DEI
DIBBLE ENGINEERS INC

SUPPLEMENTAL STRUCTURAL CALCULATIONS
PEYREE REMODEL
(OWNER REV#1)

6059 77th Ave SE
Mercer Island, WA

Prepared for Gelotte Hommas
Project # 17-291
August 10, 2020

08/10/2020

DEI
DIBBLE ENGINEERS INC

GRAVITY

Peyree Remodel #17-291

Steel Beam

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

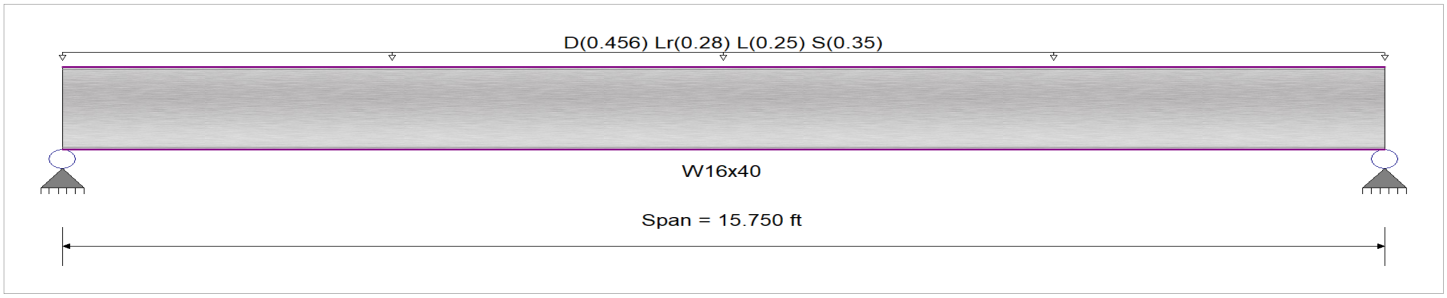
Description : MF-2 Beam Above Nanawall --- Limited to L/720 or 0.25" ---> OK

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.4560, Lr = 0.280, L = 0.250, S = 0.350 k/ft, Tributary Width = 1.0 ft, (Dead + Live)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.159** : 1	Maximum Shear Stress Ratio =	**0.076** : 1
Section used for this span	**W16x40**	Section used for this span	**W16x40**
Mu : Applied	43.572 k-ft	Vu : Applied	11.066 k
Mn * Phi : Allowable	273.750 k-ft	Vn * Phi : Allowable	146.40 k
Load Combination	+1.20D+L+1.60S+1.60H	Load Combination	+1.20D+L+1.60S+1.60H
Location of maximum on span	7.875 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	0.032 in	Ratio =	5,832 >=720
Max Upward Transient Deflection	0.000 in	Ratio =	0 <720
Max Downward Total Deflection	0.088 in	Ratio =	2158 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
+1.40D+1.60H													
Dsgn. L = 15.75 ft	1	0.079	0.037	21.53		21.53	304.17	273.75	1.00	1.00	5.47	146.40	146.40
+1.20D+0.50Lr+1.60L+1.60H													
Dsgn. L = 15.75 ft	1	0.129	0.061	35.20		35.20	304.17	273.75	1.00	1.00	8.94	146.40	146.40
+1.20D+1.60L+0.50S+1.60H													
Dsgn. L = 15.75 ft	1	0.133	0.063	36.29		36.29	304.17	273.75	1.00	1.00	9.22	146.40	146.40
+1.20D+1.60Lr+L+1.60H													
Dsgn. L = 15.75 ft	1	0.146	0.070	40.10		40.10	304.17	273.75	1.00	1.00	10.18	146.40	146.40
+1.20D+1.60Lr+0.50W+1.60H													
Dsgn. L = 15.75 ft	1	0.118	0.056	32.35		32.35	304.17	273.75	1.00	1.00	8.22	146.40	146.40
+1.20D+L+1.60S+1.60H													
Dsgn. L = 15.75 ft	1	0.159	0.076	43.57		43.57	304.17	273.75	1.00	1.00	11.07	146.40	146.40
+1.20D+1.60S+0.50W+1.60H													
Dsgn. L = 15.75 ft	1	0.131	0.062	35.82		35.82	304.17	273.75	1.00	1.00	9.10	146.40	146.40
+1.20D+0.50Lr+L+W+1.60H													
Dsgn. L = 15.75 ft	1	0.112	0.053	30.55		30.55	304.17	273.75	1.00	1.00	7.76	146.40	146.40
+1.20D+L+0.50S+W+1.60H													
Dsgn. L = 15.75 ft	1	0.116	0.055	31.63		31.63	304.17	273.75	1.00	1.00	8.03	146.40	146.40
+0.90D+W+1.60H													
Dsgn. L = 15.75 ft	1	0.051	0.024	13.84		13.84	304.17	273.75	1.00	1.00	3.52	146.40	146.40
+1.20D+L+0.20S+E+1.90H													
Dsgn. L = 15.75 ft	1	0.104	0.049	28.38		28.38	304.17	273.75	1.00	1.00	7.21	146.40	146.40
+0.90D+E+0.90H													
Dsgn. L = 15.75 ft	1	0.051	0.024	13.84		13.84	304.17	273.75	1.00	1.00	3.52	146.40	146.40

Steel Beam

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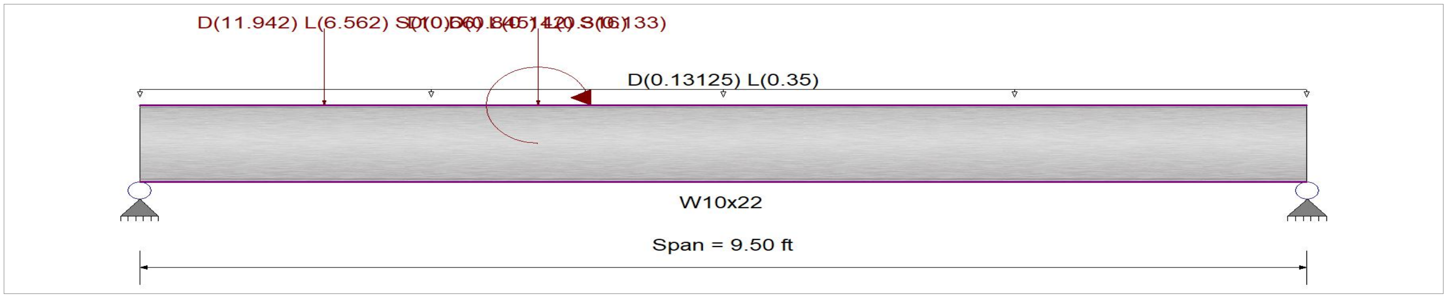
Description : MB2 Revision (Item 7) - moment per SWs from MTBs with Omega = 9.3k x 17.5' /2 =82kft * 5.5'/17.5' = 25.6kft x 2.5 = 64kft

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

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

Point Load : D = 11.942, L = 6.562, S = 10.0 k @ 1.50 ft, (UC2 abv)

Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 8.750 ft, (floor load)

Point Load : D = 0.8450, L = 0.3160 k @ 3.250 ft, (MTB2 Transfer beam reaction w SW HD)

Point Load : D = 0.660, L = 0.1420, S = 0.1330 k @ 3.250 ft, (MTB1 Transfer beam reaction w SW HD)

Moment : E = 64.0 k-ft, Loc = 3.250 ft in span, (moment per SWs from MTBs with Omega)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.865** : 1	Maximum Shear Stress Ratio =	**0.486** : 1
Section used for this span	**W10x22**	Section used for this span	**W10x22**
Ma : Applied	56.132 k-ft	Va : Applied	23.773 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+1.10D+0.750L+0.750S+0.5250E	Load Combination	+D+0.750L+0.750S
Location of maximum on span	3.257 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	0.098 in	Ratio =	1,167 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.190 in	Ratio =	599 >=360
Max Upward Total Deflection	-0.000 in	Ratio =	<360

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 Only													
Dsgn. L = 9.50 ft	1	0.269	0.240	17.48		17.48	108.33	64.87	1.00	1.00	11.77	73.44	48.96
+D+L													
Dsgn. L = 9.50 ft	1	0.437	0.393	28.33		28.33	108.33	64.87	1.00	1.00	19.26	73.44	48.96
+D+S													
Dsgn. L = 9.50 ft	1	0.466	0.414	30.21		30.21	108.33	64.87	1.00	1.00	20.28	73.44	48.96
+D+0.750L													
Dsgn. L = 9.50 ft	1	0.395	0.355	25.62		25.62	108.33	64.87	1.00	1.00	17.39	73.44	48.96
+D+0.750L+0.750S													
Dsgn. L = 9.50 ft	1	0.542	0.486	35.17		35.17	108.33	64.87	1.00	1.00	23.77	73.44	48.96
+D+0.750S													

Steel Beam

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

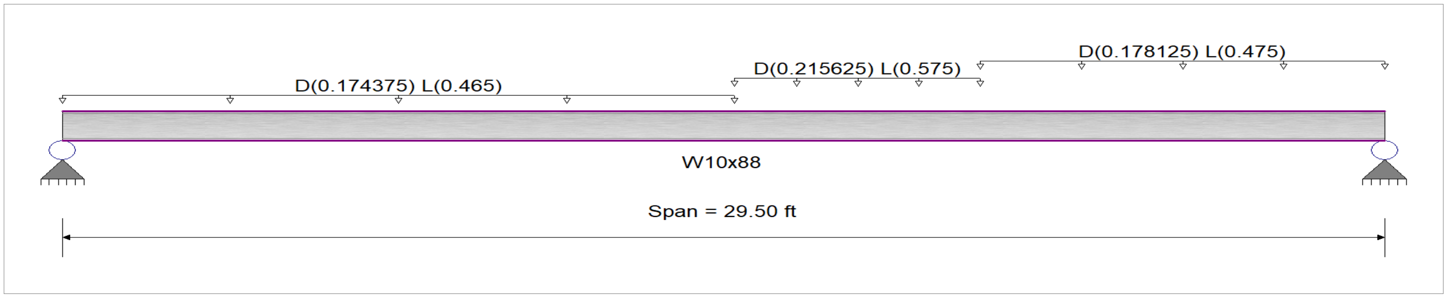
Description : BM1 - Steel Beam at Family Room (Item 7)

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 0.0 -->> 15.0 ft, Tributary Width = 11.625 ft, (floor)

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 15.0 -->> 20.50 ft, Tributary Width = 14.375 ft, (floor)

Uniform Load : D = 0.0150, L = 0.040 ksf, Extent = 20.50 -->> 29.50 ft, Tributary Width = 11.875 ft, (floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.290 : 1	Maximum Shear Stress Ratio =	0.084 : 1
Section used for this span	W10x88	Section used for this span	W10x88
Mu : Applied	123.007 k-ft	Vu : Applied	16.521 k
Mn * Phi : Allowable	423.750 k-ft	Vn * Phi : Allowable	196.020 k
Load Combination	+1.20D+1.60L	Load Combination	+1.20D+1.60L
Location of maximum on span	15.171 ft	Location of maximum on span	29.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.550 in	Ratio =	643 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.854 in	Ratio =	415 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 29.50 ft	1	0.099	0.029	41.95		41.95	470.83	423.75	1.00	1.00	5.65	196.02	196.02
+1.20D+1.60L													
Dsgn. L = 29.50 ft	1	0.290	0.084	123.01		123.01	470.83	423.75	1.00	1.00	16.52	196.02	196.02
+1.20D+L													
Dsgn. L = 29.50 ft	1	0.213	0.062	90.36		90.36	470.83	423.75	1.00	1.00	12.14	196.02	196.02
+1.20D													
Dsgn. L = 29.50 ft	1	0.085	0.025	35.96		35.96	470.83	423.75	1.00	1.00	4.84	196.02	196.02
+0.90D													
Dsgn. L = 29.50 ft	1	0.064	0.019	26.97		26.97	470.83	423.75	1.00	1.00	3.63	196.02	196.02
+1.390D+L													
Dsgn. L = 29.50 ft	1	0.227	0.066	96.05		96.05	470.83	423.75	1.00	1.00	12.91	196.02	196.02
+0.7102D													
Dsgn. L = 29.50 ft	1	0.050	0.015	21.28		21.28	470.83	423.75	1.00	1.00	2.87	196.02	196.02

Wood Beam

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

Description : BM2 - Steel Beam at reconfig walls (Item 8)

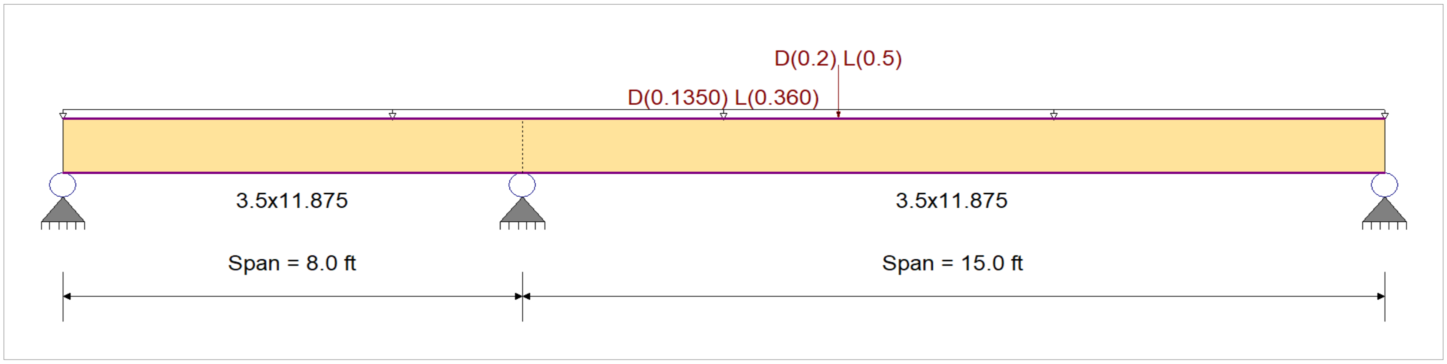
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Load Resistance Factor D	Fb +	2900 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - xx	1016.535 ksi
Wood Species	: Trus Joist	Fc - Perp	625 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 pcf

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



Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, L = 0.040 ksf, Tributary Width = 9.0 ft
Load for Span Number 2
Point Load : D = 0.20, L = 0.50 k @ 5.50 ft, (framing)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.520** : 1	Maximum Shear Stress Ratio	=	**0.490** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb : Actual	=	2,603.89 psi	fv : Actual	=	245.63 psi
FB : Allowable	=	5,011.20 psi	Fv : Allowable	=	501.12 psi
Load Combination		+1.20D+0.50Lr+1.60L+1.60H, LL Comb	Load Combination		+1.20D+0.50Lr+1.60L+1.60H, LL Comb
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	8.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.292 in	Ratio =	616 >=480
Max Upward Transient Deflection	-0.055 in	Ratio =	1742 >=480
Max Downward Total Deflection	0.401 in	Ratio =	448 >=360
Max Upward Total Deflection	-0.067 in	Ratio =	1431 >=360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	λ	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Mu	fb	Fb	Vu	fv	Fv
+1.40D+1.60H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.190	0.179	0.60	1.000	1.00	1.00	1.00	1.00	1.00	4.90	714.35	3758.40	1.87	67.38	375.84
Length = 15.0 ft	2	0.190	0.179	0.60	1.000	1.00	1.00	1.00	1.00	1.00	4.90	714.35	3758.40	1.87	67.38	375.84
+1.20D+0.50Lr+1.60L+1.60H, LL Cc					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.473	0.482	0.80	1.000	1.00	1.00	1.00	1.00	1.00	16.25	2,370.08	5011.20	6.70	241.78	501.12
Length = 15.0 ft	2	0.473	0.482	0.80	1.000	1.00	1.00	1.00	1.00	1.00	16.26	2,371.77	5011.20	6.70	241.78	501.12
+1.20D+0.50Lr+1.60L+1.60H, LL Cc					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

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

Description : RB_ Replace MF1 - 29.5' Span

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

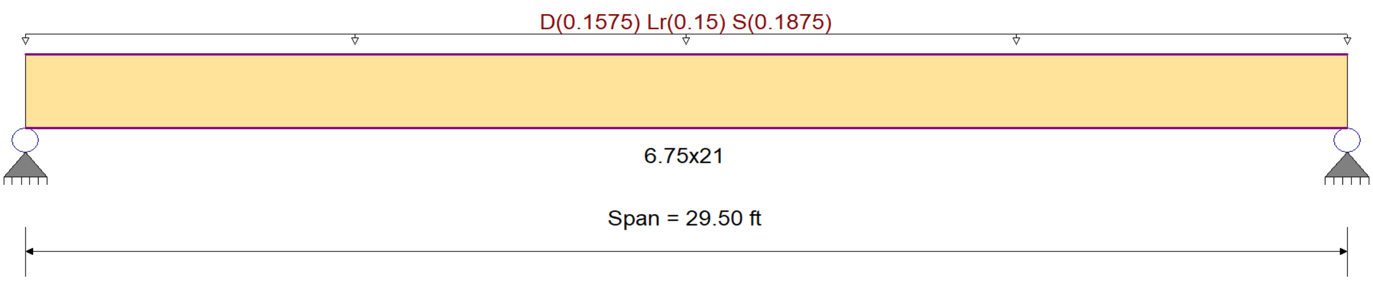
Wood Species : DF/DF
Wood Grade : 24F - V4

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	31.210	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, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 7.50 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.403** : 1	Maximum Shear Stress Ratio	=	**0.170** : 1
Section used for this span		**6.75x21**	Section used for this span		**6.75x21**
fb : Actual	=	988.58 psi	fv : Actual	=	51.80 psi
FB : Allowable	=	2,454.05 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	14.750 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	0.343 in	Ratio =	1032 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.687 in	Ratio =	515 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

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 = 29.50 ft	1	0.258	0.109	0.90	0.889	1.00	1.00	1.00	1.00	1.00	20.48	495.24	1920.56	2.45	25.95	238.50
+D+L+H					0.889	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 29.50 ft	1	0.232	0.098	1.00	0.889	1.00	1.00	1.00	1.00	1.00	20.48	495.24	2133.96	2.45	25.95	265.00
+D+Lr+H					0.889	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 29.50 ft	1	0.334	0.141	1.25	0.889	1.00	1.00	1.00	1.00	1.00	36.79	889.91	2667.45	4.41	46.63	331.25
+D+S+H					0.889	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 29.50 ft	1	0.403	0.170	1.15	0.889	1.00	1.00	1.00	1.00	1.00	40.87	988.58	2454.05	4.89	51.80	304.75
+D+0.750Lr+0.750L+H					0.889	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 29.50 ft	1	0.297	0.125	1.25	0.889	1.00	1.00	1.00	1.00	1.00	32.71	791.24	2667.45	3.92	41.46	331.25
+D+0.750L+0.750S+H					0.889	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

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

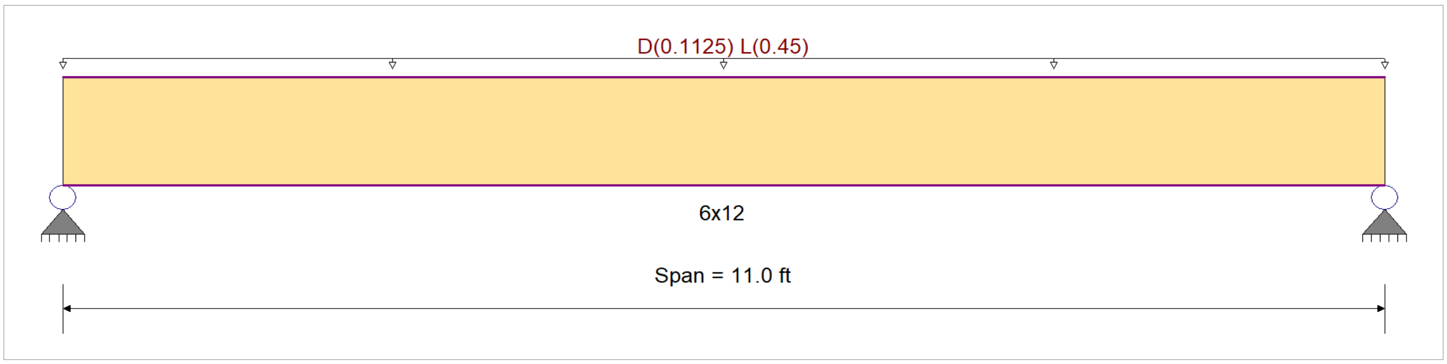
Description : FB_ Replace MF1 - 11' Span

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1350 psi	Ebend- xx	1600 ksi
		Fc - Prll	925 psi	Eminbend - xx	580 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.1	Fv	170 psi		
		Ft	675 psi	Density	31.21 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



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 = 0.060 ksf, Tributary Width = 7.50 ft, (floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.639** : 1	Maximum Shear Stress Ratio	=	**0.368** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb : Actual	=	862.68 psi	fv : Actual	=	62.54 psi
FB : Allowable	=	1,350.00 psi	Fv : Allowable	=	170.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.500 ft	Location of maximum on span	=	10.077 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.134 in	Ratio =	987 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.171 in	Ratio =	771 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

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	0.156	0.090	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.91	188.95	1215.00	0.58	13.70	153.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.639	0.368	1.00	1.000	1.00	1.00	1.00	1.00	1.00	8.72	862.68	1350.00	2.64	62.54	170.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.112	0.064	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.91	188.95	1687.50	0.58	13.70	212.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.122	0.070	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.91	188.95	1552.50	0.58	13.70	195.50
+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 = 11.0 ft	1	0.411	0.237	1.25	1.000	1.00	1.00	1.00	1.00	1.00	7.01	694.25	1687.50	2.12	50.33	212.50
+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

Dibble Engineers Inc.
Nate Moore

Wood Beam

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

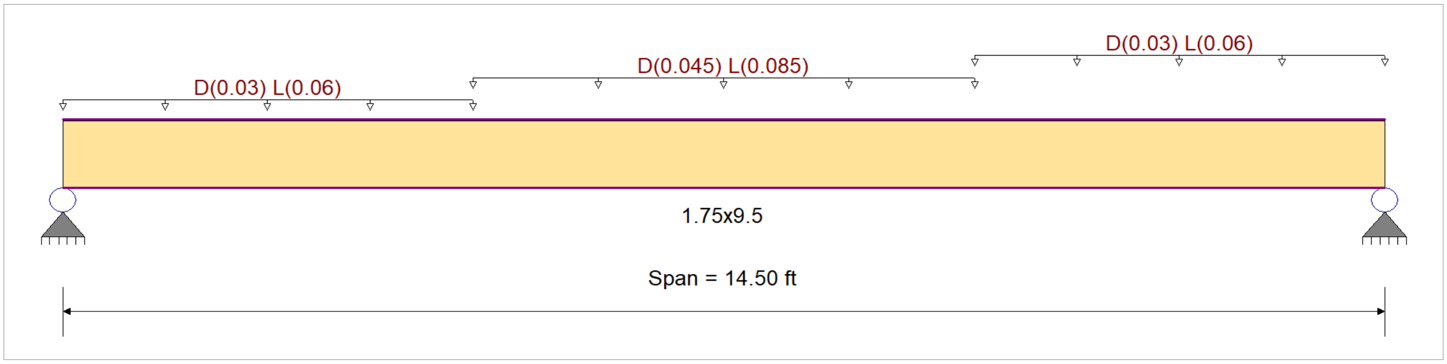
Description : Joists for Master Covered Deck w/ Hot tub

