NOTHING BEATS A GREAT TEMPLATE

Preface

Content
Interactive design aids in accordance to US codes ACI 318-11, AISC 14th edition and ASCE-7-10

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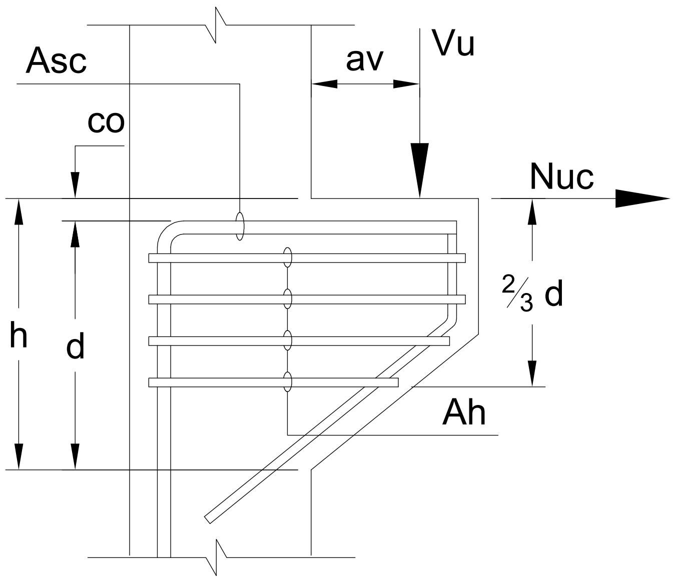
Contents

Chapter 1: Concrete Design

Design of Corbel as per ACI 318-11 Chapter 11



System

Corbel Width, b=			14.0 in
Corbel Height, h=			12.0 in
Concrete Cover, co=			1.0 in
Corbel Depth, d=	h - co = 12.0 - 1.0	=	11.0 in
Distance from Column Face to Vertical Load, a_v=			3.0 in

Load

Ultimate Vertical Load, V_u=	88.8 kips
Ultimate Horizontal Load, Nuc=	32.0 kips

Material Properties

Concrete Strength, f'_c=		5000 psi
Yield Strength of Reinforcement, f_y=		60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=		0.75
Modification Factor for Lightweight Concrete, λ=		1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= 1.4* λ	=	1.40

Check Vertical Load Capacity

V_{n1}=	0.2*f'$_c$*b*d/1000	=	154.0 Kips
V_{n2}=	(480+0.08*f'$_c$)*b*d/1000	=	135.5 Kips
V_{n3}=	1600*b*d/1000	=	246.4 Kips

Nominal Vertical Capacity (According to Cl.11.8.3.2.1 of ACI318),

ΦV_n=	$\Phi * MIN(V_{n1}; V_{n2}; V_{n3})$	=	101.6 Kips
Vertical Load Capacity=	IF(V_u>ΦV_n;"Not Pass";"Pass")	=	Pass

Determine Shear Friction Reinforcement (A_{vf})

Required Area of Reinforcement for Shear Friction (According to Cl.11.6.4.1 of ACI318),

A_{vf}=	$V_u \times 1000/(\Phi \times f_y \times \mu)$	=	1.41 in²

Determine Direct Tension Reinforcement (A_n)

Minimum Horizontal Force on Corbel, Nuc_min=	$0.2 \times V_u$	=	17.8 Kips
Horizontal Force on Corbel, Nuc_act=	MAX (Nuc ; Nuc_min)	=	32.0 kips

Required Area of Reinforcement for Direct Tension (According to Cl.11.8.3.1 of ACI318),

A_n=	$Nuc\_act * 1000/((\Phi) * f_y)$	=	0.71 in²

Determine Flexural Reinforcement (A_f)

M_u=	V_u*a_v+Nuc_act*(h-d)	=	298.4 kip*in

Required Area of Reinforcement for Flexural (According to Cl.11.8.3.3 of ACI318),

A_f=	$M_u \times 1000/(\Phi \times f_y \times 0.9 \times d)$	=	0.67 in²

Determine Primary Tension Reinforcement (A_{sc})

Required Area of Reinforcement for Primary Tension (According to Cl.11.8.3.5 of ACI318),

