Job: 24-264 5900 SW 114th Terr.

LOCATION: 5900 SW 114th Terr. Pinecrest FL 33156

STRUCTURAL CALCULATIONS

DESIGN CRITERIA:

Calculations based on:

1. 2023 Florida Building Code
2. Minimum Design Loads for Buildings and Other Structures ASCE 7-22
3. Building Code Requirements for Structural Concrete ACI 318-19
4. American Institute of Steel Construction AISC-15 Ed
5. Specifications for the Design of Cold-Formed Stainless Steel Structural Members SEI/ASCE8-02
6. 2020 Aluminum Design Manual

CALCULATION INDEX:

☑ REVIEWED AND APPROVED
☐ REVIEWED, MARKED AND APPROVED RESUBMIT CORRECTED COPY FOR FILING
☐ REVISE AND RESUBMIT
☐ NOT REVIEWED

REVIEWED ONLY FOR THE LIMITED PURPOSE OF THE CHECKING FOR CONFORMANCE WITH THE INFORMATION GIVEN AND THE DESIGN CONCEPT EXPRESSED IN THE FULL SET OF CONTRACT DOCUMENTS. IT DOES NOT AUTHORIZE CHANGES IN THE CONTRACT SUM. REVIEW OF SHOP DRAWINGS IS NOT CONDUCTED FOR DETERMINING THE ACCURACY AND COMPLETENESS OF DETAILS, QUANTITIES OR FOR SUBSTANTIATING FABRICATION INSTALLATION INSTRUCTION OR PERFORMANCE OF EQUIPMENT OF SYSTEM, ALL OF WHICH SHALL REMAIN THE SOLE RESPONSIBILITY OF THE CONTRACTOR REVIEW OF SHOP DRAWINGS DOES NOT CONSTITUTE APPROVAL OF SAFETY PRECAUTIONS, CONSTRUCTION MEANS, METHODS, SEQUENCES OR PROCEDURES, NOR DOES IT INDICATE APPROVAL OF ASSEMBLY OF WHICH THE ITEM IS A COMPONENT, COORDINATION OF ALL TRADES SHALL REMAIN THE SOLE RESPONSIBILITY OF THE CONTRACTOR. CONTRACTOR MUST REVIEW ALL QUANTITIES AND DIMENSIONS AT THE SITE.

GF CONSULTING ENGINEERS, INC.

BY: ________ DATE: 1-16-25

CALCULATION STATEMENT:

To the best of my knowledge, ability, belief and professional judgment, I hereby attest that the manual calculations and computer generated calculations are in compliance with the existing governing codes.

ALAYM AGUILAR
LICENSE
No. 92154
STATE OF FLORIDA
PROFESSIONAL ENGINEER

Alaym Aguilar

Digitally signed by Alaym Aguilar
DN: c=US, st=Florida, l=Miami, serialNumber=AATL20240903364730, o=3A ENGINEERING GROUP INC, cn=Alaym Aguilar, email=3aengdrafting@gmail.com
Date: 2025.01.17 11:45:53 -05'00'

Alaym Aguilar, Professional Engineer, State of Florida, License No. 92154. This item has been digitally signed and sealed by Alaym Aguilar, PE, on the date provided on the signature. Printed copies of this document are not considered signed and sealed and the signature must be verified on any copies.

Alaym Aguilar, PE
Lic. No. 92154

Steel Stair Design

Loads

.

Stair Tread = HSS 2"x10"x3/8"
DL= self-weight + 20 psf (imposed)
LL= 60.0 psf or PL 300 #

.

Central Stringer = HSS6x4x1/4
DL= 30 psf + 20 psf (imposed)
LL= 60.0 psf

.

Tread

.

Load Combinations

.

Comb (1)
q1 = DL + LL

.

Comb (2)
q2 = DL + 1/2 x LL w/ Ecc

.

Comb (3)
q2 = DL + 300#

See Printout for every Case-Scenario
Combination 3 is worst case scenario. Member is OK for use under the imposed load conditions

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

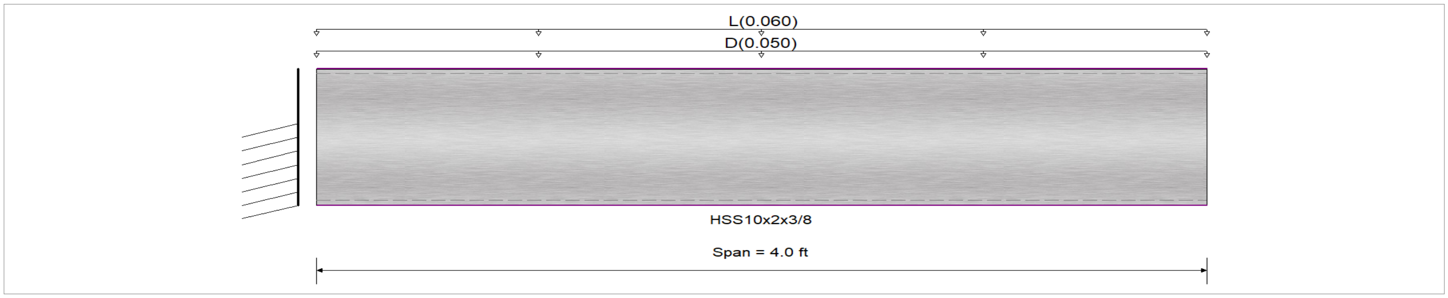
DESCRIPTION: Copy of Comb 1 DL + LL

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor 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 = 0.050 k/ft, Tributary Width = 1.0 ft

Uniform Load : L = 0.060 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.083** : 1	Maximum Shear Stress Ratio =	**0.050** : 1
Section used for this span	**HSS10x2x3/8**	Section used for this span	**HSS10x2x3/8**
Ma : Applied	1.099 k-ft	Va : Applied	0.5496 k
Mn / Omega : Allowable	13.222 k-ft	Vn/Omega : Allowable	10.994 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	3,951	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : L Only
Max Downward Total Deflection	0.056 in	Ratio =	1725	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 4.00 ft	1	0.047	0.028		-0.62	0.62	22.08	13.22	1.00	1.00	0.31	18.36	10.99
+D+L													
Dsgn. L = 4.00 ft	1	0.083	0.050		-1.10	1.10	22.08	13.22	1.00	1.00	0.55	18.36	10.99
+D+0.750L													
Dsgn. L = 4.00 ft	1	0.074	0.045		-0.98	0.98	22.08	13.22	1.00	1.00	0.49	18.36	10.99
+0.60D													
Dsgn. L = 4.00 ft	1	0.028	0.017		-0.37	0.37	22.08	13.22	1.00	1.00	0.19	18.36	10.99

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0556	4.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0313	4.000		0.0000	0.000
+D+L	1	0.0556	4.000		0.0000	0.000
+D+0.750L	1	0.0496	4.000		0.0000	0.000
+0.60D	1	0.0188	4.000		0.0000	0.000
L Only	1	0.0243	4.000		0.0000	0.000

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc. (c) ENERCALC INC 1983-2023

DESCRIPTION: Copy of Comb 1 DL + LL

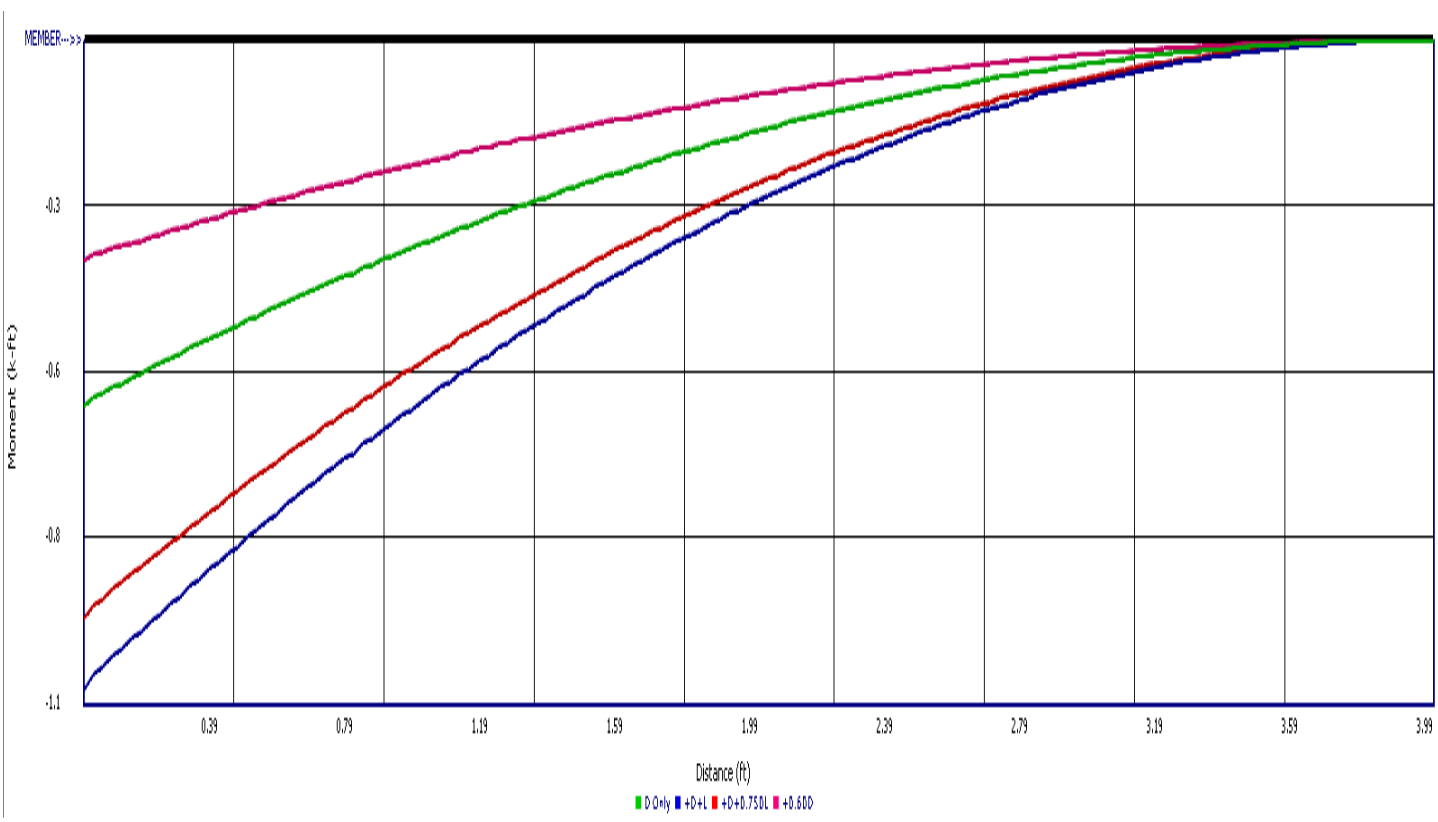
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.550	
Max Upward from Load Combinations	0.550	
Max Upward from Load Cases	0.310	
D Only	0.310	
+D+L	0.550	
+D+0.750L	0.490	
+0.60D	0.186	
L Only	0.240	

Steel Section Properties : HSS10x2x3/8

Depth	=	10.000 in	I xx	=	71.70 in^4	J	=	15.900 in^4
			S xx		14.30 in^3	Cw	=	11.00 in^6
Width	=	2.000 in	R xx	=	3.080 in			
Wall Thick	=	0.349 in	Zx	=	20.300 in^3			
Area	=	7.580 in^2	I yy	=	4.700 in^4	C	=	0.000 in^3
Weight	=	27.406 plf	S yy	=	4.700 in^3			
			R yy	=	0.787 in			
			Zy	=	5.760 in^3			
Ycg	=	5.000 in						



Steel Beam

Project File: 24-264 Calculation.ec6

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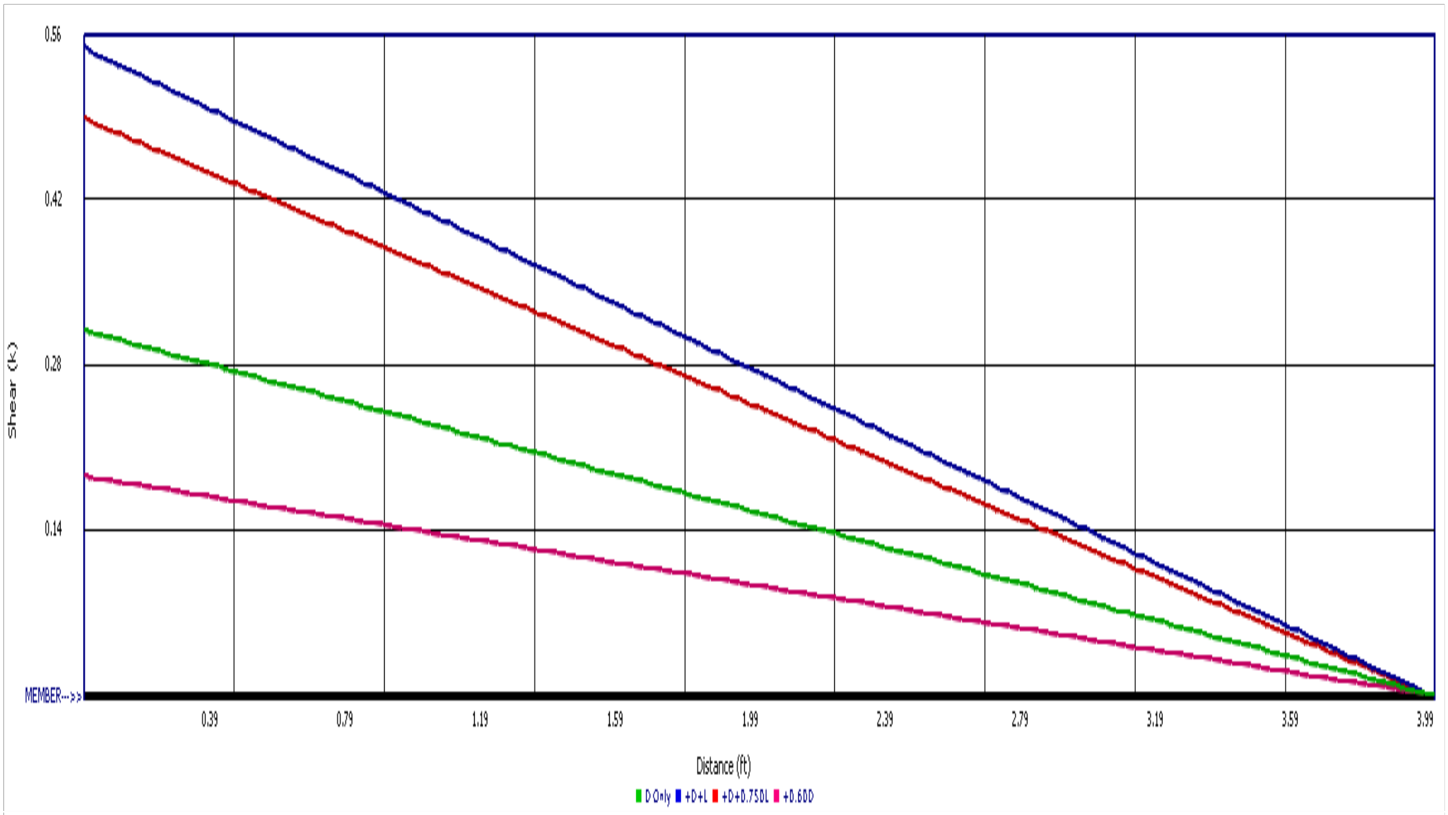
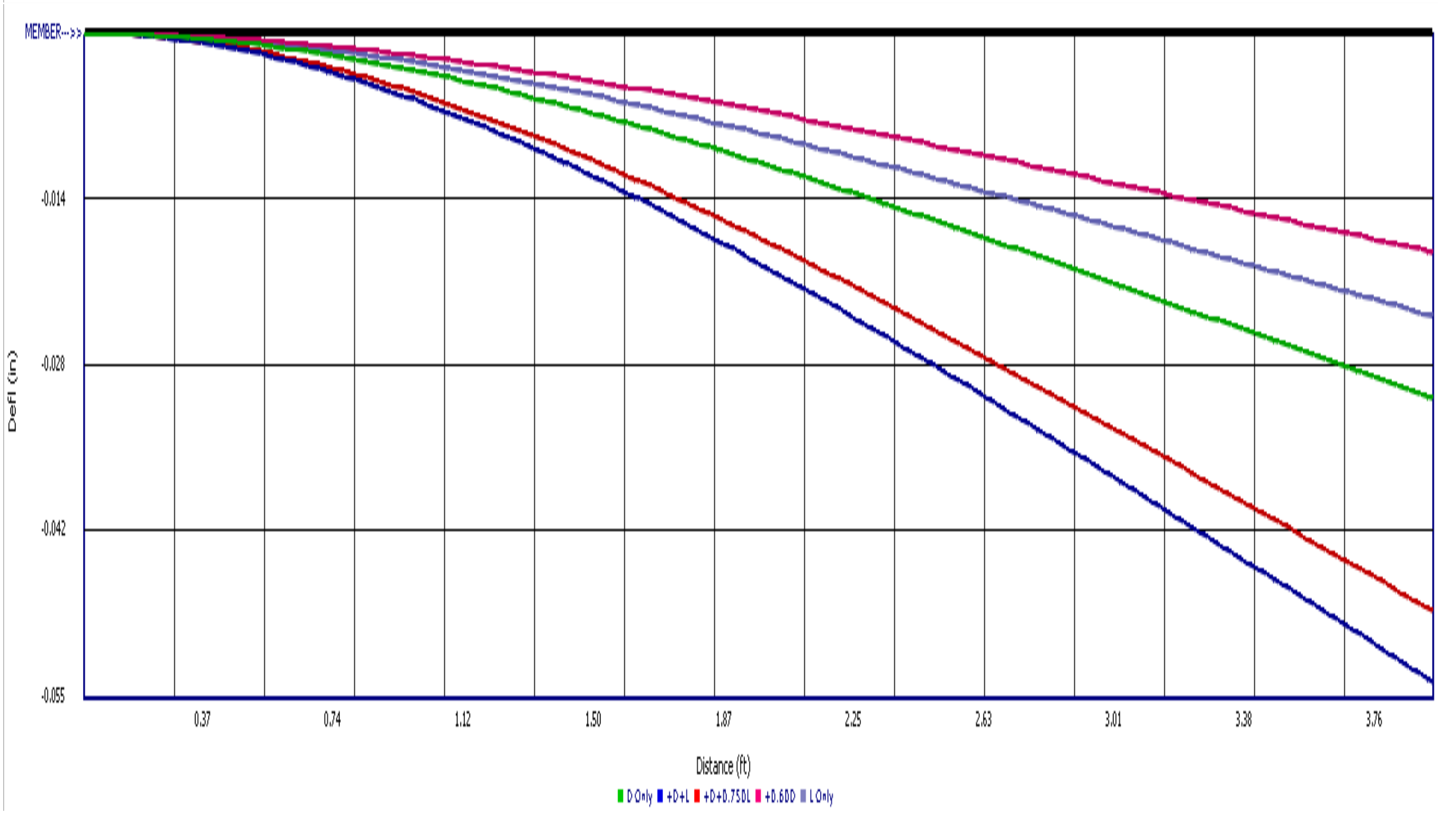
DESCRIPTION: Copy of Comb 1 DL + LL