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2600 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2600 psi	Ebend- xx	2000 ksi
		Fc - Prll	2510 psi	Eminbend - xx	1016.535 ksi
Wood Species	: Trus Joist	Fc - Perp	750 psi		
Wood Grade	: MicroLam LVL 2.0 E	Fv	285 psi		
		Ft	1555 psi	Density	42.01 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



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
Uniform Load : D = 0.030, L = 0.060 ksf, Extent = 0.0 -->> 4.50 ft, Tributary Width = 1.0 ft, (floor)
Uniform Load : D = 0.0450, L = 0.0850 k/ft, Extent = 4.50 -->> 10.0 ft, Tributary Width = 1.0 ft, (hot tub)
Uniform Load : D = 0.030, L = 0.060 ksf, Extent = 10.0 -->> 14.50 ft, Tributary Width = 1.0 ft, (floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.550** : 1	Maximum Shear Stress Ratio	=	**0.230** : 1
Section used for this span		**1.75x9.5**	Section used for this span		**1.75x9.5**
fb : Actual	=	1,431.01 psi	fv : Actual	=	65.63 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	7.250 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	0.297 in	Ratio =	586 >=480
Max Upward Transient Deflection	0.000 in	Ratio =	0 <480
Max Downward Total Deflection	0.470 in	Ratio =	370 >=360
Max Upward Total Deflection	0.000 in	Ratio =	0 <360

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 = 14.50 ft	1	0.226	0.094	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	2340.00	0.27	24.19	256.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.550	0.230	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.14	1,431.01	2600.00	0.73	65.63	285.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.162	0.068	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	3250.00	0.27	24.19	356.25
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

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

Description : Joists for Master Covered Deck w/ Hot tub

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 = 14.50 ft	1	0.177	0.074	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	2990.00	0.27	24.19	327.75
+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 = 14.50 ft	1	0.371	0.155	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.64	1,205.26	3250.00	0.61	55.27	356.25
+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 = 14.50 ft	1	0.403	0.169	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.64	1,205.26	2990.00	0.61	55.27	327.75
+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 = 14.50 ft	1	0.127	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	4160.00	0.27	24.19	456.00
+D+0.750Lr+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.127	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	4160.00	0.27	24.19	456.00
+D+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 = 14.50 ft	1	0.127	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	4160.00	0.27	24.19	456.00
+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 = 14.50 ft	1	0.076	0.032	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.69	316.81	4160.00	0.16	14.51	456.00
+D+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 = 14.50 ft	1	0.127	0.053	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.16	528.02	4160.00	0.27	24.19	456.00
+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 = 14.50 ft	1	0.290	0.121	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.64	1,205.26	4160.00	0.61	55.27	456.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.076	0.032	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.69	316.81	4160.00	0.16	14.51	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.4702	7.303		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

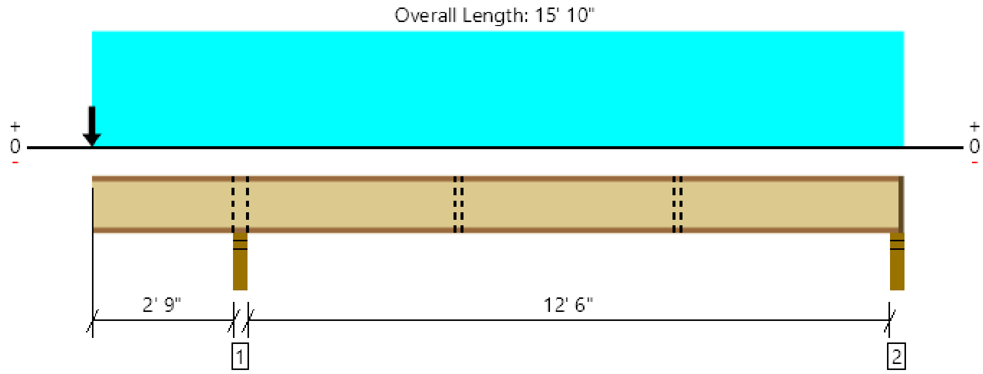
Load Combination	Support 1	Support 2
Overall MAXimum	0.798	0.798
Overall MINimum	0.504	0.504
+D+H	0.294	0.294
+D+L+H	0.798	0.798
+D+Lr+H	0.294	0.294
+D+S+H	0.294	0.294
+D+0.750Lr+0.750L+H	0.672	0.672
+D+0.750L+0.750S+H	0.672	0.672
+D+0.60W+H	0.294	0.294
+D+0.750Lr+0.450W+H	0.294	0.294
+D+0.750S+0.450W+H	0.294	0.294
+0.60D+0.60W+0.60H	0.176	0.176
+D+0.70E+0.60H	0.294	0.294
+D+0.750L+0.750S+0.5250E+H	0.672	0.672
+0.60D+0.70E+H	0.176	0.176
D Only	0.294	0.294
Lr Only		
L Only	0.504	0.504
S Only		
W Only		
E Only		
H Only		

MEMBER REPORT

G-9 PASSED

Level, Floor: Joist

1 piece(s) 11 7/8" TJI® 230 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2000 @ 2' 10 3/4"	2410 (3.50")	Passed (83%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1157 @ 2' 9"	1655	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	-2998 @ 2' 10 3/4"	3161	Passed (95%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.156 @ 0	0.200	Passed (2L/446)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.197 @ 0	0.290	Passed (2L/352)	--	1.0 D + 1.0 L (Alt Spans)
TJ-Pro™ Rating	57	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/0.2") and TL (2L/240).
- Moment capacity over cantilever support 1 has been reduced by 25% to lessen the effects of buckling.
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 3" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 4' 10" o/c based on loads applied, unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of decking_2332Edge that is gluedAndNailedDown.
- Additional considerations for the TJ-Pro™ Rating include: None.
- Permanent bracing at third points In the back span or a direct applied ceiling over the entire back span length is required at the left end of the member. See literature detail (PB1) For clarification.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	3.50"	752	1248	2000	Blocking
2 - Stud wall - SPF	3.50"	2.25"	1.75"	181	351/-149	532/-149	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 10"	16"	30.0	40.0	Default Load
2 - Point (lb)	0	N/A	300	600	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Nate Moore Dibble Engineers (425) 553-1855 nate@dibbleengineers.com	

Steel Column

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

Description : HSS 6x6 col _ Replace MF1

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name :	**HSS6x6x3/16**	Overall Column Height	32.50 ft
Analysis Method :	Load Resistance Factor	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade			
Fy : Steel Yield	42.0 ksi		
E : Elastic Bending Modulus	29,000.0 ksi		

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 32.50 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 32.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 472.225 lbs * Dead Load Factor
AXIAL LOADS . . .
Roof: Axial Load at 32.50 ft, Xecc = 3.0 in, D = 2.744, LR = 2.213, S = 2.766 k
floor: Axial Load at 13.50 ft, Xecc = 5.50 in, D = 2.145, L = 6.638 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6558** : 1

Load Combination	+1.20D+1.60L+0.50S+1.60H	
Location of max.above base	13.305	ft
At maximum location values are . . .		
Pu	18.437	k
0.9 * Pn	33.204	k
Mu-x	0.0	k-ft
0.9 * Mn-x :	26.128	k-ft
Mu-y	-2.954	k-ft
0.9 * Mn-y :	26.128	k-ft

PASS Maximum Shear Stress Ratio = **0.006163** : 1

Load Combination	+1.20D+1.60L+0.50S+1.60H	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.2220	k
Vn * Phi : Allowable	36.030	k

Maximum Load Reactions . .

Top along X-X	0.1450 k
Bottom along X-X	0.1450 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

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

Along X-X -0.1935 in at 18.322 ft above base
for load combination : +D+S+H

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D+1.60H	0.259	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.002	PASS	0.00 ft
+1.20D+0.50Lr+1.60L+1.60H	0.646	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.006	PASS	0.00 ft
+1.20D+1.60L+0.50S+1.60H	0.656	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.006	PASS	0.00 ft
+1.20D+1.60Lr+L+1.60H	0.583	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+1.60Lr+0.50W+1.60H	0.341	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.002	PASS	0.00 ft
+1.20D+L+1.60S+1.60H	0.613	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+1.60S+0.50W+1.60H	0.370	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.003	PASS	0.00 ft
+1.20D+0.50Lr+L+W+1.60H	0.501	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+L+0.50S+W+1.60H	0.510	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+0.90D+W+1.60H	0.145	PASS	0.00 ft	1.00	1.46	164.56	164.56	0.001	PASS	0.00 ft
+1.20D+L+0.20S+E+1.90H	0.483	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.004	PASS	0.00 ft
+0.90D+E+0.90H	0.145	PASS	0.00 ft	1.00	1.46	164.56	164.56	0.001	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments (k-ft) @ Base	Mx - End Moments (k-ft) @ Top	My - End Moments @ Base	My - End Moments @ Top
+D+H	5.361	0.051	0.051						
+D+L+H	11.999	0.145	0.145						

Steel Column

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

Description : HSS 6x6 col _ Replace MF1

Maximum Deflections for Load Combinations

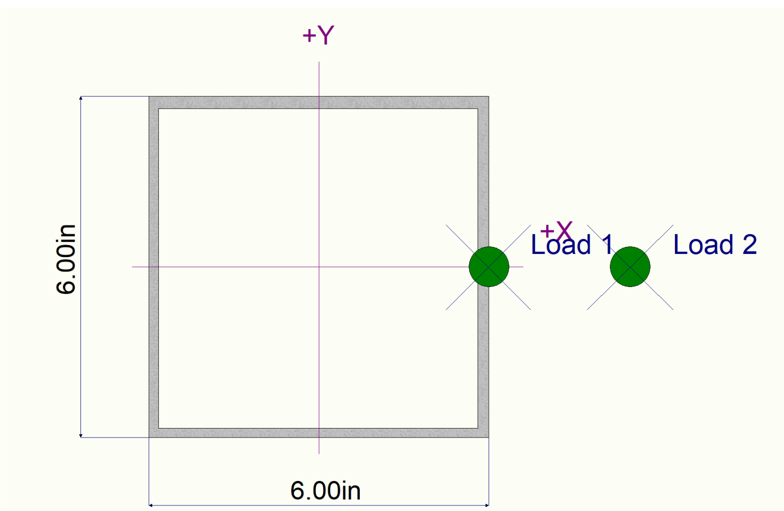
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
S Only	-0.1263	in	18.977	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

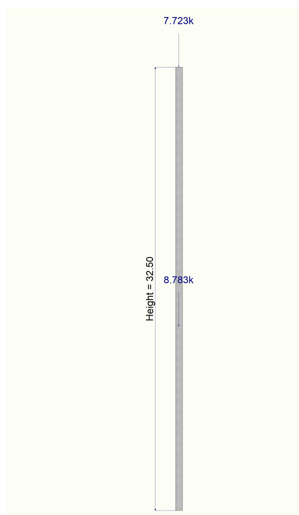
Steel Section Properties : HSS6x6x3/16

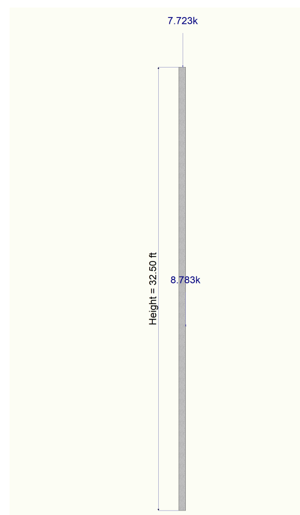
Depth	=	6.000	in	I xx	=	22.30	in^4	J	=	35.000	in^4
Design Thick	=	0.174	in	S xx	=	7.42	in^3				
Width	=	6.000	in	R xx	=	2.370	in				
Wall Thick	=	0.187	in	Zx	=	8.630	in^3				
Area	=	3.980	in^2	I yy	=	22.300	in^4	C	=	11.800	in^3
Weight	=	14.530	plf	S yy	=	7.420	in^3				
				R yy	=	2.370	in				

Ycg	=	0.000	in

Sketches







Steel Column

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

Description : HSS 6x6 col _ Replace MF1

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS6x6x3/16**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 42.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 32.50 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 32.50 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 32.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 472.225 lbs * Dead Load Factor
AXIAL LOADS . . .
Roof: Axial Load at 32.50 ft, Xecc = 3.0 in, D = 2.744, LR = 2.213, S = 2.766 k
floor: Axial Load at 13.50 ft, Xecc = 5.50 in, D = 2.145, L = 6.638 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6558** : 1
Load Combination +1.20D+1.60L+0.50S+1.60H
Location of max.above base 13.305 ft
At maximum location values are . . .
Pu 18.437 k
0.9 * Pn 33.204 k
Mu-x 0.0 k-ft
0.9 * Mn-x : 26.128 k-ft
Mu-y -2.954 k-ft
0.9 * Mn-y : 26.128 k-ft

PASS Maximum Shear Stress Ratio = **0.006163** : 1
Load Combination +1.20D+1.60L+0.50S+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.2220 k
Vn * Phi : Allowable 36.030 k

Maximum Load Reactions . .
Top along X-X 0.1450 k
Bottom along X-X 0.1450 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X -0.1935 in at 18.322 ft above base
for load combination : +D+S+H

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D+1.60H	0.259	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.002	PASS	0.00 ft
+1.20D+0.50Lr+1.60L+1.60H	0.646	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.006	PASS	0.00 ft
+1.20D+1.60L+0.50S+1.60H	0.656	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.006	PASS	0.00 ft
+1.20D+1.60Lr+L+1.60H	0.583	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+1.60Lr+0.50W+1.60H	0.341	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.002	PASS	0.00 ft
+1.20D+L+1.60S+1.60H	0.613	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+1.60S+0.50W+1.60H	0.370	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.003	PASS	0.00 ft
+1.20D+0.50Lr+L+W+1.60H	0.501	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+1.20D+L+0.50S+W+1.60H	0.510	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.005	PASS	0.00 ft
+0.90D+W+1.60H	0.145	PASS	0.00 ft	1.00	1.46	164.56	164.56	0.001	PASS	0.00 ft
+1.20D+L+0.20S+E+1.90H	0.483	PASS	13.31 ft	1.00	1.46	164.56	164.56	0.004	PASS	0.00 ft
+0.90D+E+0.90H	0.145	PASS	0.00 ft	1.00	1.46	164.56	164.56	0.001	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
+D+H	5.361	0.051	0.051						
+D+L+H	11.999	0.145	0.145						

Steel Column

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

Description : HSS 6x6 col _ Replace MF1

Maximum Deflections for Load Combinations

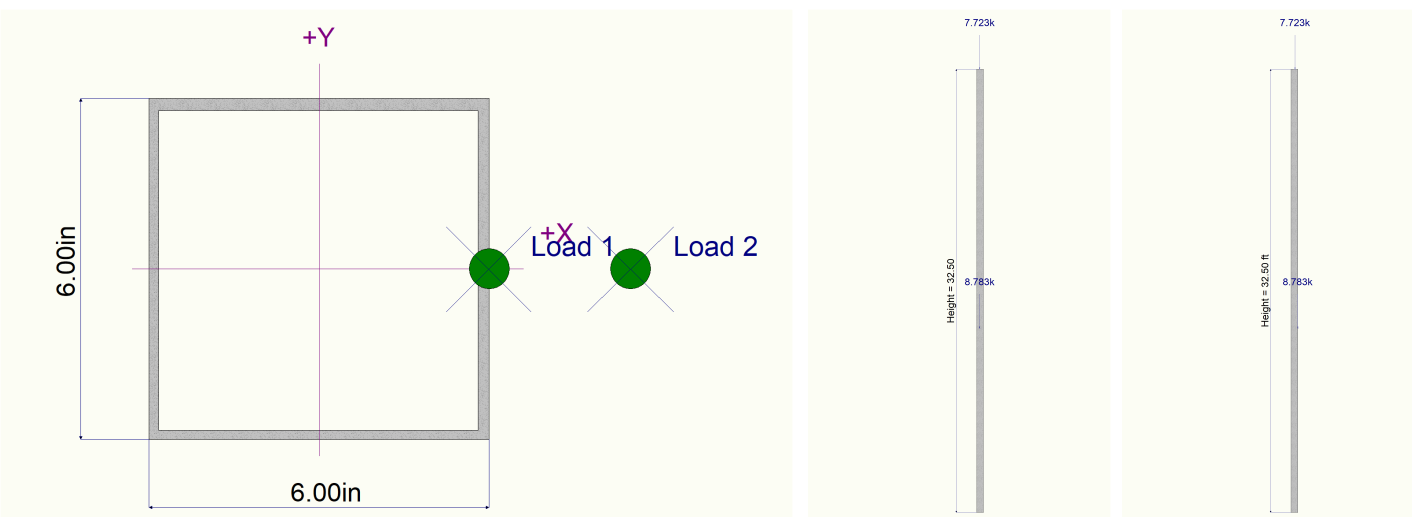
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
S Only	-0.1263	in	18.977	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS6x6x3/16

Depth	=	6.000 in		I xx	=	22.30 in^4		J	=	35.000 in^4
Design Thick	=	0.174 in		S xx	=	7.42 in^3				
Width	=	6.000 in		R xx	=	2.370 in				
Wall Thick	=	0.187 in		Zx	=	8.630 in^3				
Area	=	3.980 in^2		I yy	=	22.300 in^4		C	=	11.800 in^3
Weight	=	14.530 plf		S yy	=	7.420 in^3				
				R yy	=	2.370 in				

Ycg	=	0.000 in

Sketches



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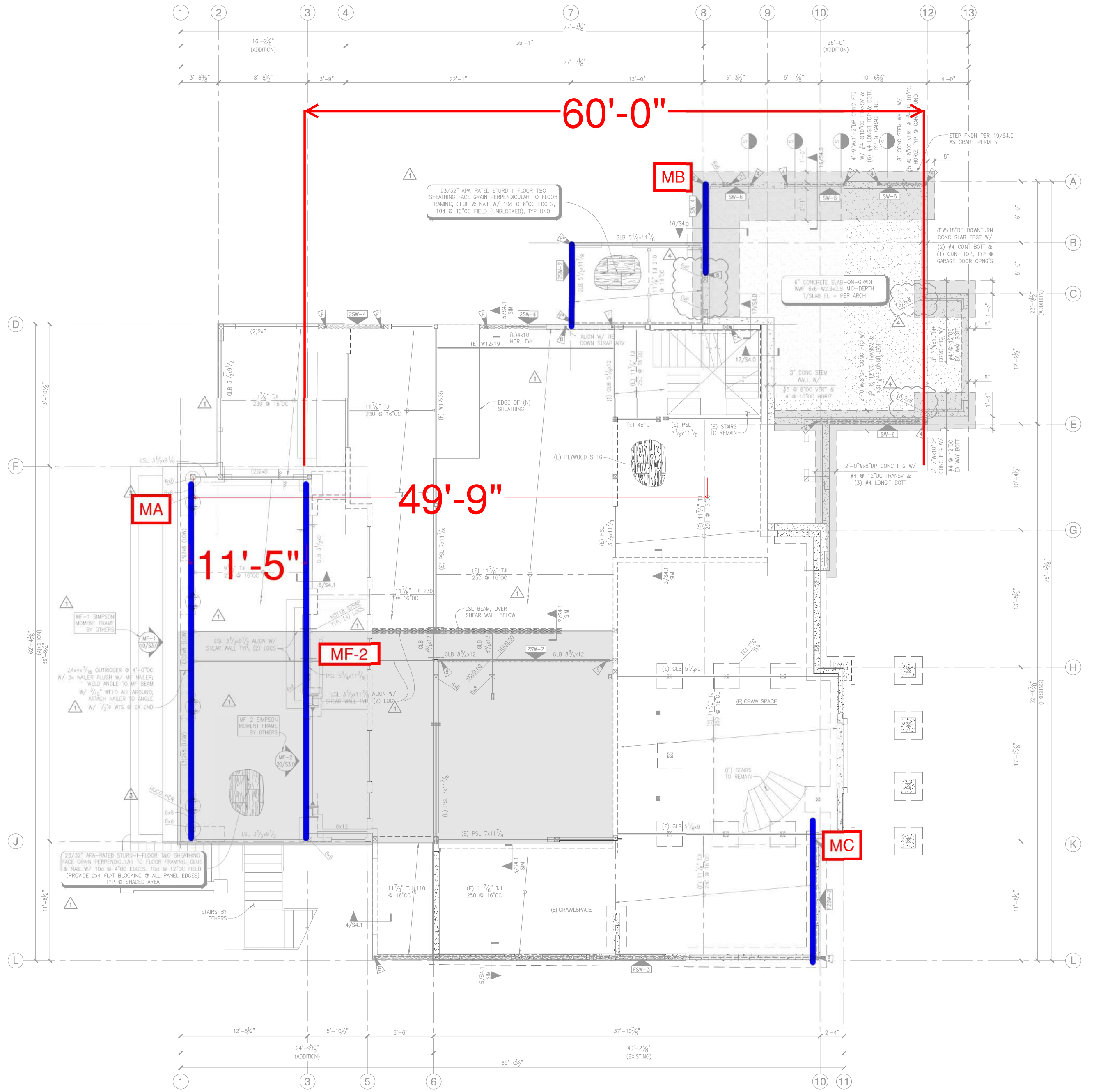
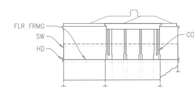
LATERAL

Peyree Remodel #17-291



MAIN FLOOR FRAMING PLAN

SCALE: 1/4" = 1'-0"



BUILDING KEY

NOTE: CONTRACTOR TO VERIFY ALL EXISTING CONDITIONS AND ARCHITECTURAL DRAWINGS PRIOR TO FABRICATION & CONSTRUCTION. NOTIFY DEI OF ANY DISCREPANCIES FOR FURTHER DIRECTION.

MAIN FLOOR FRAMING PLAN NOTES:

1. DIMENSIONS: VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS. COLUMNS AND FOOTINGS ARE CENTERED ON GRID, TYPICAL. ALL EXISTING DIMENSIONS SHALL BE FIELD VERIFIED. ALL DIMENSIONS ARE TO INSIDE FACE OF CONCRETE, OUTSIDE FACE OF CONCRETE OR CENTERLINE OF GRID/STEEL. CONTINUOUS FOOTINGS ARE CENTERED UNDER WALLS/STRUCTURAL PANELS. POSTS, BUNDLED STUDS OR COLUMNS ARE TO BE CENTERED ON FOOTING OR WALL PIER, UNO.
2. FOR ALL DUCTS, CHASES AND PIPES, REFERENCE MECHANICAL, PLUMBING, ELECTRICAL AND SPRINKLER DRAWINGS. FOR STAIR DETAILS AND GUARDRAILS, REFERENCE ARCHITECTURAL DRAWINGS.
3. AT ALL BEARING AND SHEAR WALLS, REFERENCE STUD GRADE, SIZES AND SPACING PER PLANS AND GENERAL NOTES.
4. ALL WOOD IN CONTACT WITH WEATHER-EXPOSED CONCRETE OR WITHIN 6" OF FINISHED GRADE SHALL BE PRESSURE-TREATED.
5. HANGERS: ALL 2X HANGERS TO BE SIMPSON LUS SERIES,UNO.
6. HEADERS SHOWN BUT NOT SPECIFIED ARE TO BE (2) 2X8 MINIMUM. HEADERS SHOWN SHALL BE SUPPORTED BY (2) STUDS MINIMUM, UNO ON PLAN.
7. CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY SHORING.
8. '*' INDICATES (2) HOLDOWNS REQUIRED, TYPICAL UNO ON PLAN.
9. SW-X INDICATES WOOD SHEAR WALL. REFERENCE 20/S1.2 FOR SHEAR WALL SHEATHING AND FASTENING REQUIREMENTS. REFERENCE GENERAL STRUCTURAL NOTES FOR WOOD GRADE.