A_{sc}=	MAX ((2/3 * A_{vf}) + A_n ; A_f + A_n)	=	1.65 in^2

Minimum Area of Reinforcement for Primary Tension (According to Cl.11.8.5 of ACI318),

A_{sc_min}=	$0.04 * f'_c / f_y * b * d$	=	0.51 in²
A_{sc_Req}=	MAX (A_{sc}; A_{sc_min})	=	1.65 in²

Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.9
Provided Area of Bar Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	1.00 in^2
Number of Provided Bars, n=			2
Provided Area of Reinforcement, A_{sc_Prov}=	n * A_{sb}	=	2.00 in^2
Check Validity=	IF($A_{sc\_Prov} \geq A_{sc\_Req}$; "Valid"; "Invalid")	=	Valid

Determine Horizontal Reinforcement (A_h)

Required Area of Reinforcement for Horizontal Shear (According to Cl.11.8.4 of ACI318),

A_{h_Req}=	0.5*(A_{sc_Prov}-A_n)	=	0.65 in²
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.3
Provided Area of Bar Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.11 in^2
Number of Provided Bars, n=			6
Provided Area of Reinforcement, A_{h_Prov}=	n * A_{sb}	=	0.66 in^2
Check Validity=	IF($A_{h\_Prov} \geq A_{h\_Req}$; "Valid"; "Invalid")	=	Valid

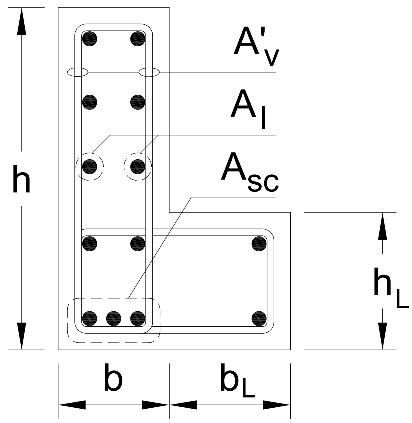
Distribute in two-thirds of Effective Corbel Depth adjacent to A_{sc}

Design Summary

Area of Reinforcement for Primary Tension A_{sc}=	A_{sc_Prov}	=	2.00 in²
Area of Reinforcement for Horizontal Shear, A_h=	A_{h_Prov}	=	0.66 in²

Distribute in two-thirds of Effective Corbel Depth adjacent to A_{sc}

Design Precast Spandrel Beam for Combined Shear and Torsion as per ACI 318-11 Chapter 11



System

Width of Beam, b=			16.0 in
Height of Beam, h=			48.0 in
Width of Beam Ledge, b_L=			8.0 in
Height of Beam Ledge, h_L=			16.0 in
Concrete Cover, co=			2.50 in
Concrete Cover to Center of Stirrup, co'=			1.50 in
Effective Depth of Beam, d=	$h - co$	=	45.50 in

Load

Ultimate Bending Moment, M_u=	1316.0 kip*ft
Ultimate Torsional Moment, T_u=	108.6 kip*ft
Ultimate Shear Force, V_u=	127.2 kips

Material Properties

Concrete Strength, f'_c=			5000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Yield Strength of Stirrups Reinforcement, f_{yt}=			60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ_s=			0.75
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ_t=			0.90
Modification Factor for Lightweight Concrete, λ=			1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ=	$1.4^* \lambda$	=	1.40

Determine Concrete Cracking Torque

Area Enclosed by Outside Perimeter of Spandrel Beam Including the Ledge,

A_{cp}= $b * h + b_L * h_L$ = 896 in^2

Outside Perimeter of Spandrel Beam Including the Ledge,

P_{cp}= $2 * (b + b_L + h)$ = 144 in

Concrete Cracking Torque, $T_{cr} = 4 * \lambda * \sqrt{f'_c} * \frac{A_{cp}^2}{P_{cp}} / 12000$ = 131.4 kip*ft

Torsional Moment should be: IF(T_u<Φ_s*T_{cr}/4;"Neglected";"Checked") = Checked

Calculation of Torsion Reinforcement

Area Enclosed by Centerline of The Outermost Closed Transverse Torsional Reinforcement (According to Cl.11.5.3.6 of ACI318),