Beam Span Moments & Shears at Incremental Locations

Load Type/Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.310	-0.619

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 | Arch-Deco Design & Construction Inc. | (c) ENERCALC INC 1983-2023

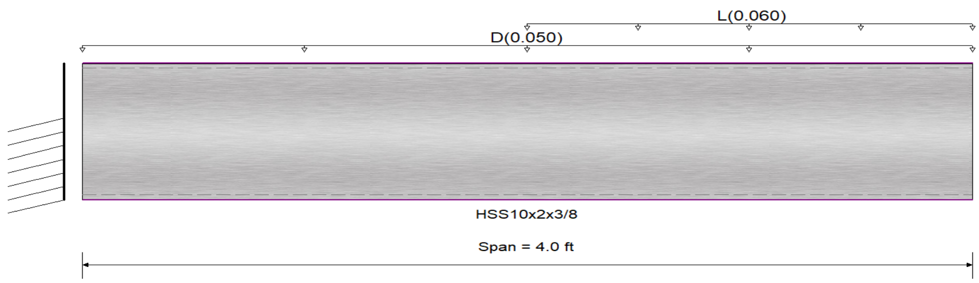
DESCRIPTION: Comb 2 DL + 0.5LL

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor 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 = 0.050 k/ft, Tributary Width = 1.0 ft

Uniform Load : L = 0.060 k/ft, Extent = 2.0 -->> 4.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.074** : 1	Maximum Shear Stress Ratio =	**0.039** : 1
Section used for this span	**HSS10x2x3/8**	Section used for this span	**HSS10x2x3/8**
Ma : Applied	0.979 k-ft	Va : Applied	0.4296 k
Mn / Omega : Allowable	13.222 k-ft	Vn/Omega : Allowable	10.994 k
Load Combination	+D+L	Load Combination	+D+L
		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	0.021 in Ratio =	4,626	>=360	
Max Upward Transient Deflection	0.000 in Ratio =	0	<360	Span: 1 : L Only
Max Downward Total Deflection	0.052 in Ratio =	1843	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in Ratio =	0	<180	

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	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 4.00 ft	1	0.047	0.028		-0.62	0.62	22.08	13.22	1.00	1.00	0.31	18.36	10.99
+D+L													
Dsgn. L = 4.00 ft	1	0.074	0.039		-0.98	0.98	22.08	13.22	1.00	1.00	0.43	18.36	10.99
+D+0.750L													
Dsgn. L = 4.00 ft	1	0.067	0.036		-0.89	0.89	22.08	13.22	1.00	1.00	0.40	18.36	10.99
+0.60D													
Dsgn. L = 4.00 ft	1	0.028	0.017		-0.37	0.37	22.08	13.22	1.00	1.00	0.19	18.36	10.99

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0521	4.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0313	4.000		0.0000	0.000
+D+L	1	0.0521	4.000		0.0000	0.000
+D+0.750L	1	0.0469	4.000		0.0000	0.000
+0.60D	1	0.0188	4.000		0.0000	0.000
L Only	1	0.0208	4.000		0.0000	0.000

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

DESCRIPTION: Comb 2 DL + 0.5LL

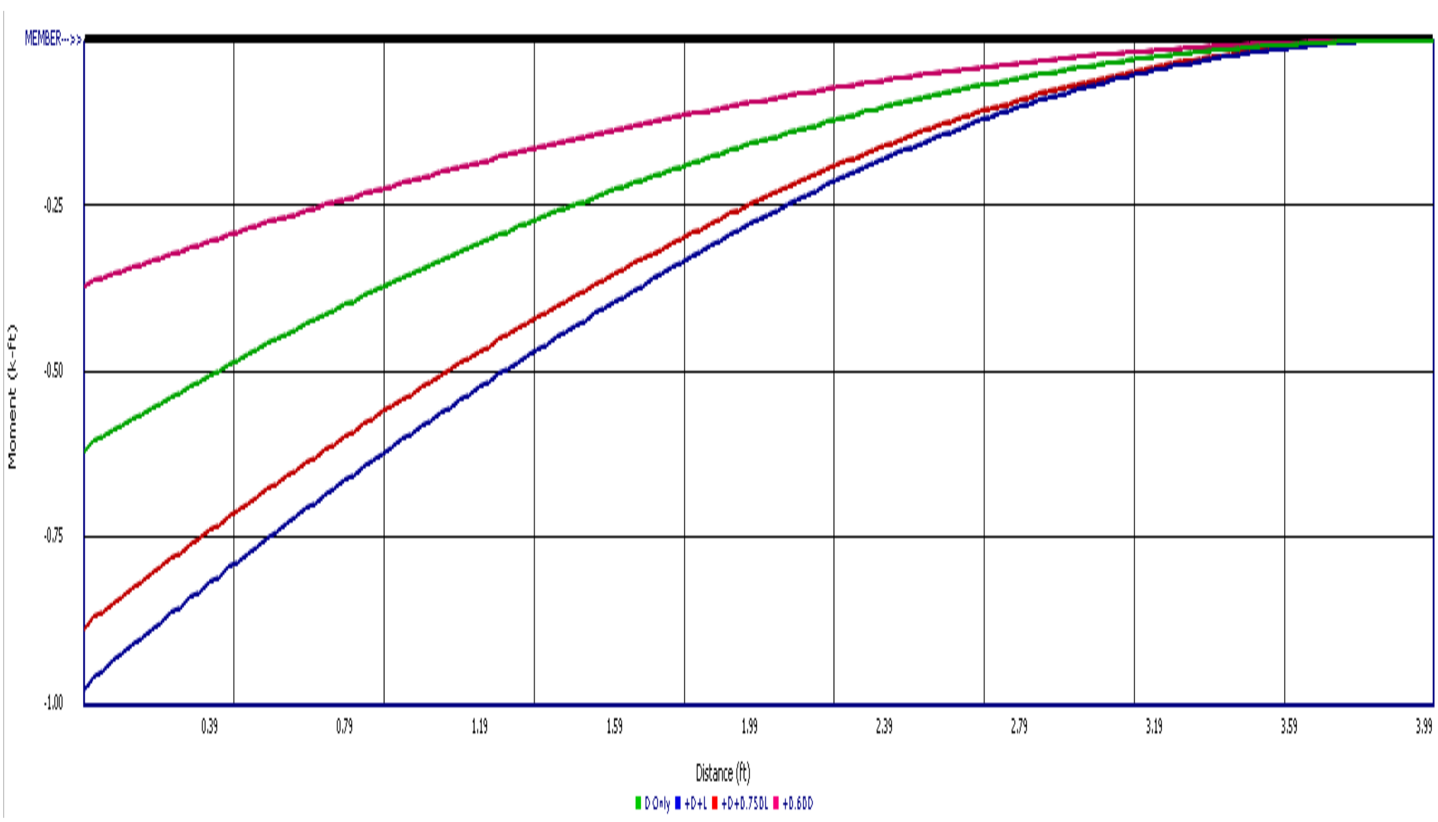
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.430	
Max Upward from Load Combinations	0.430	
Max Upward from Load Cases	0.310	
D Only	0.310	
+D+L	0.430	
+D+0.750L	0.400	
+0.60D	0.186	
L Only	0.120	

Steel Section Properties : HSS10x2x3/8

Depth	=	10.000 in	I xx	=	71.70 in^4	J	=	15.900 in^4
			S xx		14.30 in^3	Cw	=	11.00 in^6
Width	=	2.000 in	R xx	=	3.080 in			
Wall Thick	=	0.349 in	Zx	=	20.300 in^3			
Area	=	7.580 in^2	I yy	=	4.700 in^4	C	=	0.000 in^3
Weight	=	27.406 plf	S yy	=	4.700 in^3			
			R yy	=	0.787 in			
			Zy	=	5.760 in^3			
Ycg	=	5.000 in						



Steel Beam

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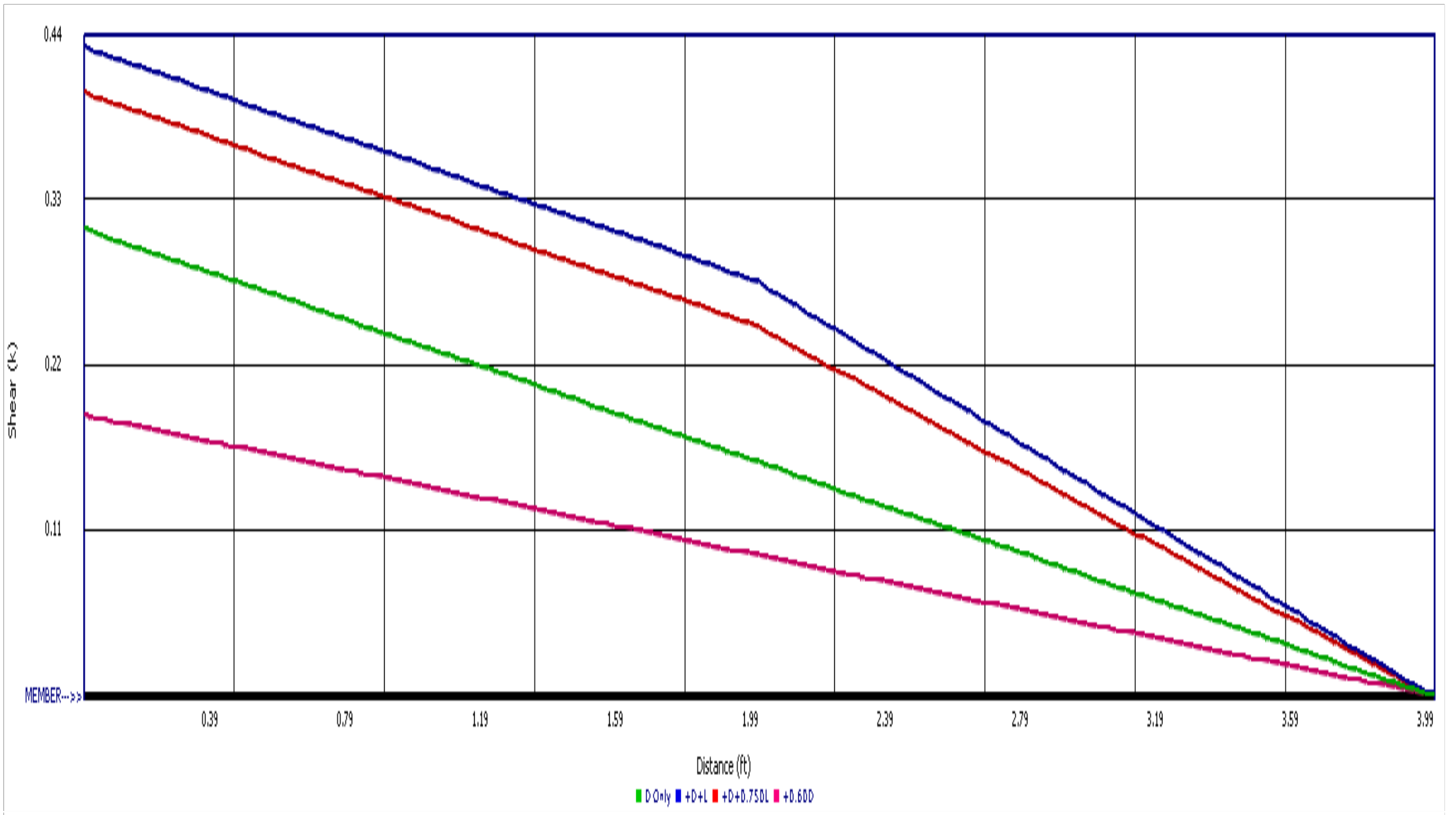
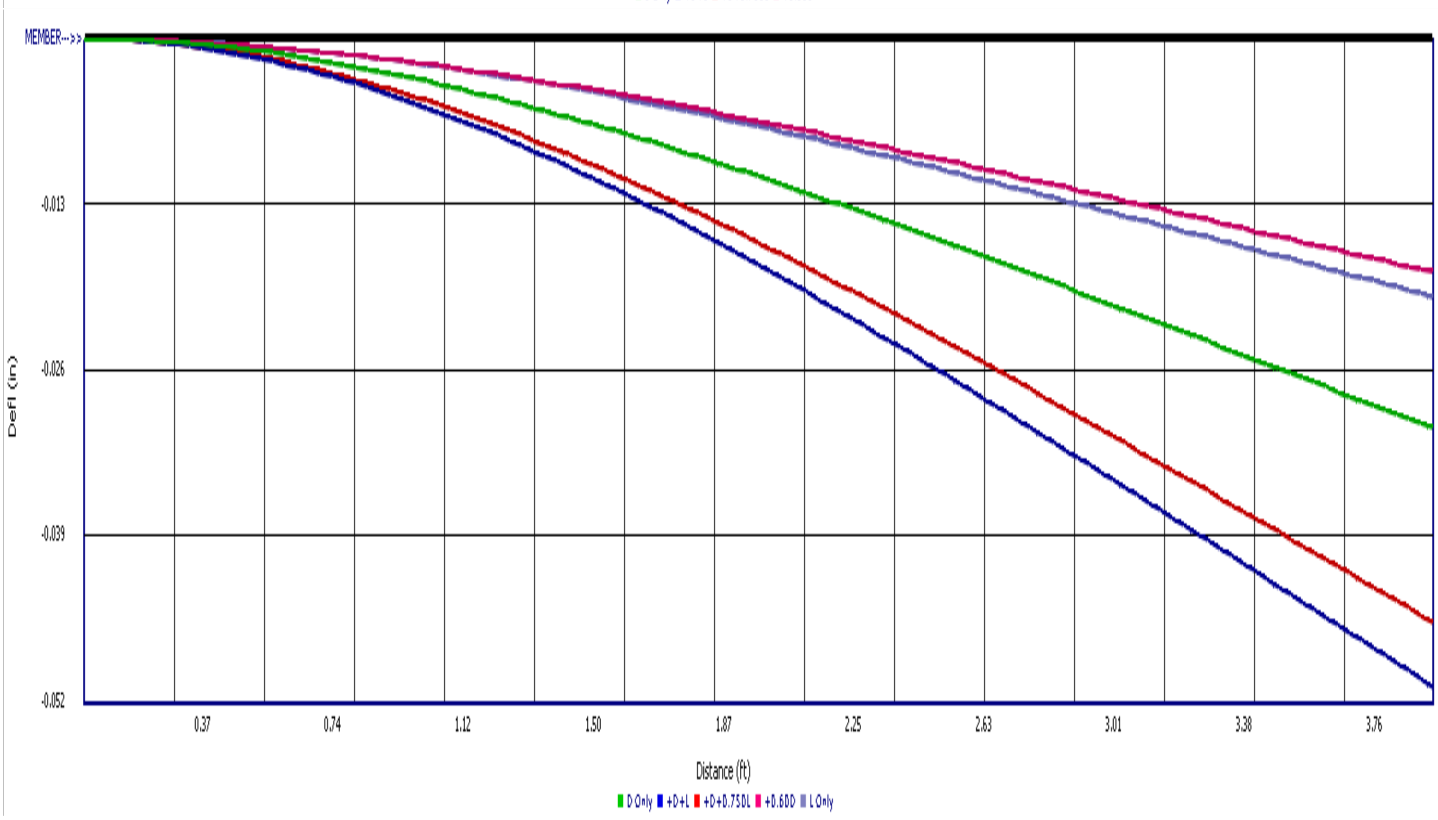
DESCRIPTION: Comb 2 DL + 0.5LL