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6/25/19

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6059 77th Ave SE
Mercer Island, WA 98040-5129

NO.	DATE	REVISION
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3	05/24/19	BLDG. DEPT. RESP.
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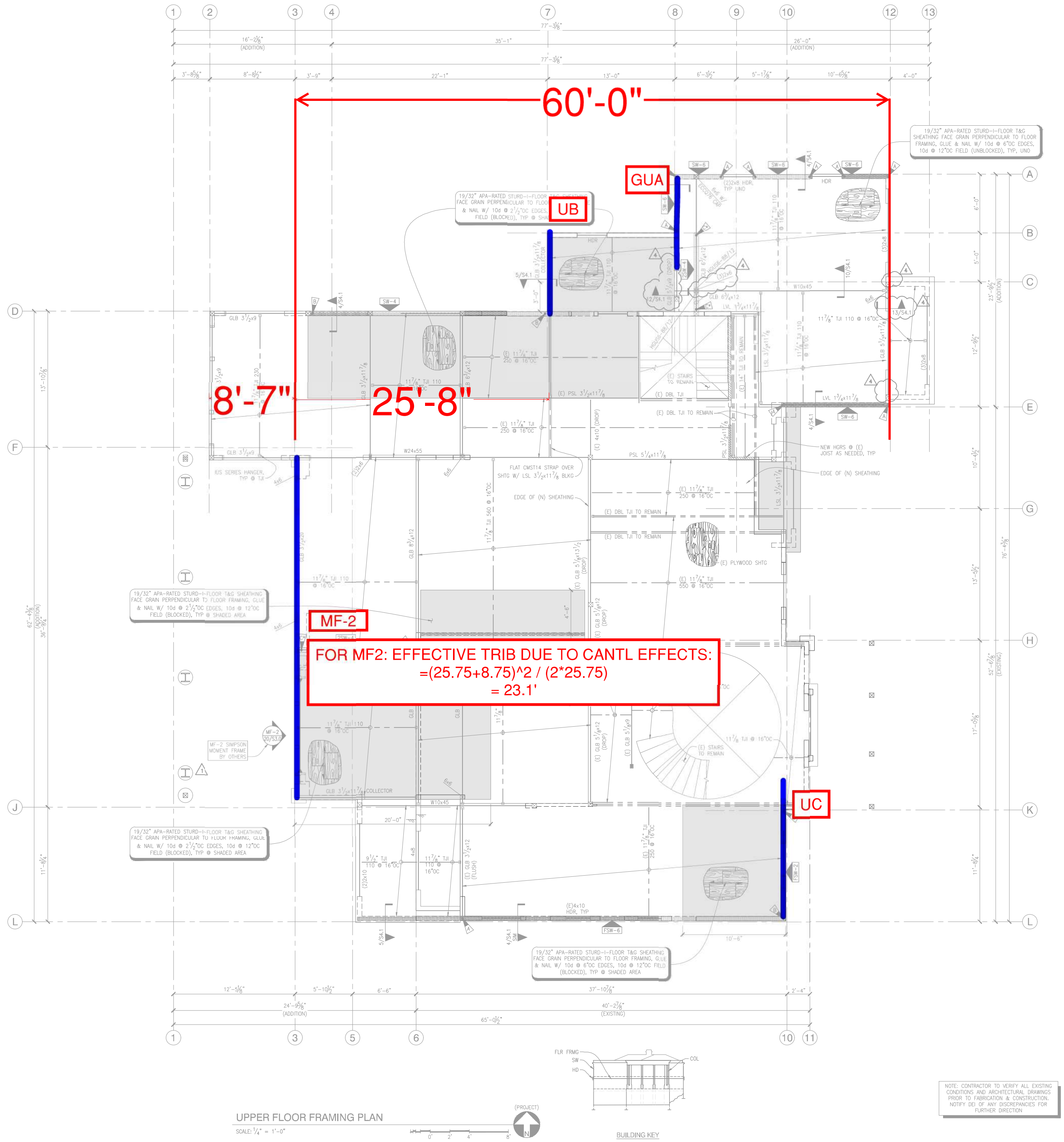
DATE: 05/19/2017
JOB NUMBER: 17-291
DRAWN BY: SAT/TLE
DESIGNED BY: JBB

L-1

STRUCTURAL MAIN FLOOR FRAMING PLAN

S2.1



UPPER FLOOR FRAMING PLAN

SCALE: 1/4" = 1'-0"

UPPER FLOOR FRAMING PLAN NOTES:

1. DIMENSIONS: VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS. COLUMNS AND FOOTINGS ARE CENTERED ON GRID, TYPICAL. ALL EXISTING DIMENSIONS SHALL BE FIELD VERIFIED. ALL DIMENSIONS ARE TO INSIDE FACE OF CONCRETE, OUTSIDE FACE OF CONCRETE OR CENTERLINE OF GRID/STEEL. CONTINUOUS FOOTINGS ARE CENTERED UNDER WALLS/STRUCTURAL PANELS. POSTS, BUNDLED STUDS OR COLUMNS ARE TO BE CENTERED ON FOOTING OR WALL PIER, UNO.
2. FOR ALL DUCTS, CHASES AND PIPES, REFERENCE MECHANICAL, PLUMBING, ELECTRICAL AND SPRINKLER DRAWINGS. FOR STAIR DETAILS AND GUARDRAILS, REFERENCE ARCHITECTURAL DRAWINGS.
3. AT ALL BEARING AND SHEAR WALLS, REFERENCE STUD GRADE, SIZES AND SPACING PER PLANS AND GENERAL NOTES.
4. ALL WOOD IN CONTACT WITH WEATHER-EXPOSED CONCRETE OR WITHIN 6" OF FINISHED GRADE SHALL BE PRESSURE-TREATED.
5. HANGERS: ALL 2X HANGERS TO BE SIMPSON LUS SERIES,UNO.
6. HEADERS SHOWN BUT NOT SPECIFIED ARE TO BE (2) 2X8 MINIMUM. HEADERS SHOWN SHALL BE SUPPORTED BY (2) STUDS MINIMUM, UNO ON PLAN.
7. CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY SHORING.
8. '*' INDICATES (2) HOLDOWNS REQUIRED, TYPICAL UNO ON PLAN.
9. SW-X INDICATES WOOD SHEAR WALL. REFERENCE 20/S1.2 FOR SHEAR WALL SHEATHING AND FASTENING REQUIREMENTS. REFERENCE GENERAL STRUCTURAL NOTES FOR WOOD GRADE.

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DATE: 05/19/2017
JOB NUMBER: 17-291
DRAWN BY: SAT/TLE
DESIGNED BY: JBB

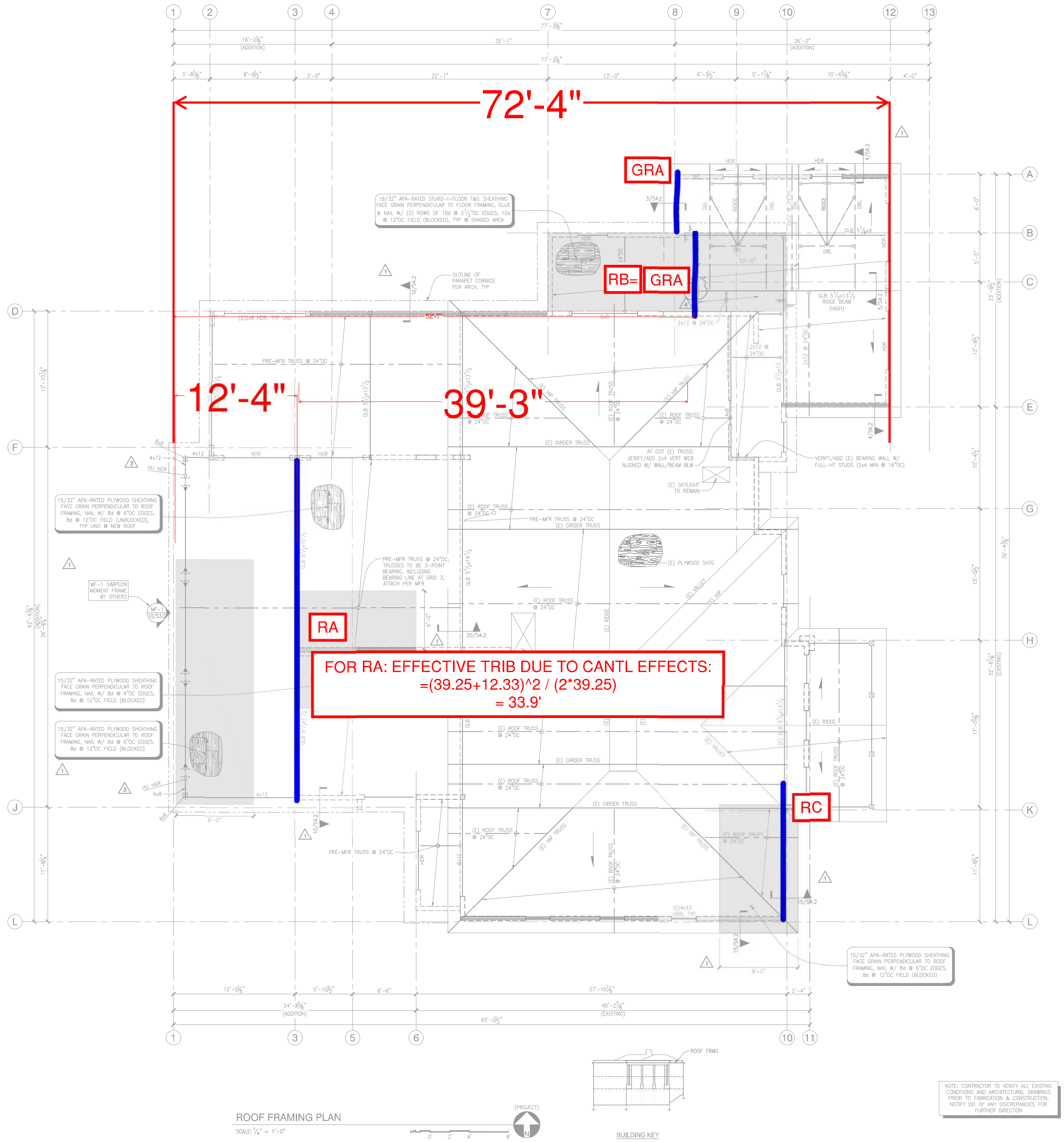
L-2

STRUCTURAL
UPPER FLOOR
FRAMING PLAN

S2.2

ROOF FRAMING PLAN NOTES:

1. FOR STRUCTURAL GENERAL NOTES, DESIGN CRITERIA, ABBREVIATIONS AND LEGEND, REFERENCE SHEET S1.0.
2. DIMENSIONS: VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS. COLUMNS AND FOOTINGS ARE CENTERED ON GRID, TYPICAL UNO. ALL EXISTING DIMENSIONS SHALL BE FIELD VERIFIED. ALL DIMENSIONS ARE TO INSIDE FACE OF CONCRETE, OUTSIDE FACE OF CONCRETE OR CENTERLINE OF GRID/STEEL. CONTINUOUS FOOTINGS ARE CENTERED UNDER WALLS/STRUCTURAL PANELS, UNO. POSTS, BUNDLED STUDS OR COLUMNS ARE TO BE CENTERED ON FOOTING OR WALL PIER, UNO.
3. FOR ALL DUCTS, CHASES AND PIPES, REFERENCE MECHANICAL, PLUMBING, ELECTRICAL AND SPRINKLER DRAWINGS. FOR STAIR DETAILS AND GUARDRAILS, REFERENCE ARCHITECTURAL DRAWINGS.
4. FOR CONCRETE SHEAR WALLS OR MOMENT FRAME MEMBER SIZES, REFER TO ELEVATIONS.
5. AT ALL WOOD-FRAMED, BEARING AND SHEAR WALLS, REFERENCE STUD GRADE, SIZES AND SPACING PER GENERAL NOTES. ALL EXTERIOR WALL STUDS 12'-0" HIGH OR GREATER, USE (2) SISTER STUDS AT 16"OC, UNO.
6. TOP PLATE BEARING WALL ELEVATION PER ARCHITECTURAL DRAWINGS.
7. BALLOON FRAME ALL WALLS GREATER THAN ONE LEVEL WITHOUT FLOOR OR ROOF SUPPORT.
8. BLOCKING: PROVIDE SOLID BLOCKING OVER ALL SHEAR WALLS AND BEARING WALLS. AT SHEAR WALLS PARALLEL TO FRAMING, ALIGN (1) ROOF TRUSS OR CONTINUOUS BLOCKING OVER SHEAR WALLS. ADDITIONAL ROOF TRUSSES MAY BE REQUIRED TO ACCOMMODATE BLOCKING.
9. HEADERS SHOWN BUT NOT SPECIFIED ARE TO BE (2) 2X8 MINIMUM. HEADERS SHOWN SHALL BE SUPPORTED BY (2) STUDS MINIMUM, UNO ON PLAN. BEAMS AND HEADERS ARE TO BE FLUSH FRAMED WITH JOISTS, UNLESS NOTED AS "DROP" INDICATING THAT DROPPED BEAM FRAMING IS REQUIRED.
10. ROOF TRUSSES SHALL BE DESIGNED FOR THE FOLLOWING CRITERIA:
    - REFER TO THE STRUCTURAL GENERAL NOTES FOR STANDARD DEAD AND LIVE LOADS AND SUBMITTAL INFORMATION.
    - TRUSS LAYOUT SHOWN IS APPROXIMATE. TRUSS SUPPLIER IS RESPONSIBLE FOR FINAL TRUSS LAYOUT AND CONFIGURATION. NOTIFY ENGINEER OF REVISIONS TO PLAN.
    - SHADED REGION INDICATES APPROXIMATE AREA OF OVER FRAMING. TRUSS MANUFACTURER IS RESPONSIBLE FOR DESIGNING THE OVER FRAMING REQUIRED. TRUSSES SHALL BE DESIGNED TO SUPPORT OVER FRAMING IN ADDITION TO THE STANDARD DESIGN LOADS.
    - PROVIDE SIMPSON H1 HURRICANE TIES AT ALL ROOF TRUSSES AND ROOF JOISTS, TYP,UNO.
12. 2000# INDICATES SHEAR TRANSFER LOAD IN ROOF TRUSS TO BE LOCATED ABOVE SHEAR WALLS TRUSS MANUFACTURER SHALL DESIGN THESE TRUSSES FOR THE LATERAL LOAD SPECIFIED ON PLAN, IN BRACKETS, IN ADDITION TO THE DESIGN DEAD AND LIVE LOADS.
13. SIMPSON STRAP TIES INDICATED ON THE SHEAR WALL PLANS ARE TO BE CENTERED OVER WALL TOP PLATE AND/OR HEADER, BLOCKING OR BEAM CONTRACTOR SHALL COORDINATE ADDITIONAL WALL FURRING REQUIRED AT BEAMS AND POSTS WITH CONNECTORS OR HOLDOWNS THAT EXCEED THE NOMINAL WALL THICKNESS.
14. CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY SHORING.



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DATE: 05/19/2017
JOB NUMBER: 17-291
DRAWN BY: SAT/TLE
DESIGNED BY: JBB

L-3

STRUCTURAL ROOF FRAMING PLAN

S2.3

DEI
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Project	Peyree Remodel	*Sheet #*	
Project #	17-291	*Date*	1/2020
Subject	Lateral - Shear Wall Design	*By*	NDM

CHANGES PER OWNER REVs

Shear Wall Design - Moment Frame Loading
North-South Direction

Lateral Shear Load (ASD)

North-South Direction

Roof	20.6	kips	
Upper	13.63	kips	
Main	3.07	kips	
Lower	0	kips	[seismic base]
	37.3	**kips**	**= Base Shear, OK**

Main Buildling

Area	3960	ft^2	
Area Ratio	1.000		
Proportional Shear Load			
Roof	20.600	kips	
Upper	13.630	kips	
Main	3.070	kips	
Lower	0	kips	[seismic base]

North-South: Roof Diaphragm - Upper Walls

Wall Height 8.5 ft
Bldg Width 72.33 ft

Wall ID	Pier Length 1	Pier Length 2	Pier Length 3	Pier Length 4	A/R Length 1	A/R Length 2	A/R Length 3	A/R Length 4	Total Length	Trib Area	Level Shear	Shear Abv	Total Shear	Unit Shear	SW-Nailing	SW Capacity
RA	2.5	2.75	2.75	2.5	2.0625	2.375	2.375	2.0625	10.5	33.9	9.65	-	9.65	0.92	2SW-3	-
RB	8.5	0	0	0	8.5	0	0	0	8.5	32.0	9.11	-	9.11	1.07	-	-
RC	11.333	0	0	0	11.333	0	0	0	11.3	14.0	3.99	-	3.99	0.35	-	-
									Total Area	79.9	**Includes RBA Cantilever Effects**					

North-South: Upper Diaphragm - Main Walls

Wall Height 9.5 ft
Bldg Width 60 ft

Wall ID	Pier Length 1	Pier Length 2	Pier Length 3	Pier Length 4	A/R Length 1	A/R Length 2	A/R Length 3	A/R Length 4	Total Length	Trib Area	Level Shear	Shear Abv	Total Shear	Unit Shear	SW-Nailing	SW Capacity
MF2	0	0	0	0	0	0	0	0	31.3	23.1	5.25	9.65	14.90	0.17	SMF	-
UB	8.5	0	0	0	8.5	0	0	0	8.5	25.6	5.82	9.11	14.93	0.68	-	-
UC	11.333	0	0	0	11.333	0	0	0	11.3	12.6	2.87	3.99	6.86	0.25	-	-
									Total Area	61.4	**Includes MF2 Cantilever Effects**					

North-South: Main Diaphragm - Lower Walls

Wall Height 11.33 ft
Diaph Width 42.5 ft [seismic base resolves rest of bldg width]

Wall ID	Pier Length 1	Pier Length 2	Pier Length 3	Pier Length 4	A/R Length 1	A/R Length 2	A/R Length 3	A/R Length 4	Total Length	Trib Area	Level Shear	Shear Abv	Total Shear	Unit Shear	SW-Nailing	SW Capacity
MA	4.5	2.25	2.25	4.5	4.03125	0	0	4.03125	9.0	6.3	0.45	-	0.45	0.05	SW-6	-
MF2	0	0	0	0	0	0	0	0	31.3	21.3	1.54	14.90	16.44	0.05	SMF	-
MB	28.333	0	0	0	28.333	0	0	0	28.3	26.3	1.90			0.07	-	-
MC	17.333	0	0	0	17.333	0	0	0	17.3	11.3	0.81			0.05	-	-
									Total Area	65.0						

Moment Frame: MF-2

Using RISA 3D for Analysis

Drift Limit

[ASCE 7-10 §12.12-1]

$\Delta_a = 0.020 h_{sx}$

$\Delta_2 = 0.020(10.5' \times 12) = 2.52''$

$\Delta_1 = 0.020(13.67' \times 12) = 3.28''$

Max Frame Deflection in RISA

$$\delta_{emax,2} = \frac{\Delta_2 (I_e)}{C_d} = \frac{2.52''(1.0)}{4.0} = 0.63''$$

$$\delta_{emax,1} = \frac{3.28''}{4} = 0.82''$$

$\underline{}$ 1.45''

Loads

RDL=21 PSF
FDL=15 PSF
LL=40 PSF
LR=20 PSF
S=25 PSF

	Gravity							Lateral*		
	Roof Trib	W_{RDL}	W_{LR}	W_S	FLR Trib	W_{FDL}	W_{LL}	V_{ABV}	V_{FLR}	$W_{NAMAWALL}$
2nd Floor	14'	294	280	350	6.25'	94	250	441	243	$=8_{PSF}(8.5') = \underline{68}$
1st Floor	—	—	—	—	11.75'	176	470	—	71	—