A_{oh}= (h-2*co')*(b-2*co')+(b_L)*(h_L-2*co') = 689.0 in^2

A_o= 0.85*A_{oh} = 585.6 in^2

Angle of Compression Diagonal Struts (According to 11.5.3.6 of ACI318),

Θ= 45 ^{o}

Required Area for Torsion Shear per Stirrups Spacing (According to Eq. 11-20, 21 of ACI318),

A'_{vt}= $\frac{T_u * 12000}{2 * \Phi_s * A_o * f_{yt} * (1 / \tan(\Theta))}$ = 0.025 in^2 per in

Calculation of Shear Reinforcement

Nominal Shear Strength Provided by Concrete (According to Eq.11-3 of ACI318),

V_c= $2 * \lambda * \sqrt{f'c} * \frac{b * d}{1000}$ = 102.95 kips

Nominal Shear Strength Provided by Reinforcement (According to Eq.11-2 of ACI318),

V_s= $V_u/\Phi_s - V_c$ = 66.65 kips

Required Area for Direct Shear per Stirrups Spacing (According to Eq. 11-1, 2 of ACI318),

A'_{vs}= $\frac{V_s * 1000}{f_{yt} * d}$ = 0.024 in^2 per in

Calculation of Combined Shear and Torsion Reinforcement

Total Required Area for Torsion & Shear per Stirrups Spacing (According to Cl.11.5.3.8 of ACI318),

A'_v= $A'_{vt}+A'_{vs}/2$ = 0.037 in^2 per in per leg

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.4

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in^2

Required Stirrups Spacing, $s_{\_Req}$= A_{sb}/A'_v = 5.41 in

Provided Stirrups Spacing, $s_{\_Prov}$= 5.00 in

Check Validity= IF($s_{\_Prov} \le s_{\_Req}$; "Valid"; "Invalid") = Valid

Perimeter of Stirrups, Ph= $2*(b-2*co'+h-2*co')+2*b_L$ = 132.00 in

Maximum Stirrups Spacing Due to Torsion (According to Cl.11.6.6 of ACI318),

s_{max_t}= MIN(Ph/8; 12) = 12.00 in

Maximum Stirrups Spacing Due to Shear (According to Cl.11.4.5 of ACI318),

s_{max_v}= MIN(d/2; 24) = 22.75 in

Maximum Stirrups Spacing, s_{max}= MIN(s_{max_t}; s_{max_v}) = 12.00 in

Check Validity= IF($s_{\_Prov} \le s_{max}$; "Valid"; "Invalid") = Valid

Calculation of Longitudinal Torsion Reinforcement

Required Area of Longitudinal Torsion Reinforcement (According to Cl.11.5.3.7 of ACI318),

A_{l_i}= $\dfrac{A'_{vt}*Ph}{\tan(\Theta)^2}*\dfrac{f_{yt}}{f_y}$ = 3.30 in^2

Minimum Area of Longidudinal Torsion Reinforcement (According to Eq.11-24 of ACI318),

A_{l_min}= $\dfrac{5*\sqrt{f'_c}*A_{cp}}{f_y}-A'_{vt}*Ph*\dfrac{f_{yt}}{f_y}$ = 1.98 in^2

A_{l_Req}= MAX(A_{l_i} ; A_{l_min}) = 3.30 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n= 12

Provided Longitudinal Reinforcement, A_{l_Prov}= A_{sb} * n = 3.72 in^2

Check Validity= IF($A_{l\_Prov} \ge A_{l\_Req}$; "Valid"; "Invalid") = Valid

Calculation of Required Flexural Reinforcement

$R_n = \frac{M_u * 12 * 1000}{\Phi_t * b * d^2}$ = 530 psi

$\rho = \frac{0.85 * f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_n}{0.85 * f'_c}}\right)$ = 0.0095

Area of Flexural Reinforcement, A_s= ρ*b*d = 6.92 in^2

Calculation of Total Bottom Reinforcement at Mid-Span

Percentage of Torsional Reinforcement Concentrated on Bottom Side, Per= 16 %

Total Area of Bottom Reinforcement at Mid-Span,

A_{sc_Req}= $A_{l\_Req} * Per / 100 + A_s$ = 7.45 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.11