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.310	-0.619

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc. (c) ENERCALC INC 1983-2023

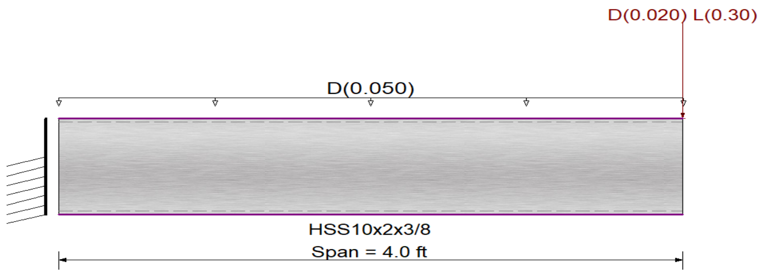
DESCRIPTION: Comb 3 DL + 300

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor 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 = 0.050 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.020, L = 0.30 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.144** : 1	Maximum Shear Stress Ratio =	**0.057** : 1
Section used for this span	**HSS10x2x3/8**	Section used for this span	**HSS10x2x3/8**
Ma : Applied	1.899 k-ft	Va : Applied	0.6296 k
Mn / Omega : Allowable	13.222 k-ft	Vn/Omega : Allowable	10.994 k
Load Combination	+D+L	Load Combination	+D+L
		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	0.081 in	Ratio =	1,185	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : L Only
Max Downward Total Deflection	0.118 in	Ratio =	816	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length		Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mny	Mny/Omega	Cb	Rm	Summary of Shear Values Va Max	Vny	Vny/Omega
D Only														
Dsgn. L =	4.00 ft	1	0.053	0.030		-0.70	0.70	22.08	13.22	1.00	1.00	0.33	18.36	10.99
+D+L														
Dsgn. L =	4.00 ft	1	0.144	0.057		-1.90	1.90	22.08	13.22	1.00	1.00	0.63	18.36	10.99
+D+0.750L														
Dsgn. L =	4.00 ft	1	0.121	0.050		-1.60	1.60	22.08	13.22	1.00	1.00	0.55	18.36	10.99
+0.60D														
Dsgn. L =	4.00 ft	1	0.032	0.018		-0.42	0.42	22.08	13.22	1.00	1.00	0.20	18.36	10.99

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.1177	4.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0367	4.000		0.0000	0.000
+D+L	1	0.1177	4.000		0.0000	0.000
+D+0.750L	1	0.0975	4.000		0.0000	0.000
+0.60D	1	0.0220	4.000		0.0000	0.000
L Only	1	0.0810	4.000		0.0000	0.000

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 | Arch-Deco Design & Construction Inc. | (c) ENERCALC INC 1983-2023

DESCRIPTION: Comb 3 DL + 300

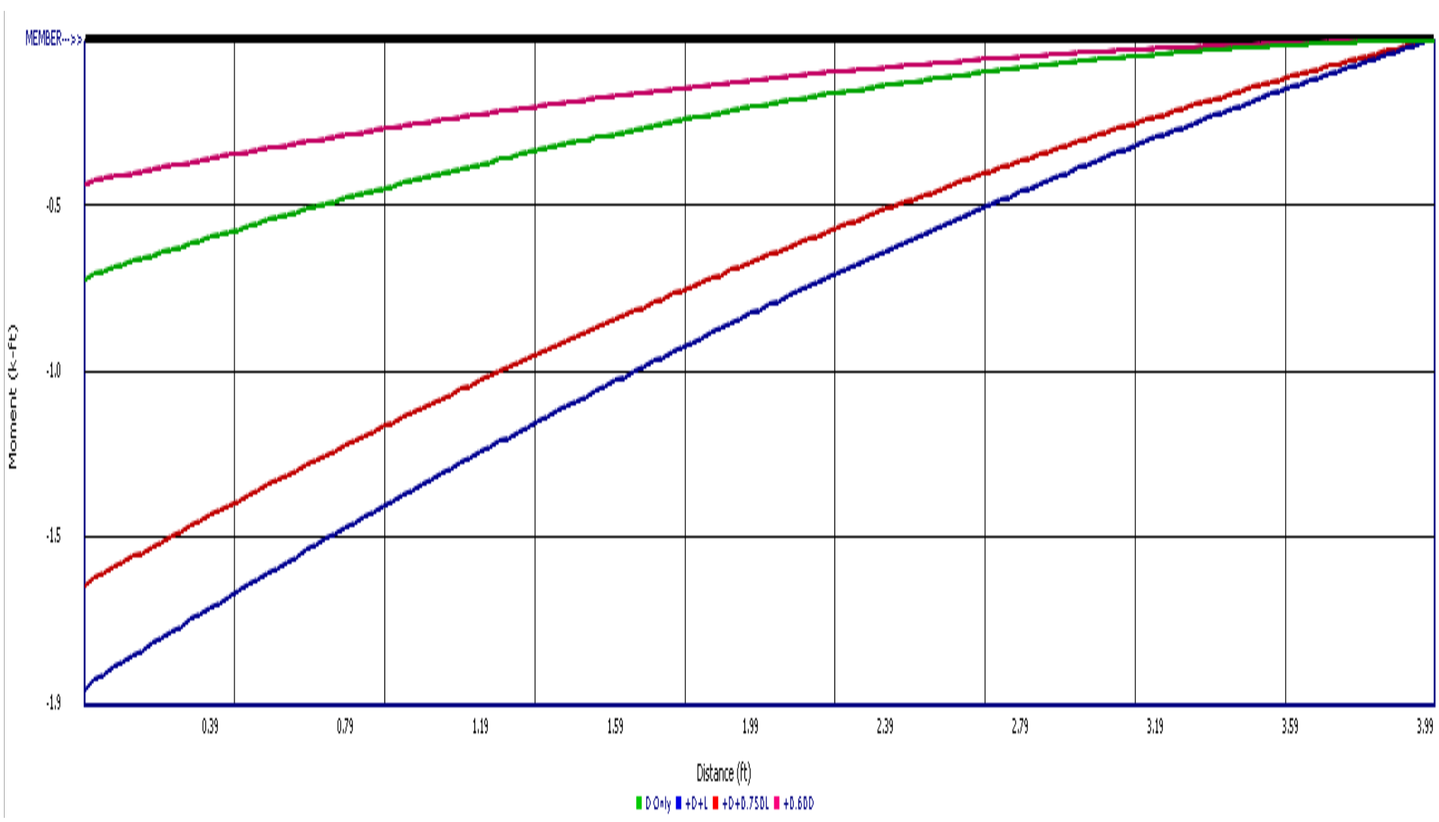
Vertical Reactions

Support notation : Far left is #1 | Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.630	
Max Upward from Load Combinations	0.630	
Max Upward from Load Cases	0.330	
D Only	0.330	
+D+L	0.630	
+D+0.750L	0.555	
+0.60D	0.198	
L Only	0.300	

Steel Section Properties : HSS10x2x3/8

Depth	=	10.000 in	I xx	=	71.70 in^4	J	=	15.900 in^4
			S xx		14.30 in^3	Cw	=	11.00 in^6
Width	=	2.000 in	R xx	=	3.080 in			
Wall Thick	=	0.349 in	Zx	=	20.300 in^3			
Area	=	7.580 in^2	I yy	=	4.700 in^4	C	=	0.000 in^3
Weight	=	27.406 plf	S yy	=	4.700 in^3			
			R yy	=	0.787 in			
			Zy	=	5.760 in^3			
Ycg	=	5.000 in						



Steel Beam

Project File: 24-264 Calculation.ec6

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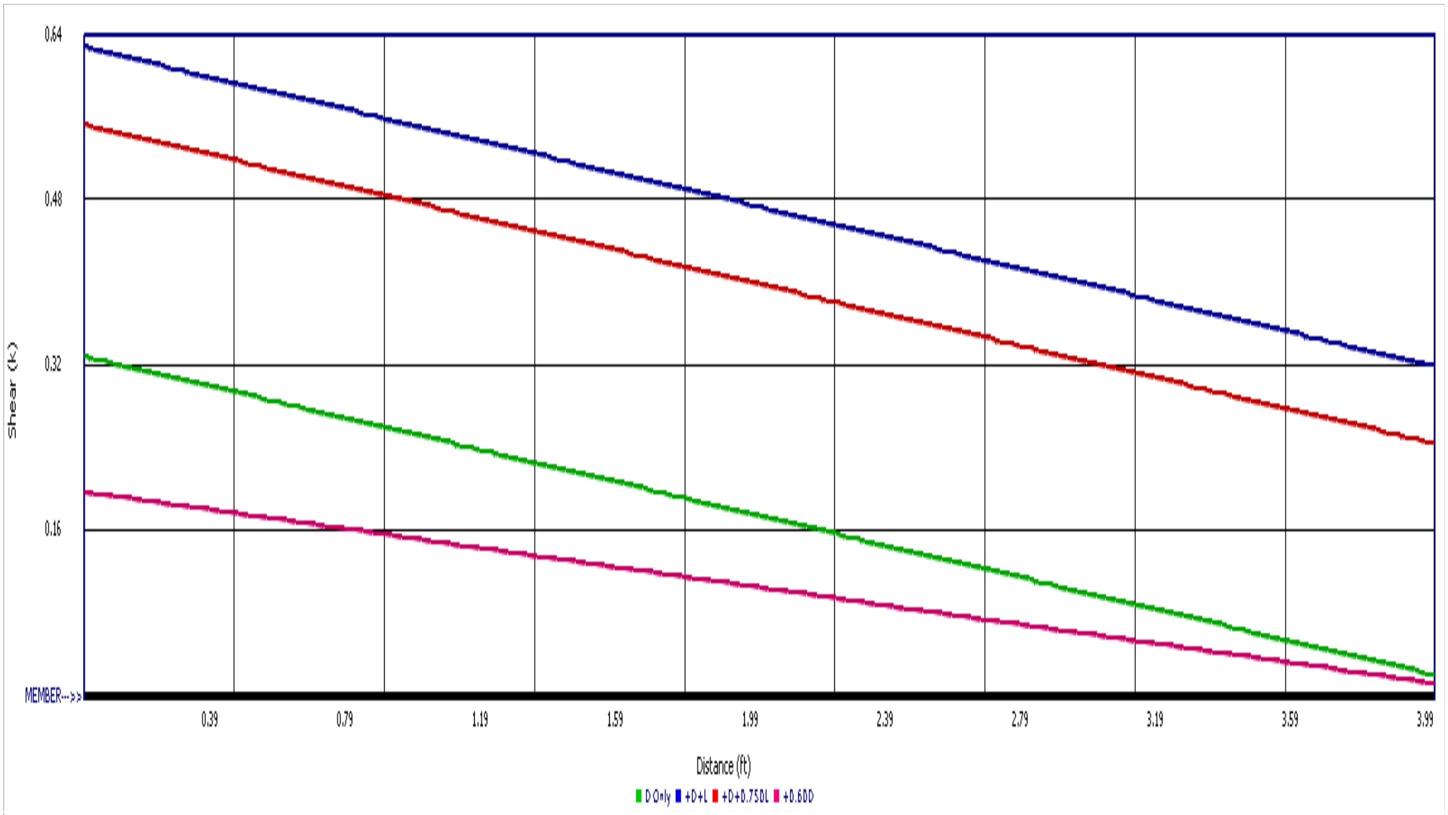
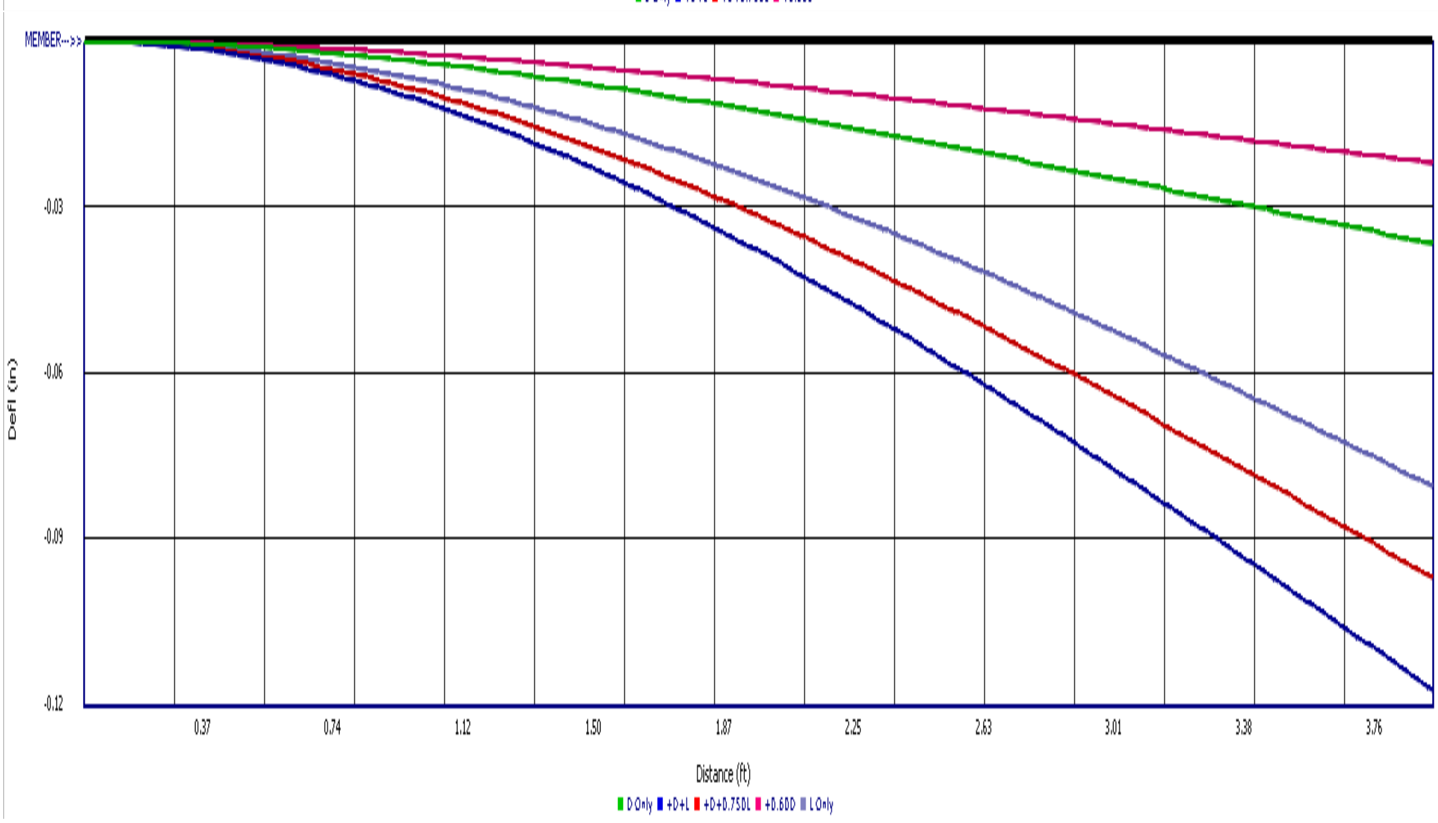
DESCRIPTION: Comb 3 DL + 300





