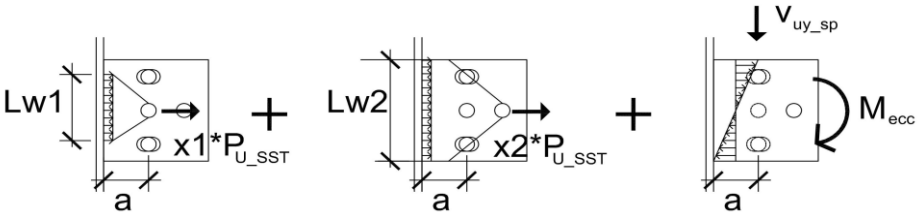
15.3a: SHEAR PLATE YIELDING (VERTICAL)

ϕyield=	0.9		
dHole_sp=	1.00	in	*= db_sp + 1/8*
Asp_Agv=	3.188	in^2	*= tsp* h_sp*
ϕVy_sp=	86.06	kips	*= ϕyield* 0.6* Asp_Agv* Fy_sp*
DCR_spYield=	0.444	OK	*= Vu_bm/ ϕVy_sp*
			OK if DCR_spYield <= DCR_allowed

15.3b: SHEAR PLATE RUPTURE (VERTICAL)

ϕrupture=	0.75		
Asp_nv=	1.69	in^2	*= hsp* tsp - dHole_sp* n_Vbolts_SST* tsp*
ϕVrupture_sp=	49.36	kips	*= ϕrupture* 0.6* Fu_shearTab* Asp_nv*
DCR_spRupture=	0.774	OK	*= Vu_bm/ ϕVrupture_sp*

15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT



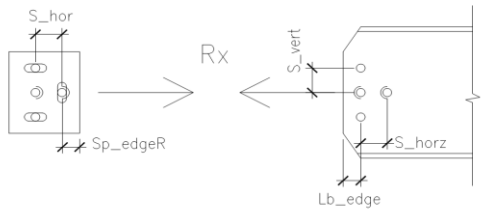
ϕb=	0.90		
a=	2.75	in	*Value previously defined*
Vuy=	38.19	kips	*= Vu_bm*
Mecc=	105.03	kips*in	*= Vuy* a*
x1 =	0.50		*assume 1/2 of axial is resisted by each bolt*
x2 =	0.50		*=1-x1*
θWhitmore=	30	deg	
Lwhitmore1 =	4.05	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore1 =	2.03	in^2	*= Lwhitmore1* tsp*
Lwhitmore2 =	6.38	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore2 =	3.19	in^2	*= Lwhitmore2* tsp*
Ssp=	3.39	in^3	*= tsp* hsp^2/ 6*
Isp=	10.80	in^4	*= tsp* hsp^3/ 12*
fb1=	31.01	ksi	*= Mecc/ Ssp*
yb=	2.03	in	*= Lwhitmore1/ 2*
fb2=	19.70	ksi	*= Mecc* yb/ Isp*
fa2=	9.20	ksi	*= x1*Pu_SST/ Awhitmore1 + x2*Pu_SST/ Awhitmore2*
ftot2=	28.90	ksi	*= fa2 + fb2*
fmax_sp=	31.01	ksi	*= max (fb1, ftot2)*
ϕb*Fy_sp=	45.00	ksi	*= ϕb* Fy_shearTab*
DCR_sp=	0.689	OK	*= fmax_sp/ (ϕb* Fy_shearTab), OK if DCR_sp <= DCR_Allowed*

15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD

tsp=	0.500	in	*value previously defined*
tw_sp_min=	0.3125	in	*= 5/8*tsp Fillet size to develop plate, Per AISC Steel Manual 14th edition*
tw_sp=	**0.3125**	in	*Each Side [0.25 default] , page 10-102*
DCR_spWeld=	1.000	OK	*= tw_sp_min/ tw_sp, OK if DCR_spWeld <= DCR_Allowed*

15.5: BEAM WEB AND SHEAR TAB BEARING

CASE 1: HORIZONTAL REACTIONS



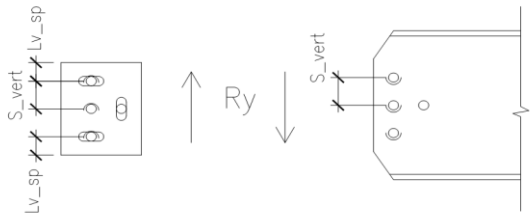
BEAM WEB:

ϕbolt=	0.75		
tbw=	0.39	in	*value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lc_bmWeb1 =	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
Lc_bmWeb2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_bmWeb=	2.531	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
ϕRn_beamWeb1=	57.75	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb*
ϕRn_beamWeb2=	79.85	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb *n_Hbolt*
ϕRn_beamWeb=	57.75	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.394	OK	*= Pu_SST/ ϕRn_beamWeb, OK if DCR_bmWebX <= DCR_Allowed*

SHEAR PLATE:

tsp=	0.5	in	*value previously defined*
Sp_edgeR =	1.75	in	*Edge distance for shear tab bolt hole*
Lc_sp1 =	1.281	in	*= Sp_edgeR - (db_sp + 1/16)* 0.5*
Lc_sp2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_sp=	2.531	in	*= Lc_sp1 + Lc_sp2*
ϕRn_sp1=	74.04	kips	*= ϕbolt* 1.2* Lc_sp* tbw* Fu_bmWeb*
ϕRn_sp2=	102.38	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕRn_sp=	74.04	kips	*= min (ϕRn_sp1, ϕRn_sp2)*
DCR_spX=	0.308	OK	*= Pu_SST/ ϕRn_sp, OK if DCR_spX <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



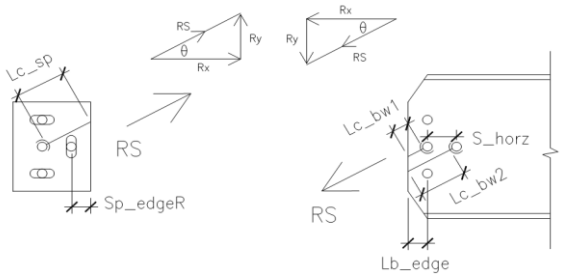
BEAM WEB:

Lc1=	**1.250**	in	*= Svert - (db_sp + 1/16)*
Lc2=	1.250	in	*= Lc1*
Lc3=	3.258	in	*= (db - 2Svert - 2tbf)*0.5*
ϕ*Rn_bmWebY1=	131.36	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc3)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY2=	119.78	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY=	119.78	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	0.319	OK	*= Vu_bm/ (ϕ* Rn_bmWebY), OK if DCR_bmWebY <= DCR_Allowed*

SHEAR PLATE:

Lc4=	0.531	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
ϕ*Rn_spY1=	88.66	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc4)* tsp* Fu_shearTab*
ϕ*Rn_spY2=	153.56	kips	*= ϕbolt* 2.4* db_sp* n_bolts_SST* tsp* Fu_shearTab*
ϕ*Rn_spY=	88.66	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	0.431	OK	*= Vu_bm/ (ϕ* Rn_spY), OK if DCR_spY <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	11.39	kips	*= Pu_SST/ n_Hbolts_SST*
Py=	12.73	kips	*= Vu_bm/ n_Vbolts_SST*
Pr=	17.08	kips	*= Sqrt (Px^2 + Py^2)*
θ=	0.841	radians	*= min(Atan (Py/ Px),1.571))*

BEAM WEB:

Method 1: T and V circular interaction

DCR_bmWebθBearing1 =	0.257		*= DCR_bmWebX^2 + DCR_bmWebY^2*

Method 2: Bearing in a diagonal line

Lvg_web1 =	2.625	in	*= Lb_edge/ (cos θ)*
Lvg_web2 =	5.906	in	*= (Lb_edge + Shorz)/ (cos θ)*
Lc_web1 =	2.125	in	*= Lvg_web1 - dHole_sp/2*
Lc_web2 =	5.406	in	*= Lvg_web2 - dHole_sp/2*
La_web=	7.531	in	*= min(d, Lc_web1, Lc_web2)*
ϕ*Rn_bmWebθ1=	171.827	kips	*= ϕbolt* 1.2* La_web* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ2=	79.853	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕ*Rn_bmWebθ=	79.85	kips	*= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)*
DCR_bmWebθBearing2 =	0.214		*= Pr/ ϕ*Rn_bmWeb θ*
			Ok if DCR_bmWebq <= DCR_Allowed
DCR_bmWebθBearing =	0.257	OK	

SHEAR PLATE:

Method 1: T and V circular interaction

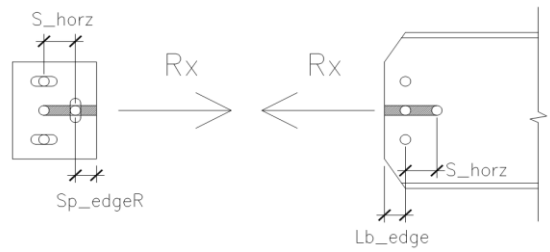
DCR_spθBearing1 =	0.280		= $DCR\_spX^2 + DCR\_spY^2$

Method 2: Bearing in a diagonal line

Lvg_sp=	2.625	in	= Sp_edgeR/ cos(θ)
La_sp=	2.125	in	= min(h_sp/2, Lvg_sp - dHole_sp/2)
ϕ*Rn_spθ1=	62.157	kips	= ϕbolt* 1.2* La_sp* tsp* Fu_shearTab
ϕ*Rn_spθ2=	102.375	kips	= ϕbolt* 2.4* db_sp* tsp* Fu_shearTab*n_boltH
ϕ*Rn_spθ=	62.16	kips	= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)
DCR_spθbearing2=	0.275		= Pr/ (ϕ* Rn_sp θ)
DCR_spθbearing=	0.280	OK	= max(DCR_bmWeb θ, DCR_sp θbearing)
			OK if DCR_sp θbearing <= DCR_Allowed

15.5b: BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK

CASE 1: HORIZONTAL REACTIONS



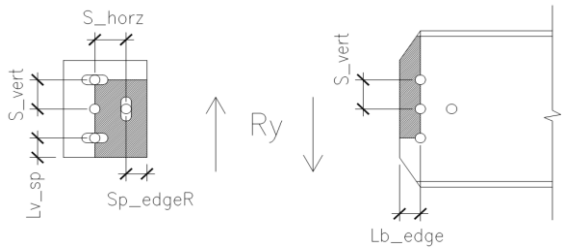
BEAM WEB:

ϕblockshear=	0.75		
Ubs=	1		
Lb_edge=	1.750	in	value previously defined
Lc_bmWeb =	2.531	in	value previously defined
Lh_bmWeb=	7.875	in	= 2* (Lb_edge+Shorz)
Agv_bmWebHorz=	3.461	in^2	= Lh_bmWeb*tbw + Lb_edge* tbw
Ant_bmWebHorz=	0.000	in^2	= 0
Anv_bmWebHorz=	3.461	in^2	= Agv_bmWebHorz
Rn_bmWebHorz1=	134.99	kips	= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz2=	103.84	kips	= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz=	77.88	kips	= ϕblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)
DCR_bmWebHorz_BS=	0.292	OK	= Pu_SST / Rn_bmWebHorz, OK if DCR_bmWebHorz <= DCR_Allowed

SHEAR PLATE:

Lsp_edge=	1.750	in	value previously defined
Lc_sp =	2.531	in	value previously defined
Lh_sp=	7.875	in	= 2* (Lsp_edge + Shorz)
Agv_spHorz=	4.438	in^2	= Lh_sp* tsp + dHole_sp*tsp
Ant_spHorz=	0.000	in^2	= 0
Anv_spHorz=	4.438	in^2	= Agv_bmWebHorz
Rn_spHorz1=	173.06	kips	= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz
Rn_spbHorz2=	133.13	kips	= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz
ϕRn_spHorz=	99.84	kips	= ϕblocksher* min (Rn_spHorz1, Rn_spbHorz2)
DCR_spHorz_BS=	0.228	OK	= Pu_SST/ Rn_spHorz, OK if DCR_spHorz <= DCR_Allowed

CASE 2: VERTICAL REACTIONS



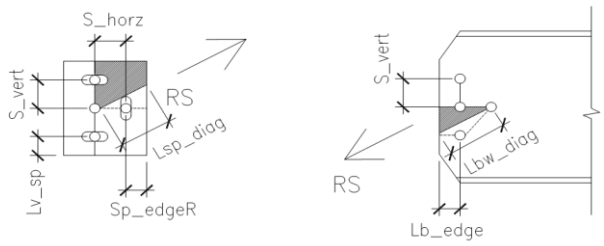
BEAM WEB:

Sbm_web=	2.19	in	= Svert
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	7.633	in	= (d- 2Svert - 2tbf)*0.5 + 2Svert
Agv_bmWeb1=	2.977	in^2	= tbw* h_bmWeb
Anv_bmWeb1=	2.002	in^2	= tbw* (h_bmWeb - 2.5* dHole_sp)
Ant_bmWeb1=	0.488	in^2	= (Lb_edge - dHole_sp/2)* tbw
Rn_bmWebVert1=	109.753	kips	= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
Rn_bmWebVert2=	120.988	kips	= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
ϕRn_bmWebVert=	82.31	kips	= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)
DCR_bmWebVert_BS=	0.464	OK	= Vu_bm/ ϕRn_bmWebVert, OK if DCR_bmWebVert_BS <= DCR_Allowed

SHEAR PLATE:

Ssp=	2.19	in	= sbm_web
Lv_sp=	1.000	in	value previously defined
hsp=	6.375	in	value previously defined
Lslot=	1.500	in	value previously defined
Agv_sp1=	3.188	in^2	= hsp* tsp
Anv_sp1=	1.938	in^2	= tsp* (hsp - 2.5* dHole_sp)
Ant_sp1=	1.594	in^2	= (Lsp_edge - LslotH/2+Shorz)* tsp
Rn_spVert1=	179.156	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	199.219	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ϕRn_spVert=	134.37	kips	= ϕblocksher* min (Rn_spVert1, Rn_spVert2)
DCR_spVert_BS=	0.284	OK	= Vu_bm/ ϕRn_spVert, OK if DCR_spVert_BS <= DCR_Allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: N and V circular interaction

DCR_bmWebθ_BS1 =	0.301		= DCR_bmWebHorz_BS^2 + DCR_bmWebVert_BS^2

Method 2: Failure along hole edges

θ=	0.841	radians	value previously defined
Lb_edge=	1.750	in	value previously defined
Lbw_diag=	4.84	in	min(sqrt(Svert^2+Shorz^2) + Lb_edge, (Lb_edge + Shorz)/cos θ)
Lvg_web=	8.78	in	= Lbw_diag + Lb_edge + Shorz
Lvn_web=	5.78	in	= IF(Lbw_diag = (Lb_edge + Shorz)/cosq, Lvg_web - 2dHole_sp, Lvg_web - 3*dHole_sp)
Ltg_web=	0.00	in	
Ltn_web=	0.00	in	
Agv_web=	3.42	in^2	= Lvg_web* tbw
Anv_web=	2.25	in^2	= Lvn_web* tbw
Agt_web=	0.00	in^2	= Ltg_web* tbw
Ant_web=	0.00	in^2	= Ltn_web* tbw
Rn_bmWeb_BS1=	87.93	kips	= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
Rn_bmWeb_BS2=	102.74	kips	= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
ϕRn_bmWeb_BS=	65.95	kips	= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)
DCR_bmWebθ_BS2=	0.518		=2* Pr/ fRn_bmWeb_BS
DCR_bmWebθ_BS=	0.518	OK	= max(DCR_bmWeb θ_BS1, DCR_bmWeb θ_BS2) <= DCR_Allowed

SHEAR PLATE:

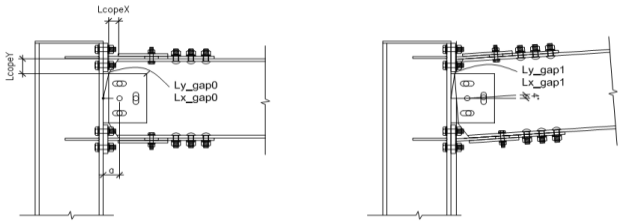
Method 1: N and V circular interaction

DCR_spθ_BS1 =	0.133		= DCR_spVert_BS^2 + DCR_spHorz_BS^2

Method 2: Failure along hole edges

Lsp_diag =	3.94	in	= min (Shorz + Sp_edgeR, (Shorz + Sp_edgeR)/cos θ)
Lsp_edge=	1.750	in	value previously defined
Lvg_sp=	7.13	in	= Lsp_diag + Svert + Lv_sp
Lvn_sp=	4.13	in	= IF(Lsp_diag = Shorz + Sp_edgeR, Lvg_sp - 3dHole_sp, Lvg_sp - 1.5dHole_sp)
Ltg_sp=	0.00	in	
Ltn_sp=	0.00	in	
Agv_sp=	3.563	in	= Lvg_sp* tsp
Anv_sp=	2.063	in	= Lvn_sp* tsp
Agt_sp=	0.000	in	= Ltg_sp* tsp
Ant_sp=	0.000	in	= Ltn_sp* tsp
Rn_sp_BS1=	80.44	kips	= 0.6* Fu_shearTab* Anv_sp + Ubs* Fu_shearTab* Ant_sp
Rn_sp_BS2=	106.88	kips	= 0.6* Fy_shearTab* Agv_sp + Ubs* Fu_shearTab* Ant_sp
ϕRn_sp_BS=	60.33	kips	= ϕshearblock* min (Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.283		= Pr/ ϕRn_sp_BS
DCR_spq_BS=	0.283	OK	= max(DCR_sp θ_BS1, DCR_sp θ_BS2)<= DCR_Allowed

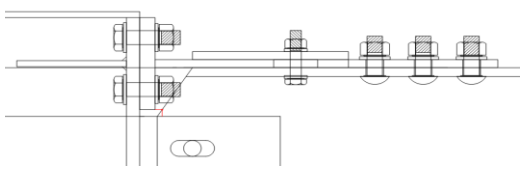
15.6: DETAIL BEAM FLANGE AND WEB COPE DISTANCE



UNDEFORMED DEFORMED TO 7 RADIANS

θrotation=	0.070	radians	
θrotation=	4.011	degrees	
Ly_gap_min =	3/16	in	
Lx_gap_min =	1/8	in	
Lb_edge=	1.75	in	*value previously defined*
t_flange=	0.875	in	*Link flange thickness*
h_flange=	5.750	in	*value previously defined*
d=	12.10	in	*value previously defined*
t_stem=	0.50		*value previously defined*
a=	2.75	in	*value previously defined*
hsp=	6.38	in	*value previously defined*
Ly_gap0 =	0.24	in	OK *= (d + t_steam - h_flange - hsp)/2*
			OK if Ly_gap0 => Ly_gap_min
Lx_gap0 =	0.125	in	OK *= (a + Lb_edge - t_flange)/2*
			OK if Lx_gap0 => Lx_gap_min
LcopeX=	1.75	in	*= Lb_edge*
			Align end of beam flange cut with beam web bolt centerline
LcopeY=	2.8750	in	*= (d - hsp)/2*
			Try to align vertical web cope with top of Shear Plate
LslotV_ck =	1.31	in	OK *= Shorz* θrotation*2 + dHole_sp*
			OK if LslotV => LslotV_ck