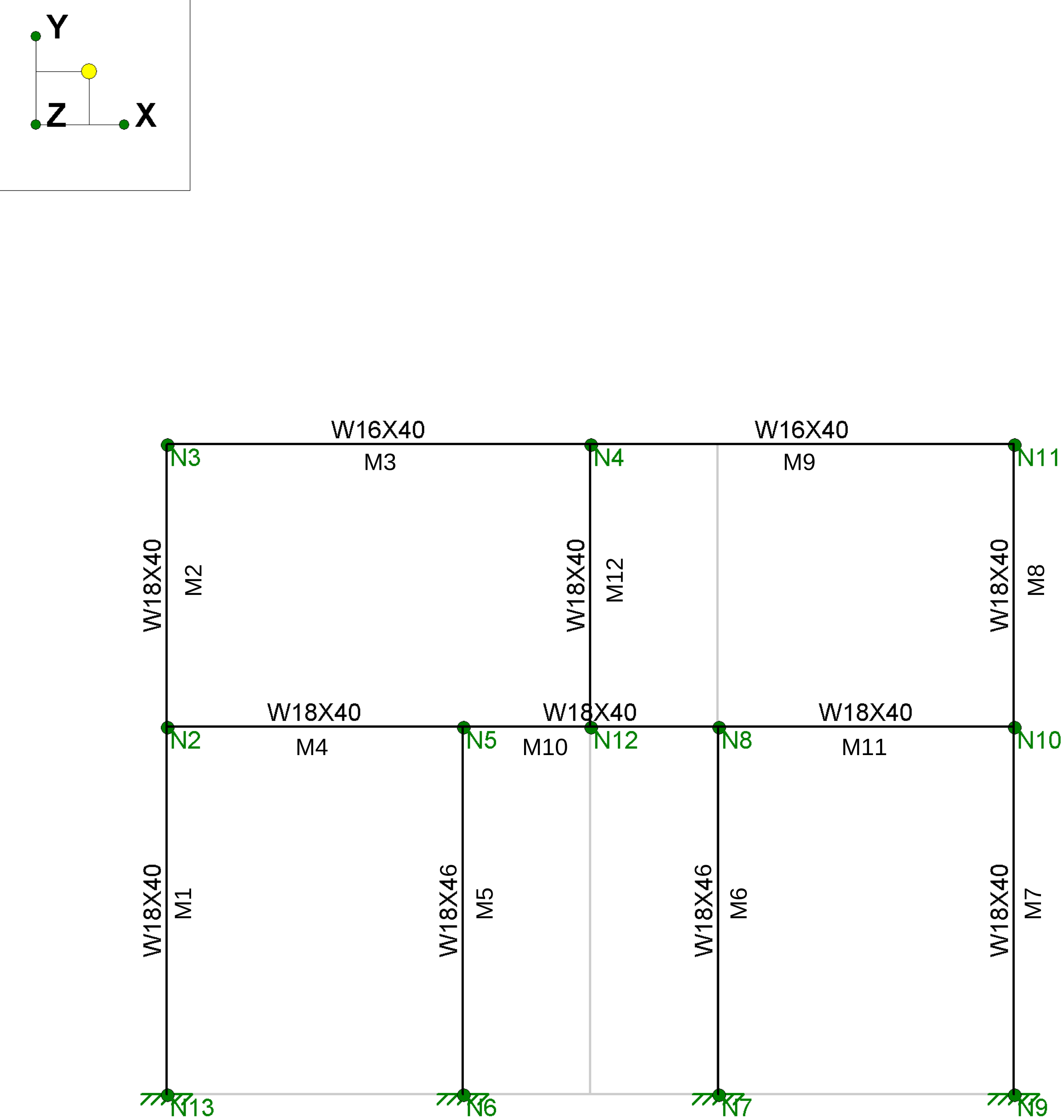
Units: Trib: FT; Else: PLF

*Loads are ultimate: per dist. spreadsheet: Dist. Load/0.7

2nd $V_{ABV} = 9.65^k / 31.25' \left(\frac{1}{0.7}\right) = 0.441$

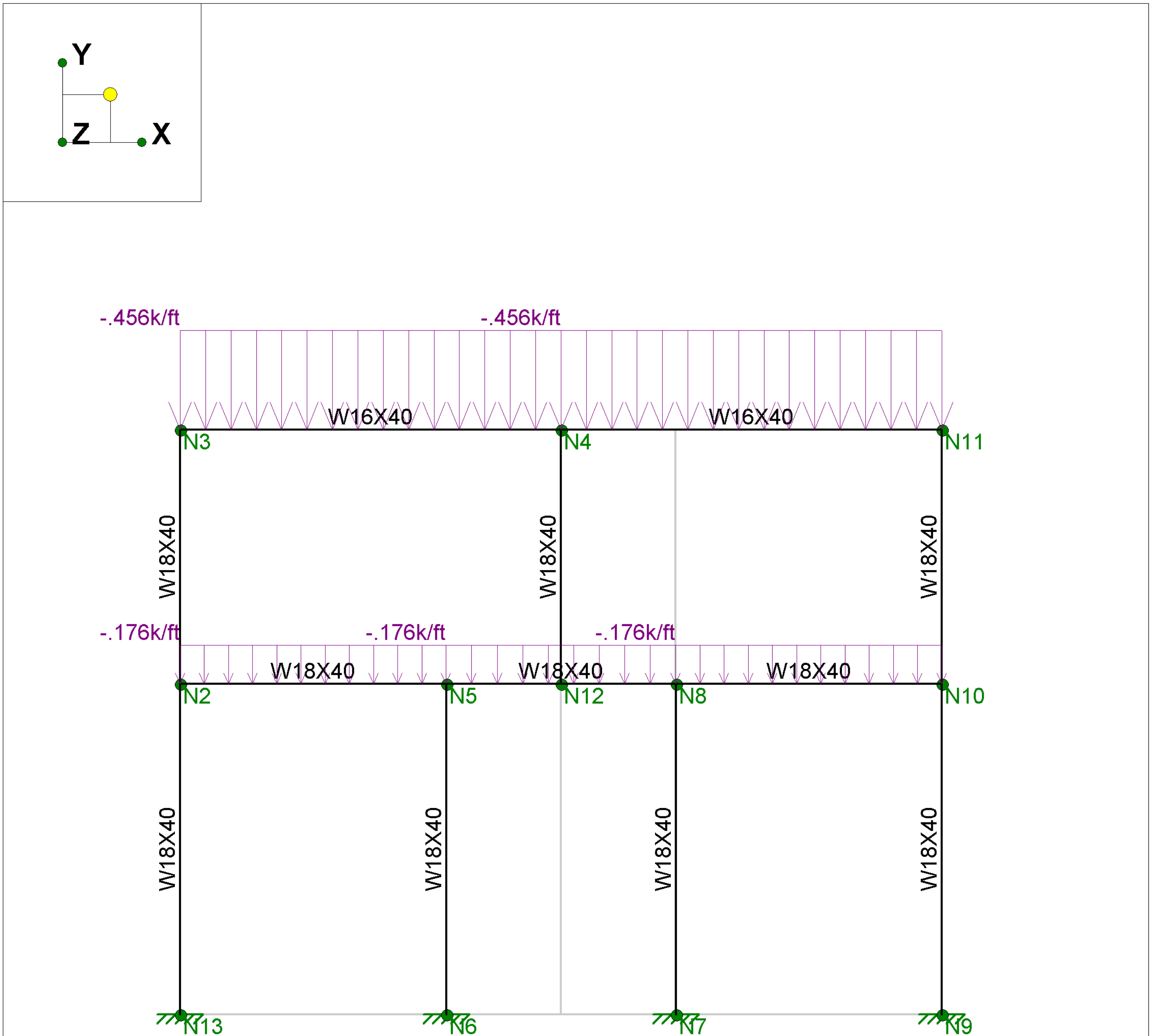
2nd $V_{FLR} = 5.25 / 31.25' (1/0.7) = 0.243$

1st $V_{FLR} = 1.54 / 31.25' (1/0.7) = 0.070$



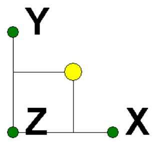
Envelope Only Solution

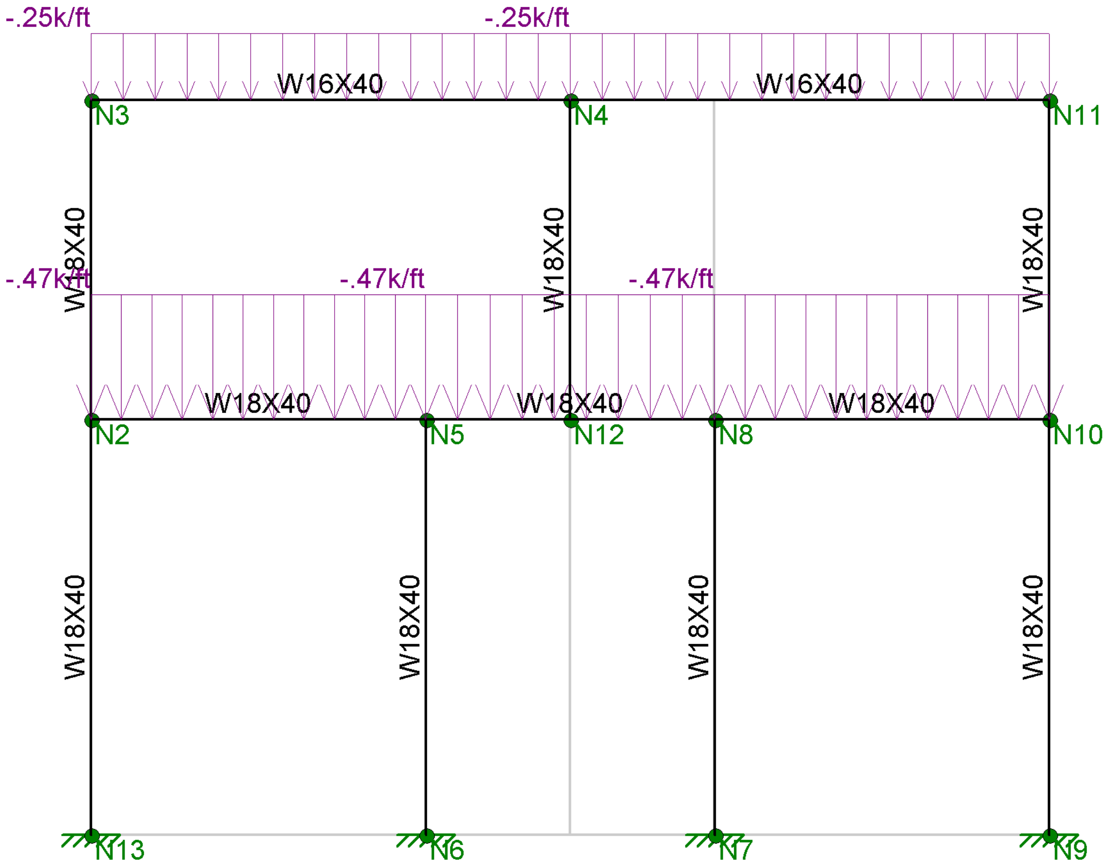
DEI		SK - 1
RH/NDM	Moment Frame 2	Jan 27, 2020 at 1:29 PM
17-291	Beam and Column Sections	MF-2 - Owner Revisions - 2 bay over...



Loads: BLC 1, DL
Envelope Only Solution

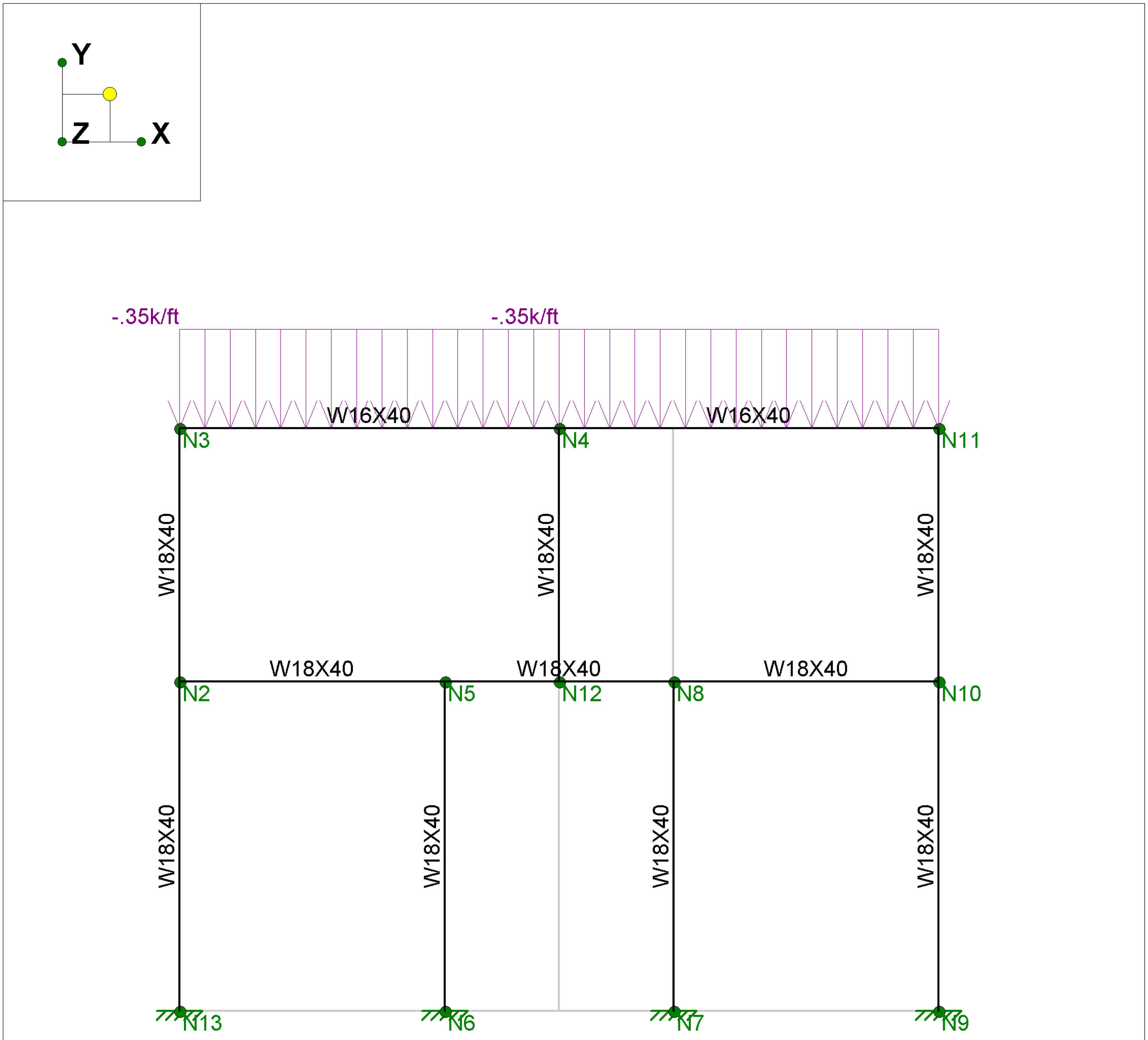
DEI		SK - 2
RH/NDM	Moment Frame 2	Jan 27, 2020 at 1:29 PM
17-291	Dead Loads	MF-2 - Owner Revisions - 2 bay over...





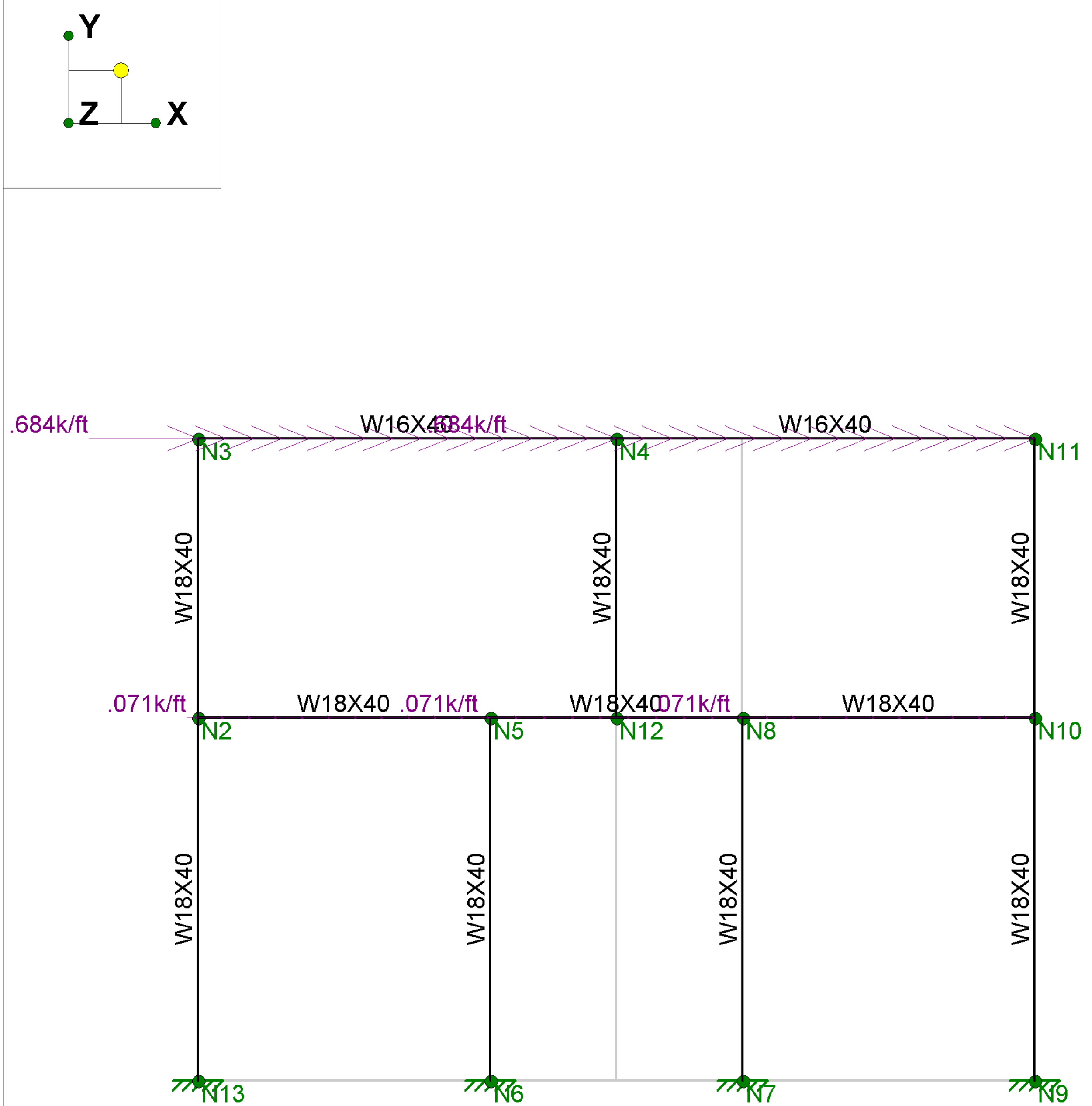
Loads: BLC 2, LL
Envelope Only Solution

DEI		SK - 3
RH/NDM	Moment Frame 2	Jan 27, 2020 at 1:29 PM
17-291	Live Loads	MF-2 - Owner Revisions - 2 bay over...



Loads: BLC 3, SL
Envelope Only Solution

DEI	Moment Frame 2	SK - 4
RH/NDM		Jan 27, 2020 at 1:30 PM
17-291	Snow Loads	MF-2 - Owner Revisions - 2 bay over...



Loads: BLC 4, E
Envelope Only Solution

DEI		SK - 5
RH/NDM	Moment Frame 2	Jan 27, 2020 at 1:30 PM
17-291	Seismic Loads	MF-2 - Owner Revisions - 2 bay over...

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N2	0	12.9167	0	0	
3	N3	0	23.5	0	0	
4	N4	15.75	23.5	0	0	
5	N5	11	12.9167	0	0	
6	N6	11	0	0	0	
7	N7	20.5	0	0	0	
8	N8	20.5	12.9167	0	0	
9	N9	31.5	0	0	0	
10	N10	31.5	12.9167	0	0	
11	N11	31.5	23.5	0	0	
12	N12	15.75	12.9167	0	0	
13	N13	0	0	0	0	

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area(...	Surface(Plate/Wall)
1	DL	DL						5		
2	LL	LL						5		
3	SL	SL						2		
4	E	EL						5		
5	W	WL								

Load Combinations

	Description	Solve	PDelta	S...	B...	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	IBC 16-1 1.4D	Yes	Y		DL	1.4																		
2	IBC 16-2 1.2D+1.6L+0...	Yes	Y		DL	1.2	LL	1.6	SL	.5														
3	IBC 16-3 1.2D+1.6S+f...	Yes	Y		DL	1.2	SL	1.6	LL	.5														
4	IBC 16-4 1.2D+1.0W+...	Yes	Y		DL	1.2	LL	.5	SL	.5	WL	1												
5	IBC 16-5 1.3952D+1.0...	Yes	Y		DL	1.3...	LL	.5	SL	.2	EL	1												
6	ASD 6b. 1.102D+.525E...		Y		DL	1.1...	LL	.75	SL	.75	EL	.525												
7	16-5 with Omega		Y		DL	1.3...	LL	.5	SL	.2	EL	2.5												
8	ASD 5. 1.137D+.7E		Y		DL	1.1...					EL	.7												
9	IBC 16-6 0.9D+1.0W	Yes	Y		DL	.9	WL	1																
10	IBC 16-7 .7048D+1.0E	Yes	Y		DL	.705	EL	1																
11	16-7 with Omega		Y		DL	.705	EL	2.5																
12	D	Yes	Y		DL	1																		
13	L	Yes	Y		LL	1																		
14	S	Yes	Y		SL	1																		
15	E	Yes	Y		EL	1																		

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	.063	13	12.915	2	0	15	0	15	0	15	0	15
2		min	-4.525	5	-15.332	15	0	1	0	1	0	1	0	1
3	N6	max	.632	3	19.93	2	0	15	0	15	0	15	0	15
4		min	-7.568	15	-5.504	15	0	1	0	1	0	1	0	1

Envelope Joint Reactions (Continued)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
5	N7	max	-.089	13	19.93	2	0	15	0	15	0	15	0	15
6		min	-7.894	5	3.109	14	0	1	0	1	0	1	0	1
7	N9	max	.135	3	23.777	5	0	15	0	15	0	15	0	15
8		min	-4.252	15	2.404	14	0	1	0	1	0	1	0	1
9	Totals:	max	0	14	65.69	2	0	15						
10		min	-23.783	5	0	15	0	1						

Envelope Joint Displacements

	Joint		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotatio...	LC	Y Rotatio...	LC	Z Rotatio...	LC
1	N1	max	0	15	0	15	0	15	0	15	0	15	1.924e-05	13
2		min	0	1	0	1	0	1	0	1	0	1	-5.781e-03	5
3	N2	max	.712	5	.009	15	0	15	0	15	0	15	8.784e-05	3
4		min	-.002	3	-.007	2	0	1	0	1	0	1	-2.013e-03	15
5	N3	max	.987	5	.011	15	0	15	0	15	0	15	-1.222e-04	13
6		min	0	14	-.011	2	0	1	0	1	0	1	-1.646e-03	5
7	N4	max	.985	5	0	15	0	15	0	15	0	15	0	14
8		min	0	1	-.049	2	0	1	0	1	0	1	-1.015e-03	5
9	N5	max	.713	5	.003	15	0	15	0	15	0	15	-4.372e-05	13
10		min	0	2	-.01	2	0	1	0	1	0	1	-9.971e-04	5
11	N6	max	0	15	0	15	0	15	0	15	0	15	1.463e-04	3
12		min	0	1	0	1	0	1	0	1	0	1	-6.259e-03	15
13	N7	max	0	15	0	15	0	15	0	15	0	15	-2.09e-05	13
14		min	0	1	0	1	0	1	0	1	0	1	-6.43e-03	5
15	N8	max	.713	5	-.002	14	0	15	0	15	0	15	3.132e-04	3
16		min	0	14	-.01	2	0	1	0	1	0	1	-7.847e-04	15
17	N9	max	0	15	0	15	0	15	0	15	0	15	2.588e-05	3
18		min	0	1	0	1	0	1	0	1	0	1	-5.771e-03	5
19	N10	max	.714	5	-.001	14	0	15	0	15	0	15	3.423e-05	13
20		min	0	13	-.013	5	0	1	0	1	0	1	-2.066e-03	5
21	N11	max	.985	5	-.003	14	0	15	0	15	0	15	6.378e-04	3
22		min	-.002	2	-.018	5	0	1	0	1	0	1	-1.212e-03	15
23	N12	max	.714	5	0	15	0	15	0	15	0	15	0	14
24		min	0	1	-.04	2	0	1	0	1	0	1	-5.2e-04	5
25	N13	max	0	15	0	15	0	15	0	15	0	15	0	15
26		min	0	1	0	1	0	1	0	1	0	1	0	1

Envelope Maximum Member Section Forces

	Member		Axial[k]	Loc[ft]	LC	y Shear...	Loc[ft]	LC	z Shear...	Loc[ft]	LC	Torque[...	Loc[ft]	LC	y-y Mome...	Loc[ft]	LC	z-z Mome...	Loc[ft]	LC
1	M1	m...	12.915	0	2	4.473	0	5	0	0	15	0	0	15	0	0	15	.816	12.9...	13
2		min	-15.3...	0	15	-.063	0	13	0	0	1	0	0	1	0	0	1	-57.773	12.9...	5
3	M2	m...	9.269	0	3	4.673	0	15	0	0	15	0	0	15	0	0	15	18.679	10.5...	3
4		min	-3.949	0	15	-2.326	0	2	0	0	1	0	0	1	0	0	1	-32.184	10.5...	15
5	M3	m...	7.708	15.75	5	9.27	0	3	0	0	15	0	0	15	0	0	15	47.219	15.75	5
6		min	-4.687	0	15	-10.784	15.75	5	0	0	1	0	0	1	0	0	1	-32.184	0	15
7	M4	m...	.994	11	15	4.398	0	2	0	0	15	0	0	15	0	0	15	62.517	11	5
8		min	-2.387	0	3	-14.759	11	5	0	0	1	0	0	1	0	0	1	-73.62	0	15
9	M5	m...	19.93	0	2	.632	0	3	0	0	15	0	0	15	0	0	15	8.162	0	3
10		min	-5.504	0	15	-7.528	0	15	0	0	1	0	0	1	0	0	1	-97.237	0	15
11	M6	m...	19.93	0	2	8.026	0	5	0	0	15	0	0	15	0	0	15	0	0	15

Envelope Maximum Member Section Forces (Continued)