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.330	-0.699

Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 2.0$ Width of Tube Steel (in)

$d := 10.0$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 1.90$ Moment (Kips-ft)

$V_{weld} := 0.63$ Shear (Kips)

$P_{weld} := 0.00$ Tension or Compression (Kips)

$T_{weld} := 0.00$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\frac{P_{weld}}{2(b+d)\cdot x} + \frac{M_{weld}\cdot 12}{\left(b\cdot d + \frac{d^2}{3}\right)\cdot x}\right]^2 + \left[\frac{1.5V_{weld}}{2(b+d)\cdot x} + \frac{T_{weld}\cdot 12\cdot\sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6}\cdot x}\right]^2} \ge F_v \\ \quad x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.03$ in

$S_{weld} := \left(b \cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 1.60$ in^3

$A_{weld} := 2\cdot(d + b)\cdot t_e$ $A_{weld} = 0.72$ in^2

$J_{weld} := \frac{(b + d)^3}{6}\cdot t_e$ $J_{weld} = 8.64$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$ $f_b = 14.25$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$ $f_v = 1.31$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$ $f_a = 0.00$ Ksi

Check Combined Stress :

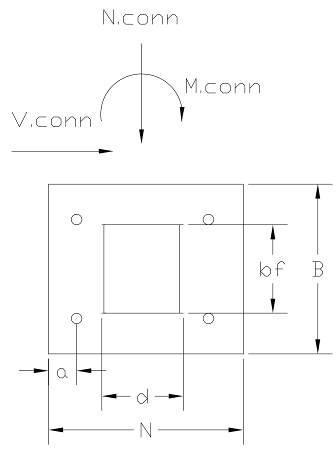
$$f_{weld} := \sqrt{\left(f_b + f_a\right)^2 + f_v^{\,2}}$$ $f_{weld} = 14.31$ ksi

$$STRESS := \begin{array}{|l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{array}$$ STRESS = "OK"

$$W_{weld} := \begin{array}{|l} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{array}$$

$W_{weld} = \text{"3/16"}$

Moment-Resisting Bearing Type Plate Connection Design



Loads Data:

$M := 1.90$ Moment in Connection (kips-ft)

$P := 0.0$ Axial in Connection (kips)

$V := 0.63$ Shear in Connection (kips)

Plate Data:

$B := 12.0$ Plate Width (in)

$N := 12.0$ Plate Lenght along Moment Direction (in)

$a := 1.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 2.0$ Depth of Column along Length (N) direction (in)

$b_f := 10.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate (Ksi)

$Fb_{plate} := 27.00$ Allowable Tensile Bending Stress of Plate (Ksi)
Fb=0.75*Fy

Allowable Support Stress Data:

$A_1 := 144.0$ Area of Plate Plate (in^2)

$A_2 := 144.0$ Area of Support conc foundation that is geometrically similar to Plate (in^2)

Allowable compressive strength of support (ksi)
0.35*f'c...for Full Area of Concrete Support
0.35*f'c*SQR(A2/A1) < or = 0.7*f'c.........for less than Full Area of Concrete Support

$(fc) := 4.00$ Concrete compressive strength (psi)

$$Fp_{supp} := 0.35 \cdot fc \cdot \sqrt{\frac{A_2}{A_1}}$$ $Fp_{supp} = 1.40$ ksi

$$e := \frac{12M}{\max(0.1, P)}$$ $e = 228.00$ in

Concrete Bearing Stress:

$$f_b := \begin{cases} \text{"INCREASE PLATE"} & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} & \text{if } e < \frac{N}{6} \\ \frac{2P}{3 \cdot \left(\frac{N}{2} - e\right) \cdot B} & \text{if } \frac{N}{6} < e < \frac{N}{2} \\ Fp_{supp} & \text{if } e \ge \frac{N}{2} \wedge \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \le Fp_{supp} \\ 0.00 & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \le 0.00 \end{cases}$$

$f_b = 1.4$ ksi

Compression Length:

$$Kd := \text{if}\left[e \le \frac{N}{6}, N, \begin{array}{l} x \leftarrow 0.01 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\frac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\frac{x^2}{3}\right) \le M \cdot 12 \\ \quad x \leftarrow x + 0.01 \end{array} \right]$$

$Kd = 0.25$

Tension in Bolts:

$$T_{bolts} := \begin{cases} 0.5 \cdot f_b \cdot Kd \cdot B - P & \\ 0 & \text{if } e \le \frac{N}{6} \end{cases}$$ $T_{bolts} = 2.10$ Kips

$$T_{Xbolt} := \frac{T_{bolts}}{2}$$ $T_{Xbolt} = 1.05$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2}$$ $m = 5.05$ in

$$n := \frac{B - 0.8 \cdot b_f}{2}$$ $n = 2.00$ in

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases}$$ $f_2 = 0$ ksi

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a) T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{2(f_b - f_2) B \cdot m^2}{3}, (m - a) T_{bolts}\right] & \text{if } Kd > m \end{cases}$$ $M_{plate} = 10.43$ kips − in

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}}$$ $t_{plate} = 0.44$ in

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Specifier's comments:

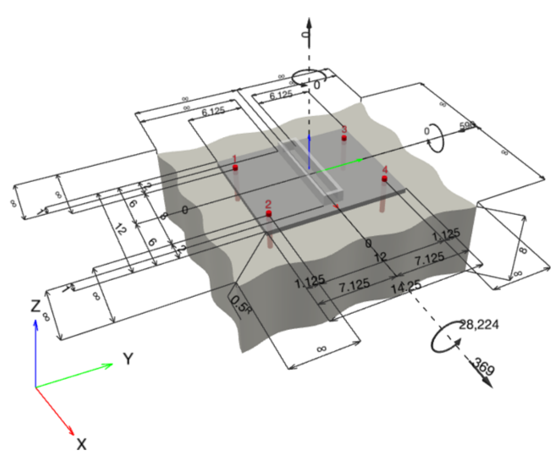
1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (4 1/4)**
Item number:	418076 KH-EZ 1/2"x5"
Specification text:	Hilti KH-EZ screw anchor with 4.25 in embedment, 1/2 (4 1/4), Carbon steel, installation per ESR-3027
Effective embedment depth:	$h_{ef,act}$ = 3.220 in., h_{nom} = 4.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 14.250 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X2X.375; (L x W x T) = 10.000 in. x 2.000 in. x 0.375 in.
Base material:	uncracked concrete, 3000, f_c' = 3,000 psi; h = 8.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Application also possible with KWIK-X 1/2 (3) hnom1 under the selected boundary conditions.
More information in section Alternative fastening data of this report.

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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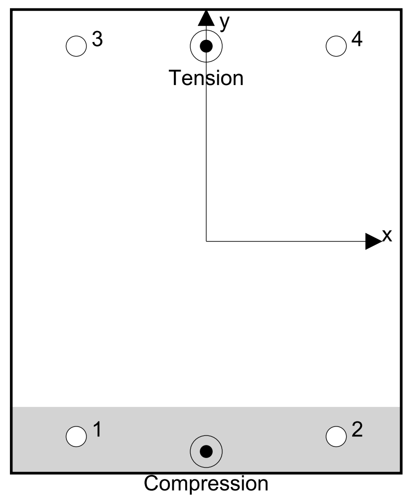
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 369; V_y = -590; M_x = 28,224; M_y = 0; M_z = 0;	no	23

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	174	92	-148
2	0	174	92	-148
3	1,133	174	92	-148
4	1,133	174	92	-148

Max. concrete compressive strain:	0.04 [‰]
Max. concrete compressive stress:	189 [psi]
Resulting tension force in (x/y)=(0.000/6.000):	2,265 [lb]
Resulting compression force in (x/y)=(0.000/-6.460):	2,265 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	**Capacity ϕ N_n [lb]**	**Utilization $\beta_N = N_{ua}/\phi$ N_n**	**Status**
Steel Strength*	1,133	11,778	10	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	2,265	10,154	23	OK

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

N_{sa} [lb]
18,120

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,120	0.650	11,778	1,133

3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.220	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.250	27	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
170.60	93.32	1.000	1.000	1.000	1.000	8,545

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
15,621	0.650	10,154	2,265

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	174	5,547	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	696	43,740	2	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor   **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

V_{sa} = ESR value	refer to ICC-ES ESR-3027
$\phi\ V_{steel} \geq V_{ua}$	ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

V_{sa} [lb]
9,245

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
9,245	0.600	5,547	174

4.2 Pryout Strength

$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.220	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	5.250	27	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
341.19	93.32	1.000	1.000	1.000	1.000	8,545

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
62,486	0.700	43,740	696

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.223	0.031	5/3	9	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc. (c) ENERCALC INC 1983-2023

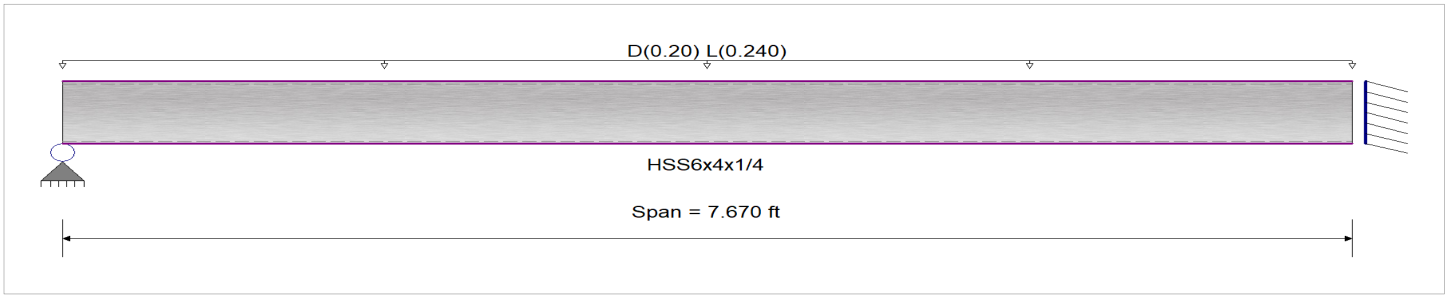
DESCRIPTION: Central Stringer DL + LL

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2006

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 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 NOT internally calculated and added
Uniform Load : D = 0.050, L = 0.060 ksf, Tributary Width = 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.165** : 1	Maximum Shear Stress Ratio =	**0.052** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	3.236 k-ft	Va : Applied	2.109 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	7.670 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.013 in	Ratio =	7,136	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : L Only
Max Downward Total Deflection	0.011 in	Ratio =	8564	>=240.	Span: 1 : D Only
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0	

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 7.67 ft	1	0.075	0.023	0.83	-1.47	1.47	32.70	19.58	1.00	1.00	0.96	68.18	40.83
+D+L													
Dsgn. L = 7.67 ft	1	0.165	0.052	1.82	-3.24	3.24	32.70	19.58	1.00	1.00	2.11	68.18	40.83
+D+0.750L													
Dsgn. L = 7.67 ft	1	0.143	0.045	1.57	-2.79	2.79	32.70	19.58	1.00	1.00	1.82	68.18	40.83
+0.60D													
Dsgn. L = 7.67 ft	1	0.045	0.014	0.50	-0.88	0.88	32.70	19.58	1.00	1.00	0.58	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	0.0129	3.243		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0107	3.243		0.0000	0.000
L Only	1	0.0129	3.243		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.266	2.109
Max Upward from Load Combinations	1.266	2.109
Max Upward from Load Cases	0.690	1.150

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc. (c) ENERCALC INC 1983-2023