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 1.56 in^2

Number of Bars, n= 5

Total Area of Bottom Reinforcement, A_{sc_Prov}= A_{sb} * n = 7.80 in^2

Check Validity= IF($A_{sc\_Prov} \geq A_{sc\_Req}$; "Valid"; "Invalid") = Valid

Design Summary

Total Required Area for Torsion & Shear per Stirrups Spacing,

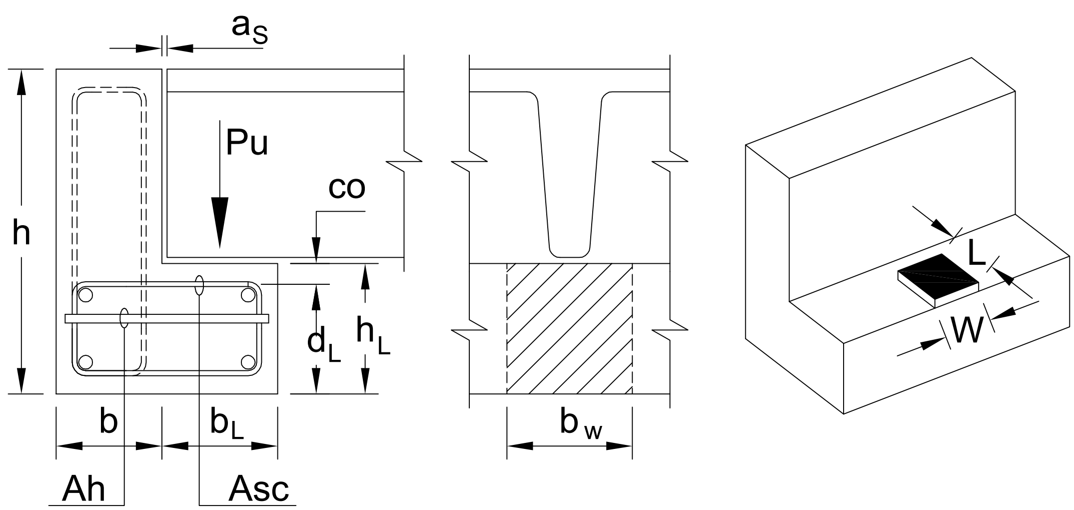
A'_v= A'_v = 0.037 in^2 per in per leg

Provided Stirrups Spacing, $s_{\_Prov}$= $s_{\_Prov}$ = 5.00 in

Provided Longitudinal Reinforcement, A_{l_Prov}= A_{l_Prov} = 3.72 in^2

Total Area of Bottom Reinforcement, A_{sc_Prov}=A_{sc_Prov} = 7.80 in^2

<u>Design of Beam Ledge as per ACI 318-11 Chapters 9 & 11</u>



System

Width of Beam, b=			7.0 in
Height of Beam, h=			36.0 in
Width of Beam Ledge, b_L=			6.0 in
Height of Beam Ledge, h_L=			12.0 in
Concrete Cover, co=			1.25 in
Width of Bearing Pad, W=			4.5 in
Length of Bearing Pad, L=			4.5 in
Thickness of Bearing Pad, t_b=			0.3 in
Gap Spacing, a_s=			1.0 in
Shear Spacing, a_v=	$2/3 * L + a_s$	=	4.0 in
Flexural Spacing, a_f=	$a_v + co$	=	5.25 in
Effective Width According to Shear Requirements, b_{ws}=	$W + 4 * a_v$	=	20.5 in
Effective Width According to Flexural Requirements, b_{wf}=	$W + 5 * a_f$	=	30.8 in
Effective Depth of Beam Ledge, d_L=	h_L -co	=	10.75 in

Load

Dead Load, P_D=			11.0 kips
Live Load, P_L=			6.5 kips
Service Load, P=	$P_D + P_L$	=	17.5 kips
Ultimate Load, P_u=	$1.2 * P_D + 1.6 * P_L$	=	23.6 kips

Material Properties

Concrete Strength, f'_c=			5000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ_s=			0.75
Bearing Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ_b=			0.65
Modification Factor for Lightweight Concrete, λ=			1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= 1.4* λ		=	1.40
Maximum Service Load for Bearing Pads, q=			1000 psi