	Member		Axial[k]	Loc[ft]	LC	y Shear...	Loc[ft]	LC	z Shear...	Loc[ft]	LC	Torque[...	Loc[ft]	LC	y-y Mome...	Loc[ft]	LC	z-z Mome...	Loc[ft]	LC
12		min	3.109	0	14	.089	0	13	0	0	1	0	0	1	0	0	1	-103.663	12.9...	5
13	M7	m...	23.777	0	5	4.379	0	5	0	0	15	0	0	15	0	0	15	1.736	12.9...	3
14		min	2.404	0	14	-.134	0	3	0	0	1	0	0	1	0	0	1	-56.562	12.9...	5
15	M8	m...	10.246	0	5	6.284	0	5	0	0	15	0	0	15	0	0	15	21.374	0	5
16		min	1.931	0	13	.577	0	14	0	0	1	0	0	1	0	0	1	-45.136	10.5...	5
17	M9	m...	6.25	15.75	5	10.137	0	3	0	0	15	0	0	15	0	0	15	45.136	15.75	5
18		min	-6.117	0	15	-10.246	15.75	5	0	0	1	0	0	1	0	0	1	-30.001	0	15
19	M10	m...	6.304	9.5	15	13.732	0	2	0	0	15	0	0	15	0	0	15	61.825	9.5	5
20		min	-7.839	0	5	-26.151	9.5	5	0	0	1	0	0	1	0	0	1	-56.971	4.75	5
21	M11	m...	-.403	11	15	6.197	0	2	0	0	15	0	0	15	0	0	15	77.937	11	5
22		min	-2.827	0	5	-13.532	11	5	0	0	1	0	0	1	0	0	1	-51.595	0	15
23	M12	m...	20.275	0	3	0	0	14	0	0	15	0	0	15	0	0	15	69.581	10.5...	5
24		min	.002	0	15	-12.278	0	5	0	0	1	0	0	1	0	0	1	-60.36	0	5

Envelope Member Section Stresses

	Member	Sec		Axial[ksi]	LC	y Shear[ksi]	LC	z Shea...	LC	y-Top[ksi]	LC	y-Bot[ksi]	LC	z-Top[...	LC	z-Bot[...	LC
1	M1	1	max	1.094	2	.793	5	0	15	0	15	0	15	0	15	0	15
2			min	-1.299	15	-.011	13	0	1	0	1	0	1	0	1	0	1
3		2	max	1.094	2	.793	5	0	15	2.535	5	.036	13	0	15	0	15
4			min	-1.299	15	-.011	13	0	1	-.036	13	-2.535	5	0	1	0	1
5		3	max	1.094	2	.793	5	0	15	5.069	5	.072	13	0	15	0	15
6			min	-1.299	15	-.011	13	0	1	-.072	13	-5.069	5	0	1	0	1
7		4	max	1.094	2	.793	5	0	15	7.604	5	.107	13	0	15	0	15
8			min	-1.299	15	-.011	13	0	1	-.107	13	-7.604	5	0	1	0	1
9		5	max	1.094	2	.793	5	0	15	10.139	5	.143	13	0	15	0	15
10			min	-1.299	15	-.011	13	0	1	-.143	13	-10.139	5	0	1	0	1
11	M2	1	max	.786	3	.829	15	0	15	1.222	2	3.032	15	0	15	0	15
12			min	-.335	15	-.412	2	0	1	-3.032	15	-1.222	2	0	1	0	1
13		2	max	.786	3	.829	15	0	15	.155	13	.889	10	0	15	0	15
14			min	-.335	15	-.412	2	0	1	-.889	10	-.155	13	0	1	0	1
15		3	max	.786	3	.829	15	0	15	1.308	15	1.186	3	0	15	0	15
16			min	-.335	15	-.412	2	0	1	-1.186	3	-1.308	15	0	1	0	1
17		4	max	.786	3	.829	15	0	15	3.478	15	2.232	3	0	15	0	15
18			min	-.335	15	-.412	2	0	1	-2.232	3	-3.478	15	0	1	0	1
19		5	max	.786	3	.829	15	0	15	5.648	15	3.278	3	0	15	0	15
20			min	-.335	15	-.412	2	0	1	-3.278	3	-5.648	15	0	1	0	1
21	M3	1	max	.197	2	1.9	3	0	15	5.965	15	3.462	3	0	15	0	15
22			min	-.397	15	-.809	15	0	1	-3.462	3	-5.965	15	0	1	0	1
23		2	max	.197	2	.905	3	0	15	4.11	5	-.267	13	0	15	0	15
24			min	-.169	15	-.809	15	0	1	.267	13	-4.11	5	0	1	0	1
25		3	max	.197	2	-.008	13	0	15	2.987	3	-.202	15	0	15	0	15
26			min	.049	14	-.869	5	0	1	.202	15	-2.987	3	0	1	0	1
27		4	max	.425	5	-.209	13	0	15	.9	3	2.68	15	0	15	0	15
28			min	.049	14	-1.539	5	0	1	-2.68	15	-.9	3	0	1	0	1
29		5	max	.653	5	-.411	13	0	15	-.892	13	8.751	5	0	15	0	15
30			min	.049	14	-2.21	5	0	1	-8.751	5	.892	13	0	1	0	1
31	M4	1	max	.018	15	.78	2	0	15	12.92	15	1.245	2	0	15	0	15
32			min	-.202	3	-2.019	15	0	1	-1.245	2	-12.92	15	0	1	0	1
33		2	max	.035	15	.31	2	0	15	7.527	5	.122	3	0	15	0	15
34			min	-.202	3	-2.019	15	0	1	-.122	3	-7.527	5	0	1	0	1

Envelope Member Section Stresses (Continued)

	Member	Sec		Axial[ksi]	LC	y Shear[ksi]	LC	z Shea...	LC	y-Top[ksi]	LC	y-Bot[ksi]	LC	z-Top[...	LC	z-Bot[...	LC
35		3	max	.051	15	-.038	14	0	15	1.999	5	.236	3	0	15	0	15
36			min	-.202	3	-2.149	5	0	1	-.236	3	-1.999	5	0	1	0	1
37		4	max	.068	15	-.038	14	0	15	-.036	13	4.167	5	0	15	0	15
38			min	-.202	3	-2.383	5	0	1	-4.167	5	.036	13	0	1	0	1
39		5	max	.084	15	-.038	14	0	15	-.422	14	10.971	5	0	15	0	15
40			min	-.202	3	-2.618	5	0	1	-10.971	5	.422	14	0	1	0	1
41	M5	1	max	1.476	2	.097	3	0	15	14.831	15	1.245	3	0	15	0	15
42			min	-.408	15	-1.155	15	0	1	-1.245	3	-14.831	15	0	1	0	1
43		2	max	1.476	2	.097	3	0	15	11.124	15	.934	3	0	15	0	15
44			min	-.408	15	-1.155	15	0	1	-.934	3	-11.124	15	0	1	0	1
45		3	max	1.476	2	.097	3	0	15	7.416	15	.622	3	0	15	0	15
46			min	-.408	15	-1.155	15	0	1	-.622	3	-7.416	15	0	1	0	1
47		4	max	1.476	2	.097	3	0	15	3.708	15	.311	3	0	15	0	15
48			min	-.408	15	-1.155	15	0	1	-.311	3	-3.708	15	0	1	0	1
49		5	max	1.476	2	.097	3	0	15	0	15	0	15	0	15	0	15
50			min	-.408	15	-1.155	15	0	1	0	1	0	1	0	1	0	1
51	M6	1	max	1.476	2	1.232	5	0	15	0	15	0	15	0	15	0	15
52			min	.23	14	.014	13	0	1	0	1	0	1	0	1	0	1
53		2	max	1.476	2	1.232	5	0	15	3.953	5	-.044	13	0	15	0	15
54			min	.23	14	.014	13	0	1	.044	13	-3.953	5	0	1	0	1
55		3	max	1.476	2	1.232	5	0	15	7.906	5	-.088	13	0	15	0	15
56			min	.23	14	.014	13	0	1	.088	13	-7.906	5	0	1	0	1
57		4	max	1.476	2	1.232	5	0	15	11.859	5	-.131	13	0	15	0	15
58			min	.23	14	.014	13	0	1	.131	13	-11.859	5	0	1	0	1
59		5	max	1.476	2	1.232	5	0	15	15.812	5	-.175	13	0	15	0	15
60			min	.23	14	.014	13	0	1	.175	13	-15.812	5	0	1	0	1
61	M7	1	max	2.015	5	.777	5	0	15	0	15	0	15	0	15	0	15
62			min	.204	14	-.024	3	0	1	0	1	0	1	0	1	0	1
63		2	max	2.015	5	.777	5	0	15	2.482	5	.076	3	0	15	0	15
64			min	.204	14	-.024	3	0	1	-.076	3	-2.482	5	0	1	0	1
65		3	max	2.015	5	.777	5	0	15	4.963	5	.152	3	0	15	0	15
66			min	.204	14	-.024	3	0	1	-.152	3	-4.963	5	0	1	0	1
67		4	max	2.015	5	.777	5	0	15	7.445	5	.229	3	0	15	0	15
68			min	.204	14	-.024	3	0	1	-.229	3	-7.445	5	0	1	0	1
69		5	max	2.015	5	.777	5	0	15	9.926	5	.305	3	0	15	0	15
70			min	.204	14	-.024	3	0	1	-.305	3	-9.926	5	0	1	0	1
71	M8	1	max	.868	5	1.115	5	0	15	-.167	14	3.751	5	0	15	0	15
72			min	.164	13	.102	14	0	1	-3.751	5	.167	14	0	1	0	1
73		2	max	.868	5	1.115	5	0	15	.14	3	.859	15	0	15	0	15
74			min	.164	13	.102	14	0	1	-.859	15	-.14	3	0	1	0	1
75		3	max	.868	5	1.115	5	0	15	2.085	5	-.144	13	0	15	0	15
76			min	.164	13	.102	14	0	1	.144	13	-2.085	5	0	1	0	1
77		4	max	.868	5	1.115	5	0	15	5.003	5	-.443	13	0	15	0	15
78			min	.164	13	.102	14	0	1	.443	13	-5.003	5	0	1	0	1
79		5	max	.868	5	1.115	5	0	15	7.921	5	-.742	13	0	15	0	15
80			min	.164	13	.102	14	0	1	.742	13	-7.921	5	0	1	0	1
81	M9	1	max	.197	2	2.077	3	0	15	5.56	15	4.727	3	0	15	0	15
82			min	-.518	15	-.809	15	0	1	-4.727	3	-5.56	15	0	1	0	1
83		2	max	.197	2	1.083	3	0	15	3.318	5	-.213	13	0	15	0	15
84			min	-.29	15	-.809	15	0	1	.213	13	-3.318	5	0	1	0	1
85		3	max	.197	2	.089	3	0	15	2.987	3	.201	15	0	15	0	15
86			min	-.062	15	-.809	15	0	1	-.201	15	-2.987	3	0	1	0	1

Envelope Member Section Stresses (Continued)

	Member	Sec		Axial[ksi]	LC	y Shear[ksi]	LC	z Shea...	LC	y-Top[ksi]	LC	y-Bot[ksi]	LC	z-Top[...	LC	z-Bot[...	LC
87		4	max	.301	5	-.194	13	0	15	1.533	3	3.081	15	0	15	0	15
88			min	.049	14	-1.429	5	0	1	-3.081	15	-1.533	3	0	1	0	1
89		5	max	.53	5	-.396	13	0	15	-.783	13	8.365	5	0	15	0	15
90			min	.049	14	-2.1	5	0	1	-8.365	5	.783	13	0	1	0	1
91	M10	1	max	-.038	14	2.435	2	0	15	8.01	15	4.155	2	0	15	0	15
92			min	-.664	5	-2.995	15	0	1	-4.155	2	-8.01	15	0	1	0	1
93		2	max	-.038	14	2.03	2	0	15	1.817	5	-.237	13	0	15	0	15
94			min	-.65	5	-2.995	15	0	1	.237	13	-1.817	5	0	1	0	1
95		3	max	-.038	14	1.798	3	0	15	9.998	5	6.067	15	0	15	0	15
96			min	-.636	5	-4.233	5	0	1	-6.067	15	-9.998	5	0	1	0	1
97		4	max	.52	15	-.514	14	0	15	1.215	3	.972	15	0	15	0	15
98			min	-.152	2	-4.436	5	0	1	-.972	15	-1.215	3	0	1	0	1
99		5	max	.534	15	-.514	14	0	15	-.859	14	10.85	5	0	15	0	15
100			min	-.152	2	-4.638	5	0	1	-10.85	5	.859	14	0	1	0	1
101	M11	1	max	-.049	13	1.099	2	0	15	9.054	15	2.981	2	0	15	0	15
102			min	-.24	5	-2.019	15	0	1	-2.981	2	-9.054	15	0	1	0	1
103		2	max	-.049	13	.629	2	0	15	3.561	15	.942	3	0	15	0	15
104			min	-.223	5	-2.019	15	0	1	-.942	3	-3.561	15	0	1	0	1
105		3	max	-.049	13	.159	2	0	15	.444	2	1.989	10	0	15	0	15
106			min	-.206	5	-2.019	15	0	1	-1.989	10	-.444	2	0	1	0	1
107		4	max	-.049	13	.038	14	0	15	.238	2	7.477	10	0	15	0	15
108			min	-.202	3	-2.166	5	0	1	-7.477	10	-.238	2	0	1	0	1
109		5	max	-.034	15	.038	14	0	15	-.012	14	13.677	5	0	15	0	15
110			min	-.202	3	-2.4	5	0	1	-13.677	5	.012	14	0	1	0	1
111	M12	1	max	1.718	3	0	14	0	15	10.593	5	0	14	0	15	0	15
112			min	0	15	-2.178	5	0	1	0	1	-10.593	5	0	1	0	1
113		2	max	1.718	3	0	14	0	15	4.892	5	0	14	0	15	0	15
114			min	0	15	-2.178	5	0	1	0	1	-4.892	5	0	1	0	1
115		3	max	1.718	3	0	14	0	15	0	14	.809	5	0	15	0	15
116			min	0	15	-2.178	5	0	1	-.809	5	0	1	0	1	0	1
117		4	max	1.718	3	0	14	0	15	0	14	6.51	5	0	15	0	15
118			min	0	15	-2.178	5	0	1	-6.51	5	0	1	0	1	0	1
119		5	max	1.718	3	0	14	0	15	0	14	12.211	5	0	15	0	15
120			min	0	15	-2.178	5	0	1	-12.211	5	0	1	0	1	0	1

Envelope Member End Reactions

	Member	Me...		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-...	LC	y-y Mome...	LC	z-z Mome...	LC
1	M1	I	max	12.915	2	4.473	5	0	15	0	15	0	15	0	15
2			min	-15.332	15	-.063	13	0	1	0	1	0	1	0	1
3		J	max	12.915	2	4.473	5	0	15	0	15	0	15	.816	13
4			min	-15.332	15	-.063	13	0	1	0	1	0	1	-57.773	5
5	M2	I	max	9.269	3	4.673	15	0	15	0	15	0	15	17.277	15
6			min	-3.949	15	-2.326	2	0	1	0	1	0	1	-6.961	2
7		J	max	9.269	3	4.673	15	0	15	0	15	0	15	18.679	3
8			min	-3.949	15	-2.326	2	0	1	0	1	0	1	-32.184	15
9	M3	I	max	2.326	2	9.27	3	0	15	0	15	0	15	18.679	3
10			min	-4.687	15	-3.949	15	0	1	0	1	0	1	-32.184	15
11		J	max	7.708	5	-2.006	13	0	15	0	15	0	15	47.219	5
12			min	.577	14	-10.784	5	0	1	0	1	0	1	4.814	13
13	M4	I	max	.213	15	4.398	2	0	15	0	15	0	15	7.095	2

Envelope Member End Reactions (Continued)

	Member	Me...		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-...	LC	y-y Mome...	LC	z-z Mome...	LC
14			min	-2.387	3	-11.383	15	0	1	0	1	0	1	-73.62	15
15		J	max	.994	15	-.213	14	0	15	0	15	0	15	62.517	5
16			min	-2.387	3	-14.759	5	0	1	0	1	0	1	2.406	14
17	M5	I	max	19.93	2	.632	3	0	15	0	15	0	15	8.162	3
18			min	-5.504	15	-7.528	15	0	1	0	1	0	1	-97.237	15
19		J	max	19.93	2	.632	3	0	15	0	15	0	15	0	15
20			min	-5.504	15	-7.528	15	0	1	0	1	0	1	0	1
21	M6	I	max	19.93	2	8.026	5	0	15	0	15	0	15	0	15
22			min	3.109	14	.089	13	0	1	0	1	0	1	0	1
23		J	max	19.93	2	8.026	5	0	15	0	15	0	15	-1.148	13
24			min	3.109	14	.089	13	0	1	0	1	0	1	-103.663	5
25	M7	I	max	23.777	5	4.379	5	0	15	0	15	0	15	0	15
26			min	2.404	14	-.134	3	0	1	0	1	0	1	0	1
27		J	max	23.777	5	4.379	5	0	15	0	15	0	15	1.736	3
28			min	2.404	14	-.134	3	0	1	0	1	0	1	-56.562	5
29	M8	I	max	10.246	5	6.284	5	0	15	0	15	0	15	21.374	5
30			min	1.931	13	.577	14	0	1	0	1	0	1	.951	14
31		J	max	10.246	5	6.284	5	0	15	0	15	0	15	-4.227	13
32			min	1.931	13	.577	14	0	1	0	1	0	1	-45.136	5
33	M9	I	max	2.326	2	10.137	3	0	15	0	15	0	15	25.506	3
34			min	-6.117	15	-3.947	15	0	1	0	1	0	1	-30.001	15
35		J	max	6.25	5	-1.931	13	0	15	0	15	0	15	45.136	5
36			min	.577	14	-10.246	5	0	1	0	1	0	1	4.227	13
37	M10	I	max	-.453	14	13.732	2	0	15	0	15	0	15	23.674	2
38			min	-7.839	5	-16.887	15	0	1	0	1	0	1	-45.644	15
39		J	max	6.304	15	-2.896	14	0	15	0	15	0	15	61.825	5
40			min	-1.798	2	-26.151	5	0	1	0	1	0	1	4.894	14
41	M11	I	max	-.581	13	6.197	2	0	15	0	15	0	15	16.987	2
42			min	-2.827	5	-11.383	15	0	1	0	1	0	1	-51.595	15
43		J	max	-.403	15	.213	14	0	15	0	15	0	15	77.937	5
44			min	-2.387	3	-13.532	5	0	1	0	1	0	1	.067	14
45	M12	I	max	20.275	3	0	14	0	15	0	15	0	15	0	14
46			min	.002	15	-12.278	5	0	1	0	1	0	1	-60.36	5
47		J	max	20.275	3	0	14	0	15	0	15	0	15	69.581	5
48			min	.002	15	-12.278	5	0	1	0	1	0	1	0	1

Envelope Beam Deflections

	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
1	M3	1	max	15.422	-.049	NC	3
2		1	min	7.547	-.072	4340	3
3	M4	1	max	2.979	-.001	NC	12
4		1	min	3.094	-.023	4402	15
5	M9	1	max	.328	-.049	NC	3
6		1	min	8.203	-.072	4340	3
7	M10	1	max	2.771	-.011	NC	12
8		1	min	0	-.004	NC	1
9		2	max	9.302	-.004	NC	10
10		2	min	4.849	-.02	NC	1
11	M11	1	max	8.021	-.001	NC	12
12		1	min	7.792	.019	4228	5

Envelope AISC 14th(360-10): ASD Steel Code Checks

	Member	Shape	Code Check	Loc[..LC	Shea...	Loc[..Dir	LC	Pnc/o...	Pnt/om...	Mnyy/o...	Mnzz/o...	Cb	Eqn
1	M1	W18X40	.310	12.9.. 15	.040	0 y	5	119.495	353.293	24.95	195.609	1.667	H1-1b
2	M2	W18X40	.170	10.5.. 15	.041	0 y	15	169.862	353.293	24.95	195.609	2.181	H1-1b
3	M3	W16X40	.291	15.75 5	.110	15.75 y	5	121.606	353.293	31.687	182.136	2.22	H1-1b
4	M4	W18X40	.377	0 15	.131	11 y	5	160.663	353.293	24.95	195.609	2.213	H1-1b
5	M5	W18X46	.436	0 15	.058	0 y	15	140.766	404.192	29.192	226.297	1.667	H1-1b
6	M6	W18X46	.522	12.9... 5	.062	0 y	5	140.766	404.192	29.192	226.297	1.667	H1-1b
7	M7	W18X40	.389	12.9... 5	.039	0 y	5	119.495	353.293	24.95	195.609	1.667	H1-1b
8	M8	W18X40	.261	10.5... 5	.056	0 y	5	169.862	353.293	24.95	195.609	2.169	H1-1b
9	M9	W16X40	.274	15.75 5	.105	15.75 y	5	121.606	353.293	31.687	182.136	2.357	H1-1b
10	M10	W18X40	.329	9.5 5	.232	9.5 y	5	193.444	353.293	24.95	195.609	1.854	H1-1b
11	M11	W18X40	.401	11 5	.120	11 y	5	160.663	353.293	24.95	195.609	2.333	H1-1b
12	M12	W18X40	.396	10.5... 5	.109	0 y	5	169.862	353.293	24.95	195.609	2.246	H1-1b

Envelope Joint Reactions (ASD COMBOS)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	-2.379	6	1.362	6	0	8	0	8	0	8	0	8
2		min	-3.162	8	-6.145	8	0	6	0	6	0	6	0	6
3	N6	max	-3.495	6	11.197	6	0	8	0	8	0	8	0	8
4		min	-5.025	8	2.805	8	0	6	0	6	0	6	0	6
5	N7	max	-4.408	6	17.029	6	0	8	0	8	0	8	0	8
6		min	-5.515	8	10.546	8	0	6	0	6	0	6	0	6
7	N9	max	-2.204	6	17.629	6	0	8	0	8	0	8	0	8
8		min	-2.946	8	15.429	8	0	6	0	6	0	6	0	6
9	Totals:	max	-12.486	6	47.217	6	0	8						
10		min	-16.648	8	22.635	8	0	6						

Envelope Joint Reactions (OMEGA COMBOS)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	-11.632	11	-30.379	7	0	11	0	11	0	11	0	11
2		min	-11.665	7	-35.577	11	0	7	0	7	0	7	0	7
3	N6	max	-18.663	7	-1.511	7	0	11	0	11	0	11	0	11
4		min	-18.905	11	-9.656	11	0	7	0	7	0	7	0	7
5	N7	max	-18.736	11	26.23	7	0	11	0	11	0	11	0	11
6		min	-18.971	7	17.943	11	0	7	0	7	0	7	0	7
7	N9	max	-10.158	7	46.98	7	0	11	0	11	0	11	0	11
8		min	-10.183	11	41.321	11	0	7	0	7	0	7	0	7
9	Totals:	max	-59.456	11	41.321	7	0	11						
10		min	-59.456	7	14.031	11	0	7						

Company : DEI
Designer : RH/NDM
Job Number : 17-291
Model Name : Moment Frame 2

Feb 4, 2020
2:40 PM
Checked By:_____

Envelope Joint Reactions (ASD COMBOS)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	-2.379	6	1.362	6	0	8	0	8	0	8	0	8
2		min	-3.162	8	-6.145	8	0	6	0	6	0	6	0	6
3	N6	max	-3.495	6	11.197	6	0	8	0	8	0	8	0	8
4		min	-5.025	8	2.805	8	0	6	0	6	0	6	0	6
5	N7	max	-4.408	6	17.029	6	0	8	0	8	0	8	0	8
6		min	-5.515	8	10.546	8	0	6	0	6	0	6	0	6
7	N9	max	-2.204	6	17.629	6	0	8	0	8	0	8	0	8
8		min	-2.946	8	15.429	8	0	6	0	6	0	6	0	6
9	Totals:	max	-12.486	6	47.217	6	0	8						
10		min	-16.648	8	22.635	8	0	6						