DESCRIPTION: Central Stringer DL + LL

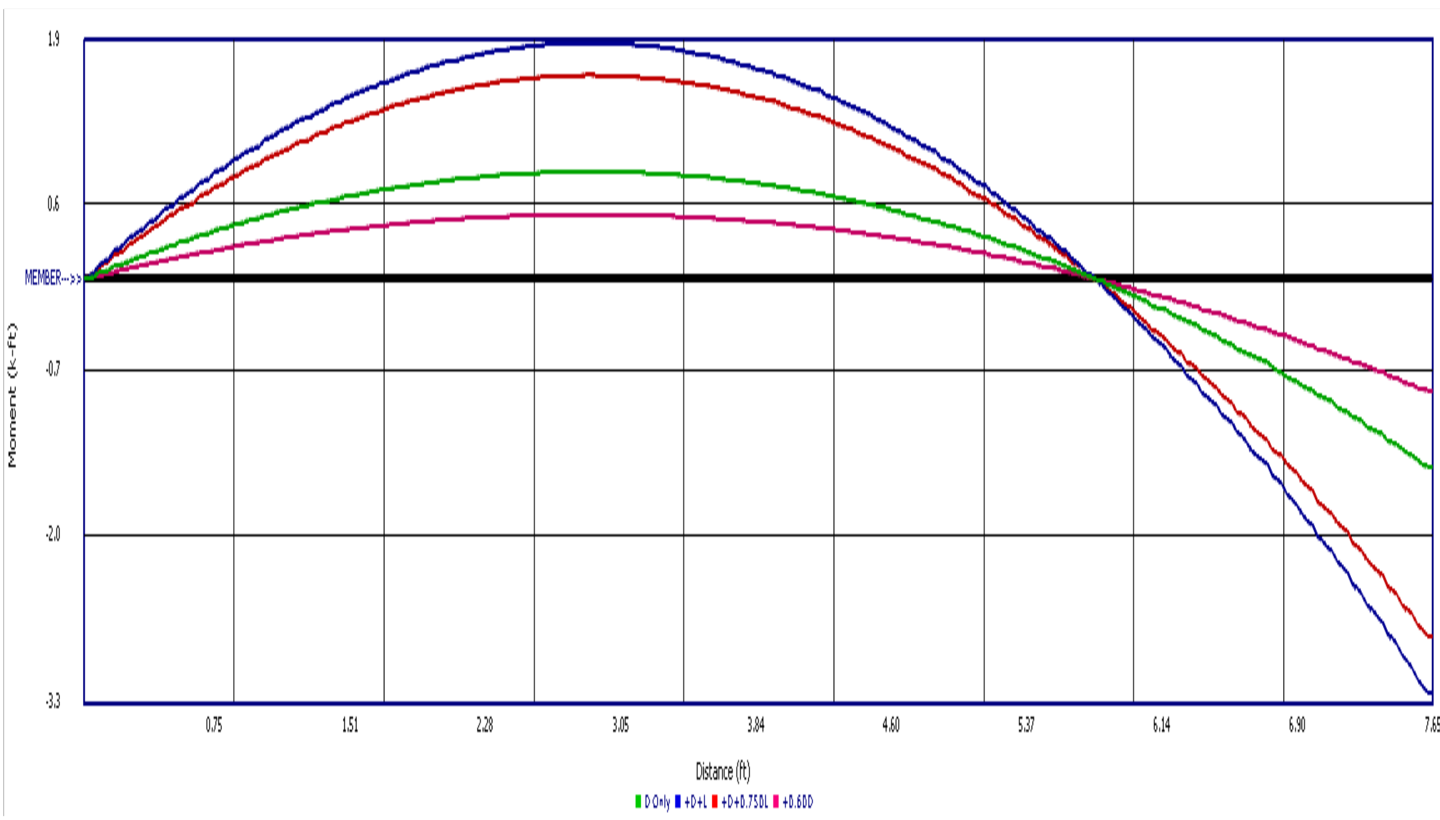
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D Only	0.575	0.959
+D+L	1.266	2.109
+D+0.750L	1.093	1.822
+0.60D	0.345	0.575
L Only	0.690	1.150

Steel Section Properties : HSS6x4x1/4

Depth	=	6.000	in	I xx	=	20.90	in^4	J	=	23.600	in^4
				S xx		6.96	in^3	Cw	=	10.10	in^6
Width	=	4.000	in	R xx	=	2.200	in				
Wall Thick	=	0.233	in	Zx	=	8.530	in^3				
Area	=	4.300	in^2	I yy	=	11.100	in^4	C	=	0.000	in^3
Weight	=	15.620	plf	S yy	=	5.560	in^3				
				R yy	=	1.610	in				
				Zy	=	6.450	in^3				
Ycg	=	3.000	in								

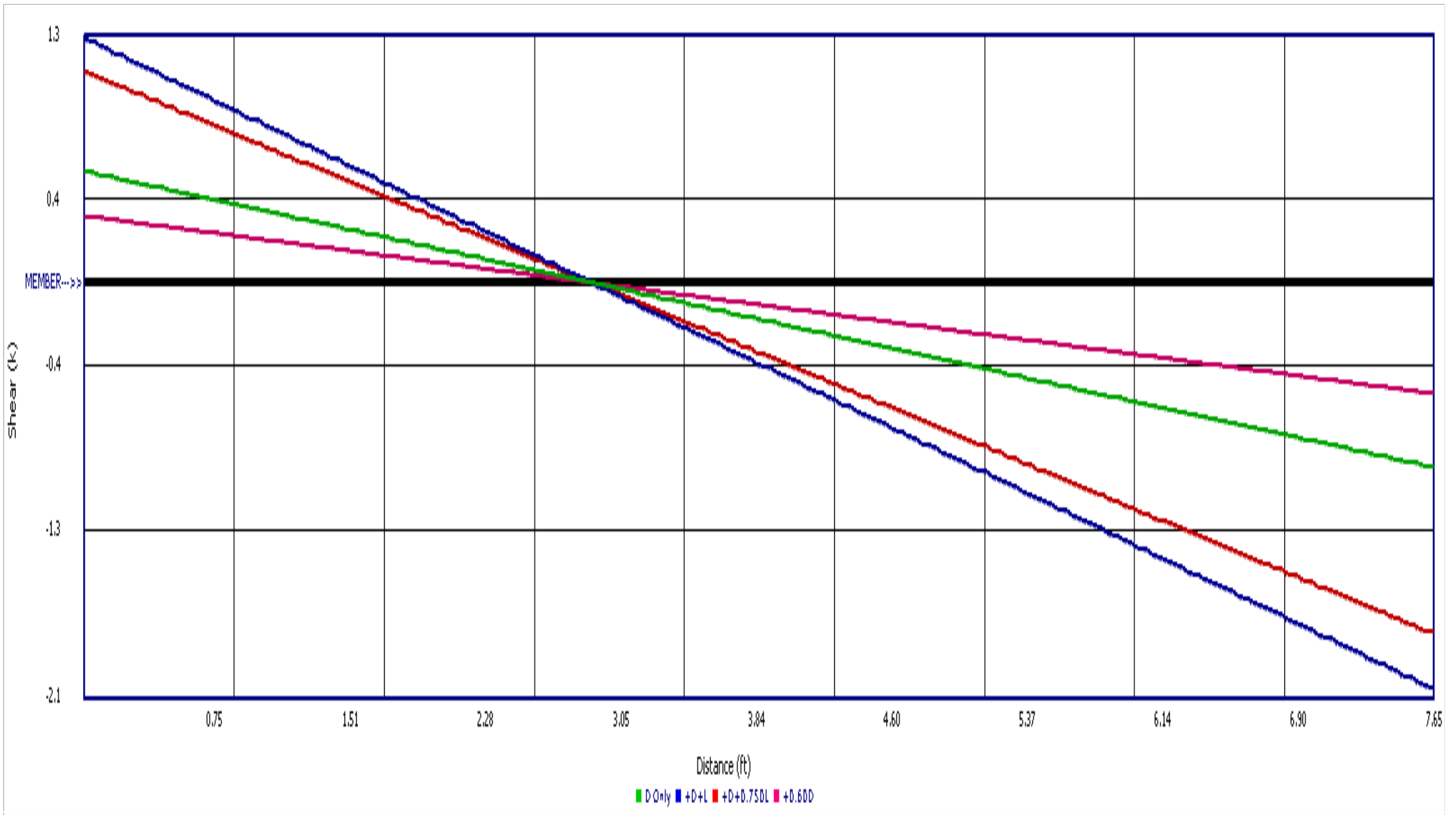


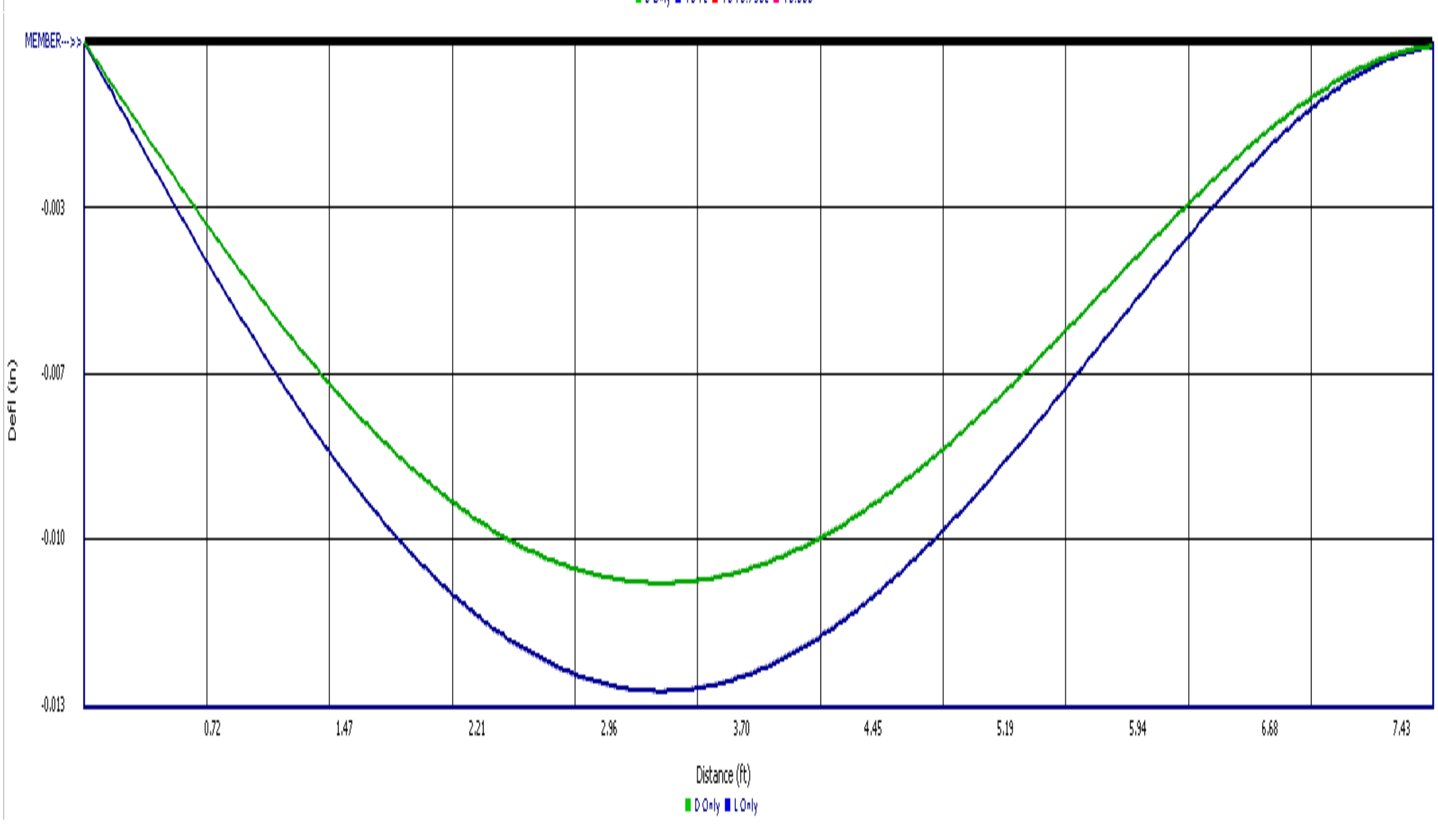
Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

DESCRIPTION: Central Stringer DL + LL





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.575	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam with Torsional Loads

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

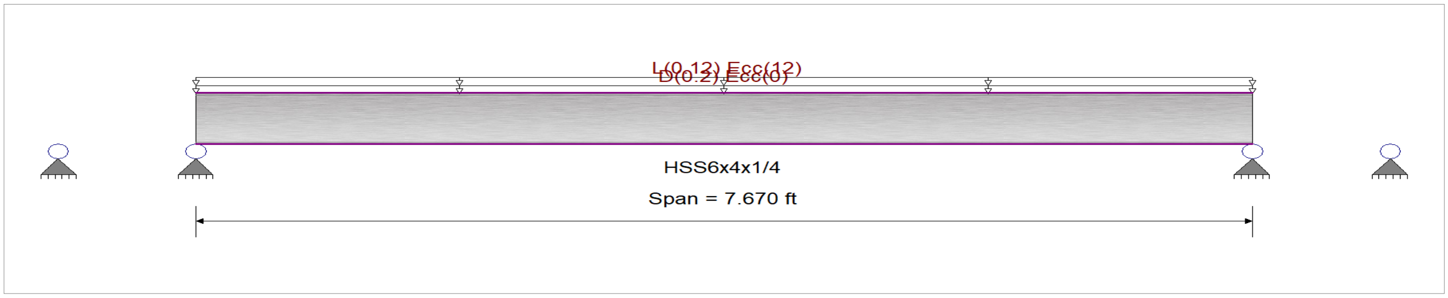
DESCRIPTION: Central Stringer DL + 0.5LL

CODE REFERENCES

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

Analysis Settings

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	42.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	ASCE 7-16		



Applied Loads

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

Load for Span Number 0
Uniform Load : D = 0.050 ksf, Tributary Width = 4.0 ft
Uniform Load : L = 0.060 ksf, Tributary Width = 2.0 ft, Tors Ecc. = 12.0 in

DESIGN SUMMARY

Design OK

Max. Flange Normal Stress Ratio =	**0.132** : 1	Maximum Shear Stress Ratio =	**0.036** : 1
Flange Normal Stress	4.06 ksi	Flange Shear Stress	0.55 ksi
Phi vn : Flange Normal Stress *(Phi Mn/Sxx)*	30.82 ksi	Web Shear Stress	0.99 ksi
		Vn/Omega : Allowable	15.09 ksi
Max Mu : Applied	2.35 k-ft		
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Load Combination	+D+L	Load Combination	+D+L

Maximum Rotation = 0.0275 deg at 3.74 ft

Maximum Deflection

Max Download Transient Load Deflection	0.016 in	Max Downward Total Deflection	0.041 in
Ratio =	5932	Ratio =	2224
Max Upward Transient Load Deflection	0.000 in	Max Upward Total Deflection	0.000 in
Ratio =	0 <360	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination	Max Stress Ratio	Bending Max Mu (kft)	Vu (k)	Flange Normal (ksi) Total	Allow	Ratio	Flange Shear (ksi) Total	Allow	Ratio	Web Shear (ksi) Total	Allow	Ratio
D Only	0.082	1.47	0.77	2.536	30.823	0.082		15.090		0.274	15.090	0.018
+D+L	0.132	2.35	1.23	4.057	30.823	0.132	0.548	15.090	0.036	0.987	15.090	0.065
+D+0.750L	0.119	2.13	1.11	3.677	30.823	0.119	0.411	15.090	0.027	0.809	15.090	0.054
+0.60D	0.049	0.88	0.46	1.521	30.823	0.049		15.090		0.165	15.090	0.011

Maximum Upward Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
span_1	1	0.0414	3.866		0.0000	0.000

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
Overall MAXimum Envelope	1	0.0005	0.031	0.0000	0.000
Overall MAXimum Envelope	1	0.0011	0.061	0.0000	0.000
Overall MAXimum Envelope	1	0.0016	0.092	0.0000	0.000
Overall MAXimum Envelope	1	0.0021	0.123	0.0000	0.000
Overall MAXimum Envelope	1	0.0026	0.153	0.0000	0.000
Overall MAXimum Envelope	1	0.0032	0.184	0.0000	0.000
Overall MAXimum Envelope	1	0.0037	0.215	0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam with Torsional Loads