Xgap between Beam web (diag cut) and edge of end plate



β =	58.67	degrees	*= atan(LcopeY/ LcopeX)*
Ly_gap0 =	0.24	in	*value previously defined*
Lx_gapβ =	0.27	in	

Check if Lx gap clear >= 0 after 4 degrees of roation

$$RM = \begin{pmatrix} \cos(\theta_{rotation}) & -\sin(\theta_{rotation}) \\ \sin(\theta_{rotation}) & \cos(\theta_{rotation}) \end{pmatrix}$$

$$RM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Rotational matrix*

$$SM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Stretch matrix*

$$X = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *= RM.SM Deformation Gradiant*

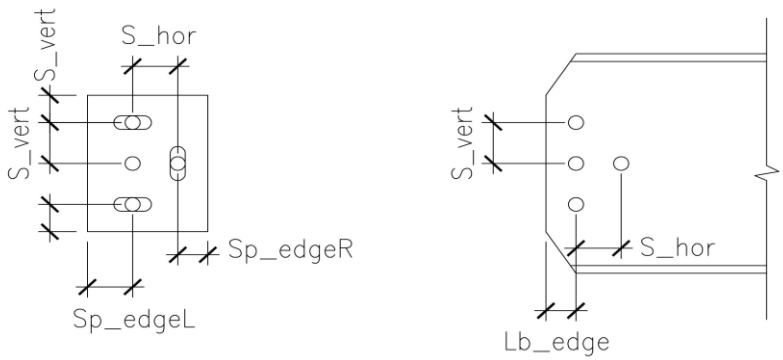
X_0 =	-1.480	in	*= a - Lx_gap β - 1*	with respect to center bolt
Y_0 =	3.425	in	*= hsp/2 + Ly_gap0*	with respect to center bolt
X_1 =	-1.717	in		with respect to center bolt
Y_1 =	3.315	in		with respect to center bolt
Lx_gap1 =	-0.237	in	*=X_1-X_0*	
Ly_gap1 =	-0.110	in	*=Y_1-Y_0*	
Lx_gap_clear =	0.086	in	OK *OK if Lx_gap_clear > 0*	
			*= Lx_gap β - abs(Lx_gap1) + tan(90° - β - θ)*abs(Ly_gap1)*	

STEP 15: DESIGN BEAM WEB TO COLUMN FLANGE CONNECTION

15.1a: SHEAR PLATE BOLT SIZE

Beam Section (bmSize) =	W12X65		*Value previously defined*
Beam Depth (db)=	12.10	in	*Value previously defined*
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Fy_bm=	50	ksi	
Fu_bm =	65	ksi	
Axial Load (Pu_SST)=	23.40	kips	*Max axial force per LCM61-64 from SAP_BeamReactions, SAP output*
Vertical Load (Vu_bm)=	36.25	kips	*Value previously defined from Step 12*
Vu_bolt=	16.8	kips	*= Sqrt ((Pu_SST/ n_Hbolt_SST)^2 + (Vu_bm/ n_Vbolt_SST)^2)*
ϕbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	A325-X		*[A325-X default]*
Bolt Dia (db_sp)=	7/8	in	
No of vertical Bolts (n_Vbolts_SST)=	3		
No of horizontal Bolts (n_Hbolts_SST)=	2		
Fnv_A325N=	68	ksi	*= if (Bolt Type = "A490-X", 84, 68)*
Anb_sp=	0.60	in^2	*= Pi* (db_sp/2)^2*
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*
ϕRn_stBolt_Shear=	30.7	kips	*= Rn_stBolt_shear* ϕbolt*
DCR_shearTabl_Cbolt=	0.548	OK	*= Vu_bolt/ ϕRn_stBolt_shear*
			OK if DCR_shearTabl_Cbolt <= 1.05

15.2: SHEAR PLATE GEOMETRY



a=	2.75	in	*Value previously defined*
tsp=	0.375	in	*[0.375 default]*
db =	12.1	in	*Value previously defined*
h_flange =	5.75	in	*Value previously defined*
t_stem =	0.50	in	*Value previously defined*
h_clear =	0.25	in	*minimum clearance between link flange and shear tab*
Bolt Spacing (S_min)=	2.33	in	*= 2.6667* max (db_sp)*
Bolt Spacing (Svert)=	2.19	in	OK *looked up value*
Bolt Spacing (Shorz)=	2.19	in	OK *looked up value*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lv_sp=	1	in	Need to Check J3.10 and J4, see Section 15.5 below
h_sp max =	6.35	in	
Plate Depth (h_sp)=	6.375	in	OK *= Lv_sp* 2 + Svert* 2*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lh_sp=	1.75	in	OK
Plate Width (W_shearTab) =	6.69	in	*= Lh_sp + a + Shorz*
Lslot_min=	1.30625	in	*= db_sp + 1/8 + 0.14* Svert*
LslotH = LslotV =	1.50	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	**OK**		*OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)*

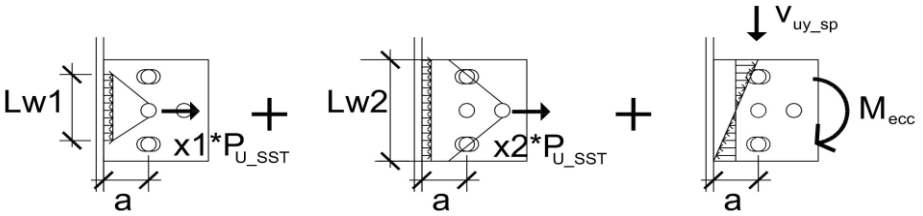
15.3a: SHEAR PLATE YIELDING (VERTICAL)

ϕyield=	0.9		
dHole_sp=	1.00	in	*= db_sp + 1/8*
Asp_Agv=	2.391	in^2	*= tsp* h_sp*
ϕVy_sp=	64.55	kips	*= ϕyield* 0.6* Asp_Agv* Fy_sp*
DCR_spYield=	0.562	OK	*= Vu_bm/ ϕVy_sp*
			OK if DCR_spYield <= DCR_allowed

15.3b: SHEAR PLATE RUPTURE (VERTICAL)

ϕrupture=	0.75		
Asp_nv=	1.27	in^2	*= hsp* tsp - dHole_sp* n_Vbolts_SST* tsp*
ϕVrupture_sp=	37.02	kips	*= ϕrupture* 0.6* Fu_shearTab* Asp_nv*
DCR_spRupture=	0.979	OK	*= Vu_bm/ ϕVrupture_sp*

15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT



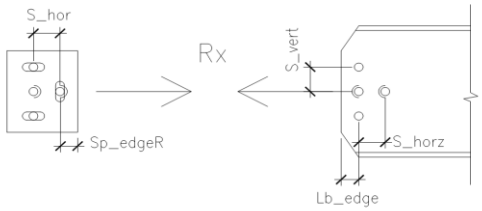
ϕb=	0.90		
a=	2.75	in	*Value previously defined*
Vuy=	36.25	kips	*= Vu_bm*
Mecc=	99.69	kips*in	*= Vuy* a*
x1 =	0.50		*assume 1/2 of axial is resisted by each bolt*
x2 =	0.50		*=1-x1*
θWhitmore=	30	deg	
Lwhitmore1 =	4.05	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore1 =	1.52	in^2	*= Lwhitmore1* tsp*
Lwhitmore2 =	6.38	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore2 =	2.39	in^2	*= Lwhitmore2* tsp*
Ssp=	2.54	in^3	*= tsp* hsp^2/ 6*
Isp=	8.10	in^4	*= tsp* hsp^3/ 12*
fb1=	39.25	ksi	*= Mecc/ Ssp*
yb=	2.03	in	*= Lwhitmore1/ 2*
fb2=	24.94	ksi	*= Mecc* yb/ Isp*
fa2=	12.60	ksi	*= x1*Pu_SST/ Awhitmore1 + x2*Pu_SST/ Awhitmore2*
ftot2=	37.53	ksi	*= fa2 + fb2*
fmax_sp=	39.25	ksi	*= max (fb1, ftot2)*
ϕb*Fy_sp=	45.00	ksi	*= ϕb* Fy_shearTab*
DCR_sp=	0.872	OK	*= fmax_sp/ (ϕb* Fy_shearTab), OK if DCR_sp <= DCR_Allowed*

15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD

tsp=	0.375	in	*value previously defined*
tw_sp_min=	0.2344	in	*= 5/8*tsp Fillet size to develop plate, Per AISC Steel Manual 14th edition*
tw_sp=	**0.2500**	in	*Each Side [0.25 default] , page 10-102*
DCR_spWeld=	0.938	OK	*= tw_sp_min/ tw_sp, OK if DCR_spWeld <= DCR_Allowed*

15.5: BEAM WEB AND SHEAR TAB BEARING

CASE 1: HORIZONTAL REACTIONS



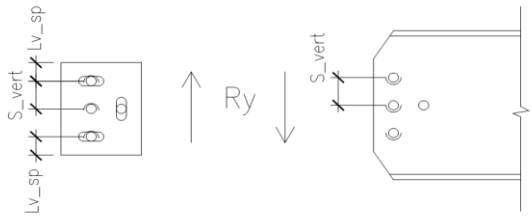
BEAM WEB:

ϕbolt=	0.75		
tbw=	0.39	in	*value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lc_bmWeb1 =	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
Lc_bmWeb2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_bmWeb=	2.531	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
ϕRn_beamWeb1=	57.75	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb*
ϕRn_beamWeb2=	79.85	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb *n_Hbolt*
ϕRn_beamWeb=	57.75	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.405	OK	*= Pu_SST/ ϕRn_beamWeb, OK if DCR_bmWebX <= DCR_Allowed*

SHEAR PLATE:

tsp=	0.375	in	*value previously defined*
Sp_edgeR =	1.75	in	*Edge distance for shear tab bolt hole*
Lc_sp1 =	1.281	in	*= Sp_edgeR - (db_sp + 1/16)* 0.5*
Lc_sp2 =	1.250	in	*= Shorz - (db_sp + 1/16)*
Lc_sp=	2.531	in	*= Lc_sp1 + Lc_sp2*
ϕRn_sp1=	55.53	kips	*= ϕbolt* 1.2* Lc_sp* tbw* Fu_bmWeb*
ϕRn_sp2=	76.78	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕRn_sp=	55.53	kips	*= min (ϕRn_sp1, ϕRn_sp2)*
DCR_spX=	0.421	OK	*= Pu_SST/ ϕRn_sp, OK if DCR_spX <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



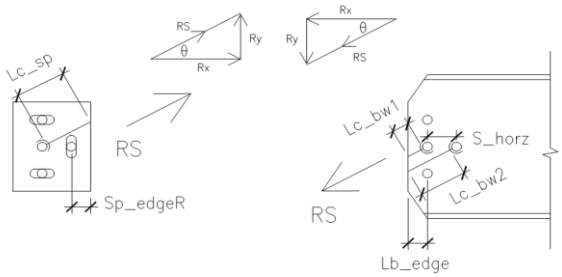
BEAM WEB:

Lc1=	**1.250**	in	*= Svert - (db_sp + 1/16)*
Lc2=	1.250	in	*= Lc1*
Lc3=	3.258	in	*= (db - 2Svert - 2tbf)*0.5*
ϕ*Rn_bmWebY1=	131.36	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc3)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY2=	119.78	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY=	119.78	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	0.303	OK	*= Vu_bm/ (ϕ* Rn_bmWebY), OK if DCR_bmWebY <= DCR_Allowed*

SHEAR PLATE:

Lc4=	0.531	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
ϕ*Rn_spY1=	66.50	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc4)* tsp* Fu_shearTab*
ϕ*Rn_spY2=	115.17	kips	*= ϕbolt* 2.4* db_sp* n_bolts_SST* tsp* Fu_shearTab*
ϕ*Rn_spY=	66.50	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	0.545	OK	*= Vu_bm/ (ϕ* Rn_spY), OK if DCR_spY <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	11.70	kips	*= Pu_SST/ n_Hbolts_SST*
Py=	12.08	kips	*= Vu_bm/ n_Vbolts_SST*
Pr=	16.82	kips	*= Sqrt (Px^2 + Py^2)*
θ=	0.802	radians	*= min(Atan (Py/ Px),1.571))*

BEAM WEB:

Method 1: T and V circular interaction

DCR_bmWebθBearing1 =	0.256		*= DCR_bmWebX^2 + DCR_bmWebY^2*

Method 2: Bearing in a diagonal line

Lvg_web1 =	2.516	in	*= Lb_edge/ (cos θ)*
Lvg_web2 =	5.661	in	*= (Lb_edge + Shorz)/ (cos θ)*
Lc_web1 =	2.016	in	*= Lvg_web1 - dHole_sp/2*
Lc_web2 =	5.161	in	*= Lvg_web2 - dHole_sp/2*
La_web=	7.177	in	*= min(d, Lc_web1, Lc_web2)*
ϕ*Rn_bmWebθ1=	163.732	kips	*= ϕbolt* 1.2* La_web* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ2=	79.853	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt*
ϕ*Rn_bmWebθ=	79.85	kips	*= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)*
DCR_bmWebθBearing2 =	0.211		*= Pr/ ϕ*Rn_bmWeb θ*
			Ok if DCR_bmWebq <= DCR_Allowed
DCR_bmWebθBearing =	0.256	OK	

SHEAR PLATE:

Method 1: T and V circular interaction

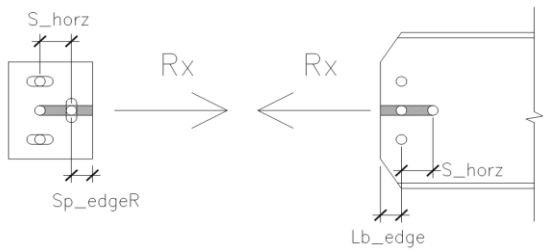
DCR_spθBearing1 =	0.475		= $DCR\_spX^2 + DCR\_spY^2$

Method 2: Bearing in a diagonal line

Lvg_sp=	2.516	in	= Sp_edgeR/ cos(θ)
La_sp=	2.016	in	= min(h_sp/2, Lvg_sp - dHole_sp/2)
ϕ*Rn_spθ1=	44.223	kips	= ϕbolt* 1.2* La_sp* tsp* Fu_shearTab
ϕ*Rn_spθ2=	76.781	kips	= ϕbolt* 2.4* db_sp* tsp* Fu_shearTab*n_boltH
ϕ*Rn_spθ=	44.22	kips	= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)
DCR_spθbearing2=	0.380		= Pr/ (ϕ* Rn_sp θ)
DCR_spθbearing=	0.475	OK	= max(DCR_bmWeb θ, DCR_sp θbearing)
			OK if DCR_sp θbearing <= DCR_Allowed

15.5b: BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK

CASE 1: HORIZONTAL REACTIONS



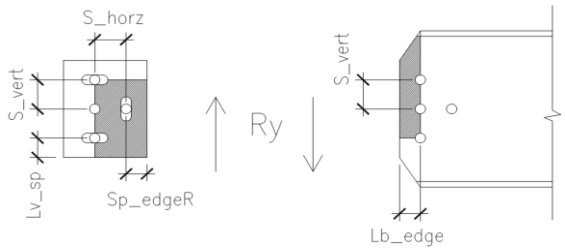
BEAM WEB:

ϕblockshear=	0.75		
Ubs=	1		
Lb_edge=	1.750	in	value previously defined
Lc_bmWeb =	2.531	in	value previously defined
Lh_bmWeb=	7.875	in	= 2* (Lb_edge+Shorz)
Agv_bmWebHorz=	3.461	in^2	= Lh_bmWeb*tbw + Lb_edge* tbw
Ant_bmWebHorz=	0.000	in^2	= 0
Anv_bmWebHorz=	3.461	in^2	= Agv_bmWebHorz
Rn_bmWebHorz1=	134.99	kips	= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz2=	103.84	kips	= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz=	77.88	kips	= ϕblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)
DCR_bmWebHorz_BS=	0.300	OK	= Pu_SST/ Rn_bmWebHorz, OK if DCR_bmWebHorz <= DCR_Allowed

SHEAR PLATE:

Lsp_edge=	1.750	in	value previously defined
Lc_sp =	2.531	in	value previously defined
Lh_sp=	7.875	in	= 2* (Lsp_edge + Shorz)
Agv_spHorz=	3.328	in^2	= Lh_sp* tsp + dHole_sp*tsp
Ant_spHorz=	0.000	in^2	= 0
Anv_spHorz=	3.328	in^2	= Agv_bmWebHorz
Rn_spHorz1=	129.80	kips	= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz
Rn_spbHorz2=	99.84	kips	= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz
ϕRn_spHorz=	74.88	kips	= ϕblocksher* min (Rn_spHorz1, Rn_spbHorz2)
DCR_spHorz_BS=	0.312	OK	= Pu_SST/ Rn_spHorz, OK if DCR_spHorz <= DCR_Allowed

CASE 2: VERTICAL REACTIONS



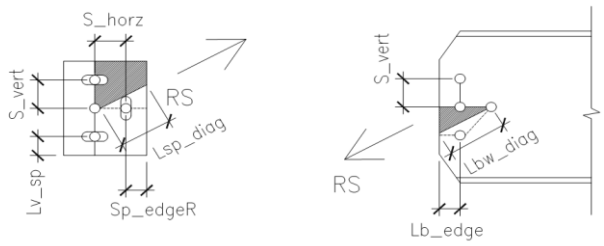
BEAM WEB:

Sbm_web=	2.19	in	= Svert
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	7.633	in	= (d- 2Svert - 2tbf)*0.5 + 2Svert
Agv_bmWeb1=	2.977	in^2	= tbw* h_bmWeb
Anv_bmWeb1=	2.002	in^2	= tbw* (h_bmWeb - 2.5* dHole_sp)
Ant_bmWeb1=	0.488	in^2	= (Lb_edge - dHole_sp/2)* tbw
Rn_bmWebVert1=	109.753	kips	= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
Rn_bmWebVert2=	120.988	kips	= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
ϕRn_bmWebVert=	82.31	kips	= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)
DCR_bmWebVert_BS=	0.440	OK	= Vu_bm/ ϕRn_bmWebVert, OK if DCR_bmWebVert_BS <= DCR_Allowed