Company	:	DEI	Feb 4, 2020
Designer	:	RH/NDM	2:40 PM
Job Number	:	17-291	Checked By:_____
Model Name	:	Moment Frame 2	

Envelope Joint Reactions (OMEGA COMBOS)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	-11.632	11	-30.379	7	0	11	0	11	0	11	0	11
2		min	-11.665	7	-35.577	11	0	7	0	7	0	7	0	7
3	N6	max	-18.663	7	-1.511	7	0	11	0	11	0	11	0	11
4		min	-18.905	11	-9.656	11	0	7	0	7	0	7	0	7
5	N7	max	-18.736	11	26.23	7	0	11	0	11	0	11	0	11
6		min	-18.971	7	17.943	11	0	7	0	7	0	7	0	7
7	N9	max	-10.158	7	46.98	7	0	11	0	11	0	11	0	11
8		min	-10.183	11	41.321	11	0	7	0	7	0	7	0	7
9	Totals:	max	-59.456	11	41.321	7	0	11						
10		min	-59.456	7	14.031	11	0	7						

Special Moment Frame Calculations: 2nd Level -Connection at Interior column to beam

Job Name:

Moment Frame (Single-Bay)

Moment Frame #1

Steel Grade ASTM A992; A572 Gr 50 or Gr 55

Properties:	Beam		Column
	W16 × 40		W18 × 40
bf	7.00		6.02
tf	0.51		0.53
tw	0.31		0.32
d	16.01		17.90
A	11.80		11.80
Aw	4.74		5.48
bf/2tf	6.93		5.73
h/tw	46.50		50.90
S	64.70		68.40
ry	1.56		1.27
Z_x	73.00		78.40
Fy	50.00		50.00
Fu	65.00		65.00
Ry	1.10		1.10
E	29,000.00	ksi	29,000.00
Lb	0.00		10.50

Column Req's
1. Columns shall be any of the rolled shapes or built-up sections permitted in 2.3 of 358-10
2. Beam shall connect to the flange of the column
3. Rolled shape limited to W36.Boxed WF columns shall not have a width or depth exceeding 24" if participating in orthogonal moment frames
4. No limit on wt/ft
5. No additional flange thickness req's
6. width-thickness ratios for flanges & webs of columns shall conform to limits of section D.1 for highly ductile members of AISC 341-10 (Table D1.1)
7. Lateral bracing shall conform to Section 4c of AISC 341-10 or SMF as applicable in Seis Prov

Beam Req's
1. Shall be hot-rolled or built-up I-shaped members comforming to Section 2.3 of 358-10
2. Depth limited to W36
3. Weight Limit of beam is 300plf
4. Flange thickness limited to 1.75"
5. Clear span-to-span depth ratio is 7+
6. Width-to-thickness ratio for flanges & web shall conform to Seis Des Param (value of bf not less than flange width at ends of center 2/3 portion of RBS, etc)
7. Lateral bracing:
   Conform to Section 4b of AISC 341-10
   Locate RBS bracing no more than d/2 beyond the end of the RBS farthest from the face of the column (ie: inside the column)

Deflection Check:

Code Deflection Limits - Special Moment Frame per ASCE7-10

1st Floor	0.82	in	First Floor Allowablw Deflection
2nd Floor	1.45	in	Total Deflection at Second Floor

Actual Analysis (2nd Order), (+10% RBS) per RISA (with Strength Design Combos)
Displacment per RISA

1st Floor	0.714	*1.1	0.7854	in	OK	First Floor Deflection
2nd Floor	0.987	*1.1	1.0857	in	OK	Total deflection at Second Floor

Member Parameters Check: AISC 358-10 Section 5.3.1 (AISC 341-10 Section D1b)

Beams:

bf/2tf:	6.93	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Beam meets allowable Criteria
h/tw:	46.50	2.45(E/Fy)^0.5:	59.00381	OK	Check if the Beam meets allowable Criteria
h/tw:	46.50				

Columns:

C_a : 0.0356 < 0.125 $C_a = P_u/\phi P_y$:

h/tw:	50.90	2.45(E/Fy)^0.5*(1-0.93Ca):	57.05069	OK	Check if the Column meets allowable Criteria
b/2tf:	5.73	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Column meets allowable Criteria
Puc	21.00	RISA Axial Column Load (Strength Combo)			

kl/r= 118.84 4.71((E/Fy)^2: 113.432
Fe: 6.447 ksi
Fcr: 5.654 ksi 0.31

Pn= 66.72

Pu/ϕPn= 0.31 OK

Beam Bracing: **6.51**
Lb 0.00 ft OK - No Beam Bracing Reqd

$$L_{b,MAX} = 0.086 r_y E / F_y$$

Member Parameters Check: AISC 358-10 Section 5.4.2 & AISC 341-10 Section E3.4a

Beam/Column Ratio:

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Column(s): (SETUP FOR SINGLE COLUMN BELOW CONDITION)

$$\sum Z_C (F_{YC} - (P_{UC} / A_g)$$

ΣMpc*				sum the moment capacities of the columns coming (which is reduced by the stress from axial loading)
	Zc	78.40		Standard Z-column Value
	Fyc	50.00		Yield Stress
	Puc	21.00	RISA Axial Column Load	
	Ag	11.80		Total Area of Steel in Column
ΣMpc*	3780.47458 k-in			Sum of Available Moment Capacities in Column

$$\boxed{\sum M_{PC} = Z_{C,mx} (F_{YC,mx} - (P_{UC,mx} / A_{g,mx}))}$$

Beam(s): WORKS FOR ONLY 1-BEAM CONDITION (NOTE THE SUM SIGN & TAILOR)

$$\sum M^*_{PB_C} = \sum (1.1 R_Y F_{YB} Z_{RBS} + M_{UV})$$

M_{PB}*									
R_Y		1.10		Constant Value; Ref: AISC Seis Des Pro Table A3.1					
F_{YB}		50.00		Yield Stress					
Z_{RBS}				Z at RBS					
	a)	3.50	(0.5*bf)= 3.4975	(see diagram)	a= (0.5 to 0.75)bf	3.4975	to	5.24625	(Typ. Detail 5602x)
	b)	12.00	(0.75*d)= 12.0075	(see diagram)	b= (0.65 to 0.85)d	10.4065	to	13.6085	
	c)	1.50	(0.2*bf)= 1.399	(see diagram)	c=(0.1b_{bf} to 0.25b_{bf})	0.6995	to	1.74875	
	Z_{RBS}	49.51 in^3							

$$Z_{RBS} = Z_X - (2ct_{bf}(d - t_{bf}))$$

M_{UV}			R= 12.75	
	V_{RBS}	12.88 kip		
	V_{MAX}	11 kip	RISA Max Beam Shear (Strength Combo)	
	L_{RBS}	14.166667		Length of the member between the RBS
	M_{UV}	276.96 k-in		

$$M_{UV} = V_{RBS}\left(a + \frac{b}{2} + \frac{dc}{2}\right)$$

$$V_{RBS} = V_{MAX}\left[\frac{\frac{L}{2} - a + \frac{b}{2} + \frac{dc}{2}}{\frac{L}{2}}\right]$$

M_{PB}* 3272.31 k-in

Check The Ratios:

		Mpc/Mpb > 1, O.K.
M_{PC}/M_{PB}	1.16	Column Bracing Required

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Max Probable Moment — AISC 358-10 — AISC 358-10 Eq:5.8-5

$$M_{PR} = C_{PR} R_Y F_Y Z_e$$

M_{PR}

Fy	50.00		
Ry	1.10		Ze - from 358-10 Eq 5.8-4 (Zrbs)
Ze	49.51		AISC SDP Table I-6-1
C_{PR}	1.15		AISC Black Book 5-3
M_{PR}	**3131.50**	k-in	

Max $V_{P,RBS}$

$$V_{P@RBS} = \frac{2M_{PR}}{L'} + \frac{wL'}{2}$$

V_P — This is the maximum shear at the area of the RBS (will be used later to get an eccentric force caused by the RBS at the Column Face)

L' — Beam Length Between RBS(VERIFY)

L_0	15.75	ft	Beam Length
w_{DL} =	0.456	k/ft	
w_{LL} =	0.25	k/ft	
w_{SL} =	0.35	k/ft	
w	0.7422	k/ft	RISA Factored Dist Load (MAX) 1.2DL+0.5LL+0.2SL
L'	170.00		
V_P	**42.10**	k	Max Shear $L' = L_0 - (a - \frac{b}{2})$

Max $M_{COL\ FACE}$

Maximum moment seen at face of column from the eccentric force at the RBS and the plastic moment at the column

M_F

$$M_F = M_{PR} + V_{P,RBS} S_h$$

$$S_h = a + \frac{b}{2}$$

S_h	9.5	
M_F	**3531.44**	k-in

Max Plastic Moment

$$M_{pe} = Z_b R_y F_y$$

M_{PE}

φ_d	1			
Zb	73.00			Z-beam (Full value)
Ry	1.10			Constant
Fy	50.00			Yield Stress
φMpe	**4015.00**	k-in	OK	$\varphi M_{pe} \geq M_F ?$

Shear Check — AISC Setion "G"

$$V_n = 0.6 * F_y A_{web} C_v$$

$V_U = V_P$ =	42.10			Actual Max shear in the beam
Vn				Allowable shear in the beam
Check:	53.95		OK	$h/t_w \leq 2.24\sqrt{\frac{29000ksi}{50ksi}}$
Fy	50.00			
Aw	4.74			
Cv	1			if "OK", then C_v & φ = 1.0
Vn	**142.05**	k	OK	$V_n \geq V_U ?$

Continuity Plate Requirements   AISC 358-10 Sec. 2.4 & AISC 341-10 Section E3.6f

Check:

MIN t_{cf}

bbf	7.00			
tbf	0.51			
Fyb	50.00			
Ryb	1.10			
Fyc	50.00			
Ryc	1.10			
MIN t_{cf}	**1.01**	in	bbf/6=	1.17

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

t_{cf} 0.53 in N.G. - Continuity Plate Reqd

k =	1.54
k_1 =	1.125
$pF_{(min)}$ =	3.04
$pC_{(max)}$ =	1.625

tbf/2= 0.2525

Minimum Continuity Plate Thickness req'd: 0.64 in

Minimum Continuity Plate Width req'd: 3.345 in bst=(bbf-tbcw)/2

Column Panel Zones   AISC 341-10 Secion E3.6e(2)

$$t_{min} = (d_2 + w_2)/90$$

tmin

d2	16.62	in	Distance between continuity plates (depth - 2*continuity plate thickness)
w2	16.85	in	distance between column flanges (depth - 2*flange thickness)
tmin	0.37	in	

t_{cw} 0.32 in N.G. Web Stiffener Required (Doubler Plate)

Min doubler plate Thickness Required = 0.057

Web Panel Shear   Per AISC 341-10 Section E3.6e(1) & AISC J10.6

$$V_U = \frac{\sum M_C}{(d - t_{bf})}$$

Vu

φ	1			LRFD for tension
ΣMc	61	k-ft	RISA Value	Sum of the moments in the columns about the joint (what the beams are resisting - all should add to zero)
	732	k-in		k-in verses k'
d	16.01	in		
tbf	0.51	in		
Vu	**47.21**	k		Design force in the web

Pr	21.00		AXIAL force in Column (strength Combos)	
Pc	274.2125			=Py(LRFD) Calculated as FyA
Fy	50.00	ksi		
Aw	5.48	in2		Area of the web taking the panel shear forces from the beam moment
Pc	274.2125	k		Allowable Shear Force in the Web
φ	0.75			Shear factor LRFD
φPc	**205.66**	k	OK (Web Stiffener NOT Required)	Check Actual shear values in beam verses allowable shear

Pr < 0.4Pc

If Pr < 0.4Pc: $R_n = 0.6F_y d_c t_w$

If Pr > 0.4Pc: Rn = 0.6Fydctw * (1.4 - Pr/Pc)

Rn

Fy	50.00	ksi		
tcf	0.53	in		
bcf	6.02	in		
db	17.90	in		
dc	17.90	in		
tw	0.32	in		
Rn	169.16			
φRn	**169.16**		OK (Web Stiffener NOT Required)	Allowable force in the web

$$V_U \geq \varphi R$$

Special Moment Frame Calculations: 2nd Level- Connection at exterior column to beam

Job Name:

Moment Frame (Single-Bay)

Moment Frame #1

Steel Grade ASTM A992; A572 Gr 50 or Gr 55

Properties:	Beam		Column
	W16 × 40		W18 × 40
bf	7.00		6.02
tf	0.51		0.53
tw	0.31		0.32
d	16.01		17.90
A	11.80		11.80
Aw	4.74		5.48
bf/2tf	6.93		5.73
h/tw	46.50		50.90
S	64.70		68.40
ry	1.56		1.27
Z_x	73.00		78.40
Fy	50.00		50.00
Fu	65.00		65.00
Ry	1.10		1.10
E	29,000.00	ksi	29,000.00
Lb	0.00		10.50

Column Req's

1. Columns shall be any of the rolled shapes or built-up sections permitted in 2.3 of 358-10
2. Beam shall connect to the flange of the column
3. Rolled shape limited to W36.Boxed WF columns shall not have a width or depth exceeding 24" if participating in orthogonal moment frames
4. No limit on wt/ft
5. No additional flange thickness req's
6. width-thickness ratios for flanges & webs of columns shall conform to limits of section D.1 for highly ductile members of AISC 341-10 (Table D1.1)
7. Lateral bracing shall conform to Section 4c of AISC 341-10 or SMF as applicable in Seis Prov

Beam Req's

1. Shall be hot-rolled or built-up I-shaped members comforming to Section 2.3 of 358-10
2. Depth limited to W36
3. Weight Limit of beam is 300plf
4. Flange thickness limited to 1.75"
5. Clear span-to-span depth ratio is 7+
6. Width-to-thickness ratio for flanges & web shall conform to Seis Des Param (value of bf not less than flange width at ends of center 2/3 portion of RBS, etc)
7. Lateral bracing:
   - Conform to Section 4b of AISC 341-10
   - Locate RBS bracing no more than d/2 beyond the end of the RBS farthest from the face of the column (ie: inside the column)

Deflection Check:

Code Deflection Limits - Special Moment Frame per ASCE7-10

1st Floor	0.82	in	First Floor Allowablw Deflection
2nd Floor	1.45	in	Total Deflection at Second Floor

Actual Analysis (2nd Order), (+10% RBS) per RISA (with Strength Design Combos)

Displacment per RISA

1st Floor	0.714	*1.1	0.7854	in	OK	First Floor Deflection
2nd Floor	0.987	*1.1	1.0857	in	OK	Total deflection at Second Floor

Member Parameters Check: AISC 358-10 Section 5.3.1 (AISC 341-10 Section D1b)

Beams:

bf/2tf:	6.93	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Beam meets allowable Criteria
h/tw:	46.50	2.45(E/Fy)^0.5:	59.00381	OK	Check if the Beam meets allowable Criteria
h/tw:	46.50				

Columns:

C_a : 0.0186 < 0.125 $C_a = P_u/\phi P_y$:

h/tw:	50.90	2.45(E/Fy)^0.5*(1-0.93Ca):	57.98075	OK	Check if the Column meets allowable Criteria
b/2tf:	5.73	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Column meets allowable Criteria
Puc	11.00	RISA Axial Column Load (Strength Combo)			

kl/r=	118.84	4.71((E/Fy)^2:	113.432	
		Fe:	6.447	ksi
		Fcr:	5.654	ksi
Pn=	66.72			

0.16

Pu/φPn= 0.16 OK

Beam Bracing: **6.51**

Lb 0.00 ft OK - No Beam Bracing Reqd

$$\boxed{L_{b,MAX} = 0.086 r_y E / F_y}$$

Member Parameters Check: AISC 358-10 Section 5.4.2 & AISC 341-10 Section E3.4a

Beam/Column Ratio:

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Column(s): (SETUP FOR SINGLE COLUMN BELOW CONDITION)

$$\sum Z_C (F_{YC} - (P_{UC} / A_g)$$

ΣM_{pc}^*			sum the moment capacities of the columns coming (which is reduced by the stress from axial loading)
Zc	78.40		Standard Z-column Value
Fyc	50.00		Yield Stress
Puc	11.00	RISA Axial Column Load	
Ag	11.80		Total Area of Steel in Column
ΣM_{pc}^*	**3846.91525** k-in		Sum of Available Moment Capacities in Column

$$\boxed{\sum M_{PC} = Z_{C,mx} (F_{YC,mx} - (P_{UC,mx} / A_{g,mx}))}$$

Beam(s): WORKS FOR ONLY 1-BEAM CONDITION (NOTE THE SUM SIGN & TAILOR)

$$\sum M^*_{PB_C} = \sum (1.1 R_Y F_{YB} Z_{RBS} + M_{UV})$$

M_{PB}^*

R_Y	1.10			Constant Value; Ref: AISC Seis Des Pro Table A3.1				
F_{YB}	50.00			Yield Stress				
Z_{RBS}				Z at RBS				
a)	3.50	(0.5*bf)=	3.4975	(see diagram)	a= (0.5 to 0.75)bf	3.4975	to	5.24625
b)	12.00	(0.75*d)=	12.0075	(see diagram)	b= (0.65 to 0.85)d	10.4065	to	13.6085
c)	1.50	(0.2*bf)=	1.399	(see diagram)	c=(0.1b_{bf} to 0.25b_{bf})	0.6995	to	1.74875
Z_{RBS}	**49.51** in^3							

$$Z_{RBS} = Z_X - (2ct_{bf}(d - t_{bf}))$$

M_{UV} R= 12.75

V_{RBS}	**12.88** kip		
V_{MAX}	11 kip	RISA Max Beam Shear (Strength Combo)	
L_{RBS}	14.166667		Length of the member between the RBS
M_{UV}	**276.96** k-in		

$$M_{UV} = V_{RBS}\left(a + \frac{b}{2} + \frac{dc}{2}\right)$$

$$V_{RBS} = V_{MAX}\left[\frac{\frac{L}{2} - a + \frac{b}{2} + \frac{dc}{2}}{\frac{L}{2}}\right]$$

M_{PB}^* **3272.31** k-in

Check The Ratios:

		Mpc/Mpb > 1, O.K.
M_{PC}/M_{PB}	**1.18**	Column Bracing Required

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Max Probable Moment — AISC 358-10

AISC 358-10 Eq:5.8-5

$$M_{PR} = C_{PR} R_Y F_Y Z_e$$

M_{PR}

Fy	50.00		
Ry	1.10		Ze - from 358-10 Eq 5.8-4 (Zrbs) / AISC SDP Table I-6-1
Ze	49.51		
C_{PR}	1.15		AISC Black Book 5-3
M_{PR}	**3131.50**	k-in	

Max $V_{P,RBS}$

$$V_{P@RBS} = \frac{2M_{PR}}{L'} + \frac{wL'}{2}$$

V_P — This is the maximum shear at the area of the RBS (will be used later to get an eccentric force caused by the RBS at the Column Face)

L' — Beam Length Between RBS(VERIFY)

L_0	15.75	ft	Beam Length
w_{DL} =	0.456	k/ft	
w_{LL} =	0.25	k/ft	
w_{SL} =	0.35	k/ft	
w	0.7422	k/ft	RISA Factored Dist Load (MAX) 1.2DL+0.5LL+0.2SL
L'	170.00		
V_P	**42.10**	k	Max Shear

$$L' = L_0 - (a - \frac{b}{2})$$

Max $M_{COL\ FACE}$

Maximum moment seen at face of column from the eccentric force at the RBS and the plastic moment at the column

M_F

$$M_F = M_{PR} + V_{P,RBS} S_h$$

$$S_h = a + \frac{b}{2}$$

S_h	9.5	
M_F	**3531.44**	k-in

Max Plastic Moment

$$M_{pe} = Z_b R_y F_y$$

M_{PE}

φ_d	1			
Zb	73.00			Z-beam (Full value)
Ry	1.10			Constant
Fy	50.00			Yield Stress
φMpe	**4015.00**	k-in	OK	

$$\varphi M_{pe} \geq M_F?$$

Shear Check — AISC Setion "G"

$$V_n = 0.6 * F_y A_{web} C_v$$

$V_U = V_P$ =	42.10			Actual Max shear in the beam
Vn				Allowable shear in the beam
Check:	53.95		OK	
Fy	50.00			
Aw	4.74			
Cv	1			if "OK", then C_v & φ = 1.0
Vn	**142.05**	k	OK	

$$h/t_w \leq 2.24\sqrt{\frac{29000ksi}{50ksi}}$$

$$V_n \geq V_U ?$$

Continuity Plate Requirements   AISC 358-10 Sec. 2.4 & AISC 341-10 Section E3.6f

Check:

MIN t_{cf}

bbf	7.00			
tbf	0.51			
Fyb	50.00			
Ryb	1.10			
Fyc	50.00			
Ryc	1.10			
MIN t_{cf}	**1.01**	in	bbf/6=	1.17
t_{cf}	0.53	in	N.G. - Continuity Plate Reqd	

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

k =	1.54
k_1 =	1.125
$pF_{(min)}$ =	3.04
$pC_{(max)}$ =	1.625

tbf/2= 0.2525

Minimum Continuity Plate Thickness req'd: 0.64 in

Minimum Continuity Plate Width req'd: 3.345 in   bst=(bbf-tbcw)/2

Column Panel Zones   AISC 341-10 Secion E3.6e(2)

$$t_{min} = (d_2 + w_2) / 90$$

tmin

d2	16.62	in	Distance between continuity plates (depth - 2*continuity plate thickness)
w2	16.85	in	distance between column flanges (depth - 2*flange thickness)
tmin	0.37	in	
t_{cw}	0.32	in	N.G. Web Stiffener Required (Doubler Plate)

Min doubler plate Thickness Required = 0.057

Web Panel Shear   Per AISC 341-10 Section E3.6e(1) & AISC J10.6

$$V_U = \frac{\sum M_C}{(d - t_{bf})}$$

Vu

φ	1			LRFD for tension
ΣMc	45	k-ft	RISA Value	Sum of the moments in the columns about the joint (what the beams are resisting - all should add to zero)
	540	k-in		k-in verses k'
d	16.01	in		
tbf	0.51	in		
Vu	**34.83**	k		Design force in the web
Pr	11.00		AXIAL force in Column (strength Combos)	
Pc	274.2125			=Py(LRFD) Calculated as FyA

Fy	50.00	ksi	
Aw	5.48	in2	Area of the web taking the panel shear forces from the beam moment
Pc	274.2125	k	Allowable Shear Force in the Web

φ	0.75			Shear factor LRFD
φPc	**205.66**	k	OK (Web Stiffener NOT Required)	Check Actual shear values in beam verses allowable shear

Pr < 0.4Pc

If Pr < 0.4Pc: Rn = $0.6F_y d_c t_w$

If Pr > 0.4Pc: Rn = 0.6Fydctw * (1.4 - Pr/Pc)

Rn

Fy	50.00	ksi		
tcf	0.53	in		
bcf	6.02	in		
db	17.90	in		
dc	17.90	in		
tw	0.32	in		
Rn	169.16			
φRn	**169.16**		OK (Web Stiffener NOT Required)	Allowable force in the web

$$V_u \geq \varphi R$$

Special Moment Frame Calculations: 1st Level -Connection at Interior column to beam

Job Name:

Moment Frame (Single-Bay)