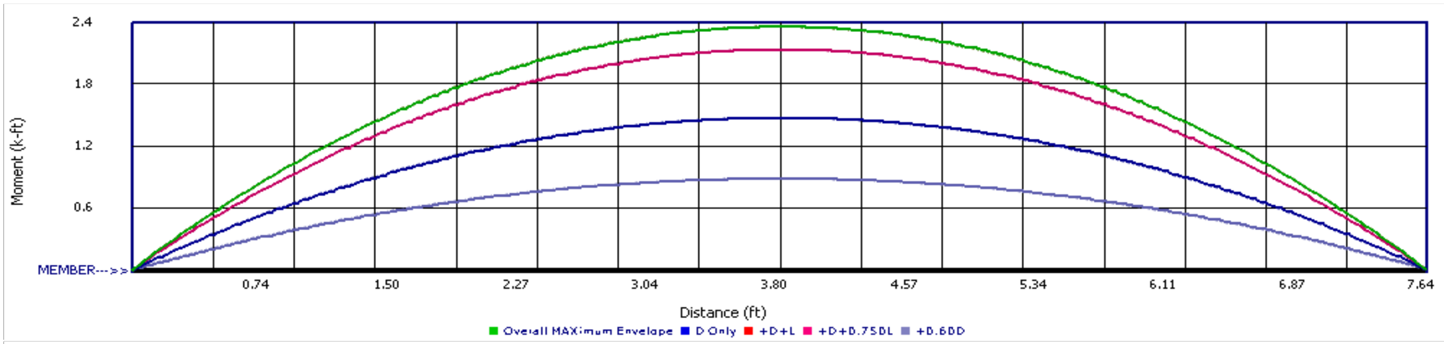
Project File: 24-264 Calculation.ec6

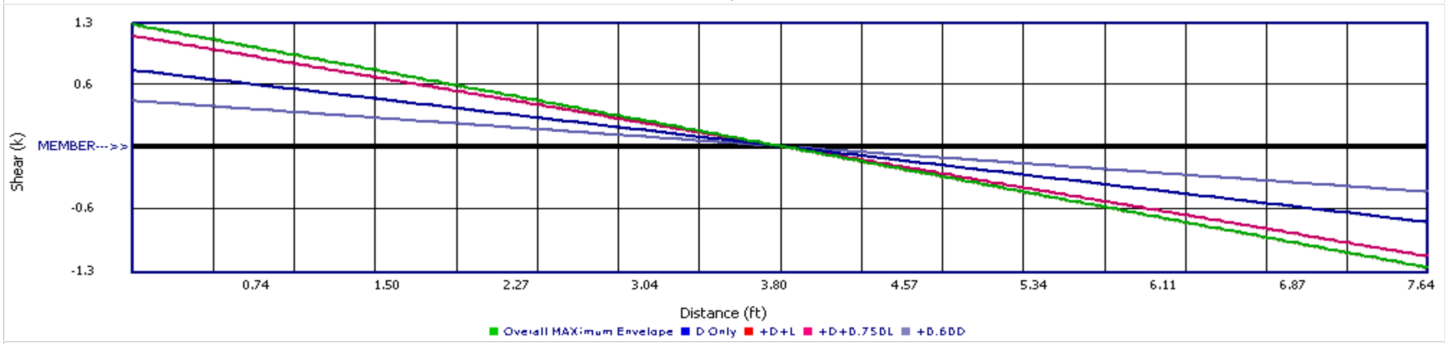
LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

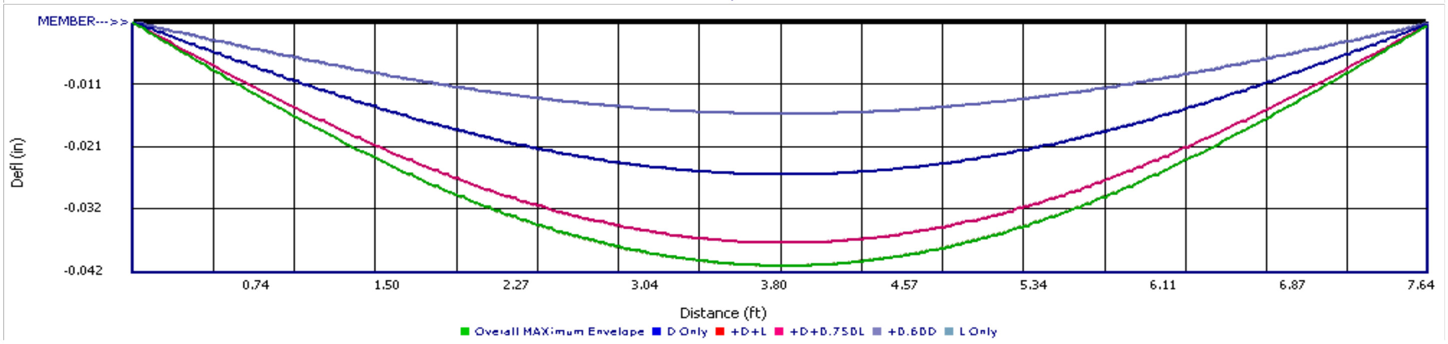
DESCRIPTION: Central Stringer DL + 0.5LL

Steel Section Properties HSS6x4x1/4

Depth	=	6.000 in	I xx	=	20.90 in^4	J	=	23.600 in^4
			S xx		6.96 in^3	Cw	=	10.10 in^6
Width	=	4.000 in	R xx	=	2.200 in			
Wall Thick	=	0.233 in	Zx	=	8.530 in^3			
Area	=	4.300 in^2	I yy	=	11.100 in^4			
Weight	=	15.620 plf	S yy	=	5.560 in^3			
			R yy	=	1.610 in			
			Zy	=	6.450 in^3			
Ycg	=	3.000 in						
						Ao	=	21.619 in^2





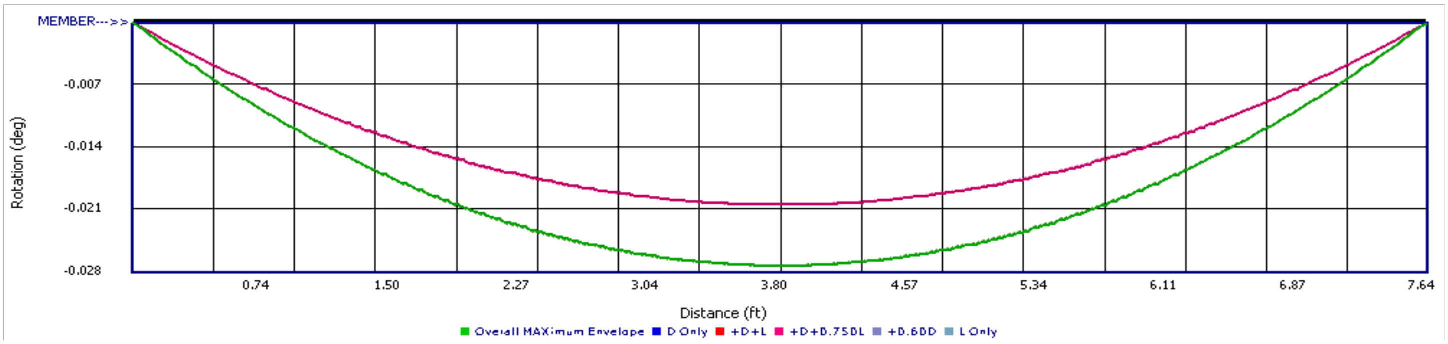


Steel Beam with Torsional Loads

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 | Arch-Deco Design & Construction Inc. | (c) ENERCALC INC 1983-2023

DESCRIPTION: Central Stringer DL + 0.5LL



Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc. (c) ENERCALC INC 1983-2023

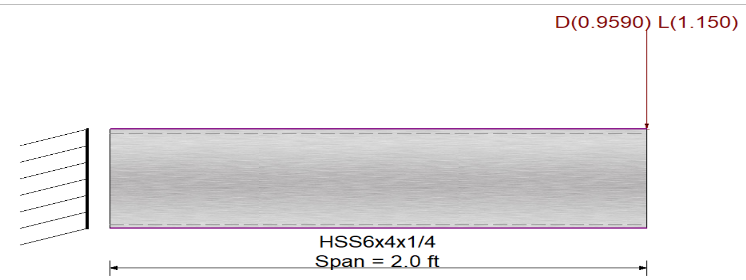
DESCRIPTION: Wall Support DL + LL

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2006

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 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 = 0.9590, L = 1.150 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.217** : 1	Maximum Shear Stress Ratio =	**0.052** : 1
Section used for this span	**HSS6x4x1/4**	Section used for this span	**HSS6x4x1/4**
Ma : Applied	4.249 k-ft	Va : Applied	2.140 k
Mn / Omega : Allowable	19.580 k-ft	Vn/Omega : Allowable	40.826 k
Load Combination	+D+L	Load Combination	+D+L
		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	0.009 in	Ratio =	5,501	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : L Only
Max Downward Total Deflection	0.007 in	Ratio =	6518	>=240.	Span: 1 : D Only
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0	

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 2.00 ft	1	0.100	0.024		-1.95	1.95	32.70	19.58	1.00	1.00	0.99	68.18	40.83
+D+L													
Dsgn. L = 2.00 ft	1	0.217	0.052		-4.25	4.25	32.70	19.58	1.00	1.00	2.14	68.18	40.83
+D+0.750L													
Dsgn. L = 2.00 ft	1	0.188	0.045		-3.67	3.67	32.70	19.58	1.00	1.00	1.85	68.18	40.83
+0.60D													
Dsgn. L = 2.00 ft	1	0.060	0.015		-1.17	1.17	32.70	19.58	1.00	1.00	0.59	68.18	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	0.0087	2.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0074	2.000		0.0000	0.000
L Only	1	0.0087	2.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.140	
Max Upward from Load Combinations	2.140	

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 | Arch-Deco Design & Construction Inc. | (c) ENERCALC INC 1983-2023

DESCRIPTION: Wall Support DL + LL

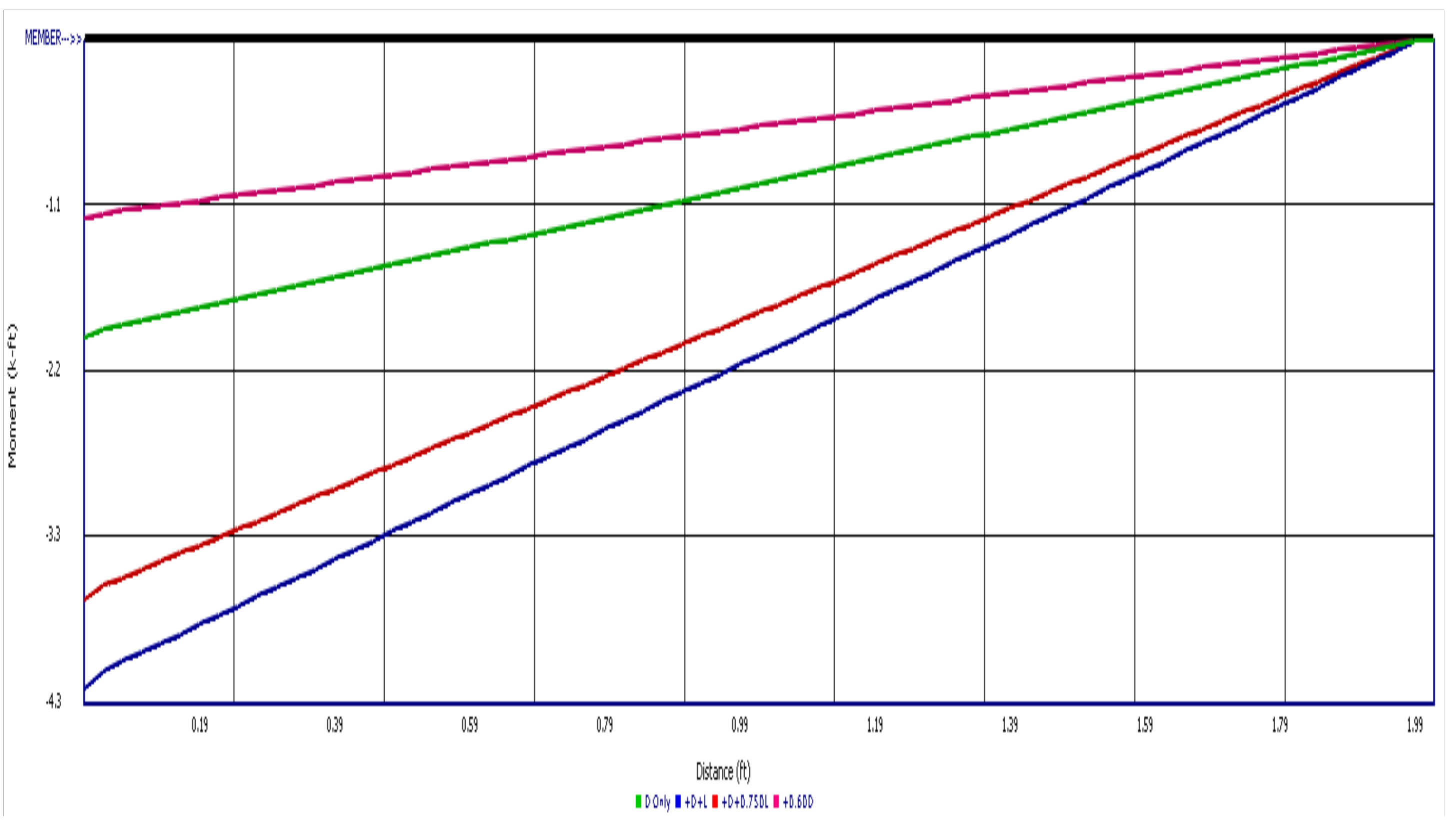
Vertical Reactions

Support notation : Far left is #1 | Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Cases	1.150	
D Only	0.990	
+D+L	2.140	
+D+0.750L	1.853	
+0.60D	0.594	
L Only	1.150	

Steel Section Properties : HSS6x4x1/4

Depth	=	6.000 in	I xx	=	20.90 in^4	J	=	23.600 in^4
			S xx		6.96 in^3	Cw	=	10.10 in^6
Width	=	4.000 in	R xx	=	2.200 in			
Wall Thick	=	0.233 in	Zx	=	8.530 in^3			
Area	=	4.300 in^2	I yy	=	11.100 in^4	C	=	0.000 in^3
Weight	=	15.620 plf	S yy	=	5.560 in^3			
			R yy	=	1.610 in			
			Zy	=	6.450 in^3			
Ycg	=	3.000 in						