SHEAR PLATE:

Ssp=	2.19	in	= sbm_web
Lv_sp=	1.000	in	value previously defined
hsp=	6.375	in	value previously defined
Lslot=	1.500	in	value previously defined
Agv_sp1=	2.391	in^2	= hsp* tsp
Anv_sp1=	1.453	in^2	= tsp* (hsp - 2.5* dHole_sp)
Ant_sp1=	1.195	in^2	= (Lsp_edge - LslotH/2+Shorz)* tsp
Rn_spVert1=	134.367	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	149.414	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ϕRn_spVert=	100.78	kips	= ϕblocksher* min (Rn_spVert1, Rn_spVert2)
DCR_spVert_BS=	0.360	OK	= Vu_bm/ ϕRn_spVert, OK if DCR_spVert_BS <= DCR_Allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: N and V circular interaction

DCR_bmWebθ_BS1 =	0.284		= DCR_bmWebHorz_BS^2 + DCR_bmWebVert_BS^2

Method 2: Failure along hole edges

θ=	0.802	radians	value previously defined
Lb_edge=	1.750	in	value previously defined
Lbw_diag=	4.84	in	min(sqrt(Svert^2+Shorz^2) + Lb_edge, (Lb_edge + Shorz)/cos θ)
Lvg_web=	8.78	in	= Lbw_diag + Lb_edge + Shorz
Lvn_web=	5.78	in	= IF(Lbw_diag = (Lb_edge + Shorz)/cosq, Lvg_web - 2dHole_sp, Lvg_web - 3*dHole_sp)
Ltg_web=	0.00	in	
Ltn_web=	0.00	in	
Agv_web=	3.42	in^2	= Lvg_web* tbw
Anv_web=	2.25	in^2	= Lvn_web* tbw
Agt_web=	0.00	in^2	= Ltg_web* tbw
Ant_web=	0.00	in^2	= Ltn_web* tbw
Rn_bmWeb_BS1=	87.93	kips	= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
Rn_bmWeb_BS2=	102.74	kips	= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
ϕRn_bmWeb_BS=	65.95	kips	= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)
DCR_bmWebθ_BS2=	0.510		= 2*Pr/ fRn_bmWeb_BS
DCR_bmWebθ_BS=	0.510	OK	= max(DCR_bmWeb θ_BS1, DCR_bmWeb θ_BS2) <= DCR_Allowed

SHEAR PLATE:

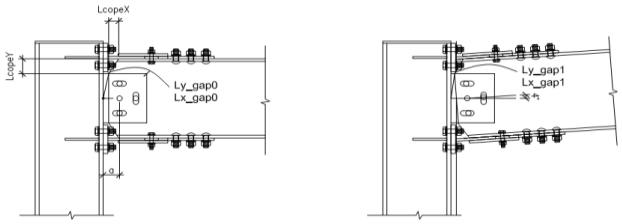
Method 1: N and V circular interaction

DCR_spθ_BS1 =	0.227		= DCR_spVert_BS^2 + DCR_spHorz_BS^2

Method 2: Failure along hole edges

Lsp_diag =	3.94	in	= min (Shorz + Sp_edgeR, (Shorz + Sp_edgeR)/cos θ)
Lsp_edge=	1.750	in	value previously defined
Lvg_sp=	7.13	in	= Lsp_diag + Svert + Lv_sp
Lvn_sp=	4.13	in	= IF(Lsp_diag = Shorz + Sp_edgeR, Lvg_sp - 3dHole_sp, Lvg_sp - 1.5dHole_sp)
Ltg_sp=	0.00	in	
Ltn_sp=	0.00	in	
Agv_sp=	2.672	in	= Lvg_sp* tsp
Anv_sp=	1.547	in	= Lvn_sp* tsp
Agt_sp=	0.000	in	= Ltg_sp* tsp
Ant_sp=	0.000	in	= Ltn_sp* tsp
Rn_sp_BS1=	60.33	kips	= 0.6* Fu_shearTab* Anv_sp + Ubs* Fu_shearTab* Ant_sp
Rn_sp_BS2=	80.16	kips	= 0.6* Fy_shearTab* Agv_sp + Ubs* Fu_shearTab* Ant_sp
ϕRn_sp_BS=	45.25	kips	= ϕshearblock* min (Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.372		= Pr/ ϕRn_sp_BS
DCR_spq_BS=	0.372	OK	= max(DCR_sp θ_BS1, DCR_sp θ_BS2)<= DCR_Allowed

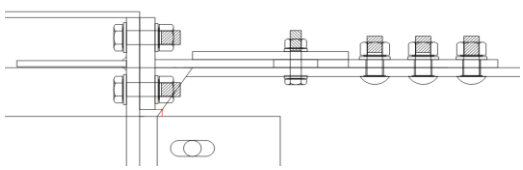
15.6: DETAIL BEAM FLANGE AND WEB COPE DISTANCE



UNDEFORMED DEFORMED TO 7 RADIANS

θrotation=	0.070	radians	
θrotation=	4.011	degrees	
Ly_gap_min =	3/16	in	
Lx_gap_min =	1/8	in	
Lb_edge=	1.75	in	*value previously defined*
t_flange=	0.875	in	*Link flange thickness*
h_flange=	5.750	in	*value previously defined*
d=	12.10	in	*value previously defined*
t_stem=	0.50		*value previously defined*
a=	2.75	in	*value previously defined*
hsp=	6.38	in	*value previously defined*
Ly_gap0 =	0.24	in	OK *= (d + t_steam - h_flange - hsp)/2*
			OK if Ly_gap0 => Ly_gap_min
Lx_gap0 =	0.125	in	OK *= (a + Lb_edge - t_flange)/2*
			OK if Lx_gap0 => Lx_gap_min
LcopeX=	1.75	in	*= Lb_edge*
			Align end of beam flange cut with beam web bolt centerline
LcopeY=	2.8750	in	*= (d - hsp)/2*
			Try to align vertical web cope with top of Shear Plate
LslotV_ck =	1.31	in	OK *= Shorz* θrotation*2 + dHole_sp*
			OK if LslotV => LslotV_ck

Xgap between Beam web (diag cut) and edge of end plate



β =	58.67	degrees	*= atan(LcopeY/ LcopeX)*
Ly_gap0 =	0.24	in	*value previously defined*
Lx_gapβ =	0.27	in	

Check if Lx gap clear >= 0 after 4 degrees of roation

$$RM = \begin{pmatrix} \cos(\theta_{rotation}) & -\sin(\theta_{rotation}) \\ \sin(\theta_{rotation}) & \cos(\theta_{rotation}) \end{pmatrix}$$

$$RM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Rotational matrix*

$$SM = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *Stretch matrix*

$$X = \begin{pmatrix} 0.998 & -0.07 \\ 0.07 & 0.998 \end{pmatrix}$$ *= RM.SM Deformation Gradiant*

X_0 =	-1.480	in	*= a - Lx_gap β - 1*	with respect to center bolt
Y_0 =	3.425	in	*= hsp/2 + Ly_gap0*	with respect to center bolt
X_1 =	-1.717	in		with respect to center bolt
Y_1 =	3.315	in		with respect to center bolt
Lx_gap1 =	-0.237	in	*=X_1-X_0*	
Ly_gap1 =	-0.110	in	*=Y_1-Y_0*	
Lx_gap_clear =	0.086	in	OK *OK if Lx_gap_clear > 0*	
			*= Lx_gap β - abs(Lx_gap1) + tan(90° - β - θ)*abs(Ly_gap1)*	

Base Reaction (kips, in)												
Load Combos	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2 (Moment)
1	1	LCM1	0.10	0.00	6.79	5.36	8	LCM1	-0.10	0.00	6.79	-5.36
2	1	LCM2	0.19	0.00	4.83	9.71	8	LCM2	-0.19	0.00	4.83	-9.71
3	1	LCM3	0.00	0.00	0.00	0.00	8	LCM3	0.00	0.00	0.00	0.00
4	1	LCM4	0.00	0.00	0.00	0.00	8	LCM4	0.00	0.00	0.00	0.00
5	1	LCM5	0.00	0.00	0.00	0.00	8	LCM5	0.00	0.00	0.00	0.00
6	1	LCM6	0.00	0.00	0.00	0.00	8	LCM6	0.00	0.00	0.00	0.00
7	1	LCM7	-17.58	0.00	-26.77	-2223.99	8	LCM7	-17.58	0.00	26.77	-2223.99
8	1	LCM8	-0.42	0.00	5.99	-61.21	8	LCM8	-0.63	0.00	7.59	-71.93
9	1	LCM9	-0.24	0.00	10.82	-51.50	8	LCM9	-0.82	0.00	12.42	-81.64
10	1	LCM10	-0.42	0.00	5.99	-61.21	8	LCM10	-0.63	0.00	7.59	-71.93
11	1	LCM11	-0.42	0.00	5.99	-61.21	8	LCM11	-0.63	0.00	7.59	-71.93
12	1	LCM12	-0.42	0.00	5.99	-61.21	8	LCM12	-0.63	0.00	7.59	-71.93
13	1	LCM13	-0.28	0.00	9.61	-53.93	8	LCM13	-0.77	0.00	11.21	-79.21
14	1	LCM14	-0.28	0.00	9.61	-53.93	8	LCM14	-0.77	0.00	11.21	-79.21
15	1	LCM15	-0.28	0.00	9.61	-53.93	8	LCM15	-0.77	0.00	11.21	-79.21
16	1	LCM16	0.10	0.00	6.79	5.36	8	LCM16	-0.10	0.00	6.79	-5.36
17	1	LCM17	0.10	0.00	6.79	5.36	8	LCM17	-0.10	0.00	6.79	-5.36
18	1	LCM18	-12.19	0.00	-11.02	-1550.70	8	LCM18	-12.42	0.00	26.46	-1562.88
19	1	LCM19	12.42	0.00	26.46	1562.88	8	LCM19	12.19	0.00	-11.02	1550.70
20	1	LCM20	0.24	0.00	10.41	12.64	8	LCM20	-0.24	0.00	10.41	-12.64
21	1	LCM21	0.24	0.00	10.41	12.64	8	LCM21	-0.24	0.00	10.41	-12.64
22	1	LCM22	0.24	0.00	10.41	12.64	8	LCM22	-0.24	0.00	10.41	-12.64
23	1	LCM23	0.24	0.00	10.41	12.64	8	LCM23	-0.24	0.00	10.41	-12.64
24	1	LCM24	0.24	0.00	10.41	12.64	8	LCM24	-0.24	0.00	10.41	-12.64
25	1	LCM25	0.24	0.00	10.41	12.64	8	LCM25	-0.24	0.00	10.41	-12.64
26	1	LCM26	-8.97	0.00	-2.95	-1154.40	8	LCM26	-9.48	0.00	25.16	-1180.79
27	1	LCM27	9.48	0.00	25.16	1180.79	8	LCM27	8.97	0.00	-2.95	1154.40
28	1	LCM28	-8.97	0.00	-2.95	-1154.40	8	LCM28	-9.48	0.00	25.16	-1180.79
29	1	LCM29	9.48	0.00	25.16	1180.79	8	LCM29	8.97	0.00	-2.95	1154.40
30	1	LCM30	-8.97	0.00	-2.95	-1154.40	8	LCM30	-9.48	0.00	25.16	-1180.79
31	1	LCM31	9.48	0.00	25.16	1180.79	8	LCM31	8.97	0.00	-2.95	1154.40
32	1	LCM32	0.06	0.00	4.07	3.21	8	LCM32	-0.06	0.00	4.07	-3.21
33	1	LCM33	0.06	0.00	4.07	3.21	8	LCM33	-0.06	0.00	4.07	-3.21
34	1	LCM34	-12.26	0.00	-15.59	-1554.31	8	LCM34	-12.35	0.00	21.88	-1559.28
35	1	LCM35	12.35	0.00	21.88	1559.28	8	LCM35	12.26	0.00	-15.59	1554.31
36	1	LCM36	-0.38	0.00	8.71	-59.07	8	LCM36	-0.67	0.00	10.31	-74.07
37	1	LCM37	-0.10	0.00	15.07	-44.60	8	LCM37	-0.95	0.00	16.67	-88.54
38	1	LCM38	-0.10	0.00	15.07	-44.60	8	LCM38	-0.95	0.00	16.67	-88.54
39	1	LCM39	-0.10	0.00	15.07	-44.60	8	LCM39	-0.95	0.00	16.67	-88.54
40	1	LCM40	-0.31	0.00	9.76	-55.28	8	LCM40	-0.75	0.00	11.36	-77.86
41	1	LCM41	0.12	0.00	8.15	6.43	8	LCM41	-0.12	0.00	8.15	-6.43
42	1	LCM42	0.12	0.00	8.15	6.43	8	LCM42	-0.12	0.00	8.15	-6.43
43	1	LCM43	-0.31	0.00	9.76	-55.28	8	LCM43	-0.75	0.00	11.36	-77.86
44	1	LCM44	0.12	0.00	8.15	6.43	8	LCM44	-0.12	0.00	8.15	-6.43
45	1	LCM45	0.12	0.00	8.15	6.43	8	LCM45	-0.12	0.00	8.15	-6.43
46	1	LCM46	-0.31	0.00	9.76	-55.28	8	LCM46	-0.75	0.00	11.36	-77.86
47	1	LCM47	0.12	0.00	8.15	6.43	8	LCM47	-0.12	0.00	8.15	-6.43
48	1	LCM48	0.12	0.00	8.15	6.43	8	LCM48	-0.12	0.00	8.15	-6.43
49	1	LCM49	0.22	0.00	10.56	11.29	8	LCM49	-0.22	0.00	10.56	-11.29
50	1	LCM50	0.22	0.00	10.56	11.29	8	LCM50	-0.22	0.00	10.56	-11.29
51	1	LCM51	0.22	0.00	10.56	11.29	8	LCM51	-0.22	0.00	10.56	-11.29
52	1	LCM52	0.22	0.00	10.56	11.29	8	LCM52	-0.22	0.00	10.56	-11.29
53	1	LCM53	0.22	0.00	10.56	11.29	8	LCM53	-0.22	0.00	10.56	-11.29
54	1	LCM54	0.22	0.00	10.56	11.29	8	LCM54	-0.22	0.00	10.56	-11.29
55	1	LCM55	-17.34	0.00	-14.88	-2211.66	8	LCM55	-17.82	0.00	38.66	-2236.32
56	1	LCM56	17.82	0.00	38.66	2236.32	8	LCM56	17.34	0.00	-14.88	2211.66
57	1	LCM57	0.09	0.00	6.11	4.82	8	LCM57	-0.09	0.00	6.11	-4.82
58	1	LCM58	0.09	0.00	6.11	4.82	8	LCM58	-0.09	0.00	6.11	-4.82
59	1	LCM59	-17.51	0.00	-21.98	-2220.21	8	LCM59	-17.65	0.00	31.55	-2227.77
60	1	LCM60	17.65	0.00	31.55	2227.77	8	LCM60	17.51	0.00	-21.98	2220.21
61	1	LCM61	-52.50	0.00	-68.41	-6659.64	8	LCM61	-52.97	0.00	92.19	-6684.30
62	1	LCM62	52.97	0.00	92.19	6684.30	8	LCM62	52.50	0.00	-68.41	6659.64
63	1	LCM63	-52.66	0.00	-75.52	-6668.19	8	LCM63	-52.81	0.00	85.09	-6675.74
64	1	LCM64	52.81	0.00	85.09	6675.74	8	LCM64	52.66	0.00	-75.52	6668.19

Note: Negative (-) values for axial load indicate tension

SMF BASE PLATE CONNECTION DESIGN (COLUMN 1)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W12X106

Base Plate =	**12.25" x 14" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 62 **Design Code:** IBC 2015

Left Column:

V=	52.974	kips	
P=	92.190	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	12.250	in	Base plate width
N =	14.000	in	Base plate length
t =	0.625	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	12.90	in	Column depth
t_w =	0.61	in	Column web thickness
h_{web} =	10.92	in	Column web depth
b_f =	12.20	in	Column flange width
t_f =	0.99	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	5.0	in	Anchor bolt spacing
g =	5.0	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	171.5	in^2	Area of base plate
A_2 =	171.5	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	364	kips	Nominal strength for bearing
ϕP_p =	237	kips	Axial design strength for bearing
DCR_conc=	**0.389**	OK	

Plate Yielding:

m =	0.87	in	Base plate cantilever dimension
n =	1.25	in	Base plate cantilever dimension
X =	0.999		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	3.14	in	Base plate cantilever dimension
l =	3.14	in	Critical base plate cantilever dimension
ϕP_n =	153	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.602**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.61	in	Column web thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.99	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	9.92	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	11.70	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	52.974	kips
t_weld_web=	0.25	in
ϕRn_web=	110.48	kips
DCR_V=	**0.48**	OK

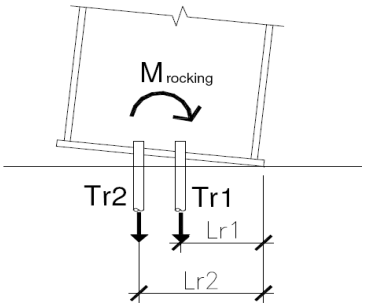
Tension Only

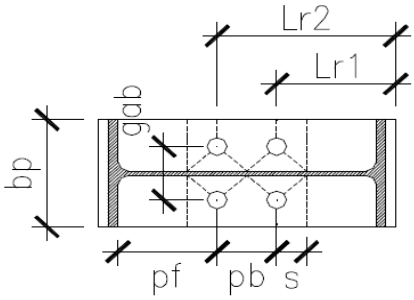
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	162.88	kips
ϕRn=	273.36	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	52.97	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	318.96	kips	
DCR_V+T=	**0.166**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	4.5	in	
Lr2=	9.5	in	
Tr1=	46.10	kips	Assume equal to 1/2 of P
Tr2=	46.10	kips	Assume equal to 1/2 of P
Mrocking=	645.3	kip-in	Rocking moment about Bp tip
treq=	0.542	in	Plate thickness required for tension
tbp=	0.625	in	
DCR_bp=	**0.868**	OK	

bp=	12.250	in
pb=	5.0	in
gab=	5.0	in
pf=	2.96	in
s_tmp=	3.91	in
s=	3.913118961	in