Check Bearing Plate Dimension

Capacity of Bearing Plate, B_p=	W * L * q /1000	=	20.25 kips
Check Validity=	IF(B_p>P;"Valid" ;"Increase Dimension")	=	Valid

Check Concrete Bearing Strength

Bearing Strength of Concrete, ΦP_{nb}=	Φ_b * 0.85 * f'_c * L * W / 1000	=	55.9 kips
Check Validity=	IF(ΦP_{nb}>Pu;"Valid" ;"Invalid")	=	Valid

Check Maximum Nominal Shear-Transfer by Effective Section

Nominal Shear by Effective Section (According to Cl.11.9.3.2.1 of ACI318),

V_{n1}=	0.2 * f'_c * b_{ws} * d_L /1000	=	220.4 kips
V_{n2}=	(480 + 0.08 * f'_c) * b_{ws} * d_L /1000	=	193.9 kips
V_{n3}=	1600 * b_{ws} * d_L /1000	=	352.6 kips
ΦV_n=	Φ_s * MIN(V_{n1}; V_{n2}; V_{n3})	=	145.4 kips
Check Validity=	IF(ΦV_n>P_u;"Valid" ;"Increase Dimension")	=	Valid

Determine Shear Friction Reinforcement (A_{vf})

Required Reinforcement for Shear Friction (According to Cl.11.6.4.1 of ACI318),

A_{vf}=	P_u * 1000 / (Φ_s * f_y * μ)	=	0.37 in^2 per bws

Determine Direct Tension Reinforcement (A_n)

Required Reinforcement for Direct Tension (According to Cl.11.8.3.4 of ACI318),

A_n=	0.2 * P_u * 1000 / (Φ_s * f_y)	=	0.10 in^2 per bwf

Determine Flexural Reinforcement (A_f)

M_u=	P_u * a_f + 0.2 * P_u * (h_L - d_L)	=	129.8 kip*in

Required Reinforcement for Flexural (According to Cl.11.8.3.3 of ACI318),

A_f=	M_u * 1000 / (Φ_s * f_y * 0.8 * d_L)	=	0.34 in^2 per bwf

Determine Primary Tension Reinforcement (A_{sc})

Required Area of Reinforcement for Primary Tension (According to Cl.11.8.3.5 of ACI318),

A_{sc}=	MAX (2/3 * $A_{vf}/b_{ws} + A_n/b_{wf}$; $A_f/b_{wf} + A_n/b_{wf}$)	=	0.015 in^2 per in

Minimum Area of Reinforcement for Primary Tension (According to Cl.11.8.5 of ACI318),

A_{sc_min}=	$0.04 * f'_c / f_y * d_L$	=	0.036 in^2 per in
A_{sc_req}=	MAX (A_{sc}; A_{sc_min})	=	0.036 in^2 per in
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.5
Spacing between Bars, s=			8.0 in
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.31 in^2
Check Validity=	IF(A_{sb}/A_{sc_req}>s; "Valid"; "Invalid")	=	Valid

Determine Horizontal Reinforcement (A_h)

Required Area of Reinforcement for Horizontal Shear (According to Cl.11.8.4 of ACI318),

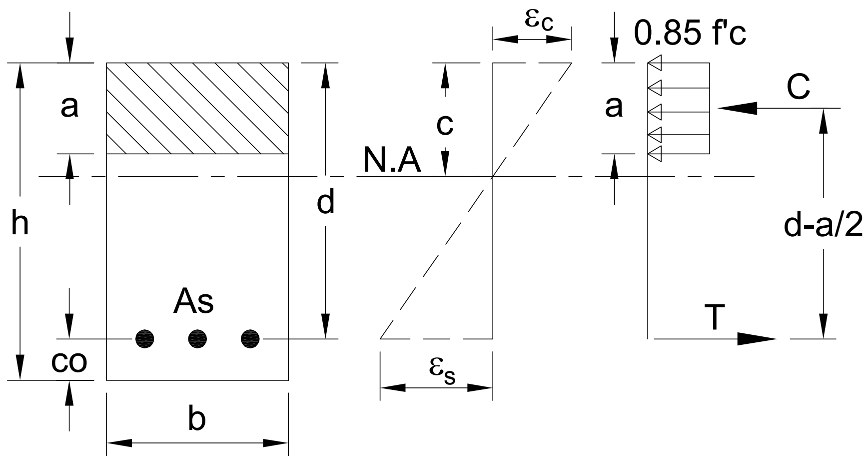
A_h=	0.5*(A_{sc_req}-A_n/b_{wf})	=	0.016 in^2 per in
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.4
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.20 in^2
Check Validity=	IF(A_{sb}/A_h>s; "Valid"; "Invalid")	=	Valid