Moment Frame #1

Steel Grade ASTM A992; A572 Gr 50 or Gr 55

Properties:	Beam	Column
	W18 × 40	W18 × 46
bf	6.02	6.06
tf	0.53	0.61
tw	0.32	0.36
d	17.90	18.06
A	11.80	13.50
Aw	5.48	6.17
bf/2tf	5.73	5.01
h/tw	50.90	44.60
S	68.40	78.80
ry	1.27	1.29
Z_x	78.40	90.70
Fy	50.00	50.00
Fu	65.00	65.00
Ry	1.10	1.10
E	29,000.00 ksi	29,000.00
Lb	0.00	13.67

Column Req's

1 Columns shall be any of the rolled shapes or built-up sections permitted in 2.3 of 358-10
2 Beam shall connect to the flange of the column
3 Rolled shape limited to W36.Boxed WF columns shall not have a width or depth exceeding 24" if participating in orthogonal moment frames
4 No limit on wt/ft
5 No additional flange thickness req's
6 width-thickness ratios for flanges & webs of columns shall conform to limits of section D.1 for highly ductile members of AISC 341-10 (Table D1.1)
7 Lateral bracing shall conform to Section 4c of AISC 341-10 or SMF as applicable in Seis Prov

Beam Req's

1 Shall be hot-rolled or built-up I-shaped members comforming to Section 2.3 of 358-10
2 Depth limited to W36
3 Weight Limit of beam is 300plf
4 Flange thickness limited to 1.75"
5 Clear span-to-span depth ratio is 7+
6 Width-to-thickness ratio for flanges & web shall conform to Seis Des Param (value of bf not less than flange width at ends of center 2/3 portion of RBS, etc)
7 Lateral bracing:
Conform to Section 4b of AISC 341-10
Locate RBS bracing no more than d/2 beyond the end of the RBS farthest from the face of the column (ie: inside the column)

Deflection Check:

Code Deflection Limits - Special Moment Frame per ASCE7-10

1st Floor	0.82	in	First Floor Allowablw Deflection
2nd Floor	1.45	in	Total Deflection at Second Floor

Actual Analysis (2nd Order), (+10% RBS) per RISA (with Strength Design Combos)

Displacment per RISA

1st Floor	0.714	*1.1	0.7854	in	OK	First Floor Deflection
2nd Floor	0.987	*1.1	1.0857	in	OK	Total deflection at Second Floor

Member Parameters Check: AISC 358-10 Section 5.3.1 (AISC 341-10 Section D1b)

Beams:

bf/2tf:	5.73	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Beam meets allowable Criteria
h/tw:	50.90	2.45(E/Fy)^0.5:	59.00381	OK	Check if the Beam meets allowable Criteria
h/tw:	50.90				

Columns:

C_a : 0.0296 < 0.125 $C_a = P_u/\phi P_y$:

h/tw:	44.60	2.45(E/Fy)^0.5*(1-0.93Ca):	57.37793	OK	Check if the Column meets allowable Criteria
b/2tf:	5.01	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Column meets allowable Criteria
Puc	20.00	RISA Axial Column Load (Strength Combo)			

kl/r= 152.48 4.71((E/Fy)^2: 113.432

Fe: 3.917 ksi

Fcr: 3.435 ksi 0.43

Pn= 46.37

$Pu/\phi Pn=$ 0.43 OK

Beam Bracing: **5.29**

Lb 0.00 ft OK - No Beam Bracing Reqd

$$\boxed{L_{b,MAX} = 0.086 r_y E / F_y}$$

Member Parameters Check: AISC 358-10 Section 5.4.2 & AISC 341-10 Section E3.4a

Beam/Column Ratio:

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Column(s): (SETUP FOR SINGLE COLUMN BELOW CONDITION)

$$\sum Z_C (F_{YC} - (P_{UC} / A_g)$$

ΣMpc*				sum the moment capacities of the columns coming (which is reduced by the stress from axial loading)
Zc	90.70			Standard Z-column Value
Fyc	50.00			Yield Stress
Puc	20.00		RISA Axial Column Load	
Ag	13.50			Total Area of Steel in Column
ΣMpc*	**4400.62963**	k-in		Sum of Available Moment Capacities in Column

$$\boxed{\sum M_{PC} = Z_{C,mx} (F_{YC,mx} - (P_{UC,mx} / A_{g,mx}))}$$

Beam(s): WORKS FOR ONLY 1-BEAM CONDITION (NOTE THE SUM SIGN & TAILOR)

$$\sum M^*_{PB_C} = \sum (1.1 R_Y F_{YB} Z_{RBS} + M_{UV})$$

M_{PB}*									
R_Y	1.10			Constant Value; Ref: AISC Seis Des Pro Table A3.1					
F_{YB}	50.00			Yield Stress					
Z_{RBS}				Z at RBS					
a)	3.50	(0.5*bf)=	3.0075	(see diagram)	a= (0.5 to 0.75)bf	3.0075	to	4.51125	(Typ. Detail 5602x)
b)	12.00	(0.75*d)=	13.425	(see diagram)	b= (0.65 to 0.85)d	11.635	to	15.215	
c)	1.50	(0.2*bf)=	1.203	(see diagram)	c=(0.1b_{bf} to 0.25b_{bf})	0.6015	to	1.50375	
Z_{RBS}	**51.03** in^3								
M_{UV}		R=	**12.75**						
V_{RBS}	**26.71** kip								
V_{MAX}	20 kip	RISA Max Beam Shear (Strength Combo)							
L_{RBS}	7.9166667			Length of the member between the RBS					
M_{UV}	**612.22** k-in								
M_{PB}*	**3699.80** k-in								

$$Z_{RBS} = Z_X - (2ct_{bf}(d - t_{bf}))$$

$$M_{UV} = V_{RBS}(a + \frac{b}{2} + \frac{dc}{2})$$

$$V_{RBS} = V_{MAX}\left[\frac{\frac{L}{2} - a + \frac{b}{2} + \frac{dc}{2}}{\frac{L}{2}}\right]$$

Check The Ratios:

		Mpc/Mpb > 1, O.K.
M_{PC}/M_{PB}	**1.19**	Column Bracing Required

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Max Probable Moment — AISC 358-10, AISC 358-10 Eq:5.8-5

$$M_{PR} = C_{PR} R_Y F_Y Z_e$$

M_{PR}				
Fy	50.00			
Ry	1.10			Ze - from 358-10 Eq 5.8-4 (Zrbs)
Ze	51.03			AISC SDP Table I-6-1
C_{PR}	1.15			AISC Black Book 5-3
M_{PR}	**3227.92**	k-in		

Max $V_{P,RBS}$

$$V_{P@RBS} = \frac{2M_{PR}}{L'} + \frac{wL'}{2}$$

V_P				This is the maximum shear at the area of the RBS (will be used later to get an eccentric force caused by the RBS at the Column Face)
L'				Beam Length Between RBS(VERIFY)
L_0	9.5	ft		Beam Length
w_{DL} =	0.176	k/ft		
w_{LL} =	0.47	k/ft		
w_{SL} =	0	k/ft		
w	0.4462	k/ft	RISA Factored Dist Load (MAX) 1.2DL+0.5LL+0.2SL	
L'	95.00			
V_P	**69.72**	k		Max Shear $L' = L_0 - (a - \frac{b}{2})$

Max $M_{COL\ FACE}$

Maximum moment seen at face of column from the eccentric force at the RBS and the plastic moment at the column

$$M_F = M_{PR} + V_{P,RBS} S_h$$

$$S_h = a + \frac{b}{2}$$

M_F		
S_h	9.5	
M_F	**3890.29**	k-in

Max Plastic Moment

$$M_{pe} = Z_b R_y F_y$$

M_{PE}				
φ_d	1			
Zb	78.40			Z-beam (Full value)
Ry	1.10			Constant
Fy	50.00			Yield Stress
φMpe	**4312.00**	k-in	OK	$\varphi M_{pe} \geq M_F$?

Shear Check — AISC Setion "G"

$$V_n = 0.6 * F_y A_{web} C_v$$

$V_U = V_P$ =	69.72			Actual Max shear in the beam
Vn				Allowable shear in the beam
Check:	53.95		OK	$h/t_w \leq 2.24\sqrt{\frac{29000ksi}{50ksi}}$
Fy	50.00			
Aw	5.48			
Cv	1			if "OK", then C_v & φ = 1.0
Vn	**164.53**	k	OK	$V_n \geq V_U$?

Continuity Plate Requirements — AISC 358-10 Sec. 2.4 & AISC 341-10 Section E3.6f

Check:

MIN t_{cf}

bbf	6.02			
tbf	0.53			
Fyb	50.00			
Ryb	1.10			
Fyc	50.00			
Ryc	1.10			
MIN t_{cf}	**0.95**	in	bbf/6=	1.00
t_{cf}	0.61	in	N.G. - Continuity Plate Reqd	

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

tbf/2= 0.2625

k = 1.54
k_1 = 1.125
$pF_{(min)}$ = 3.04
$pC_{(max)}$ = 1.625

Minimum Continuity Plate Thickness req'd: 0.40 in

Minimum Continuity Plate Width req'd: 2.85 in — bst=(bbf-tbcw)/2

Column Panel Zones — AISC 341-10 Secion E3.6e(2)

$$t_{min} = (d_2 + w_2)/90$$

tmin

d2	17.27	in	Distance between continuity plates (depth - 2*continuity plate thickness)
w2	16.85	in	distance between column flanges (depth - 2*flange thickness)
tmin	0.38	in	
t_{cw}	0.36	in	N.G. Web Stiffener Required (Doubler Plate)

Min doubler plate Thickness Required = 0.019

Web Panel Shear — Per AISC 341-10 Section E3.6e(1) & AISC J10.6

$$V_U = \frac{\sum M_C}{(d - t_{bf})}$$

Vu

φ	1			LRFD for tension
ΣMc	104	k-ft	RISA Value	Sum of the moments in the columns about the joint (what the beams are resisting - all should add to zero)
	1248	k-in		k-in verses k'
d	17.90	in		
tbf	0.53	in		
Vu	**71.83**	k		Design force in the web
Pr	20.00		AXIAL force in Column (strength Combos)	
Pc	308.37			=Py(LRFD) Calculated as FyA
Fy	50.00	ksi		
Aw	6.17	in2		Area of the web taking the panel shear forces from the beam moment
Pc	308.37	k		Allowable Shear Force in the Web
φ	0.75			Shear factor LRFD
φPc	**231.28**	k	OK (Web Stiffener NOT Required)	Check Actual shear values in beam verses allowable shear

Pr < 0.4Pc

If Pr < 0.4Pc: Rn = $0.6F_y d_c t_w$

If Pr > 0.4Pc: Rn = 0.6Fydctw * (1.4 - Pr/Pc)

Rn

Fy	50.00	ksi		
tcf	0.61	in		
bcf	6.06	in		
db	18.06	in		
dc	18.06	in		
tw	0.36	in		
Rn	195.05			
φRn	**195.05**		OK (Web Stiffener NOT Required)	Allowable force in the web

$$V_u \geq \varphi R$$

Special Moment Frame Calculations: 1st Level- Connection at exterior column to beam

Job Name:

Moment Frame (Single-Bay)

Moment Frame #1

Steel Grade ASTM A992; A572 Gr 50 or Gr 55

	Beam	Column
Properties:	W18 × 40	W18 × 40
bf	6.02	6.02
tf	0.53	0.53
tw	0.32	0.32
d	17.90	17.90
A	11.80	11.80
Aw	5.48	5.48
bf/2tf	5.73	5.73
h/tw	50.90	50.90
S	68.40	68.40
ry	1.27	1.27
Z_x	78.40	78.40
Fy	50.00	50.00
Fu	65.00	65.00
Ry	1.10	1.10
E	29,000.00 ksi	29,000.00
Lb	0.00	13.67

Column Req's

1. Columns shall be any of the rolled shapes or built-up sections permitted in 2.3 of 358-10
2. Beam shall connect to the flange of the column
3. Rolled shape limited to W36.Boxed WF columns shall not have a width or depth exceeding 24" if participating in orthogonal moment frames
4. No limit on wt/ft
5. No additional flange thickness req's
6. width-thickness ratios for flanges & webs of columns shall conform to limits of section D.1 for highly ductile members of AISC 341-10 (Table D1.1)
7. Lateral bracing shall conform to Section 4c of AISC 341-10 or SMF as applicable in Seis Prov

Beam Req's

1. Shall be hot-rolled or built-up I-shaped members comforming to Section 2.3 of 358-10
2. Depth limited to W36
3. Weight Limit of beam is 300plf
4. Flange thickness limited to 1.75"
5. Clear span-to-span depth ratio is 7+
6. Width-to-thickness ratio for flanges & web shall conform to Seis Des Param (value of bf not less than flange width at ends of center 2/3 portion of RBS, etc)
7. Lateral bracing:
   Conform to Section 4b of AISC 341-10
   Locate RBS bracing no more than d/2 beyond the end of the RBS farthest from the face of the column (ie: inside the column)

Deflection Check:

Code Deflection Limits - Special Moment Frame per ASCE7-10

1st Floor	0.82	in	First Floor Allowablw Deflection
2nd Floor	1.45	in	Total Deflection at Second Floor

Actual Analysis (2nd Order), (+10% RBS) per RISA (with Strength Design Combos)

Displacment per RISA

1st Floor	0.714	*1.1	0.7854	in	OK	First Floor Deflection
2nd Floor	0.987	*1.1	1.0857	in	OK	Total deflection at Second Floor

Member Parameters Check: AISC 358-10 Section 5.3.1 (AISC 341-10 Section D1b)

Beams:

bf/2tf:	5.73	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Beam meets allowable Criteria
h/tw:	50.90	2.45(E/Fy)^0.5:	59.00381	OK	Check if the Beam meets allowable Criteria
h/tw:	50.90				

Columns:

C_a : 0.0407 < 0.125 $C_a = P_u/\phi P_y$:

h/tw:	50.90	2.45(E/Fy)^0.5*(1-0.93Ca):	56.77167	OK	Check if the Column meets allowable Criteria
b/2tf:	5.73	0.3(E/Fy)^0.5:	7.224957	OK	Check if the Column meets allowable Criteria
Puc	24.00	RISA Axial Column Load (Strength Combo)			

kl/r= 154.72 4.71((E/Fy)^2: 113.432

Fe: 3.804 ksi

Fcr: 3.336 ksi 0.61

Pn= 39.36

Pu/ϕPn= 0.61 OK

Beam Bracing: **5.29**

Lb 0.00 ft OK - No Beam Bracing Reqd

$$L_{b,MAX} = 0.086 r_y E / F_y$$

Member Parameters Check: AISC 358-10 Section 5.4.2 & AISC 341-10 Section E3.4a

Beam/Column Ratio:

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Column(s): (SETUP FOR SINGLE COLUMN BELOW CONDITION)

$$\sum Z_C (F_{YC} - (P_{UC} / A_g)$$

ΣMpc*			sum the moment capacities of the columns coming (which is reduced by the stress from axial loading)
Zc	78.40		Standard Z-column Value
Fyc	50.00		Yield Stress
Puc	24.00	RISA Axial Column Load	
Ag	11.80		Total Area of Steel in Column
ΣMpc*	**3760.54237** k-in		Sum of Available Moment Capacities in Column

$$\boxed{\sum M_{PC} = Z_{C,mx} (F_{YC,mx} - (P_{UC,mx} / A_{g,mx}))}$$

Beam(s): WORKS FOR ONLY 1-BEAM CONDITION (NOTE THE SUM SIGN & TAILOR)

$$\sum M^*_{PB_C} = \sum (1.1 R_Y F_{YB} Z_{RBS} + M_{UV})$$

M_{PB}*

R_Y	1.10			Constant Value; Ref: AISC Seis Des Pro Table A3.1				
F_{YB}	50.00			Yield Stress				
Z_{RBS}				Z at RBS				
a)	3.50	(0.5*bf)=	3.0075	(see diagram)	a= (0.5 to 0.75)bf	3.0075	to	4.51125
b)	12.00	(0.75*d)=	13.425	(see diagram)	b= (0.65 to 0.85)d	11.635	to	15.215
c)	1.50	(0.2*bf)=	1.203	(see diagram)	c=(0.1b_{bf} to 0.25b_{bf})	0.6015	to	1.50375
Z_{RBS}	**51.03** in^3							

$$Z_{RBS} = Z_X - (2ct_{bf}(d - t_{bf}))$$

M_{UV}		R= 12.75	
V_{RBS}	**14.10** kip		
V_{MAX}	11 kip	RISA Max Beam Shear (Strength Combo)	
L_{RBS}	9.4166667		Length of the member between the RBS
M_{UV}	**323.25** k-in		

$$M_{UV} = V_{RBS}\left(a + \frac{b}{2} + \frac{dc}{2}\right)$$

$$V_{RBS} = V_{MAX}\left[\frac{\frac{L}{2} - a + \frac{b}{2} + \frac{dc}{2}}{\frac{L}{2}}\right]$$

M_{PB}* **3410.83** k-in

Check The Ratios:

		Mpc/Mpb > 1, O.K.
M_{PC}/M_{PB}	**1.10**	Column Bracing Required

$$\frac{\Sigma M_{PC}}{\Sigma M_{PB}} \geq 2$$

Max Probable Moment

AISC 358-10
AISC 358-10 Eq:5.8-5

$$M_{PR} = C_{PR} R_Y F_Y Z_e$$

M_{PR}

Fy	50.00		
Ry	1.10		Ze - from 358-10 Eq 5.8-4 (Zrbs)
Ze	51.03		AISC SDP Table I-6-1
C_{PR}	1.15		AISC Black Book 5-3
M_{PR}	**3227.92**	k-in	

Max $V_{P,RBS}$

$$V_{P@RBS} = \frac{2M_{PR}}{L'} + \frac{wL'}{2}$$

V_P — This is the maximum shear at the area of the RBS (will be used later to get an eccentric force caused by the RBS at the Column Face)

L' — Beam Length Between RBS(VERIFY)

L_0	11	ft	Beam Length
w_{DL} =	0.176	k/ft	
w_{LL} =	0.47	k/ft	
w_{SL} =	0	k/ft	
w	0.4462	k/ft	RISA Factored Dist Load (MAX) 1.2DL+0.5LL+0.2SL
L'	113.00		
V_P	**59.23**	k	Max Shear

$$L' = L_0 - (a - \frac{b}{2})$$

Max $M_{COL\ FACE}$

M_F — Maximum moment seen at face of column from the eccentric force at the RBS and the plastic moment at the column

$$M_F = M_{PR} + V_{P,RBS} S_h$$

$$S_h = a + \frac{b}{2}$$

S_h	9.5	
M_F	**3790.63**	k-in

Max Plastic Moment

$$M_{pe} = Z_b R_y F_y$$

M_{PE}

φd	1			
Zb	78.40			Z-beam (Full value)
Ry	1.10			Constant
Fy	50.00			Yield Stress
φMpe	**4312.00**	k-in	OK	

$$\varphi M_{pe} \geq M_F?$$

Shear Check

AISC Setion "G"

$$V_n = 0.6 * F_y A_{web} C_v$$

$V_U = V_P$ =	59.23			Actual Max shear in the beam
Vn				Allowable shear in the beam
Check:	53.95		OK	
Fy	50.00			
Aw	5.48			
Cv	1			if "OK", then C_v & φ = 1.0
Vn	**164.53**	k	OK	

$$h/t_w \leq 2.24\sqrt{\frac{29000ksi}{50ksi}}$$

$$V_n \geq V_U?$$

Continuity Plate Requirements — AISC 358-10 Sec. 2.4 & AISC 341-10 Section E3.6f

Check:

MIN t_{cf}

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

bbf	6.02			
tbf	0.53			
Fyb	50.00			
Ryb	1.10			
Fyc	50.00			
Ryc	1.10			
MIN t_{cf}	**0.95**	in	bbf/6=	1.00
t_{cf}	0.53	in	N.G. - Continuity Plate Reqd	

k =	1.54
k_1 =	1.125
$pF_{(min)}$ =	3.04
$pC_{(max)}$ =	1.625

tbf/2= 0.2625

Minimum Continuity Plate Thickness req'd: 0.48 in

Minimum Continuity Plate Width req'd: 2.85 in — bst=(bbf-tbcw)/2

Column Panel Zones — AISC 341-10 Secion E3.6e(2)

$$t_{min} = (d_2 + w_2)/90$$

tmin

d2	16.95	in	Distance between continuity plates (depth - 2*continuity plate thickness)
w2	16.85	in	distance between column flanges (depth - 2*flange thickness)
tmin	0.38	in	
t_{cw}	0.32	in	N.G. Web Stiffener Required (Doubler Plate)

Min doubler plate Thickness Required = 0.061

Web Panel Shear — Per AISC 341-10 Section E3.6e(1) & AISC J10.6

$$V_U = \frac{\sum M_C}{(d - t_{bf})}$$

Vu

φ	1			LRFD for tension
ΣMc	78	k-ft	RISA Value	Sum of the moments in the columns about the joint (what the beams are resisting - all should add to zero)
	936	k-in		k-in verses k'
d	17.90	in		
tbf	0.53	in		
Vu	**53.87**	k		Design force in the web
Pr	24.00		AXIAL force in Column (strength Combos)	
Pc	274.2125			=Py(LRFD) Calculated as FyA
Fy	50.00	ksi		
Aw	5.48	in2		Area of the web taking the panel shear forces from the beam moment
Pc	274.2125	k		Allowable Shear Force in the Web
φ	0.75			Shear factor LRFD
φPc	**205.66**	k	OK (Web Stiffener NOT Required)	Check Actual shear values in beam verses allowable shear

Pr < 0.4Pc

If Pr < 0.4Pc: $R_n = 0.6 F_y d_c t_w$

If Pr > 0.4Pc: Rn = 0.6Fydctw * (1.4 - Pr/Pc)

Rn

Fy	50.00	ksi		
tcf	0.53	in		
bcf	6.02	in		
db	17.90	in		
dc	17.90	in		
tw	0.32	in		
Rn	169.16			
φRn	**169.16**		OK (Web Stiffener NOT Required)	Allowable force in the web

$V_u \geq \varphi R$

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E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.63
C_{min} (inch): 1.88
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 30.00
State: Cracked
Compressive strength, f'_c (psi): 5500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 30.00 x 16.00 x 1.00
Yield stress: 36000 psi