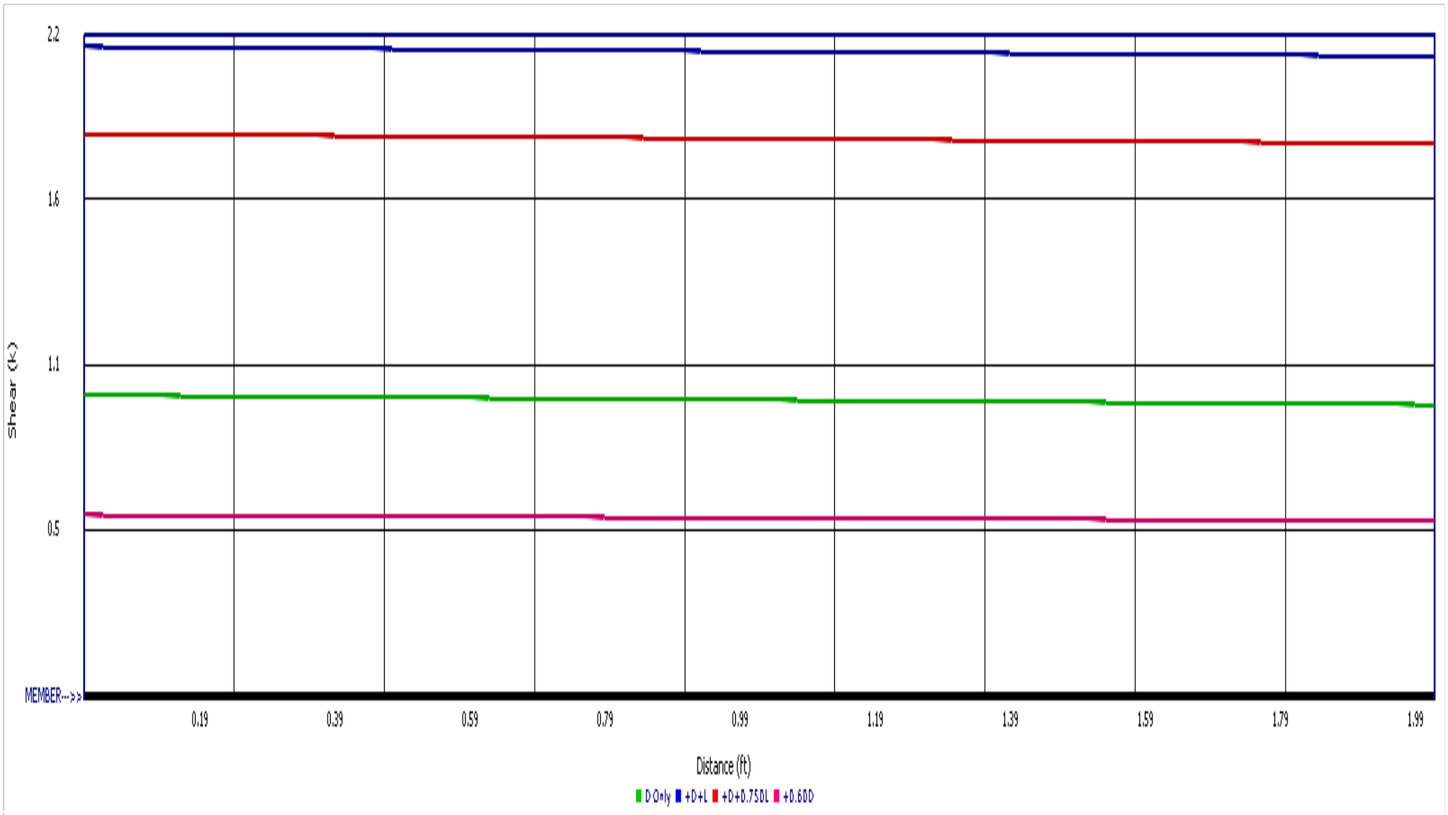


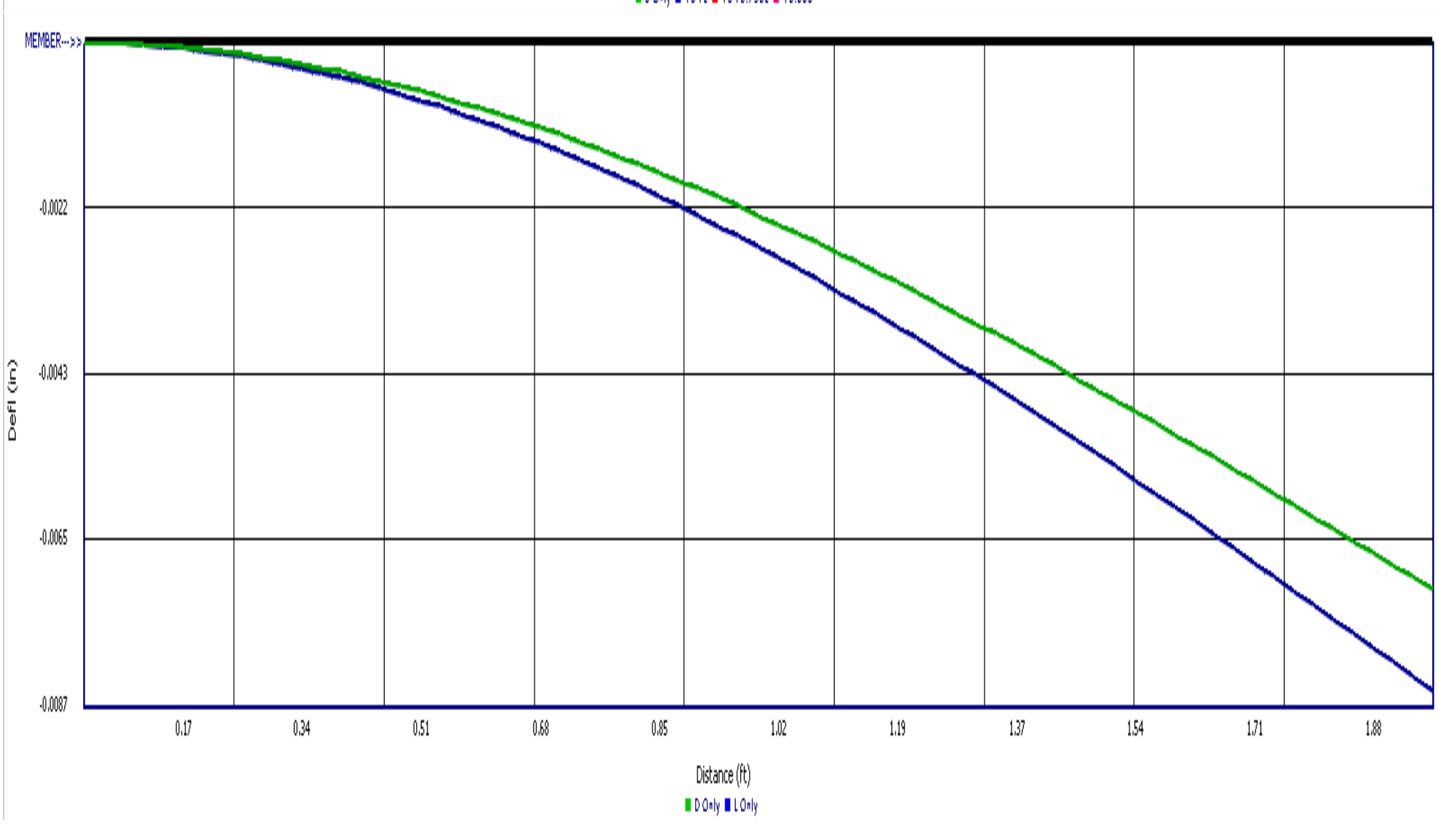
Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 Arch-Deco Design & Construction Inc.

DESCRIPTION: Wall Support DL + LL





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.990	-1.949

Steel Beam

DESCRIPTION: Wall Support DL + LL

Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.20	Span 1	0.987	-1.752
D Only	0.40	Span 1	0.984	-1.554
D Only	0.60	Span 1	0.981	-1.358
D Only	0.80	Span 1	0.978	-1.162
D Only	1.00	Span 1	0.975	-0.967
D Only	1.20	Span 1	0.971	-0.772
D Only	1.40	Span 1	0.968	-0.578
D Only	1.60	Span 1	0.965	-0.385
D Only	1.80	Span 1	0.962	-0.192
D Only	2.00	Span 1	0.959	0.000
+D+L	0.00	Span 1	2.140	-4.249
+D+L	0.20	Span 1	2.137	-3.822
+D+L	0.40	Span 1	2.134	-3.394
+D+L	0.60	Span 1	2.131	-2.968
+D+L	0.80	Span 1	2.128	-2.542
+D+L	1.00	Span 1	2.125	-2.117
+D+L	1.20	Span 1	2.121	-1.692
+D+L	1.40	Span 1	2.118	-1.268
+D+L	1.60	Span 1	2.115	-0.845
+D+L	1.80	Span 1	2.112	-0.422
+D+L	2.00	Span 1	2.109	0.000
+D+0.750L	0.00	Span 1	1.853	-3.674
+D+0.750L	0.20	Span 1	1.850	-3.304
+D+0.750L	0.40	Span 1	1.846	-2.934
+D+0.750L	0.60	Span 1	1.843	-2.565
+D+0.750L	0.80	Span 1	1.840	-2.197
+D+0.750L	1.00	Span 1	1.837	-1.829
+D+0.750L	1.20	Span 1	1.834	-1.462
+D+0.750L	1.40	Span 1	1.831	-1.096
+D+0.750L	1.60	Span 1	1.828	-0.730
+D+0.750L	1.80	Span 1	1.825	-0.365
+D+0.750L	2.00	Span 1	1.822	0.000
+0.60D	0.00	Span 1	0.594	-1.170
+0.60D	0.20	Span 1	0.592	-1.051
+0.60D	0.40	Span 1	0.590	-0.933
+0.60D	0.60	Span 1	0.589	-0.815
+0.60D	0.80	Span 1	0.587	-0.697
+0.60D	1.00	Span 1	0.585	-0.580
+0.60D	1.20	Span 1	0.583	-0.463
+0.60D	1.40	Span 1	0.581	-0.347
+0.60D	1.60	Span 1	0.579	-0.231
+0.60D	1.80	Span 1	0.577	-0.115
+0.60D	2.00	Span 1	0.575	0.000

Beam Span Deflections at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Deflection (in)
Overall MAXimum Envelope	0.00	Span 1	0.000
Overall MAXimum Envelope	0.21	Span 1	0.000
Overall MAXimum Envelope	0.41	Span 1	0.001
Overall MAXimum Envelope	0.62	Span 1	0.001
Overall MAXimum Envelope	0.82	Span 1	0.002
Overall MAXimum Envelope	1.03	Span 1	0.003
Overall MAXimum Envelope	1.23	Span 1	0.004
Overall MAXimum Envelope	1.44	Span 1	0.005
Overall MAXimum Envelope	1.65	Span 1	0.006
Overall MAXimum Envelope	1.85	Span 1	0.008
Overall MAXimum Envelope	2.00	Span 1	0.009
D Only	0.00	Span 1	0.000
D Only	0.21	Span 1	0.000
D Only	0.41	Span 1	0.000
D Only	0.62	Span 1	0.001

Steel Beam

Project File: 24-264 Calculation.ec6

LIC# : KW-06011763, Build:20.23.05.01 | Arch-Deco Design & Construction Inc. | (c) ENERCALC INC 1983-2023

DESCRIPTION: Wall Support DL + LL

Beam Span Deflections at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Deflection (in)
D Only	0.82	Span 1	0.002
D Only	1.03	Span 1	0.002
D Only	1.23	Span 1	0.003
D Only	1.44	Span 1	0.004
D Only	1.65	Span 1	0.005
D Only	1.85	Span 1	0.007
D Only	2.00	Span 1	0.007
L Only	0.00	Span 1	0.000
L Only	0.21	Span 1	0.000
L Only	0.41	Span 1	0.001
L Only	0.62	Span 1	0.001
L Only	0.82	Span 1	0.002
L Only	1.03	Span 1	0.003
L Only	1.23	Span 1	0.004
L Only	1.44	Span 1	0.005
L Only	1.65	Span 1	0.006
L Only	1.85	Span 1	0.008
L Only	2.00	Span 1	0.009

Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 6.0$ Width of Tube Steel (in)

$d := 4.0$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 1.8$ Moment (Kips-ft)

$V_{weld} := 1.75$ Shear (Kips)

$P_{weld} := 0.00$ Tension or Compression (Kips)

$T_{weld} := 3.2$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\frac{P_{weld}}{2(b+d)\cdot x} + \frac{M_{weld}\cdot 12}{\left(b\cdot d + \frac{d^2}{3}\right)\cdot x}\right]^2 + \left[\frac{1.5V_{weld}}{2(b+d)\cdot x} + \frac{T_{weld}\cdot 12\cdot\sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6}\cdot x}\right]^2} \ge F_v \\ \quad x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.06$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 1.76$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $A_{weld} = 1.20$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $J_{weld} = 10.00$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$ $f_b = 12.27$ ksi

Check Shear Stress :

$$f_v := \frac{1.5V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$ $f_v = 16.03$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$ $f_a = 0.00$ Ksi

Check Combined Stress :

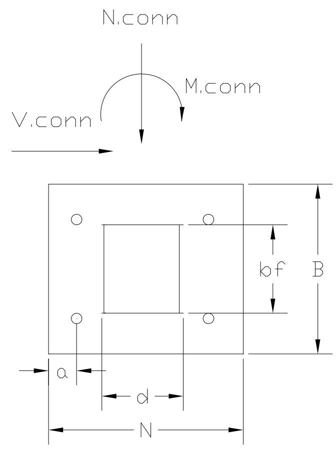
$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^{\,2}}$$ $f_{weld} = 20.19$ ksi

$$STRESS := \begin{array}{|l} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{array}$$ STRESS = "OK"

$$W_{weld} := \begin{array}{|l} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{array}$$

$W_{weld} =$ "3/16"

Moment-Resisting Bearing Type Plate Connection Design



Loads Data:

$M := 1.8$ Moment in Connection (kips-ft)

$P := 0.0$ Axial in Connection (kips)

$V := 1.75$ Shear in Connection (kips)

Plate Data:

$B := 12.0$ Plate Width (in)

$N := 20.0$ Plate Lenght along Moment Direction (in)

$a := 2.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 2.0$ Depth of Column along Length (N) direction (in)

$b_f := 10.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate (Ksi)

$Fb_{plate} := 27.00$ Allowable Tensile Bending Stress of Plate (Ksi)
Fb=0.75*Fy

Allowable Support Stress Data:

$A_1 := 240.0$ Area of Plate Plate (in^2)

$A_2 := 240.0$ Area of Support conc foundation that is geometrically similar to Plate (in^2)

Allowable compressive strength of support (ksi)
0.35*f'c...for Full Area of Concrete Support
0.35*f'c*SQR(A2/A1) < or = 0.7*f'c.........for less than Full Area of Concrete Support

$(fc) := 4.00$ Concrete compressive strength (psi)

$$Fp_{supp} := 0.35 \cdot fc \cdot \sqrt{\frac{A_2}{A_1}}$$ $Fp_{supp} = 1.40$ ksi

$$e := \frac{12M}{\max(0.1, P)}$$ $e = 216.00$ in

Concrete Bearing Stress:

$$f_b := \begin{cases} \text{"INCREASE PLATE"} & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} & \text{if } e < \frac{N}{6} \\ \frac{2P}{3 \cdot \left(\frac{N}{2} - e\right) \cdot B} & \text{if } \frac{N}{6} < e < \frac{N}{2} \\ Fp_{supp} & \text{if } e \ge \frac{N}{2} \wedge \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \le Fp_{supp} \\ 0.00 & \text{if } \frac{P}{B \cdot N} + \frac{6(M \cdot 12)}{B \cdot N^2} \le 0.00 \end{cases}$$

$f_b = 1.4$ ksi

Compression Length:

$$Kd := \text{if}\left[e \le \frac{N}{6}, N, \begin{array}{|l} x \leftarrow 0.01 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\frac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\frac{x^2}{3}\right) \le M \cdot 12 \\ \quad x \leftarrow x + 0.01 \end{array}\right]$$

$Kd = 0.15$

Tension in Bolts:

$$T_{bolts} := \begin{cases} 0.5 \cdot f_b \cdot Kd \cdot B - P \\ 0 & \text{if } e \le \frac{N}{6} \end{cases}$$ $T_{bolts} = 1.26$ Kips

$$T_{Xbolt} := \frac{T_{bolts}}{2}$$ $T_{Xbolt} = 0.63$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2} \qquad m = 9.05 \quad \text{in}$$

$$n := \frac{B - 0.8 \cdot b_f}{2} \qquad n = 2.00 \quad \text{in}$$

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases} \qquad f_2 = 0 \quad \text{ksi}$$