Design Summary

Primary Tension Reinforcement, A_{sc_req}=	A_{sc_req}	=	0.036 in^2 per in
Horizontal Shear Reinforcement, A_h=	A_h	=	0.016 in^2 per in

Distribute in two-thirds of Effective Ledge Depth adjacent to A_{sc}

Design of Rectangular Section with Tension Reinforcement only as per ACI 318-11 Chapters 9 & 10



System

Width of Concrete Section, b=		12.0 in
Depth of Concrete Section, h=		16.0 in
Concrete Cover, co=		2.5 in
Effective Depth of Concrete Section, d= $h - co = 16.0 - 2.5$	=	13.5 in

Load

Bending Moment due to Dead Load, M_D=			56.0 kip*ft
Bending Moment due to Live Load, M_L=			35.0 kip*ft
Ultimate Bending Moment, M_U=	$(1.2 * M_D) + (1.6 * M_L)$	=	123.2 kip*ft

Material Properties

Concrete Strength, f'_c=			4000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=			0.90
Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3),			
β_1=	IF($f'_c \le$4000;0.85;IF($f'_c \ge$8000;0.65;1.05-0.00005*f'_c))	=	0.85

Area of Reinforcement

$$R_n = \frac{M_U * 12000}{\Phi * b * d^2} = 751.1 \text{ psi}$$

$$\rho = 0.85 * \frac{f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_n}{0.85 * f'_c}}\right) = 0.0143$$

Area of Reinforcement, A_s= $\rho * b * d$ = 2.32 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$A_{s\_min1}= \dfrac{3*\sqrt{f'_c}*b*d}{f_y}$ = 0.51 in^2

$A_{s\_min2}= \dfrac{200*b*d}{f_y}$ = 0.54 in^2

$A_{s\_min}=$ MAX(A_{s_min1} ; A_{s_min2}) = 0.54 in^2

Required Area of Reinforcement, A_{sc_Req}= MAX(A_s; A_{s_min}) = 2.32 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.10

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 1.27 in^2

Number of Bars, n= 2

Vertical Reinforcement, A_{sc_Prov}= A_{sb} * n = 2.54 in^2

Check Validity= IF($A_{sc\_Prov} \geq A_{sc\_Req}$; "Valid"; "Invalid") = Valid

Check Tension Controlled

Depth of Rectangular Stress Block, a= $\dfrac{A_{sc\_Prov}*f_y}{0.85*f'_c*b}$ = 3.74 in

Distance from Extreme Compression Fiber to Neutral Axis, c= a/β_1 = 4.40 in

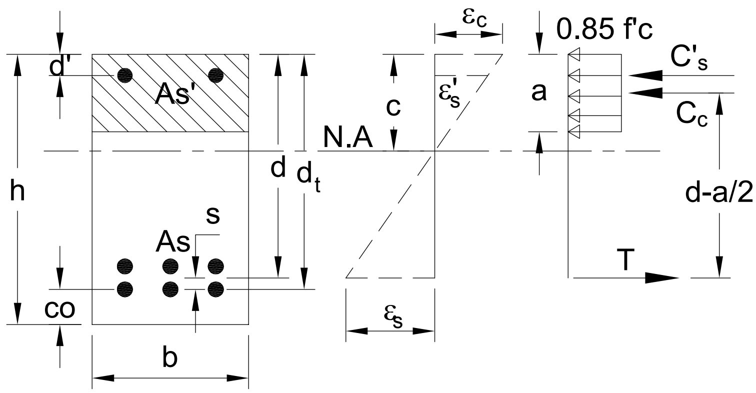
c/d = $c/d = 4.40/13.5$ = 0.326

IF(c/d>0.375; "Add Com. RFT"; "Tension Controlled") = Tension Controlled

Design Summary

Required Area of Reinforcement, A_{sc}= A_{sc_Prov} = 2.54 in^2

Design of Rectangular Section with Compression Reinforcement as per ACI 318-11 Chapters 9 & 10