Y=	67.62	in
gamma_r=	1.25	
ϕb=	0.9	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.037	0.057	0.003	0.267
37	1.2 D + 1.6 L + 0.5 Lr	0.064	0.098	0.001	0.351
38	1.2 D + 1.6 L + 0.5 S	0.064	0.098	0.001	0.351
39	1.2 D + 1.6 L + 0.5 R	0.064	0.098	0.001	0.351
40	1.2 D + 1.6 Lr + 0.5 L	0.041	0.064	0.003	0.282
41	1.2 D + 1.6 Lr + 0.5 W	0.034	0.053	0.001	0.258
42	1.2 D + 1.6 Lr - 0.5 W	0.034	0.053	0.001	0.258
43	1.2 D + 1.6 S + 0.5 L	0.041	0.064	0.003	0.282
44	1.2 D + 1.6 S + 0.5 W	0.034	0.053	0.001	0.258
45	1.2 D + 1.6 S - 0.5 W	0.034	0.053	0.001	0.258
46	1.2 D + 1.6 R + 0.5 L	0.041	0.064	0.003	0.282
47	1.2 D + 1.6 R + 0.5 W	0.034	0.053	0.001	0.258
48	1.2 D + 1.6 R - 0.5 W	0.034	0.053	0.001	0.258
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.045	0.069	0.002	0.294
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.045	0.069	0.002	0.294
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.045	0.069	0.002	0.294
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.045	0.069	0.002	0.294
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.045	0.069	0.002	0.294
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.045	0.069	0.002	0.294
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.000	0.000	0.157	0.349
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.163	0.252	0.161	0.562
57	0.9 D + 1.0 W	0.026	0.040	0.001	0.223
58	0.9 D - 1.0 W	0.026	0.040	0.001	0.223
59	(0.9 - 0.2 SDS) D + E	0.000	0.000	0.158	0.424
60	(0.9 - 0.2 SDS) D - E	0.133	0.206	0.160	0.508
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.000	0.000	0.475	0.748
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.389	0.602	0.479	0.868
63	(0.9 - 0.2 SDS) D + Omega*E	0.000	0.000	0.477	0.786
64	(0.9 - 0.2 SDS) D - Omega*E	0.359	0.555	0.478	0.834
	Max=	0.389	0.602	0.479	0.868
	Check=	OK	OK	OK	OK

SMF BASE PLATE CONNECTION DESIGN (COLUMN 2)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W12X106

Base Plate =	**12.25" x 14" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**5/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 61 **Design Code:** IBC 2015

Left Column:

V=	-52.974	kips	
P=	92.190	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	12.250	in	Base plate width
N =	14.000	in	Base plate length
t =	0.625	in	Base plate thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.3125	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

Column:

d =	12.90	in	Column depth
t_w =	0.61	in	Column web thickness
h_{web} =	10.92	in	Column web depth
b_f =	12.20	in	Column flange width
t_f =	0.99	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.75	in	Anchor bolt spacing
g =	3.75	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	171.5	in^2	Area of base plate
A_2 =	171.5	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	364	kips	Nominal strength for bearing
ϕP_p =	237	kips	Axial design strength for bearing
DCR_conc=	**0.389**	OK	

Plate Yielding:

m =	0.87	in	Base plate cantilever dimension
n =	1.25	in	Base plate cantilever dimension
X =	0.999		Factor assuming $P_u = \phi P_p$
λ=	1.000		
$\lambda n'$ =	3.14	in	Base plate cantilever dimension
l =	3.14	in	Critical base plate cantilever dimension
ϕP_n =	153	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.602**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.61	in	Column web thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.99	in	Column flange thickness
w_{min} =	5/16	in	AISC Table J2.4
w_{flg} =	5/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	9.92	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	11.70	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	52.974	kips
t_weld_web=	0.25	in
ϕRn_web=	110.48	kips
DCR_V=	**0.48**	OK

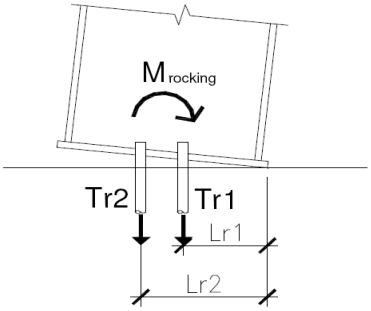
Tension Only

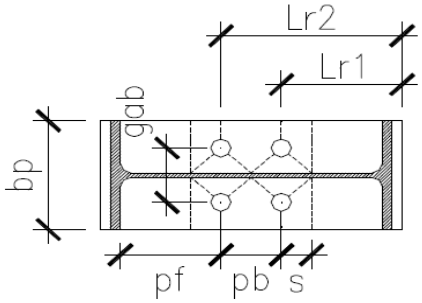
T=	0.00	kips
t_weld_flg=	0.31	in
ϕRn_flg=	162.88	kips
ϕRn=	273.36	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	52.97	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	436.23	kips	
DCR_V+T=	**0.121**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	5.125	in	
Lr2=	8.875	in	
Tr1=	46.10	kips	Assume equal to 1/2 of P
Tr2=	46.10	kips	Assume equal to 1/2 of P
Mrocking=	645.3	kip-in	Rocking moment about Bp tip
treq=	0.523	in	Plate thickness required for tension
tbp=	0.625	in	
DCR_bp=	**0.837**	OK	

bp=	12.250	in
pb=	3.75	in
gab=	3.75	in
pf=	3.585	in
s_tmp=	3.39	in
s=	3.585	in

Y=	72.69	in
gamma_r=	1.25	
ϕb=	0.9	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension
36	1.4 D	0.044	0.067	0.006	0.280
37	1.2 D + 1.6 L + 0.5 Lr	0.070	0.109	0.009	0.356
38	1.2 D + 1.6 L + 0.5 S	0.070	0.109	0.009	0.356
39	1.2 D + 1.6 L + 0.5 R	0.070	0.109	0.009	0.356
40	1.2 D + 1.6 Lr + 0.5 L	0.048	0.074	0.007	0.294
41	1.2 D + 1.6 Lr + 0.5 W	0.034	0.053	0.001	0.249
42	1.2 D + 1.6 Lr - 0.5 W	0.034	0.053	0.001	0.249
43	1.2 D + 1.6 S + 0.5 L	0.048	0.074	0.007	0.294
44	1.2 D + 1.6 S + 0.5 W	0.034	0.053	0.001	0.249
45	1.2 D + 1.6 S - 0.5 W	0.034	0.053	0.001	0.249
46	1.2 D + 1.6 R + 0.5 L	0.048	0.074	0.007	0.294
47	1.2 D + 1.6 R + 0.5 W	0.034	0.053	0.001	0.249
48	1.2 D + 1.6 R - 0.5 W	0.034	0.053	0.001	0.249
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.045	0.069	0.002	0.283
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.045	0.069	0.002	0.283
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.045	0.069	0.002	0.283
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.045	0.069	0.002	0.283
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.045	0.069	0.002	0.283
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.045	0.069	0.002	0.283
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.163	0.252	0.161	0.542
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.000	0.000	0.157	0.336
57	0.9 D + 1.0 W	0.026	0.040	0.001	0.216
58	0.9 D - 1.0 W	0.026	0.040	0.001	0.216
59	(0.9 - 0.2 SDS) D + E	0.133	0.206	0.160	0.490
60	(0.9 - 0.2 SDS) D - E	0.000	0.000	0.158	0.409
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.389	0.602	0.479	0.837
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.000	0.000	0.475	0.721
63	(0.9 - 0.2 SDS) D + Omega*E	0.359	0.555	0.478	0.804
64	(0.9 - 0.2 SDS) D - Omega*E	0.000	0.000	0.477	0.758
	Max=	0.389	0.602	0.479	0.837
	Check=	OK	OK	OK	OK

SMF COLUMN FIXED BASE DESIGN (COLUMN 1& 2)

Ref: AISC Seismic Design Manual, AISC Steel Design Manual, AISC 360-10

Column Fixed base Summary

Column = W12X106

Bearing Plate =	**10.875" x 5.75" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Transfer Reinforcement =	**(2) 0.75 in. x 36 in. bars**
Bearing Plate to Column Flange Weld Size =	**0.5 in**
Bearing Plate to Column Web Weld Size =	**0.5 in**
Longitudinal Reinforcement Requirement =	**1.59 in^2**

1. DESIGN DATA

Design Forces:

LC =	64	
V_{u1} =	52.809	kips
P_u =	85.088	kips

Bearing Plate

Grade =	ASTM A572 Gr 50		*Face plate material grade*
F_{yP} =	50	ksi	*Face plate yield stress*
B_P =	5.750	in	*Bearing plate width*
L_P =	10.875	in	*Bearing plate length*
t_P =	**0.625**	in	*Flange splice plate thickness*

Reinforcement Material:

Grade =	ASTM A615 Gr 60		*Reinforcement material grade*
F_{yR} =	60	ksi	*Reinforcement yield stress*
n_{bar} =	2	bars	*Number of bars in each region*
d_{bar} =	**3/4**	in.	*bar's diameter for transfer reinf.*
L_{bar} =	36	in	*default = 36 inch per ACI 318*

Concrete Material:

f'_c =	**2,500**	psi	*Concrete compressive stress*
b_w =	48.000	in	*Grade beam width*

Column:

Grade =	ASTM A572 Gr 50		
F_{yC} =	50	ksi	*Column yield stress*
F_u =	65	ksi	*Column ultimate stress*
H =	**171.75**	in	*Story Height*
L_e =	22.5	in.	*Column minimum embedment length*
d =	12.90	in	*Column depth*
t_w =	0.61	in	*Column web thickness*
b_f =	12.20	in	*Column flange width*
t_f =	0.99	in	*Exterior column flange thickness*
Z_x =	164.00	in^3	*Plastic section modulus X-axis*
A =	31.20	in^2	*Cross section area of column.*

Include Anchorage Moment:

No

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{min} =	0.4783	in	5/8*tsp Fillet size per AISC Steel Manual 14th edition and to develop Fy*Ast per AISC 341-16.
w_{size} =	1/2	in	*Fillet weld size on each size*

2. AXIAL DESIGN

2.1 Concrete Bearing:

A_1 =	125.0625	in^2	*Area of base plate*
A_2 =	125.0625	in^2	*Area of concrete support concentric with base plate*
Φ_{brg} =	0.65		*Resistance factor for concrete bearing*
$\sqrt{A_2/A_1}$ =	1.0		*=√A2/A1*
$f_{p(max)}$ =	4.25	ksi	*=0.85*f'c/ (1000*min(√A2/A1,2.0))*
P_p =	532	kips	*=A2*$f_{p(max)}$*
ϕP_p =	345	kips	*=Φbrg*Pp*
DCR_conc=	**0.246**	OK	*=P_{u1} / ϕP_P, OK if DCR_conc <= 1.03*

2.2 Plate Yeilding

N =	12.90		=d
B =	12.20		=bf
λ =	1.0		Conservative per AISC Manual Part 14
m =	0.32		=(N - 0.95 * d) / 2
n =	1.22		=(B - 0.8 * bf) / 2
λ_n' =	3.14		=(λ * sqrt(d * bf)) / 4
t_{minP1} =	0.486	in.	= λ n' * sqrt(2 * Pu /(0.9* F_{yP} * B *N))
t_{minP2} =	0.458	in.	=0.75*tw
DCR_Pyield =	**0.778**	OK	=max(t_{minP1}, t_{minP2})/t_P, OK if DCR_Pyield <= 1.03

3. SHEAR DESIGN

3.1 Column Shear Strength

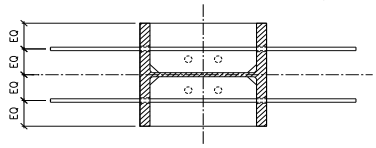
M_{pc} =	8200.0	kip-ft	=Fy * Zx (Nominal plastic flexural strength of the column)
V_{u2} =	95.49	kips	=2*M_{pc} / H
V_u =	95.49	kips	=max(V_{u1}, V_{u2})
β_1 =	0.85		from ACI 318 Section 10.2.7.3
g =	171.75		g=H
V_{n1} =	99.11	kips	=1.54*sqrt(f'c)*(bw/bf)$^{0.66}$ * β1*bf*(Le-2")*(0.58-0.22* β1)/(0.88+g/2/(Le-2"))
P =	59.28	kips	= ϕ_{brg}*(tf + 2)*bf*f'c
M_2 =	382.34	kip.in	= P*d/2
V_{n2} =	16.99	kips	= M_2/Le
V_n =	99.11	kips	= If (Include Anchorage Moment = "Yes", V_{n1} + V_{n2}, V_{n1})
DCR_ColV =	**0.963**	OK	=V_u/V_n, OK if DCR_Col <= 1.03

3.2 Required transfer reinforcement:

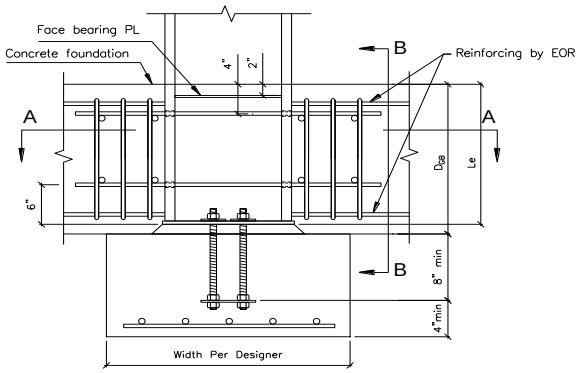
F_{ysr} =	60.00	ksi	value defined previously
A_{tbmin} =	0.313	in.^{2}	=0.03*f'c*(Le-2")*bf/ Fysr
A_{tbmax} =	77.129	in.^{2}	=0.08*(Le-2")*bw-As
A_{tb} =	0.884	in.^{2}	=n_{bar} * π * d_{bar}^2 / 4
DCR_Bar =	**0.354**	OK	=A_{tb}/ min(A_{tbmin}, A_{tbmax}), OK if DCR_Bar <= 1.03

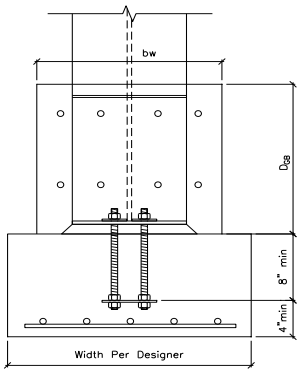
3.2 Longitudinal Foundation Reinforcement Requirement (EOR to provide calculations):

A_s =	1.59	in.^{2}	=V_u / F_{yR}



SECTION A-A





SECTION B-B

4. DESIGN SUMMARY:

Axial DCR = 0.778 Shear DCR = 0.963

Check = OK Check = OK

b_w = 48.000 in. (Min.)

L_e = 22.500 in. (Min.)

D_{GB} = 24.000 in. (Min.)

		Axial Check			Shear Check			
Combo	Load Combinations	P_{u1} (kips)	DCR	Check	V_{u1} (kips)	V_u (kips)	DCR	Check
36	1.4 D	10.31	0.73	OK	0.67	95.49	0.96	OK
37	1.2 D + 1.6 L + 0.5 Lr	16.67	0.73	OK	0.95	95.49	0.96	OK
38	1.2 D + 1.6 L + 0.5 S	16.67	0.73	OK	0.95	95.49	0.96	OK
39	1.2 D + 1.6 L + 0.5 R	16.67	0.73	OK	0.95	95.49	0.96	OK
40	1.2 D + 1.6 Lr + 0.5 L	11.36	0.73	OK	0.75	95.49	0.96	OK
41	1.2 D + 1.6 Lr + 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
42	1.2 D + 1.6 Lr - 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
43	1.2 D + 1.6 S + 0.5 L	11.36	0.73	OK	0.75	95.49	0.96	OK
44	1.2 D + 1.6 S + 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
45	1.2 D + 1.6 S - 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
46	1.2 D + 1.6 R + 0.5 L	11.36	0.73	OK	0.75	95.49	0.96	OK
47	1.2 D + 1.6 R + 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
48	1.2 D + 1.6 R - 0.8 W	8.15	0.73	OK	0.12	95.49	0.96	OK
49	1.2 D + 1.6 W + 0.5 L + 0.5 Lr	10.56	0.73	OK	0.22	95.49	0.96	OK
50	1.2 D - 1.6 W + 0.5 L + 0.5 Lr	10.56	0.73	OK	0.22	95.49	0.96	OK
51	1.2 D + 1.6 W + 0.5 L + 0.5 S	10.56	0.73	OK	0.22	95.49	0.96	OK
52	1.2 D - 1.6 W + 0.5 L + 0.5 S	10.56	0.73	OK	0.22	95.49	0.96	OK
53	1.2 D + 1.6 W + 0.5 L + 0.5 R	10.56	0.73	OK	0.22	95.49	0.96	OK
54	1.2 D - 1.6 W + 0.5 L + 0.5 R	10.56	0.73	OK	0.22	95.49	0.96	OK
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2*S	38.66	0.73	OK	17.82	95.49	0.96	OK
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	38.66	0.73	OK	17.82	95.49	0.96	OK
57	0.9 D + 1.6 W	6.11	0.73	OK	0.09	95.49	0.96	OK
58	0.9 D - 1.6 W	6.11	0.73	OK	0.09	95.49	0.96	OK
59	(0.9 - 0.2 SDS) D + E	31.55	0.73	OK	17.65	95.49	0.96	OK
60	(0.9 - 0.2 SDS) D - E	31.55	0.73	OK	17.65	95.49	0.96	OK
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	92.19	0.81	OK	52.97	95.49	0.96	OK
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	92.19	0.81	OK	52.97	95.49	0.96	OK
63	(0.9 - 0.2 SDS) D + Omega*E	85.09	0.78	OK	52.81	95.49	0.96	OK
64	(0.9 - 0.2 SDS) D - Omega*E	85.09	0.78	OK	52.81	95.49	0.96	OK

Simpson Strong-Tie
5956 West Las Positas Boulevard
Pleasanton, CA 94588

Date: 5/3/2019
By: bchi

SMF BEAM TOP WOOD NAILER ATTACHMENT DESIGN

BOLT LATERAL DESIGN VALUES

Wood Data		Steel Data		Bolt Data	
Species:	**DFL**	**Steel Gauge:**	**0.5**	**Bolt Size:**	**3/4**
Main Member	**Single 4X**	**F_u:**	**65000**	**Angle to Grain**	**0**
Countersink	**3/4"**				

ASTM A572 Gr 50 Steel

Bolt Design Information

D	F_{yb}	l_m	R_t	K_θ	R_e	R_d		
						I_m, I_s	II	III_m, III_s, IV
0.75	45000	2.750	5.500	1.00	0.0627	4	3.6	3.2

Steel Design Information

F_u	t_{steel}
65000	0.5

Wood Design Information

t_{wood}	SG	$F_{em(\theta)}$	F_{es}
2.75	0.5	5600	89375

Design Calculations

k_1	K_2	K_3	Z (I_m)	Z (I_s):	Z (II):	Z (III_m):	Z (III_s):	Z (IV):
0.180	0.604	6.667	2888	8379	1679	1938	2121	2210

	ALLOWABLE LOADS				
	100%	***115%***	***125%***	***133%***	***160%***
Single Shear:	**1679**	**1930**	**2098**	**2233**	**2686**