Profile type/size: W18X40

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

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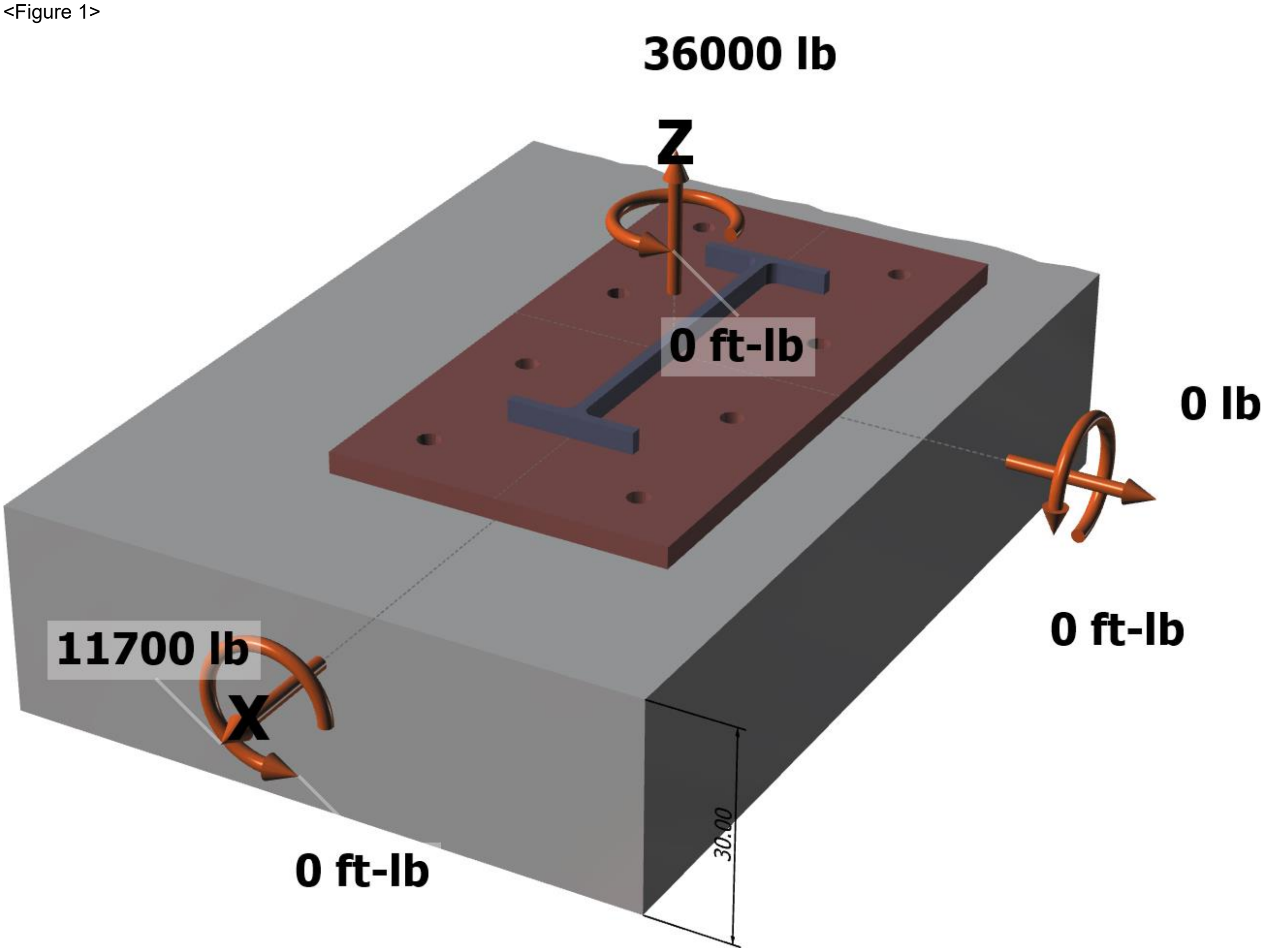
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 36000
V_{uax} [lb]: 11700
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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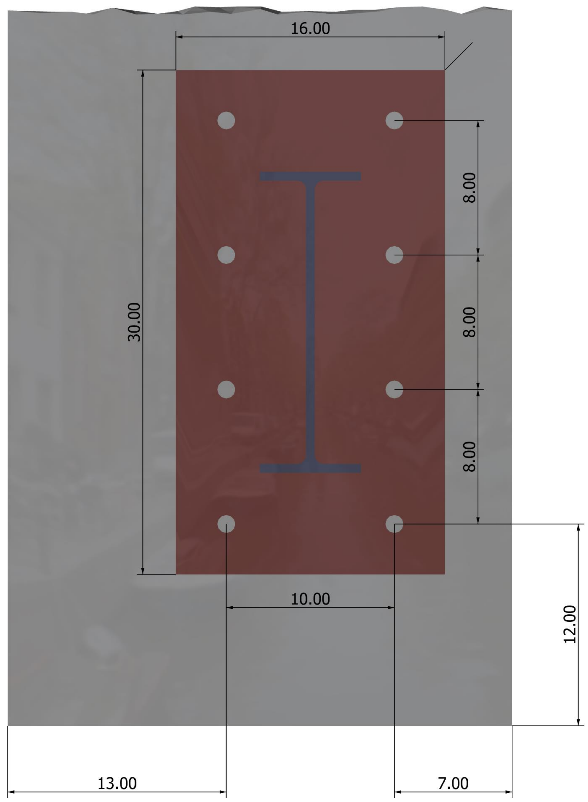
<Figure 2>

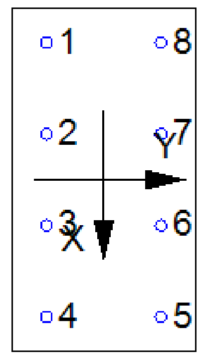


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4500.0	1462.5	0.0	1462.5
2	4500.0	1462.5	0.0	1462.5
3	4500.0	1462.5	0.0	1462.5
4	4500.0	1462.5	0.0	1462.5
5	4500.0	1462.5	0.0	1462.5
6	4500.0	1462.5	0.0	1462.5
7	4500.0	1462.5	0.0	1462.5
8	4500.0	1462.5	0.0	1462.5
Sum	36000.0	11700.0	0.0	11700.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 36000
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	5500	8.667	43390

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in²)	A_{Nco} (in²)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
1511.25	676.00	7.00	1.000	0.862	1.00	1.000	43390	0.70	43874

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in²)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	5500	0.70	119057

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sbg} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1, Eq. 17.4.4.1 & 17.4.4.2)

s (in)	c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sb}$ (lb)
24.00	7.00	12.00	5.15	1.00	5500	0.70	105565

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
21090	1.0	0.65	13709

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1511.25	676.00	1.000	0.862	1.000	1.000	43390	0.70	116999

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4500	26363	0.17	Pass
Concrete breakout	**36000**	**43874**	**0.82**	**Pass (Governs)**
Pullout	4500	119057	0.04	Pass
Side-face blowout	18000	105565	0.17	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**1463**	**13709**	**0.11**	**Pass (Governs)**
Pryout	11700	116999	0.10	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.82	0.00	82.1%	1.0	Pass

PAB8 (1"Ø) with hef = 18.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.488 inch

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.63
C_{min} (inch): 1.88
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 30.00
State: Cracked
Compressive strength, f'_c (psi): 5500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 30.00 x 16.00 x 1.00
Yield stress: 36000 psi

Profile type/size: W18X46

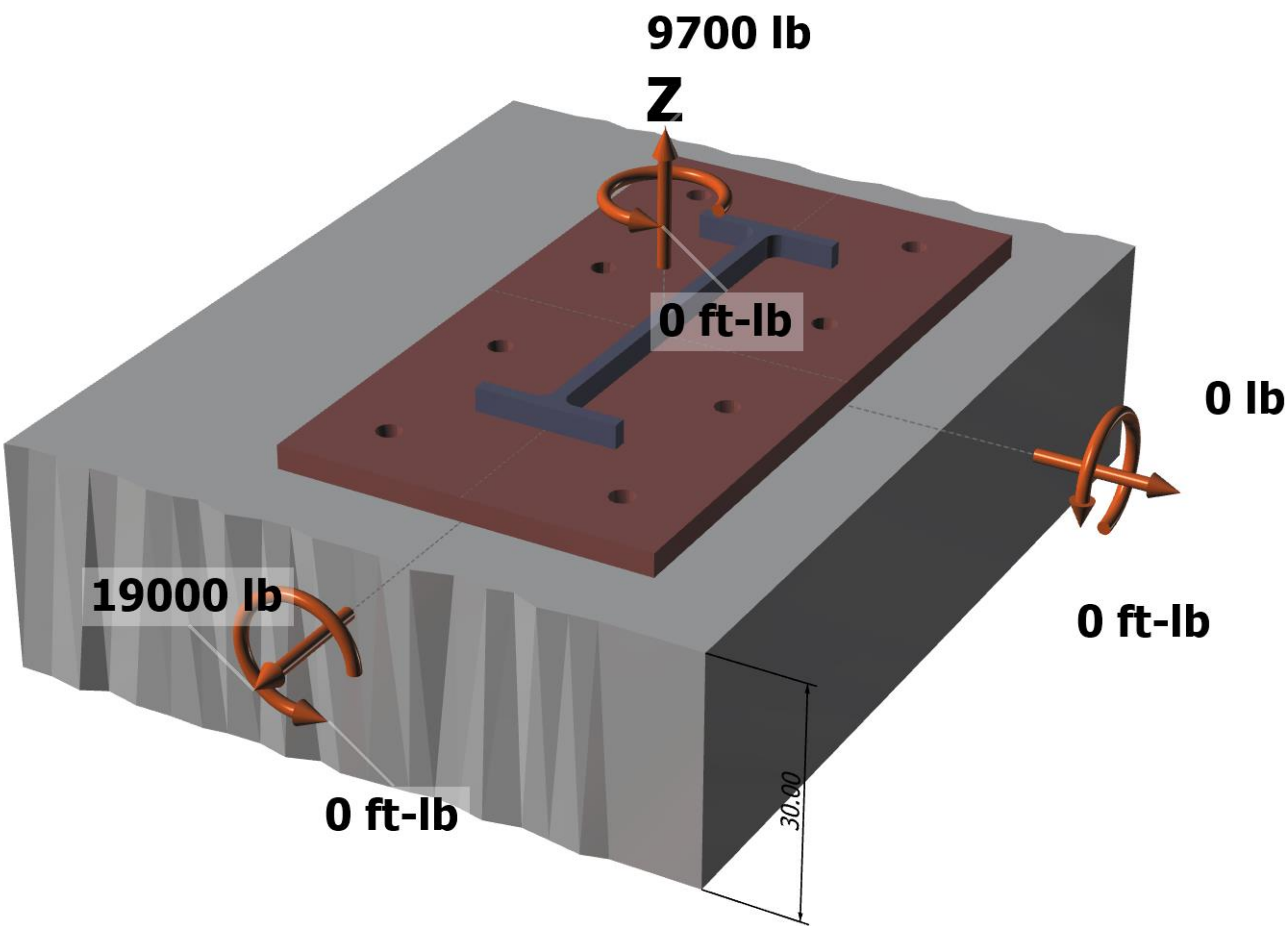
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 9700
V_{uax} [lb]: 19000
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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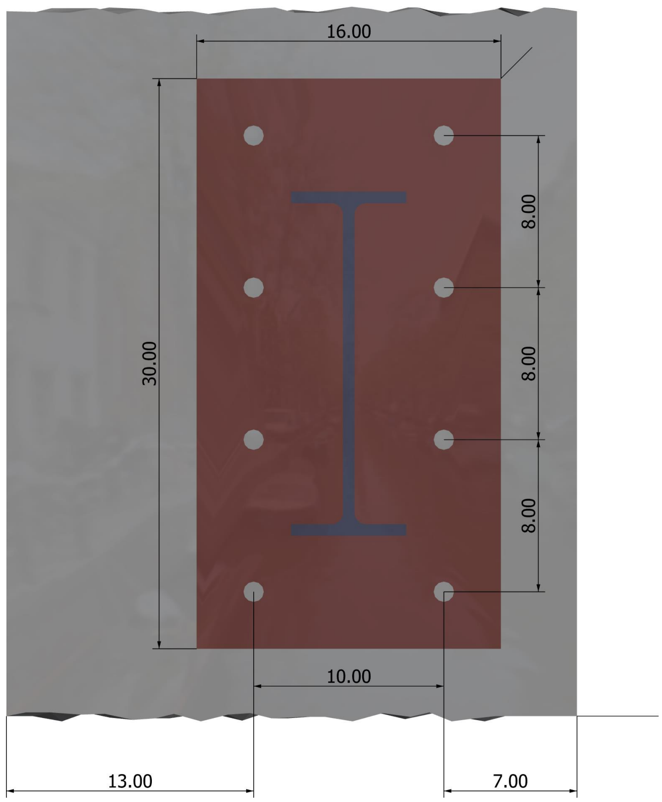
<Figure 2>

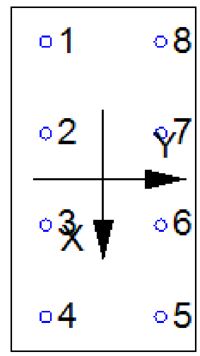


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1212.5	2375.0	0.0	2375.0
2	1212.5	2375.0	0.0	2375.0
3	1212.5	2375.0	0.0	2375.0
4	1212.5	2375.0	0.0	2375.0
5	1212.5	2375.0	0.0	2375.0
6	1212.5	2375.0	0.0	2375.0
7	1212.5	2375.0	0.0	2375.0
8	1212.5	2375.0	0.0	2375.0
Sum	9700.0	19000.0	0.0	19000.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 9700
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	5500	18.000	146697

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
2422.50	2916.00	7.00	1.000	0.778	1.00	1.000	146697	0.70	49764

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	5500	0.70	119057

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sbg} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1, Eq. 17.4.4.1 & 17.4.4.2)

s (in)	c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sb}$ (lb)
24.00	7.00	99999.00	5.15	1.00	5500	0.70	155570

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
21090	1.0	0.65	13709

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	5500	7.00	12361

$\phi V_{cbgx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
472.50	220.50	1.000	1.000	1.000	1.000	12361	0.70	37084

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	2422.50	2916.00	1.000	0.778	1.000	1.000	146697	0.70	132703

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	1213	26363	0.05	Pass
Concrete breakout	**9700**	**49764**	**0.19**	**Pass (Governs)**
Pullout	1213	119057	0.01	Pass
Side-face blowout	4850	155570	0.03	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	2375	13709	0.17	Pass
|| Concrete breakout y+	**9500**	**37084**	**0.26**	**Pass (Governs)**
Pryout	19000	132703	0.14	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	0.26	25.6%	1.0	Pass

PAB8 (1"Ø) with hef = 18.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.329 inch

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

Beam on Elastic Foundation

DESCRIPTION: MF-2 FOoting on (E) Footings

CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : IBC 2018

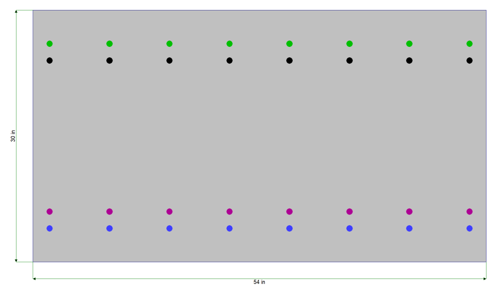
Material Properties

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 Lt Wt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi			

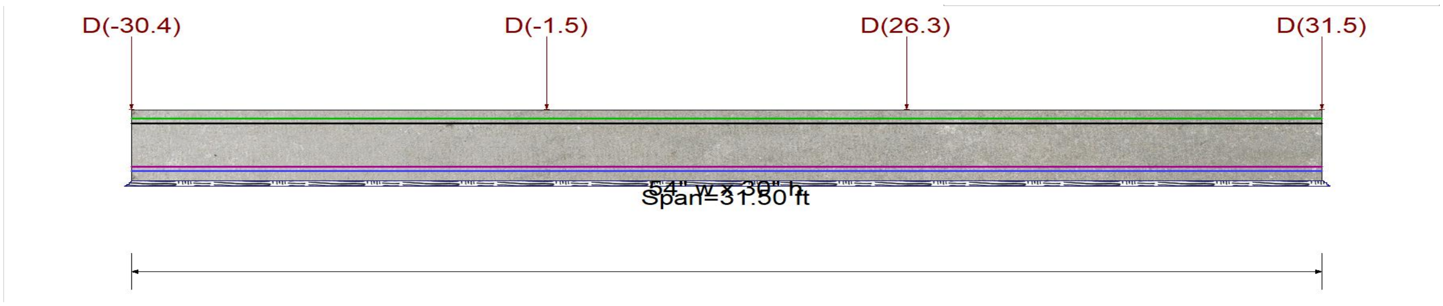
Soil Subgrade Modulus = 250.0 psi / (inch deflection)

Load Combination IBC 2018

fy - Main Rebar	=	60.0 ksi	Fy - Stirrups	=	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups	=	29,000.0 ksi
			Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup		2



Beam is supported on an elastic foundation.



Cross Section & Reinforcing Details

Rectangular Section, Width = 54.0 in, Height = 30.0 in
Span #1 Reinforcing....
8-#6 at 4.0 in from Bottom, from 0.0 to 31.50 ft in this span
8-#6 at 6.0 in from Bottom, from 0.0 to 31.50 ft in this span
8-#6 at 4.0 in from Top, from 0.0 to 31.50 ft in this span
8-#6 at 6.0 in from Top, from 0.0 to 31.50 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 = -30.40 k @ 0.0 ft
Point Load : D = -1.50 k @ 11.0 ft
Point Load : D = 26.30 k @ 20.50 ft
Point Load : D = 31.50 k @ 31.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.143** : 1	**Maximum Deflection**	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.000 in
Mu : Applied	109.695 k-ft	Max Upward L+Lr+S Deflection	0.000 in
Mn * Phi : Allowable	764.65 k-ft	Max Downward Total Deflection	0.023 in
Load Combination	+1.40D+1.60H	Max Upward Total Deflection	-0.036 in
Location of maximum on span	6.300 ft		
Span # where maximum occurs	Span # 1		

Maximum Soil Pressure = **0.829** ksf at 20.65 ft LdComb: +D+H
Allowable Soil Pressure = **1.50** ksf OK

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination				Bending Stress Results (k-ft)		
	Segment Length	Span #	Location (ft) in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum Bending Envelope						

Beam on Elastic Foundation

DESCRIPTION: MF-2 FOoting on (E) Footings

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 1	1	6.300	109.69	764.65	0.14
+1.40D+1.60H					
Span # 1	1	6.300	109.69	764.65	0.14
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+1.60L+0.50S+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+1.60Lr+0.50L+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+1.60Lr+0.50W+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+0.50L+1.60S+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+1.60S+0.50W+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+0.50Lr+0.50L+W+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+0.50L+0.50S+W+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+1.20D+0.50L+0.70S+E+1.60H					
Span # 1	1	6.300	94.02	764.65	0.12
+0.90D+W+0.90H					
Span # 1	1	6.300	70.52	764.65	0.09
+0.90D+E+0.90H					
Span # 1	1	6.300	70.52	764.65	0.09

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0230	20.650	Span 1	-0.0364	0.000

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Spacing (in) Req'd	Suggest
+1.40D+1.60H	1	0.00	26.00	-1.65	1.65	0.00	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	0.37	26.00	37.17	37.17	15.05	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	0.74	26.00	33.60	33.60	28.71	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	1.11	26.00	30.21	30.21	41.05	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	1.48	26.00	27.00	27.00	52.13	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	1.85	26.00	23.95	23.95	62.03	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	2.22	26.00	21.07	21.07	70.79	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	2.59	26.00	18.36	18.36	78.49	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	2.96	26.00	15.81	15.81	85.18	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	3.34	26.00	13.42	13.42	90.93	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	3.71	26.00	11.19	11.19	95.79	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	4.08	26.00	9.10	9.10	99.82	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	4.45	26.00	7.17	7.17	103.08	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	4.82	26.00	5.38	5.38	105.63	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	5.19	26.00	3.73	3.73	107.51	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	5.56	26.00	2.22	2.22	108.78	0.61	125.78	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	5.93	26.00	0.85	0.85	109.49	0.23	115.70	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	6.30	26.00	-0.40	0.40	109.69	0.11	112.49	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	6.67	26.00	-1.53	1.53	109.43	0.42	120.64	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	7.04	26.00	-2.53	2.53	108.76	0.70	128.02	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	7.41	26.00	-3.42	3.42	107.71	0.95	134.72	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	7.78	26.00	-4.19	4.19	106.33	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	8.15	26.00	-4.86	4.86	104.66	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	8.52	26.00	-5.42	5.42	102.75	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	8.89	26.00	-5.89	5.89	100.62	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	9.26	26.00	-6.25	6.25	98.33	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	9.64	26.00	-6.52	6.52	95.90	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	10.01	26.00	-6.71	6.71	93.37	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00

Beam on Elastic Foundation

DESCRIPTION: MF-2 FOoting on (E) Footings

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)	Spacing (in) Req'd	Spacing (in) Suggest
+1.40D+1.60H	1	10.38	26.00	-6.80	6.80	90.77	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	10.75	26.00	-6.82	6.82	88.14	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	11.12	26.00	-4.65	4.65	85.75	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	11.49	26.00	-4.51	4.51	83.91	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	11.86	26.00	-4.30	4.30	82.13	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	12.23	26.00	-4.02	4.02	80.42	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	12.60	26.00	-3.67	3.67	78.82	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	12.97	26.00	-3.25	3.25	77.35	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	13.34	26.00	-2.78	2.78	76.03	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	13.71	26.00	-2.24	2.24	74.89	0.90	133.32	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	14.08	26.00	-1.66	1.66	73.95	0.67	127.32	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	14.45	26.00	-1.02	1.02	73.22	0.42	120.57	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	14.82	26.00	-0.33	0.33	72.73	0.13	113.13	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	15.19	26.00	0.41	0.41	72.50	0.17	114.08	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	15.56	26.00	1.19	1.19	72.54	0.49	122.60	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	15.94	26.00	2.02	2.02	72.87	0.83	131.48	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	16.31	26.00	2.88	2.88	73.51	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	16.68	26.00	3.77	3.77	74.46	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	17.05	26.00	4.70	4.70	75.75	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	17.42	26.00	5.67	5.67	77.38	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	17.79	26.00	6.65	6.65	79.37	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	18.16	26.00	7.67	7.67	81.72	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	18.53	26.00	8.70	8.70	84.45	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	18.90	26.00	9.75	9.75	87.56	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	19.27	26.00	10.82	10.82	91.07	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	19.64	26.00	11.90	11.90	94.96	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	20.01	26.00	12.99	12.99	99.26	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	20.38	26.00	14.08	14.08	103.96	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	20.75	26.00	-21.65	21.65	99.75	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	21.12	26.00	-20.56	20.56	91.62	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	21.49	26.00	-19.48	19.48	83.88	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	21.86	26.00	-18.41	18.41	76.55	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	22.24	26.00	-17.36	17.36	69.62	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	22.61	26.00	-16.31	16.31	63.07	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	22.98	26.00	-15.29	15.29	56.91	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	23.35	26.00	-14.28	14.28	51.14	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	23.72	26.00	-13.30	13.30	45.73	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	24.09	26.00	-12.34	12.34	40.69	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	24.46	26.00	-11.41	11.41	36.00	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	24.83	26.00	-10.50	10.50	31.66	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	25.20	26.00	-9.62	9.62	27.66	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	25.57	26.00	-8.77	8.77	23.98	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	25.94	26.00	-7.95	7.95	20.61	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	26.31	26.00	-7.16	7.16	17.56	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	26.68	26.00	-6.41	6.41	14.79	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	27.05	26.00	-5.68	5.68	12.30	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	27.42	26.00	-4.98	4.98	10.09	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	27.79	26.00	-4.32	4.32	8.13	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	28.16	26.00	-3.69	3.69	6.41	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	28.54	26.00	-3.10	3.10	4.93	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	28.91	26.00	-2.53	2.53	3.67	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	29.28	26.00	-2.00	2.00	2.62	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	29.65	26.00	-1.51	1.51	1.77	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	30.02	26.00	-1.05	1.05	1.10	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00

Beam on Elastic Foundation

File: MF Grade Beam and Reactions for Pins.ec6

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

DESCRIPTION: MF-2 FOoting on (E) Footings

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)	Spacing (in) Req'd	Spacing (in) Suggest
+1.40D+1.60H	1	30.39	26.00	-0.62	0.62	0.60	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+1.40D+1.60H	1	30.76	26.00	-0.22	0.22	0.26	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00
+0.90D+E+0.90H	1	31.13	26.00	0.20	0.20	0.04	1.00	135.98	Vu < PhiVc/2	Not Reqd	0.00	0.00