System

Width of Concrete Section, b=		12.0 in
Depth of Concrete Section, h=		32.5 in
Concrete Cover, co=		2.5 in
Effective Depth of Concrete Section to Extreme Layer , $d_t = h - co = 32.5 - 2.5$	=	30.0 in
Distance between C.G of Tension Reinforcement and Extreme Layer, s=		1.2 in
Effective Depth of Concrete Section to C.G of Tension Reinforcement, $d = d_t - s$	=	28.8 in
Depth of Compression Reinforcement, d'=		2.5 in

Load

Bending Moment due to Dead Load, M_D=		430.0 kip*ft
Bending Moment due to Live Load, M_L=		175.0 kip*ft
Ultimate Bending Moment, M_U= $(1.2 * M_D) + (1.6 * M_L)$	=	796.0 kip*ft

Material Properties

Concrete Strength, f'_c=		4000 psi
Yield Strength of Reinforcement, f_y=		60000 psi
Modulus of Elasticity of Reinforcement, E_s=		29000000 psi
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=		0.90
Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3),		
β_1= IF(f'_c≤4000;0.85;IF(f'_c≥8000;0.65;1.05-0.00005*f'_c))	=	0.85

Check If Compression Reinforcement is Required

ω_t=	$0.31875*\beta_1$	=	0.271
R_{nt}=	$\omega_t*(1-0.59*\omega_t)*f'_c$	=	910.7 pci
R_n=	$\dfrac{M_U*12000}{\Phi*b*d_t^2}$	=	982.7 psi
Compression Reinforcement is:	IF(R_n>R_{nt};"Required";"Not Required")	=	Required

Determine Required Moment Resisted by Compression Reinforcement

ω_t=	$0.31875*\beta_1$	=	0.271
ρ_t =	$0.31875 * f'_c * \beta_1 / f_y$	=	0.01806
ρ=	ρ_t*d_t/d	=	0.01881
ω=	$\rho*f_y/f'_c$	=	0.28215
Moment Resisted by Tension RFT, M_{nt}=	$\omega*(1-0.59*\omega)*\dfrac{f'_c*b*d^2}{12000}$	=	780.3 kip*ft
Moment Resisted by Compression RFT, $M'_n = M_U/\Phi - M_{nt}$		=	104.1 kip*ft

Required Area of Compression Reinforcement

d'/c_{limit}=	$1-\dfrac{f_y}{E_s*0.003}$	=	0.31
c_{limit}=	$\left(1-\dfrac{(f_y)}{E_s*0.003}\right)*d_t$	=	9.3 in
c_{cal}=	$0.375*d_t$	=	11.3 in
d'/c_{cal}=	$d'/(c_{cal})$	=	0.22
f'_{si}=	MIN($0.003*E_s*(1-d'/c_{cal})$; f_y)	=	60000 psi
f'_s=	IF($d'/c_{cal} \le d'/c_{limit}$; f_y ; f'_{si})	=	60000 psi
Required Reinforcement Area for Compression, A'_s=	$\dfrac{M'_n*12000}{f'_s*(d-d')}$	=	0.79 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.6
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.44 in^2
Number of Bars, n=			2
Vertical Reinforcement, A'_{s_Prov}=	A_{sb} * n	=	0.88 in^2
Check Validity=	IF($A'_{s\_Prov} \ge A'_s$; "Valid"; "Invalid")	=	Valid
Required Reinforcement Area for Tension, A_s=	$A'_s + (\rho*b*d)$	=	7.29 in^2