ASD Bolt Z_{allow} = **2686 lbs** **Per NDS: 3/4" Diameter Bolts, Single Shear, C_D = 1.60**

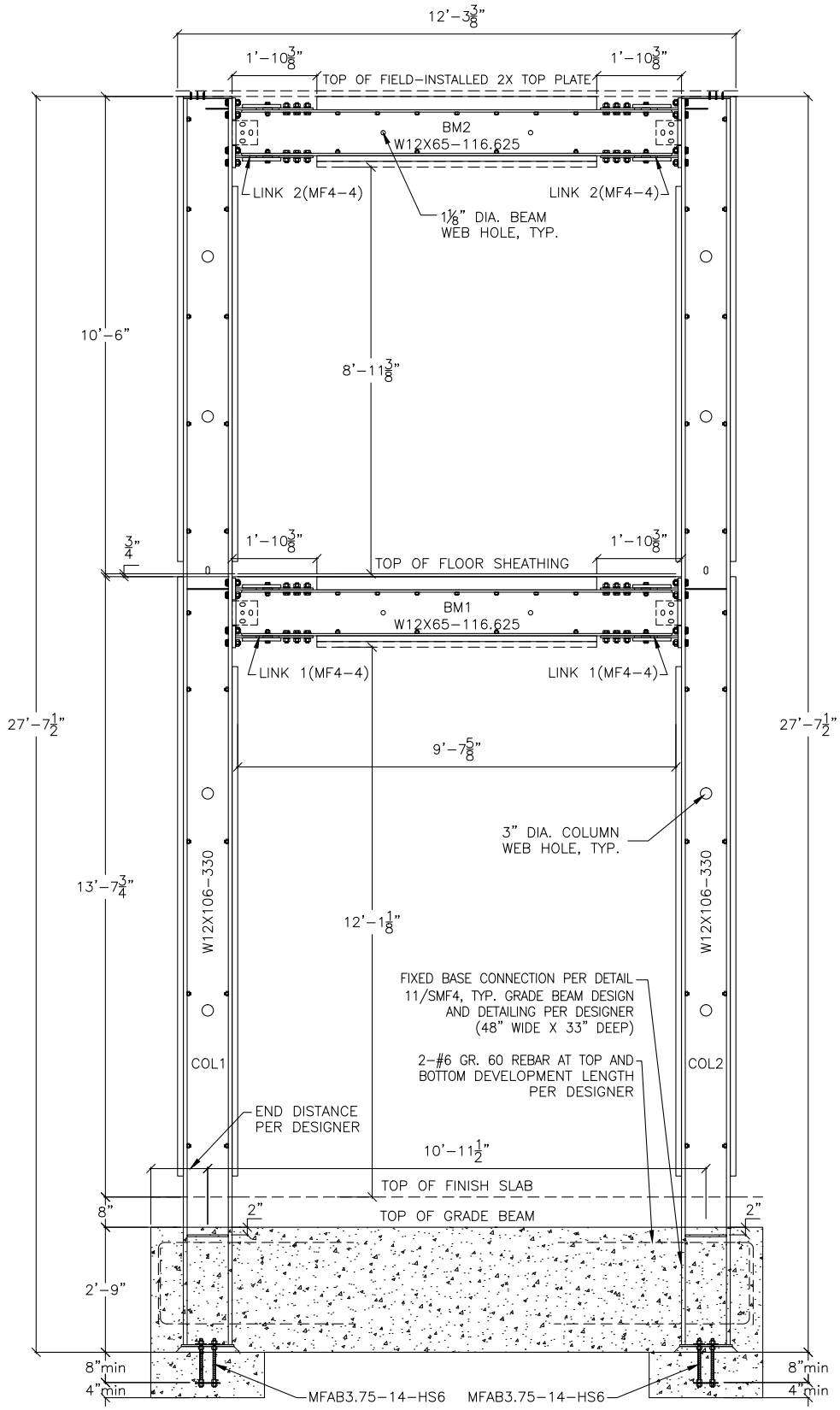
1.2 x ASD Bolt Z_{allow} = **3223 lbs** **1.2 Increase for Overstrength Forces per ASCE 7-10 12.4.3.3**

BEAM 1:

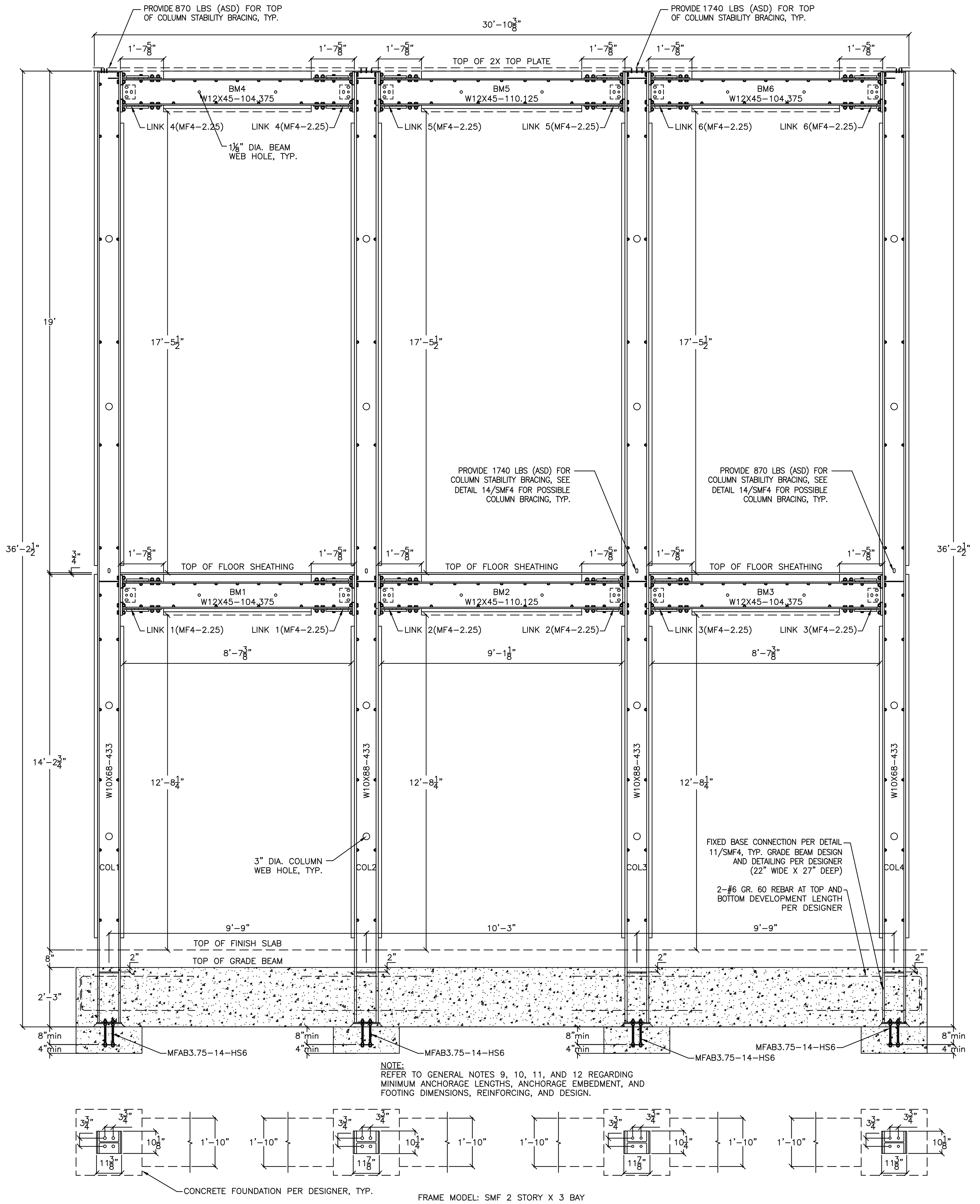
Seismic Lateral Load = 12,200 lbs
Wind Lateral Load = 0 lbs
Ωo = **3**
Vreq = 37,230 lbs
n_bolts_req'd = 11.55
n_bolt_min = **12** 3/4" dia. A307 Bolts

BEAM 2:

Seismic Lateral Load = 12,410 lbs
Wind Lateral Load = 0 lbs
Link = MF4-4 in
Lbm = 116.63 in
Lnailer = 64.875 in
S_bolt = 12.9375
Check = **OK**



Note: Footing sizes shown is based on anchorage design only. Actual footing/grade beam size and reinforcing must be determined by Designer based on project specific geometry and allowable soil pressures.

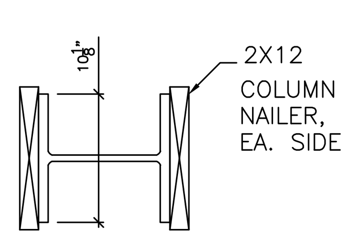


NOTE:
REFER TO GENERAL NOTES 9, 10, 11, AND 12 REGARDING MINIMUM ANCHORAGE LENGTHS, ANCHORAGE EMBEDMENT, AND FOOTING DIMENSIONS, REINFORCING, AND DESIGN.

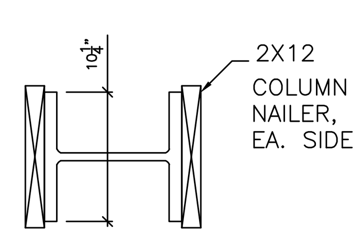
FRAME MODEL: SMF 2 STORY X 3 BAY

FRAME ELEVATION

SCALE:3/8" = 1'-0"



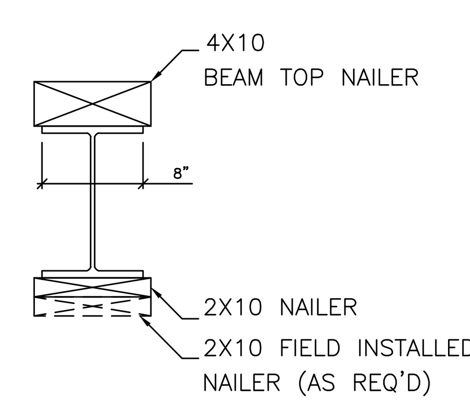
SECTION AT COLUMNS 1,4



SECTION AT COLUMNS 2,3

COLUMN SECTION

SCALE:1" = 1'-0"



SECTION AT BEAMS

BEAM SECTION

SCALE:1" = 1'-0"

Simpson Strong–Tie® Strong Frame® and the Yield–Link™ structural fuse are protected under one or more of the following US patents and applications: US patent No. 8,001,734 B2, US patent No. 8,375,652 B2, and US patent publication No. 2015/0159362, and must be supplied or licensed through Simpson Strong–tie.

GENERAL NOTES:

1. SIMPSON STRONG–TIE® STRONG FRAME® AND THE YIELD–LINK™ STRUCTURAL FUSE ARE PROTECTED UNDER ONE OR MORE OF THE FOLLOWING US PATENTS AND APPLICATIONS: US PATENT NO. 8,001,734 B2, US PATENT NO. 8,375,652 B2, AND US PATENT PUBLICATION NO. 2015/0159362, AND MUST BE SUPPLIED OR LICENSED THROUGH SIMPSON STRONG–TIE.
2. STRONG FRAME® SPECIAL MOMENT FRAME IS MANUFACTURED AND TRADEMARKED BY "SIMPSON STRONG–TIE COMPANY INC." HOME OFFICE: 5956 W. LAS POSITAS BLVD. , PLEASANTON, CA 94588 TEL: (800) 999–5099, FAX: (925) 847–1597. "SIMPSON STRONG–TIE COMPANY INC." IS AN ISO 9001 REGISTERED COMPANY.
3. DESIGN FOR STRONG FRAME® MOMENT FRAMES ARE IN ACCORDANCE WITH THE FOLLOWING:
   - 2015 AND 2012 INTERNATIONAL BUILDING CODE
   - AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360–05, 360–10)
   - AISC SEISMIC PROVISIONS (ANSI/AISC 341–05, 341–10)
   - RCSC SPECIFICATION FOR STRUCTURAL JOINTS USING ASTM A325 OR A490 BOLTS
   - BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI318–11, ACI318–14)
4. USE OF THIS PRODUCT IS SUBJECT TO THE APPROVAL OF THE LOCAL BUILDING DEPARTMENT.
5. THIS PRODUCT IS PART OF THE OVERALL LATERAL FORCE RESISTING SYSTEM OF THE STRUCTURE. DESIGN OF THE BUILDING'S LATERAL FORCE RESISTING SYSTEM, INCLUDING THE LOAD PATH TO TRANSFER LATERAL FORCES FROM THE STRUCTURE TO THE GROUND, IS THE RESPONSIBILITY OF THE DESIGNER.
6. THE DESIGNER MUST SPECIFY THE REQUIRED COMPONENTS OF THE COMPLETE LOAD TRANSFER PATH INCLUDING DIAPHRAGMS, SHEAR TRANSFER, CHORDS AND COLLECTORS AND FOUNDATIONS.
7. ALL CONNECTED MEMBERS AND RELATED ELEMENTS SHALL BE DESIGNED BY THE DESIGNER.
8. DESIGNER IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS. SEE LIMITATIONS NOTED ON SHEET SMF3.
9. ANCHORAGE LENGTHS PROVIDED ARE SHOWN FOR MINIMUM EMBEDMENT INTO FOOTING BASED ON TENSION ANCHORAGE DESIGN ONLY. ACTUAL LENGTH OF ANCHORAGE SHALL BE PER DESIGNER'S SPECIFICATIONS AND PROJECT SPECIFIC INSTALLATION REQUIREMENTS.
10. PRE–ASSEMBLED ANCHORAGE KITS PROVIDED BY SIMPSON (MFSL OR MFAB) SHALL BE SPECIFIED BY DESIGNER AND SHOULD INCLUDE ANCHORAGE TYPE, ROD GRADE, AND LENGTH OF ASSEMBLY. REFER TO DETAIL 2 FOR AVAILABLE LENGTHS OF FULLY ASSEMBLED ANCHORAGE ASSEMBLIES. EXTENSION KITS IN 36" LENGTHS ARE AVAILABLE FOR USE IN STEMWALLS OR APPLICATIONS WHERE DEEPER EMBEDMENT IS REQUIRED.
11. FOOTING DIMENSIONS SHOWN ARE THE MINIMUMS REQUIRED FOR CONCRETE ANCHORAGE REQUIREMENTS ONLY. THE DESIGNER MUST DETERMINE REQUIRED FOOTING SIZE AND REINFORCING FOR OTHER DESIGN LIMITS, SUCH AS FOUNDATION SHEAR AND BENDING, SOIL BEARING SHEAR TRANSFER, AND FRAME STABILITY / OVERTURNING.
12. DESIGNER MUST DETAIL ACTUAL FOOTING / GRADE BEAM SIZE AND REINFORCING.
13. HOLES IN BASE PLATES ARE OVER–SIZED FOR ERECTION TOLERANCE. DESIGNER MUST EVALUATE EFFECTS OF OVER–SIZED HOLES AND PROVIDE PLATE WASHER WITH STANDARD–SIZE HOLES WELDED TO BASE PLATE OR REQUEST BASE PLATES WITH STANDARD SIZE HOLES WHERE REQUIRED.
14. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, CONDITIONS, ELEVATIONS, ETC. PRIOR TO INSTALLATION OF ANY COMPONENTS FOR THE STEEL STRONG FRAME SYSTEM. IF ANY DISCREPANCIES ARE FOUND, THEY SHALL BE BROUGHT TO THE ATTENTION OF THE DESIGNER FOR CLARIFICATION PRIOR TO CONSTRUCTION.
15. INSTALLATION OF PRODUCT SHALL BE DONE IN CONFORMANCE WITH THESE DRAWINGS AND ICC ESR–2802. THE PERFORMANCE OF MODIFIED PRODUCTS OR ALTERED INSTALLATION PROCEDURES ARE THE SOLE RESPONSIBILITY OF THE DESIGNER.
16. SIMPSON STRONG–TIE® COMPANY, INC. RESERVES THE RIGHT TO CHANGE SPECIFICATIONS, DESIGNS, AND MODELS WITHOUT NOTICE OR LIABILITY FOR SUCH CHANGES.
17. ALL HARDWARE CALLED OUT IS SIMPSON STRONG–TIE®.
18. USE OF A SIMPSON STRONG–TIE PRODUCT DOES NOT IMPLY THAT SIMPSON STRONG–TIE ENDORSES ANY PROJECT, STRUCTURE OR USE. NO LICENSE IS GRANTED WITH RESPECT TO ANY SIMPSON STRONG–TIE TRADEMARK OR OTHER INTELLECTUAL PROPERTY RIGHTS. WRITTEN PERMISSION MUST BE OBTAINED PRIOR TO USING ANY SIMPSON STRONG–TIE TRADEMARKS OR PROPRIETARY DOCUMENTS AND MATERIALS.
19. SIMPSON STRONG–TIE IS NOT AFFILIATED WITH, AND DOES NOT SPONSOR OR ENDORSE, THE DESIGNER, INSTALLER OR USERS OF THIS DRAWING, NOR DOES SIMPSON STRONG–TIE HAVE ANY JOINT VENTURE, PARTNERSHIP, AGENCY, EMPLOYMENT OR FIDUCIARY RELATIONSHIP WITH SUCH PERSONS.

MATERIAL:

1. BARS/PLATES: ASTM 572 GR. 50, ASTM A529 GR. 50, OR ASTM A1011 HSLAS GR. 50
2. W–SECTIONS (HOT ROLLED SECTIONS): ASTM A992
3. LINK TO COLUMN FLANGE HIGH STRENGTH BOLTS: 7/8" DIA. ASTM A325, TYPE 1 (SNUG–TIGHT)
4. BRP TO BEAM FLANGE AND SHEAR PLATE TO BEAM WEB HIGH STRENGTH BOLTS: ASTM A325, TYPE 1 (SNUG–TIGHT)
5. LINK TO BEAM FLANGE HIGH STRENGTH BOLTS: ASTM F2280 TWIST OFF TYPE (A490 EQUIVALENT) (PRETENSIONED)
6. BEAM TOP FLANGE WOOD NAILER BOLT: ASTM A307 GR. A
7. CARRIAGE BOLTS: ASTM A307 GR. A
8. ANCHOR RODS: ASTM F1554 GR 36 OR A36 (MFAB, MFSL, AND MF–ATR6EXT–LS); ASTM A449 (MFAB–HS, MFSL–HS, AND MF–ATR6EXT–HS)
9. GROUT: ASTM C1107, MINIMUM 5,000 PSI COMPRESSIVE STRENGTH

INSTALLATION AND FIELD MODIFICATIONS:

THESE GENERAL INSTRUCTIONS FOR THE INSTALLER ARE PROVIDED TO ENSURE PROPER SELECTION AND INSTALLATION OF SIMPSON STRONG–TIE COMPANY INC. PRODUCTS AND MUST BE FOLLOWED CAREFULLY. THESE GENERAL INSTRUCTIONS ARE IN ADDITION TO THE SPECIFIC INSTALLATION INSTRUCTIONS AND NOTES PROVIDED FOR EACH PARTICULAR PRODUCT, ALL OF WHICH SHOULD BE CONSULTED PRIOR TO AND DURING INSTALLATION OF SIMPSON STRONG–TIE COMPANY INC. PRODUCTS.