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a)T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{2(f_b - f_2)B \cdot m^2}{3}, (m - a)T_{bolts}\right] & \text{if } Kd > m \end{cases} \qquad M_{plate} = 11.34 \quad \text{kips} - \text{in}$$

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}} \qquad t_{plate} = 0.46 \quad \text{in}$$

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Oct 4, 2024	Date:	10/4/2024
Fastening point:			

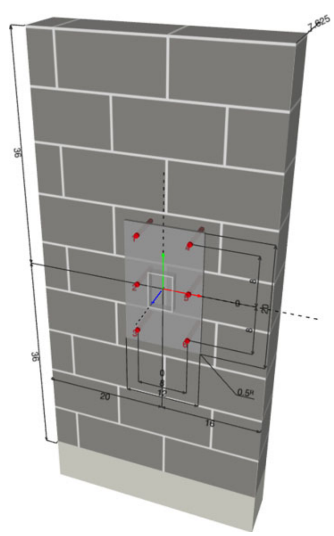
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/4 (6 1/4)**
Item number:	418084 KH-EZ 3/4"x5 1/2"
Specification text:	Hilti KH-EZ screw anchor with 4.84 in embedment, 3/4 (6 1/4), Steel galvanized, installation per ESR-3056
Effective embedment depth:	h_{ef} = 4.840 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3056
Issued I Valid:	11/1/2023 | 10/1/2024
Proof:	Design Method LRFD (AC01) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 12.000 in. x 20.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS6X4X.250; (L x W x T) = 6.000 in. x 4.000 in. x 0.250 in.
Base material:	uncracked Grout-filled CMU, f=3000, f = 2,999 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Solid Head Joint: no; open ended unit: no
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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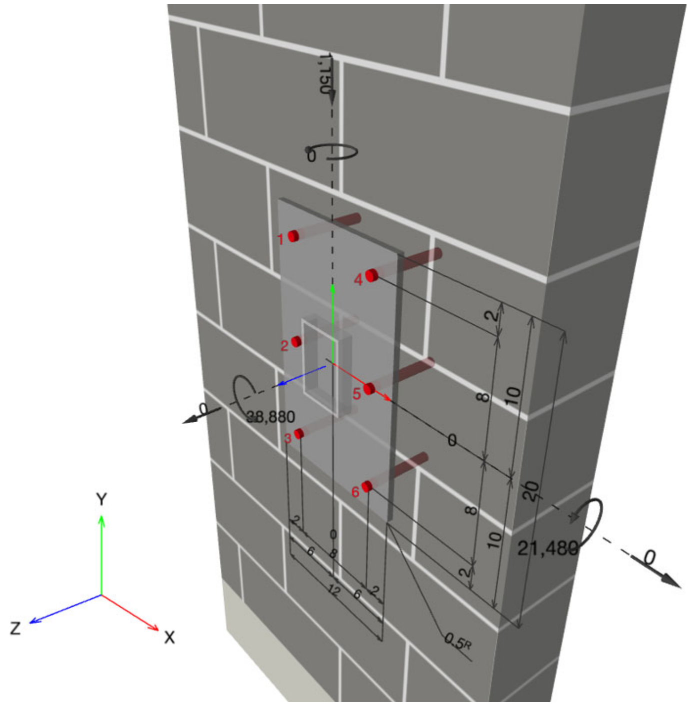
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 0; V_y = -1,150; M_x = 21,480; M_y = 0; M_z = 38,880;	no	90

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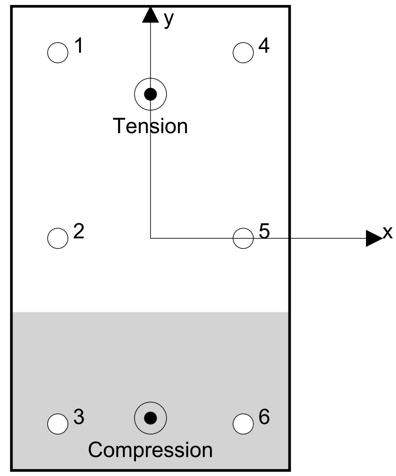
2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	597	1,087	-884	-633
2	172	633	0	-633
3	0	1,087	884	-633
4	597	918	-884	250
5	172	250	0	250
6	0	918	884	250

Max. concrete compressive strain: 0.03 [‰]
Max. concrete compressive stress: 38 [psi]
Resulting tension force in (x/y)=(0.000/6.212): 1,539 [lb]
Resulting compression force in (x/y)=(0.000/-7.745): 1,539 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	597	20,809	3	OK
Pullout	597	3,514	18	OK
Masonry breakout	770	3,063	26	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ESR-3056
$\phi\ N_{sa} \geq N_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

N_{sa} [lb]	ϕ	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
32,013	0.650	20,809	597

3.2 Pullout strength

$\phi\ N_{pn} \geq N_{ua}$ AC01 Table 3.2 + ACI 318-19 + Table 17.5.2

N_p [lb]	ϕ	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
6,390	0.550	3,514	597

3.3 Masonry breakout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec,N,m}\ \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ — ACI 318-19 Eq. (17.6.5.1b)

$\phi\ N_{mbg} \geq N_{ua}$ — AC01 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec,N,m} = \left(\frac{1}{1 + \frac{2\ e_N'}{3 \cdot h_{ef}}} \right) \leq 1.00$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f_m'} \cdot h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N,m}$ [in.]	$e_{c2,N,m}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_c
4.840	0.000	2.212	3.813	1.000	17

λ_a	f_m' [psi]
1.000	2,999

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
180.16	210.83	1.000	0.767	0.858	9,914

Results

N_{mbg} [lb]	ϕ	$\phi\ N_{mbg}$ [lb]	N_{ua} [lb]
5,568	0.550	3,063	770

4 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	1,087	6,180	18	OK
Pryout	1,087	5,085	22	OK
Masonry crushing strength	1,087	5,124	22	OK
Masonry breakout in direction y-	2,096	2,414	87	OK

4.1 Steel strength

V_{sa} = ESR value refer to ICC-ES ESR-3056

$\phi\ V_{sa} \geq V_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
10,299	0.600	6,180	1,087

4.2 Pryout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec1,N,m} \cdot \psi_{ec2,N,m} \cdot \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ ACI 318-19 Eq. (17.6.5.1b)

$\phi\ V_{mpg} \geq V_{ua}$ AC01 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nm0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c1,N}}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ec1,N,m} = \left(\frac{1}{1 + \frac{e_{c2,N}}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N,m} = 0.7 + 0.3 \left(\frac{c}{c_{cr,N}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_c	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	4.840	0.000	0.000	3.813	1.000

k_m	λ_a
17	1.000

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N}$	$N_{b,m}$ [lb]
90.08	210.83	1.000	1.000	0.858	9,914

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
7,265	0.700	5,085	1,087

4.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ — AC01 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ — AC01 Eq. (3-1)

Variables

f'_m [psi]	$A_{se,v}$ [in.^{2}]
2,999	0.39

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
10,247	0.500	5,124	1,087

4.4 Masonry breakout strength y-

$V_{mbg} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{ec,V,m}\ \psi_{ed,V,m}\ \psi_{c,V,m}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{mbg} \geq V_{ua}$ — AC01 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
4.840	0.750	5.083	3.813	61.00	116.28	2,999

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V,m}$	$\psi_{c,V,m}$	$\psi_{h,V,m}$	$V_{b,m}$ [lb]
0.850	1.000	0.000	1.000	1.400	1.000	5,525

Results

V_{mbg} [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
3,449	0.700	2,414	2,096

5 Required verifications under combined tension and shear forces

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.251	0.868	5/3	90	OK

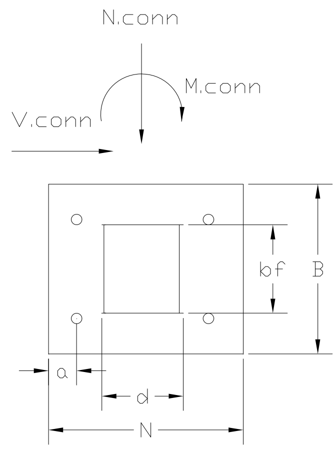
$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!

Fastening meets the design criteria!

Moment-Resisting Bearing Type Plate Connection Design



Loads Data:

$M := 0.0$ Moment in Connection (kips-ft)

$P := 1.27$ Axial in Connection (kips)

$V := 0.0$ Shear in Connection (kips)

Plate Data:

$B := 12.0$ Plate Width (in)

$N := 22.0$ Plate Lenght along Moment Direction (in)

$a := 2.0$ Bolts Distance from plate Edge along Length (N) direction (in)

$d := 2.0$ Depth of Column along Length (N) direction (in)

$b_f := 10.0$ Flange Width of Column along Width (B) direction (in)

$Fy_{plate} := 36.00$ Yield Strength of Plate (Ksi)

$Fb_{plate} := 27.00$ Allowable Tensile Bending Stress of Plate (Ksi)
Fb=0.75*Fy

Allowable Support Stress Data:

$A_1 := 264.0$ Area of Plate Plate (in^2)

$A_2 := 264.0$ Area of Support conc foundation that is geometrically similar to Plate (in^2)

Allowable compressive strength of support (ksi)
0.35*f'c...for Full Area of Concrete Support
0.35*f'c*SQR(A2/A1) < or = 0.7*f'c.........for less than Full Area of Concrete Support

$(fc) := 4.00$ Concrete compressive strength (psi)

$$Fp_{supp} := 0.35 \cdot fc \cdot \sqrt{\frac{A_2}{A_1}}$$ $Fp_{supp} = 1.40$ ksi

$$e := \frac{12M}{\max(0.1, P)}$$ $e = 0.00$ in

Concrete Bearing Stress:

$$f_b := \begin{array}{|l} \text{"INCREASE PLATE"} \quad \text{if } \dfrac{P}{B \cdot N} + \dfrac{6(M \cdot 12)}{B \cdot N^2} > Fp_{supp} \\ \dfrac{P}{B \cdot N} + \dfrac{6(M \cdot 12)}{B \cdot N^2} \quad \text{if } e < \dfrac{N}{6} \\ \dfrac{2P}{3 \cdot \left(\dfrac{N}{2} - e\right) \cdot B} \quad \text{if } \dfrac{N}{6} < e < \dfrac{N}{2} \\ Fp_{supp} \quad \text{if } e \ge \dfrac{N}{2} \wedge \dfrac{P}{B \cdot N} + \dfrac{6(M \cdot 12)}{B \cdot N^2} \le Fp_{supp} \\ 0.00 \quad \text{if } \dfrac{P}{B \cdot N} + \dfrac{6(M \cdot 12)}{B \cdot N^2} \le 0.00 \end{array}$$

$f_b = 4.811 \times 10^{-}$ ksi

Compression Length:

$$Kd := \text{if}\left[e \le \frac{N}{6}, N, \begin{array}{|l} x \leftarrow 0.01 \\ \text{while } (N - a) \cdot (0.5 \cdot f_b \cdot x \cdot B - P) + P \cdot \left(\dfrac{N}{2}\right) - 0.5 \cdot f_b \cdot B \cdot \left(\dfrac{x^2}{3}\right) \le M \cdot 12 \\ \quad x \leftarrow x + 0.01 \end{array} \right]$$

$Kd = 22.00$

Tension in Bolts:

$$T_{bolts} := \begin{array}{|l} 0.5 \cdot f_b \cdot Kd \cdot B - P \\ 0 \quad \text{if } e \le \dfrac{N}{6} \end{array}$$

$T_{bolts} = 0.00$ Kips

$$T_{Xbolt} := \frac{T_{bolts}}{2}$$

$T_{Xbolt} = 0.00$ Kips

Plate Design:

$$m := \frac{N - 0.95 \cdot d}{2} \qquad m = 10.05 \quad \text{in}$$

$$n := \frac{B - 0.8 \cdot b_f}{2} \qquad n = 2.00 \quad \text{in}$$

$$f_2 := \begin{cases} 0 & \text{if } Kd \le m \\ \dfrac{(Kd - m) \cdot f_b}{Kd} & \text{if } Kd > m \wedge e > \dfrac{N}{6} \\ f_b & \text{if } Kd > m \wedge e \le \dfrac{N}{6} \end{cases} \qquad f_2 = 4.811 \times 1\,\text{ksi}^3$$

$$M_{plate} := \begin{cases} \max\left[0.5 f_b \cdot Kd \cdot B \cdot \left(m - \dfrac{Kd}{3}\right), (m - a) T_{bolts}\right] & \text{if } Kd \le m \\ \max\left[\dfrac{f_2 \cdot m^2 \cdot B}{2} + \dfrac{2(f_b - f_2) B \cdot m^2}{3}, (m - a) T_{bolts}\right] & \text{if } Kd > m \end{cases} \qquad M_{plate} = 2.92 \quad \text{kips} - \text{in}$$

$$t_{plate} := \sqrt{\frac{6 \cdot M_{plate}}{Fb_{plate} \cdot B}} \qquad t_{plate} = 0.23 \quad \text{in}$$

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Specifier's comments:

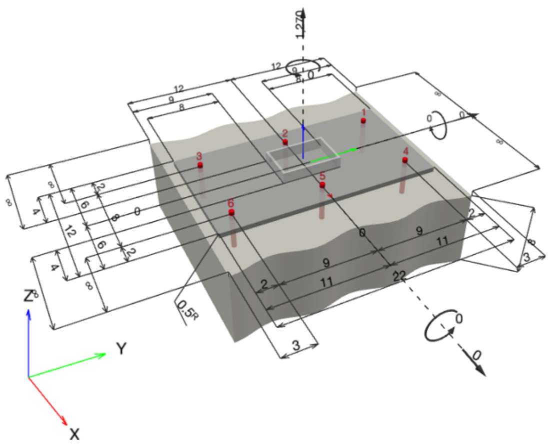
1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (4 1/4)**
Item number:	418076 KH-EZ 1/2"x5"
Specification text:	Hilti KH-EZ screw anchor with 4.25 in embedment, 1/2 (4 1/4), Carbon steel, installation per ESR-3027
Effective embedment depth:	$h_{ef,act}$ = 3.220 in., h_{nom} = 4.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 12.000 in. x 22.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS6X4X.250; (L x W x T) = 6.000 in. x 4.000 in. x 0.250 in.
Base material:	uncracked concrete, 3000, f_c' = 3,000 psi; h = 8.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Application also possible with KWIK-X 1/2 (3) hnom1 under the selected boundary conditions.
More information in section Alternative fastening data of this report.

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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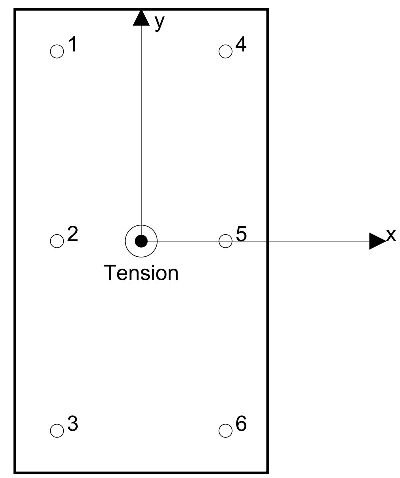
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1,270$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	7

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	212	0	0	0
2	212	0	0	0
3	212	0	0	0
4	212	0	0	0
5	212	0	0	0
6	212	0	0	0

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(0.000/0.000): 1,270 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	212	11,778	2	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,270	20,571	7	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

N_{sa} [lb]
18,120

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,120	0.650	11,778	212

3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.220	0.000	0.000	3.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.250	27	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
423.84	93.32	1.000	1.000	0.886	0.920	8,545

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
31,648	0.650	20,571	1,270

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Design:	Concrete - Oct 4, 2024	Date:	10/4/2024
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!