1. PROPER PRODUCT INSTALLATION REQUIRES CAREFUL ATTENTION TO ALL NOTES AND INSTRUCTIONS. IN ADDITIONAL TO THE NOTES, WARNINGS, AND INSTRUCTIONS PROVIDED IN THE CATALOG, INSTALLERS, DESIGNERS, ENGINEERS AND CONSUMERS SHOULD CONSULT THE SIMPSON STRONG–TIE COMPANY INC. WEBSITE AT WWW.STRONGTIE.COM TO OBTAIN ADDITIONAL INFORMATION FOR INSTALLATION, SPECIFICATIONS, CODE REPORTS, TECHNICAL FLIERS AND BULLETINS, FAQS, AND OTHER PERTINENT INFORMATION.
2. PROVIDE TEMPORARY DIAGONAL BRACING OF STRONG FRAME® AS REQUIRED UNTIL FRAME IS TIED INTO THE FLOOR OR ROOF FRAMING ABOVE.
3. USE PROPER SAFETY AND INSTALLATION EQUIPMENT DURING INSTALLATION OF STRONG FRAME®.
4. ALL SPECIFIED FASTENERS MUST BE INSTALLED ACCORDING TO THE INSTRUCTIONS PROVIDED IN THE CATALOG, CODE REPORT, AND INSTALLATION DETAILS. INCORRECT FASTENER QUANTITY, SIZE, PLACEMENT, TYPE, MATERIAL, OR FINISH MAY CAUSE THE CONNECTION TO FAIL.
5. FILL ALL FASTENER HOLES AS SPECIFIED IN THE INSTALLATION INSTRUCTIONS FOR THE SPECIFIED PRODUCT. INSTALL ALL FASTENERS BEFORE LOADING THE FRAME. SOME PRE–INSTALLED ITEMS MAY NOT USE ALL HOLES.
6. NUTS SHALL BE INSTALLED SUCH THAT THE END OF THE THREADED ROD OR BOLT IS AT LEAST FLUSH WITH THE TOP OF THE NUT.
7. REFER TO DETAIL 12/SMF3 FOR ALLOWABLE HOLE OPENINGS IN BEAM AND COLUMNS.
8. REFER TO DETAIL 11/SMF3 FOR CONNECTION PROTECTED ZONE.
9. WELDING SHALL BE IN ACCORDANCE WITH AWS D1.1 AND AWS D1.8 (AS APPLICABLE FOR SEISMIC). WELDS SHALL BE SPECIFIED BY THE DESIGNER. PROVIDE WELDING SPECIAL INSPECTION AS REQUIRED BY THE LOCAL BUILDING DEPARTMENT.

INSPECTIONS:

1. WELDING OF FRAME MEMBERS AND APPLICABLE WELDING SPECIAL INSPECTIONS REQUIRED BY IBC SECTION 1707 ARE PERFORMED ON THE PREMISES OF A FABRICATOR REGISTERED AND APPROVED IN ACCORDANCE WITH THE REQUIREMENTS OF IBC SECTION 1704.2.5 FOR FABRICATOR APPROVAL.
2. PRE–INSTALLATION VERIFICATION TESTING IS PERFORMED ON HIGH–STRENGTH FASTENER ASSEMBLIES.
3. INSPECTION REQUIREMENTS OUTSIDE THE SHOP MANUFACTURING AND ASSEMBLY PROCESS SHALL BE IN ACCORDANCE WITH THE LOCAL CODE, BASED ON BUILDING OCCUPANCY, CONCRETE STRENGTH, REQUIREMENTS OF THE LOCAL BUILDING OFFICIAL, AND OTHER CONSIDERATIONS AND SHALL BE SPECIFIED BY THE DESIGNER.
4. GROUTING UNDER COLUMN BASE PLATE MAY REQUIRE SPECIAL INSPECTION, CONTACT THE LOCAL BUILDING DEPARTMENT FOR COMPLIANCE REQUIREMENTS.
5. CONTACT SIMPSON STRONG–TIE® AT 800–999–5099 TO REQUEST PRE–INSTALLATION TESTING, WELDING REPORTS, MILL CERTS, ETC. WHEN REQUIRED.

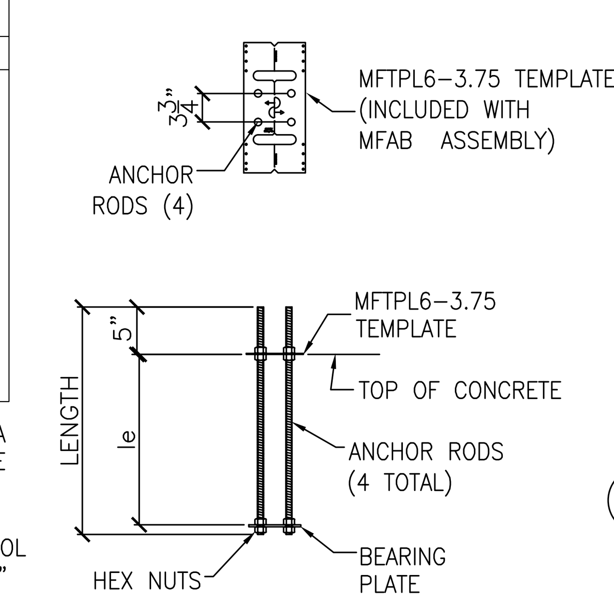
GENERAL NOTES — 1

MODEL NO.	ROD SIZE & NUMBER	LENGTH (in)	l_e (in)	BEARING PLATE (in)
ALL SMF COLUMNS				
MFAB3.75–14–6	4 – ¾	14	8	⅜ x 7 x 7
MFAB3.75–14–HS6	4 – ¾	14	8	
MFAB3.75–18–6	4 – ¾	18	12	
MFAB3.75–18–HS6	4 – ¾	18	12	
MFAB3.75–24–6	4 – ¾	24	18	
MFAB3.75–24–HS6	4 – ¾	24	18	
MFAB3.75–30–6	4 – ¾	30	24	
MFAB3.75–30–HS6	4 – ¾	30	24	
MFAB3.75–36–6	4 – ¾	36	30	
MFAB3.75–36–HS6	4 – ¾	36	30	

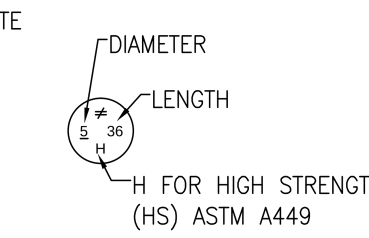
THE MFAB ANCHOR ASSEMBLIES HAVE BEEN ENGINEERED TO PROVIDE A SOLUTION MEETING THE 2012 AND 2015 INTERNATIONAL BUILDING CODE REQUIREMENTS FOR BOTH TENSION AND SHEAR.

INSPECTION IS EASY; THE HEAD IS STAMPED WITH A "NO EQUAL" SYMBOL FOR IDENTIFICATION, BOLT LENGTH, BOLT DIAMETER, AND OPTIONAL "HS" FOR HIGH STRENGTH IF SPECIFIED.

MFAB ANCHOR ASSEMBLIES REQUIRE ADDITIONAL REINFORCING TO PROVIDE A COMPLETE TENSION AND SHEAR SOLUTION. DESIGNER MUST SPECIFY ADDITIONAL REINFORCING TIES OR HAIRPINS AS REQUIRED FOR SHEAR.



MFAB3.75

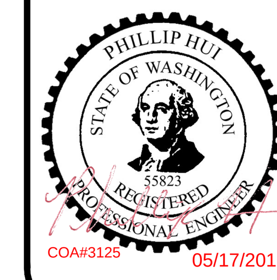


ANCHOR ROD STAMP

MFAB ANCHORAGE ASSEMBLIES — 2

A92

REVISIONS		
NO.	DATE	



SIMPSON STRONG–TIE, CO. INC.
- 5956 W. Las Positas Blvd. Pleasanton, CA 94588
- Tel: (800) 999–5099
- Fax: (925) 847–15977
- Web site: www.strongtie.com

SIMPSON Strong-Tie® THERE IS NO EQUAL

THE MOMENT FRAME(S) IS DESIGNED FOLLOWING INFORMATION PROVIED BY:

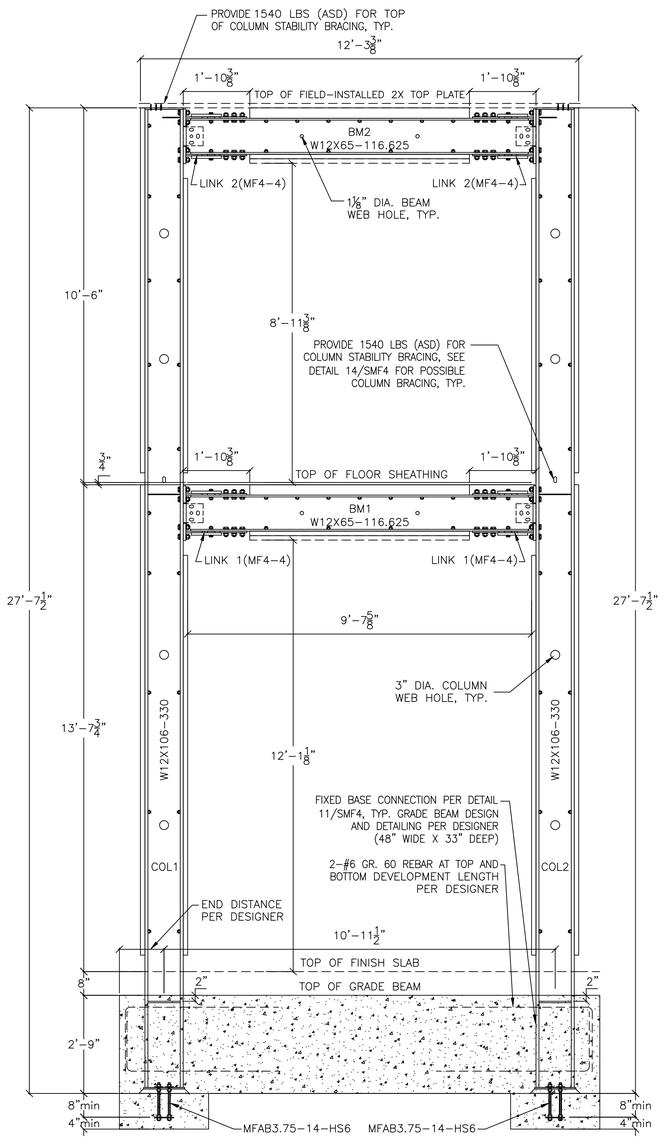
DATE OF PLANS (D.O.P.): 4/17/2019

PROVIDED TO SIMPSON STRONG-TIE BY: JCW

STRONG-FRAME®
FRAME ELEVATION DRAWING
ENGINEERED DESIGN
6059 77TH AVE SE - MF-1 (GRID 1)

NAME: P.H.
DATE: 05/17/2019
SCALE: N.T.S.
SHEET:
MF-1

JOB NO. ES–191628

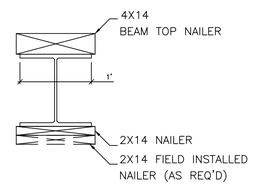


NOTE:
REFER TO GENERAL NOTES 9, 10, 11, AND 12 REGARDING MINIMUM ANCHORAGE LENGTHS, ANCHORAGE EMBEDMENT, AND FOOTING DIMENSIONS, REINFORCING, AND DESIGN.

SMF-2X12z12z12z-116.625x171.75x330-(MF4-4)-(MF4-4)

FRAME ELEVATION

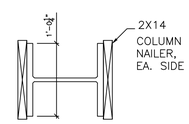
SCALE:1/2" = 1'-0"



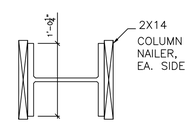
SECTION AT BEAMS

BEAM SECTION

SCALE:1" = 1'-0"



SECTION AT COLUMN 1



SECTION AT COLUMN 2

COLUMN SECTION

SCALE:1" = 1'-0"

Simpson Strong-Tie® Strong Frame® and the Yield-Link™ structural fuse are protected under one or more of the following US patents and applications: US patent No. 8,001,734 B2, US patent No. 8,375,652 B2, and US patent publication No. 2015/0159362, and must be supplied or licensed through Simpson Strong-tie.

GENERAL NOTES:

1. SIMPSON STRONG-TIE® STRONG FRAME® AND THE YIELD-LINK™ STRUCTURAL FUSE ARE PROTECTED UNDER ONE OR MORE OF THE FOLLOWING US PATENTS AND APPLICATIONS: US PATENT NO. 8,001,734 B2, US PATENT NO. 8,375,652 B2, AND US PATENT PUBLICATION NO. 2015/0159362, AND MUST BE SUPPLIED OR LICENSED THROUGH SIMPSON STRONG-TIE.
2. STRONG FRAME® SPECIAL MOMENT FRAME IS MANUFACTURED AND TRADEMARKED BY "SIMPSON STRONG-TIE COMPANY INC." HOME OFFICE: 5956 W. LAS POSITAS BLVD. , PLEASANTON, CA 94588 TEL: (800) 999-5099, FAX: (925) 847-1597. "SIMPSON STRONG-TIE COMPANY INC." IS AN ISO 9001 REGISTERED COMPANY.
3. DESIGN FOR STRONG FRAME® MOMENT FRAMES ARE IN ACCORDANCE WITH THE FOLLOWING:
   - 2015 AND 2012 INTERNATIONAL BUILDING CODE
   - AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360-05, 360-10)
   - AISC SEISMIC PROVISIONS (ANSI/AISC 341-05, 341-10)
   - RCSC SPECIFICATION FOR STRUCTURAL JOINTS USING ASTM A325 OR A490 BOLTS
   - BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI318-11, ACI318-14)
4. USE OF THIS PRODUCT IS SUBJECT TO THE APPROVAL OF THE LOCAL BUILDING DEPARTMENT.
5. THIS PRODUCT IS PART OF THE OVERALL LATERAL FORCE RESISTING SYSTEM OF THE STRUCTURE. DESIGN OF THE BUILDING'S LATERAL FORCE RESISTING SYSTEM, INCLUDING THE LOAD PATH TO TRANSFER LATERAL FORCES FROM THE STRUCTURE TO THE GROUND, IS THE RESPONSIBILITY OF THE DESIGNER.
6. THE DESIGNER MUST SPECIFY THE REQUIRED COMPONENTS OF THE COMPLETE LOAD TRANSFER PATH INCLUDING DIAPHRAGMS, SHEAR TRANSFER, CHORDS AND COLLECTORS AND FOUNDATIONS.
7. ALL CONNECTED MEMBERS AND RELATED ELEMENTS SHALL BE DESIGNED BY THE DESIGNER.
8. DESIGNER IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS. SEE LIMITATIONS NOTED ON SHEET SMF3.
9. ANCHORAGE LENGTHS PROVIDED ARE SHOWN FOR MINIMUM EMBEDMENT INTO FOOTING BASED ON TENSION ANCHORAGE DESIGN ONLY. ACTUAL LENGTH OF ANCHORAGE SHALL BE PER DESIGNER'S SPECIFICATIONS AND PROJECT SPECIFIC INSTALLATION REQUIREMENTS.
10. PRE-ASSEMBLED ANCHORAGE KITS PROVIDED BY SIMPSON (MFSL OR MFAB) SHALL BE SPECIFIED BY DESIGNER AND SHOULD INCLUDE ANCHORAGE TYPE, ROD GRADE, AND LENGTH OF ASSEMBLY. REFER TO DETAIL 2 FOR AVAILABLE LENGTHS OF FULLY ASSEMBLED ANCHORAGE ASSEMBLIES. EXTENSION KITS IN 36" LENGTHS ARE AVAILABLE FOR USE IN STEMWALLS OR APPLICATIONS WHERE DEEPER EMBEDMENT IS REQUIRED.
11. FOOTING DIMENSIONS SHOWN ARE THE MINIMUMS REQUIRED FOR CONCRETE ANCHORAGE REQUIREMENTS ONLY. THE DESIGNER MUST DETERMINE REQUIRED FOOTING SIZE AND REINFORCING FOR OTHER DESIGN LIMITS, SUCH AS FOUNDATION SHEAR AND BENDING, SOIL BEARING SHEAR TRANSFER, AND FRAME STABILITY / OVERTURNING.
12. DESIGNER MUST DETAIL ACTUAL FOOTING / GRADE BEAM SIZE AND REINFORCING.
13. HOLES IN BASE PLATES ARE OVER-SIZED FOR ERECTION TOLERANCE. DESIGNER MUST EVALUATE EFFECTS OF OVER-SIZED HOLES AND PROVIDE PLATE WASHER WITH STANDARD-SIZE HOLES WELDED TO BASE PLATE OR REQUEST BASE PLATES WITH STANDARD SIZE HOLES WHERE REQUIRED.
14. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, CONDITIONS, ELEVATIONS, ETC. PRIOR TO INSTALLATION OF ANY COMPONENTS FOR THE STEEL STRONG FRAME SYSTEM. IF ANY DISCREPANCIES ARE FOUND, THEY SHALL BE BROUGHT TO THE ATTENTION OF THE DESIGNER FOR CLARIFICATION PRIOR TO CONSTRUCTION.
15. INSTALLATION OF PRODUCT SHALL BE DONE IN CONFORMANCE WITH THESE DRAWINGS AND ICC ESR-2802. THE PERFORMANCE OF MODIFIED PRODUCTS OR ALTERED INSTALLATION PROCEDURES ARE THE SOLE RESPONSIBILITY OF THE DESIGNER.
16. SIMPSON STRONG-TIE® COMPANY, INC. RESERVES THE RIGHT TO CHANGE SPECIFICATIONS, DESIGNS, AND MODELS WITHOUT NOTICE OR LIABILITY FOR SUCH CHANGES.
17. ALL HARDWARE CALLED OUT IS SIMPSON STRONG-TIE®.
18. USE OF A SIMPSON STRONG-TIE PRODUCT DOES NOT IMPLY THAT SIMPSON STRONG-TIE ENDORSES ANY PROJECT, STRUCTURE OR USE. NO LICENSE IS GRANTED WITH RESPECT TO ANY SIMPSON STRONG-TIE TRADEMARK OR OTHER INTELLECTUAL PROPERTY RIGHTS. WRITTEN PERMISSION MUST BE OBTAINED PRIOR TO USING ANY SIMPSON STRONG-TIE TRADEMARKS OR PROPRIETARY DOCUMENTS AND MATERIALS.
19. SIMPSON STRONG-TIE IS NOT AFFILIATED WITH, AND DOES NOT SPONSOR OR ENDORSE, THE DESIGNER, INSTALLER OR USERS OF THIS DRAWING, NOR DOES SIMPSON STRONG-TIE HAVE ANY JOINT VENTURE, PARTNERSHIP, AGENCY, EMPLOYMENT OR FIDUCIARY RELATIONSHIP WITH SUCH PERSONS.

MATERIAL:

1. BARS/PLATES: ASTM 572 GR. 50, ASTM A529 GR. 50, OR ASTM A1011 HSLAS GR. 50
2. W-SECTIONS (HOT ROLLED SECTIONS): ASTM A992
3. LINK TO COLUMN FLANGE HIGH STRENGTH BOLTS: 7/8" DIA. ASTM A325, TYPE 1 (SNUG-TIGHT)
4. BRP TO BEAM FLANGE AND SHEAR PLATE TO BEAM WEB HIGH STRENGTH BOLTS: ASTM A325, TYPE 1 (SNUG-TIGHT)
5. LINK TO BEAM FLANGE HIGH STRENGTH BOLTS: ASTM F2280 TWIST OFF TYPE (A490 EQUIVALENT) (PRETENSIONED)
6. BEAM TOP FLANGE WOOD NAILER BOLT: ASTM A307 GR. A
7. CARRIAGE BOLTS: ASTM A307 GR. A
8. ANCHOR RODS: ASTM F1554 GR 36 OR A36 (MFAB, MFSL, AND MF-ATR6EXT-LS); ASTM A449 (MFAB-HS, MFSL-HS, AND MF-ATR6EXT-HS)
9. GROUT: ASTM C1107, MINIMUM 5,000 PSI COMPRESSIVE STRENGTH

INSTALLATION AND FIELD MODIFICATIONS:

THESE GENERAL INSTRUCTIONS FOR THE INSTALLER ARE PROVIDED TO ENSURE PROPER SELECTION AND INSTALLATION OF SIMPSON STRONG-TIE COMPANY INC. PRODUCTS AND MUST BE FOLLOWED CAREFULLY. THESE GENERAL INSTRUCTIONS ARE IN ADDITION TO THE SPECIFIC INSTALLATION INSTRUCTIONS AND NOTES PROVIDED FOR EACH PARTICULAR PRODUCT, ALL OF WHICH SHOULD BE CONSULTED PRIOR TO AND DURING INSTALLATION OF SIMPSON STRONG-TIE COMPANY INC. PRODUCTS.

1. PROPER PRODUCT INSTALLATION REQUIRES CAREFUL ATTENTION TO ALL NOTES AND INSTRUCTIONS. IN ADDITIONAL TO THE NOTES, WARNINGS, AND INSTRUCTIONS PROVIDED IN THE CATALOG, INSTALLERS, DESIGNERS, ENGINEERS AND CONSUMERS SHOULD CONSULT THE SIMPSON STRONG-TIE COMPANY INC. WEBSITE AT WWW.STRONGTIE.COM TO OBTAIN ADDITIONAL INFORMATION FOR INSTALLATION, SPECIFICATIONS, CODE REPORTS, TECHNICAL FLIERS AND BULLETINS, FAQS, AND OTHER PERTINENT INFORMATION.
2. PROVIDE TEMPORARY DIAGONAL BRACING OF STRONG FRAME® AS REQUIRED UNTIL FRAME IS TIED INTO THE FLOOR OR ROOF FRAMING ABOVE.
3. USE PROPER SAFETY AND INSTALLATION EQUIPMENT DURING INSTALLATION OF STRONG FRAME®.
4. ALL SPECIFIED FASTENERS MUST BE INSTALLED ACCORDING TO THE INSTRUCTIONS PROVIDED IN THE CATALOG, CODE REPORT, AND INSTALLATION DETAILS. INCORRECT FASTENER QUANTITY, SIZE, PLACEMENT, TYPE, MATERIAL, OR FINISH MAY CAUSE THE CONNECTION TO FAIL.
5. FILL ALL FASTENER HOLES AS SPECIFIED IN THE INSTALLATION INSTRUCTIONS FOR THE SPECIFIED PRODUCT. INSTALL ALL FASTENERS BEFORE LOADING THE FRAME. SOME PRE-INSTALLED ITEMS MAY NOT USE ALL HOLES.
6. NUTS SHALL BE INSTALLED SUCH THAT THE END OF THE THREADED ROD OR BOLT IS AT LEAST FLUSH WITH THE TOP OF THE NUT.
7. REFER TO DETAIL 12/SMF3 FOR ALLOWABLE HOLE OPENINGS IN BEAM AND COLUMNS.
8. REFER TO DETAIL 11/SMF3 FOR CONNECTION PROTECTED ZONE.
9. WELDING SHALL BE IN ACCORDANCE WITH AWS D1.1 AND AWS D1.8 (AS APPLICABLE FOR SEISMIC). WELDS SHALL BE SPECIFIED BY THE DESIGNER. PROVIDE WELDING SPECIAL INSPECTION AS REQUIRED BY THE LOCAL BUILDING DEPARTMENT.

INSPECTIONS:

1. WELDING OF FRAME MEMBERS AND APPLICABLE WELDING SPECIAL INSPECTIONS REQUIRED BY IBC SECTION 1707 ARE PERFORMED ON THE PREMISES OF A FABRICATOR REGISTERED AND APPROVED IN ACCORDANCE WITH THE REQUIREMENTS OF IBC SECTION 1704.2.5 FOR FABRICATOR APPROVAL.
2. PRE-INSTALLATION VERIFICATION TESTING IS PERFORMED ON HIGH-STRENGTH FASTENER ASSEMBLIES.
3. INSPECTION REQUIREMENTS OUTSIDE THE SHOP MANUFACTURING AND ASSEMBLY PROCESS SHALL BE IN ACCORDANCE WITH THE LOCAL CODE, BASED ON BUILDING OCCUPANCY, CONCRETE STRENGTH, REQUIREMENTS OF THE LOCAL BUILDING OFFICIAL, AND OTHER CONSIDERATIONS AND SHALL BE SPECIFIED BY THE DESIGNER.
4. GROUTING UNDER COLUMN BASE PLATE MAY REQUIRE SPECIAL INSPECTION, CONTACT THE LOCAL BUILDING DEPARTMENT FOR COMPLIANCE REQUIREMENTS.
5. CONTACT SIMPSON STRONG-TIE® AT 800-999-5099 TO REQUEST PRE-INSTALLATION TESTING, WELDING REPORTS, MILL CERTS, ETC. WHEN REQUIRED.

GENERAL NOTES

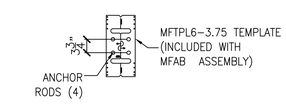
1

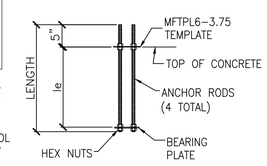
MODEL NO.	ROD SIZE & NUMBER	LENGTH (in)	l_e (in)	BEARING PLATE (in)
ALL SMF COLUMNS				
MFAB3.75-14-6	4 - ¾	14	8	⅜ x 7 x 7
MFAB3.75-14-HS6	4 - ¾	14	8	
MFAB3.75-18-6	4 - ¾	18	12	
MFAB3.75-18-HS6	4 - ¾	18	12	
MFAB3.75-24-6	4 - ¾	24	18	
MFAB3.75-24-HS6	4 - ¾	24	18	
MFAB3.75-30-6	4 - ¾	30	24	
MFAB3.75-30-HS6	4 - ¾	30	24	
MFAB3.75-36-6	4 - ¾	36	30	
MFAB3.75-36-HS6	4 - ¾	36	30	

THE MFAB ANCHOR ASSEMBLIES HAVE BEEN ENGINEERED TO PROVIDE A SOLUTION MEETING THE 2012 AND 2015 INTERNATIONAL BUILDING CODE REQUIREMENTS FOR BOTH TENSION AND SHEAR.

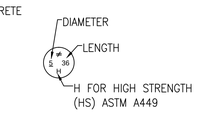
INSPECTION IS EASY; THE HEAD IS STAMPED WITH A "NO EQUAL" SYMBOL FOR IDENTIFICATION, BOLT LENGTH, BOLT DIAMETER, AND OPTIONAL "HS" FOR HIGH STRENGTH IF SPECIFIED.

MFAB ANCHOR ASSEMBLIES REQUIRE ADDITIONAL REINFORCING TO PROVIDE A COMPLETE TENSION AND SHEAR SOLUTION. DESIGNER MUST SPECIFY ADDITIONAL REINFORCING TIES OR HAIRPINS AS REQUIRED FOR SHEAR.





MFAB3.75



ANCHOR ROD STAMP

MFAB ANCHORAGE ASSEMBLIES

2

A93

NO.	DATE	REVISIONS

PHILLIP HU, STATE OF WASHINGTON, REGISTERED PROFESSIONAL ENGINEER
COA#3125
05/17/2019

SIMPSON STRONG-TIE, CO. INC.
• 5956 W. Las Positas Blvd.
Pleasanton, CA 94588
• Tel: (800) 999-5099
• Fax: (925) 847-1597
• Web site: www.strongtie.com

SIMPSON Strong-Tie ≠ THERE IS NO EQUAL

THE MOMENT FRAME(S) IS DESIGNED FOLLOWING INFORMATION PROVIDED BY: Dible Engineers, Inc.
DATE OF PLANS (D.O.P.): 2019-05-03
PROVIDED TO SIMPSON STRONG-TIE BY: Dible Engineers, Inc.

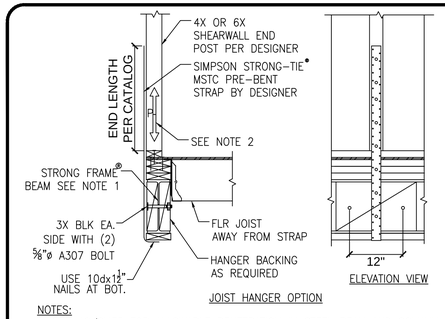
STRONG-FRAME®

FRAME ELEVATION DRAWING
ENGINEERED DESIGN
6059 77TH AVE SE - MF-2 (GRID 3)

NAME: P.H.
DATE: 05/17/2019
SCALE: N.T.S.
SHEET:

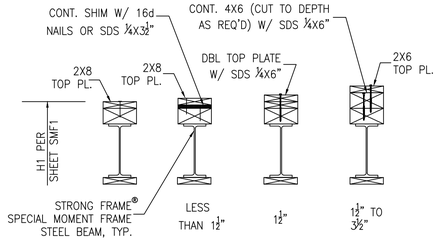
MF-2

JOB NO. ES-191628



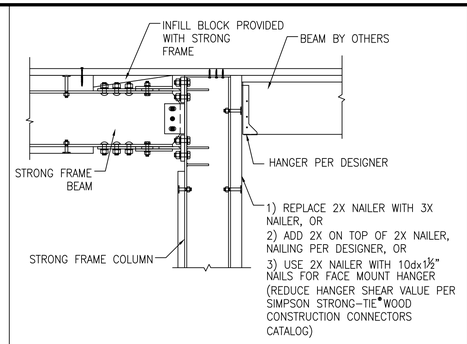
NOTES:
1. SEE 12/SMF3 FOR ALLOWABLE PENETRATION IN STEEL BEAM AND COLUMN
2. DESIGNER TO VERIFY TOTAL BEAM LOADING INCLUDING REACTION "P" DOES NOT EXCEED "Pmax" VALUE PER CATALOG

HOLDOWN POST TO SMF BEAM 1

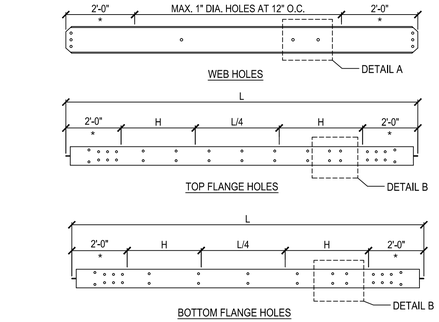


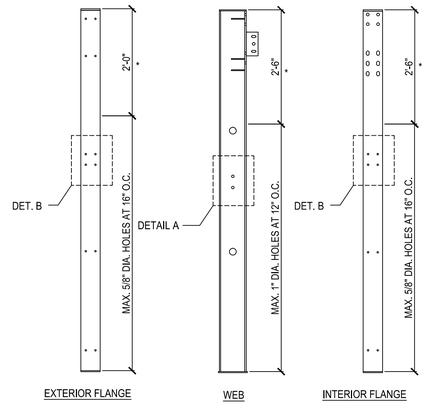
NOTES:
1. FASTENER SPACING BETWEEN STRONG FRAME® SPECIAL MOMENT FRAME BEAM DOUBLE 2X NAILER AND PLATE/SHIM/4X PER DESIGNER.
2. TOP OF CONCRETE TO TOP OF FIELD INSTALLED TOP PLATE HEIGHT (H1) CAN ALSO BE ADJUSTED BY THE USE OF THICKER NON-SHRINK GROUT (2" MAX), OR USE NEXT NOMINAL HEIGHT MODEL AND LOWER THE STEM WALL OR FOOTING (SEE SMF2 FOR DETAILS)

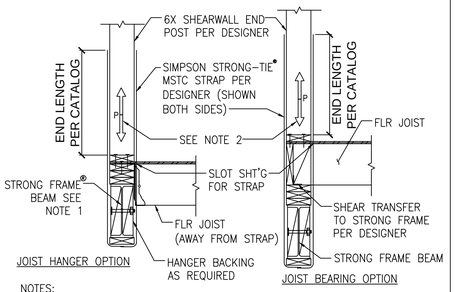
TOP OF FRAME ADJUSTMENT 5



WOOD BM TO SMF COL. CONN. 8

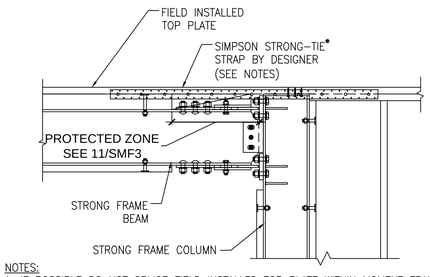






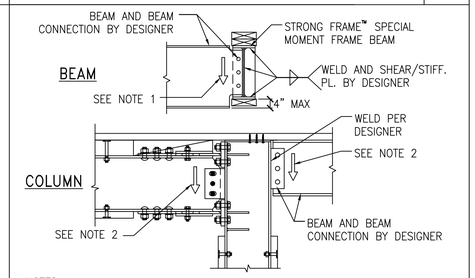
NOTES:
1. SEE 12/SMF3 FOR ALLOWABLE PENETRATION IN STEEL BEAM AND COLUMN
2. DESIGNER TO VERIFY TOTAL BEAM LOADING INCLUDING REACTION "P" DOES NOT EXCEED "Pmax" VALUE PER CATALOG
3. SEE 1/SMF3 FOR INFO NOT NOTED

6x HOLDOWN POST TO SMF BEAM 2



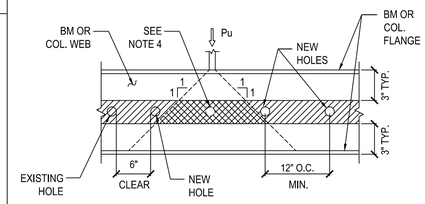
NOTES:
1. IF POSSIBLE DO NOT SPLICE FIELD INSTALLED TOP PLATE WITHIN MOMENT FRAME.
2. WHEN TOP PLACE SPLICE ARE REQUIRED WITHIN MOMENT FRAME, DESIGNER TO DESIGN/SPECIFY NO NAILING WITHIN PROTECTED ZONE.

TOP PLATE SPLICE DETAIL 6

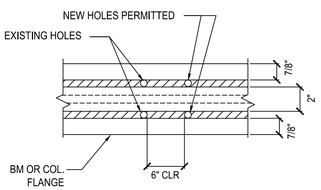


NOTES:
1. BEAM REACTIONS SHALL NOT EXCEED 50% OF "Pmax" LOAD AS NOTED ON CATALOG. DESIGNER TO VERIFY TOTAL GRAVITY LOAD IMPOSED ON STRONG FRAME SPECIAL MOMENT FRAME BEAM DOSE NOT EXCEED "Pmax" AS NOTED IN CATALOG.
2. SUMMATION OF BEAM REACTIONS (FRAMING INTO STEEL COLUMN) SHALL NOT EXCEED 50% OF "Pmax" LOADS NOTED IN CATALOG.

STEEL BEAM TO SMF BEAM/COL. 9



A: WEB PENETRATION



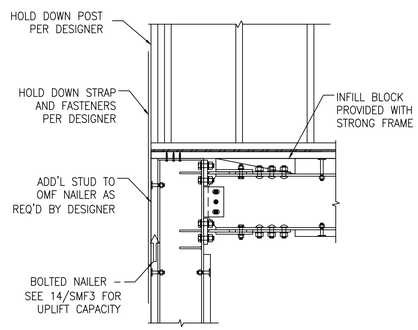
B: FLANGE PENETRATION

NOTES:
1. * DENOTES NO PENETRATIONS PERMITTED.
2. H DENOTES MAXIMUM 3/4" Ø HOLES AT 16" O.C. ARE PERMITTED.
3. LOCATE NEW HOLES 6" CLEAR OF EXISTING HOLES
4. PENETRATION PERMITTED UNDER Pu (MAXIMUM FACTORED POINT LOAD) REGION ONLY IF ALLOWABLE GRAVITY LOAD "Pmax" IS REDUCED BY 15%.
5. SEE 9/SMF3 FOR MAXIMUM COLUMN REACTIONS.
6. HOLES MAY BE BORED THROUGH WOOD NAILERS AT LOCATIONS CORRESPONDING TO AN ACCESS HOLE IN THE FLANGE.

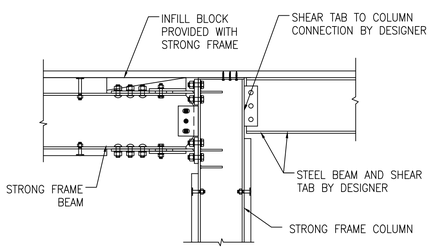
DENOTE REGIONS WHERE PENETRATIONS ARE PERMITTED

DENOTE BEAM WEB REGIONS WHERE PENETRATIONS ARE PERMITTED IF "Pmax" IS REDUCED PER NOTE 4.

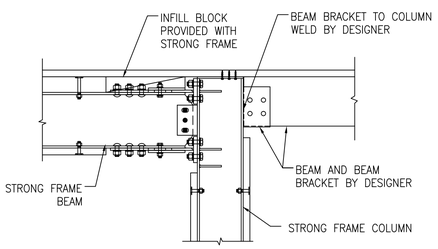
ALLOWABLE BEAM AND COLUMN PENETRATIONS 12



HOLDOWN POST TO SMF COL. 3

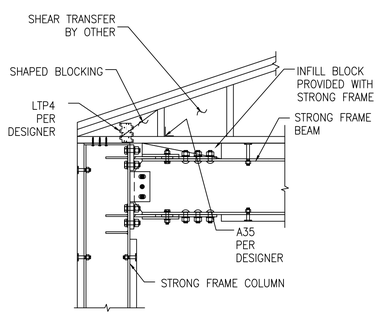


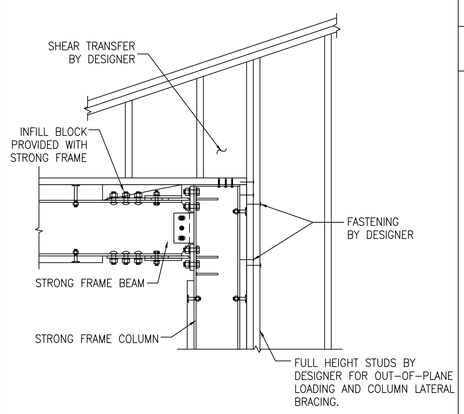
B: CONNECT STEEL BEAM TO STEEL COLUMN



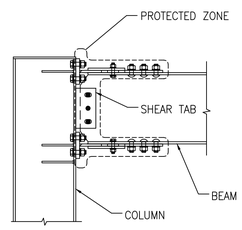
B: CONNECT WOOD BEAM TO STEEL COLUMN

COLLECTOR DETAILS 7





RAKE WALL DETAILS 10

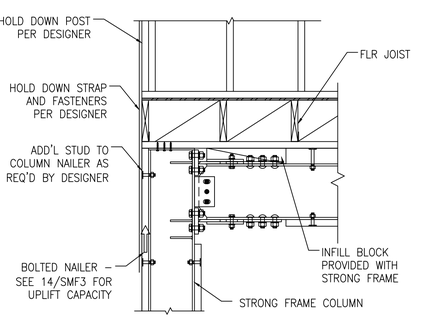


NOTE:
1. PROTECTED ZONE INCLUDED THE FOLLOWING ELEMENTS:
   A. LINK FLANGE AND LINK STEM
   B. BRP PLATES
   C. BEAM FLANGE AREAS CONNECTED TO THE LINK STEM
   D. COLUMN FLANGE AREAS CONNECTED TO THE LINK FLANGE
   C. SHEAR TAB AND BEAM WEB AT SHEAR TAB
2. NO ATTACHMENT SHALL BE MADE TO THE PROTECTED ZONE.

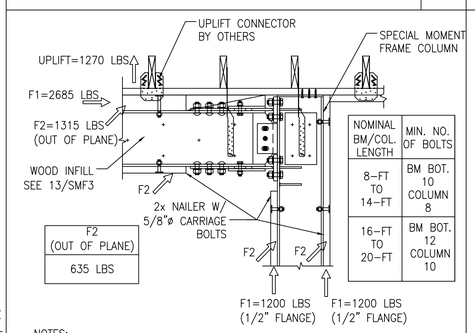
PROTECTED ZONE 11

NOTE:
1. SEE WOOD CONNECTOR CATALOG FOR TB SCREWS AND C-SAS-2012 FOR PDPH FASTENERS
2. SEE DETAIL 11 FOR PROTECTED ZONE REQUIREMENTS

WOOD INFILLS 13



HOLDOWN POST TO SMF COL. 4

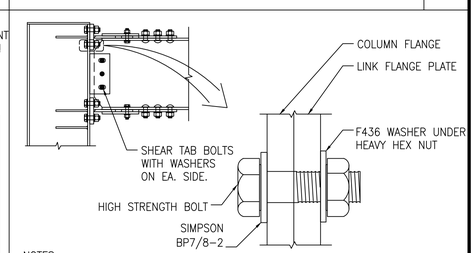


F2 (OUT OF PLANE)
635 LBS

NOMINAL BM/COL. LENGTH	MIN. NO. OF BOLTS
8-FT TO 14-FT	BM BOT. 10 COLUMN 8
16-FT TO 20-FT	BM BOT. 12 COLUMN 10

NOTES:
1. WOOD NAILERS ARE DOUGLAS-FIR LARCH NO. 2 GRADE.
2. ALLOWABLE LOADS LISTED ARE FOR ONE BOLT.

NAILER BOLT ALLOWABLE LOADS 14



NOTES:
1. INSTALL BOLTS AND TIGHTEN IN ACCORDANCE WITH INSTALLATION INSTRUCTIONS PROVIDED WITH EACH FRAME.
2. THE TYPE OF JOINT (SNUG-TIGHT OR PRETENSIONED) MUST BE DETERMINED BY THE DESIGNER AND SHOWN ON THE PLANS.
3. DRAW PLATES TOGETHER UNTIL THEY ARE IN FIRM CONTACT BY TIGHTENING BOLTS. GAPS AWAY FROM THE BOLT HEADS ARE PERMITTED. IF CONNECTION PLATES CAN NOT BE DRAWN TOGETHER SUFFICIENTLY, SHIMS ARE REQUIRED. TOTAL THICKNESS OF SHIMS UNDER A BOLT HEAD MUST NOT EXCEED 1/4".

BEAM-TO-COLUMN CONNECTION 15

NO.	DATE	REVISIONS

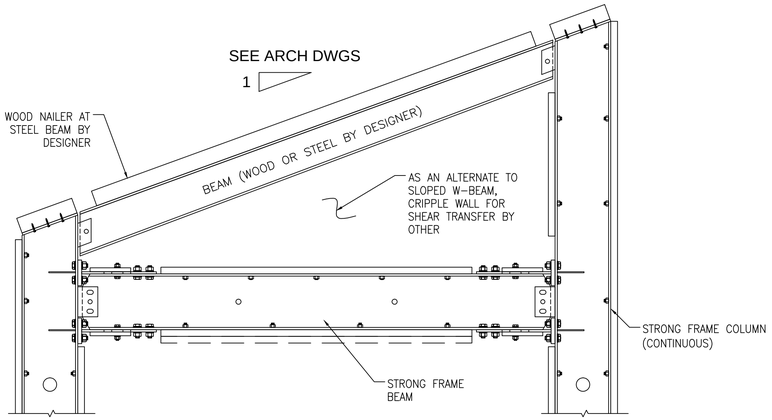
PHILLIP HUI — STATE OF WASHINGTON — 35621 — REGISTERED PROFESSIONAL ENGINEER
COA#3125 05/17/2019

SIMPSON STRONG-TIE, CO. INC.
• 5956 W. Las Positas Blvd. Pleasanton, CA 94588
• Tel: (800) 999-5099
• Fax: (925) 847-15977
• Web site: www.stronglie.com

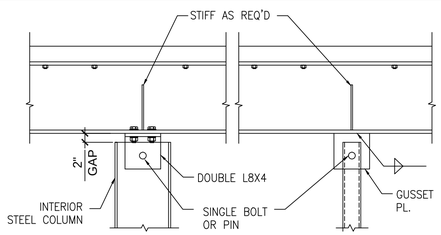
SIMPSON Strong-Tie® THERE IS NO EQUAL

STRONG-FRAME®
SMF INSTALLATION DETAILS
ENGINEERED DESIGN
6059 77TH AVE SE

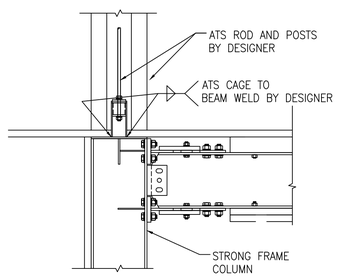
NAME: P.H.
DATE: 05/17/2019
SCALE: N.T.S.
SHEET: **SMF3**
JOB NO. ES-191628



SST SMF WITH SLOPED ROOF BEAM — 1

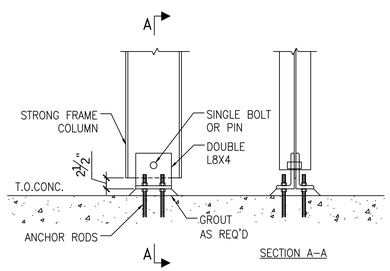


INTERIOR PINNED BEARING COL. — 2



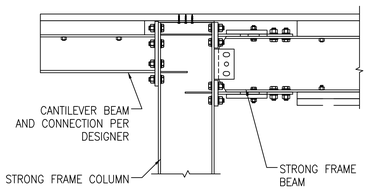
NOTE:
1. COORDINATE WITH SIMPSON STRONG-TIE IF ADDITIONAL HARDWARE NEEDS TO BE WELDED TO THE SIMPSON STRONG FRAME.
2. SEE DETAIL 11/SMF3 FOR PROTECTED ZONE REQUIREMENTS.

ATS ROD//HD ON STEEL COLUMN — 3



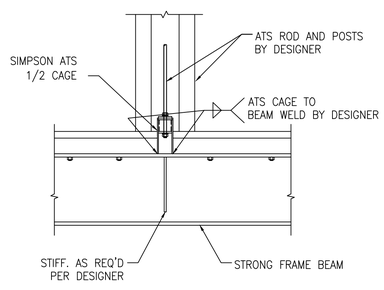
NOTES:
1. CONTACT SIMPSON STRONG-TIE FOR AVAILABLITY OF SINGLE BOLT PINNED BASE CONNECTION.

SINGLE BOLT PINNED BASE DETAIL — 4



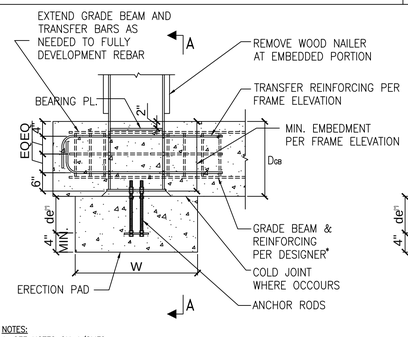
NOTE:
1. COORDINATE WITH SIMPSON STRONG-TIE IF ADDITIONAL HARDWARE NEEDS TO BE WELDED TO THE SIMPSON STRONG FRAME
2. SEE DETAIL 11/SMF3 FOR PROTECTED ZONE REQUIREMENTS.

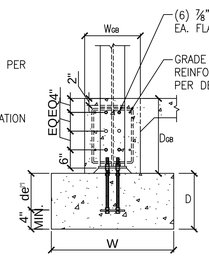
CANTILEVER BEAM TO COLUMN — 5

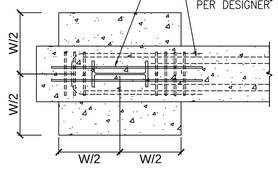


NOTE:
1. COORDINATE WITH SIMPSON STRONG-TIE IF ADDITIONAL HARDWARE NEEDS TO BE WELDED TO THE SIMPSON STRONG FRAME

ATS ROD ON STEEL BEAM — 6

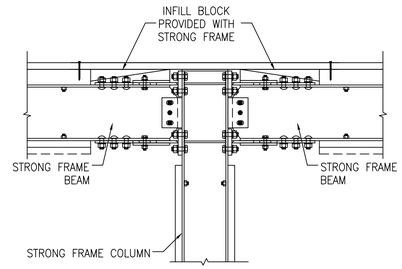






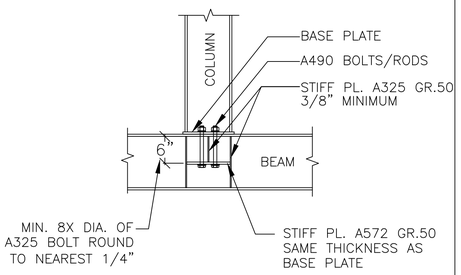
NOTES:
1. SEE NOTES ON 1/SMF2
2. ACTUAL FOOTING/GRADE BEAM SIZE, REINFORCING, AND DETAILING PER DESIGNER

FIXED BASE CONNECTION — 11

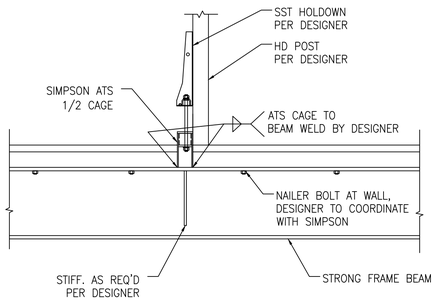


NOTE:
1. FOR FLOOR JOIST TO BEAM CONNECTION, SEE 1/SMF3 OR 4/SMF3

INT. COL WITH BEAMS ON EA. SIDE — 8

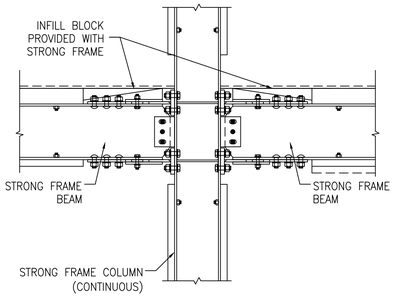


SMF COLUMN ON STEEL BEAM — 9



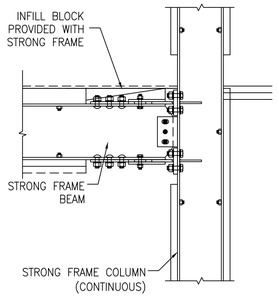
NOTE:
1. COORDINATE WITH SIMPSON STRONG-TIE IF ADDITIONAL HARDWARE NEEDS TO BE WELDED TO THE SIMPSON STRONG FRAME

SHEAR WALL HD ON STEEL BEAM — 10



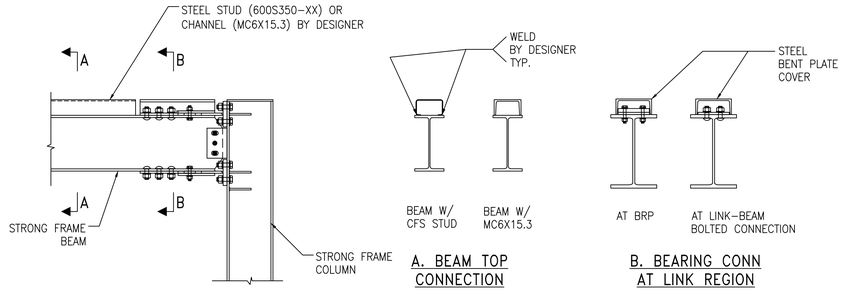
NOTE:
1. FOR FLOOR JOIST TO BEAM CONNECTION, SEE 1/SMF3 OR 4/SMF3

MULTI-STORY COLUMNS (2 BEAMS) — 12

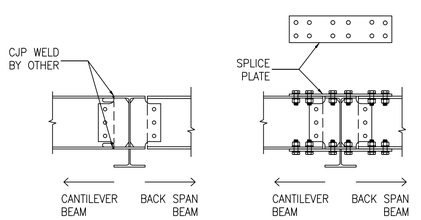


NOTE:
1. FOR FLOOR JOIST TO BEAM CONNECTION, SEE 1/SMF3 OR 4/SMF3

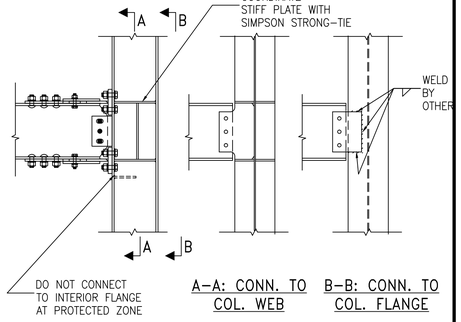
MULTI-STORY COLUMNS (1-BEAM) — 15



SMF BEAM ATTACHEMENT FOR COLD-FORMED STEEL CONSTRUCTION — 16

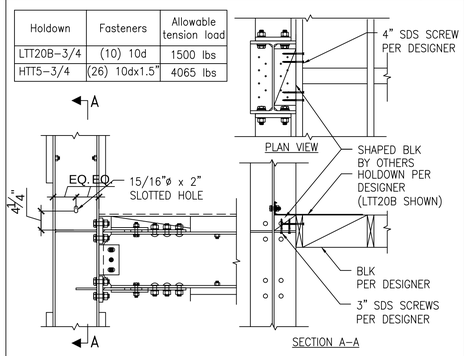


CANTILEVER STEEL BEAM CON. — 13

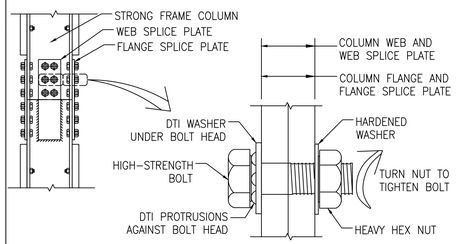


BEAM CONNECTION TO COLUMN — 17

Holdown	Fasteners	Allowable tension load
LTT20B-3/4	(10) 10d	1500 lbs
HTT5-3/4	(26) 10dx1.5"	4065 lbs



COL. BRACING AT FLOOR LEVEL — 14



NOTES:
1. INSTALL BOLTS AND TIGHTEN IN ACCORDANCE WITH INSTALLATION INSTRUCTIONS PROVIDED WITH EACH FRAME.
2. HOLD HEAD OF BOLT AND TURN NUT TO TIGHTEN.
3. THE TYPE OF JOINT IS PRETENSIONED FOR THIS CONNECTION.
4. NUMBER OF FLANGE AND WEB CONNECTION BOLTS PER DESIGN DRAWINGS.
5. DRAW PLATES TOGETHER UNTIL THEY ARE IN FIRM CONTACT BY TIGHTENING BOLTS. GAPS AWAY FROM THE BOLT HEADS ARE PERMITTED.

BOLTED COLUMN SPLICE — 18

NO.	DATE	REVISIONS

PHILLIP HUNG — STATE OF WASHINGTON — 53923 — REGISTERED PROFESSIONAL ENGINEER
COA#3125 — 05/17/2019

SIMPSON STRONG-TIE, CO. INC.
5956 W. Las Positas Blvd.
Pleasanton, CA 94588
Tel: (800) 999-5099
Fax: (925) 847-15977
Web site: www.stronglie.com

SIMPSON Strong-Tie — THERE IS NO EQUAL

STRONG-FRAME®
SMF INSTALLATION DETAILS
ENGINEERED DESIGN
6059 77TH AVE SE

NAME: P.H.
DATE: 05/17/2019
SCALE: N.T.S.
SHEET: **SMF4**
JOB NO. ES-191628