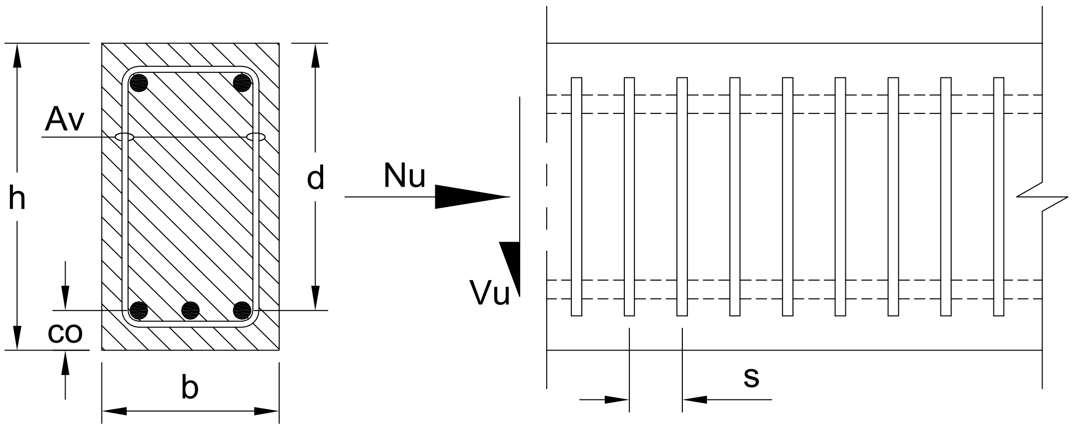
Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

A_{s_min1}=	$\dfrac{3 * \sqrt{f'_c} * b * d}{f_y}$	=	1.09 in^2
A_{s_min2}=	$\dfrac{200 * b * d}{f_y}$	=	1.15 in^2
A_{s_min}=	MAX(A_{s_min1} ; A_{s_min2})	=	1.15 in^2
Required Area of Reinforcement, A_{sc_Req}= MAX(A_s; A_{s_min})		=	7.29 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.10
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	1.27 in^2
Number of Bars, n=			6
Vertical Reinforcement, A_{sc_Prov}=	A_{sb} * n	=	7.62 in^2
Check Validity=	IF($A_{sc\_Prov} \geq A_{sc\_Req}$; "Valid"; "Invalid")	=	Valid

Design Summary

Required Reinforcement Area for Compression, A'_s=	A'_{s_Prov}	=	0.88 in^2
Required Area of Reinforcement, A_{sc}=	A_{sc_Prov}	=	7.62 in^2

Design of Shear Reinforcement for Section Subject to Shear & Axial Compression As per ACI318-11



System

Width of Concrete Section, b=			12.0 in
Depth of Concrete Section, h=			16.0 in
Concrete Cover, co=			2.25 in
Effective Depth of Concrete Section, d=	h - co	=	13.75 in

Load

Shear Force due to Dead Load, V_D=			10.0 kips
Shear Force due to Live Load, V_L=			5.0 kips
Ultimate Shear Force, V_u=	$(1.2*V_D)+(1.6*V_L)$	=	20.0 kips
Axial Compression Force due to Dead Load, N_D=			4.2 kips
Axial Compression Force due to Live Load, N_L=			3.1 kips
Ultimate Axial Compression Force, N_u=	$(1.2*N_D)+(1.6*N_L)$	=	10.0 kips

Material Properties

Concrete Strength, f'_c=	4000 psi
Yield Strength of Reinforcement, f_y=	60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Determine Concrete Shear Strength

Nominal Shear Strength provided by Concrete (According to Eq. 11-4 of ACI318),

$V_c =$ $2 * \left(1 + \frac{N_u * 1000}{2000 * h * b}\right) * \lambda * \frac{\sqrt{f'_c} * b * d}{1000}$ = 21.4 kips

Shear Reinforcement is : IF(V_u>$\Phi * V_c$;"Required";"Not Required") = Required

Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),

$V_s =$ $\frac{V_u - \Phi * V_c}{\Phi}$ = 5.3 kips

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI318),

$V_{s\_max} =$ $8 * \lambda * \frac{\sqrt{f'_c} * b * d}{1000}$ = 83.5 kips

IF(V_s>V_{s_max}; "Increase Beam Dimension"; "OK") = OK

Spacing of Provided Stirrups, s= 6.75 in

Required Area of Reinforcement, $A_v =$ $\frac{V_s * s * 1000}{f_y * d}$ = 0.04 in^2

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI318),

$A_{v\_min1} =$ $\frac{0.75 * \sqrt{f'_c} * b * s}{f_y}$ = 0.06 in^2

$A_{v\_min2} =$ $\frac{50 * b * s}{f_y}$ = 0.07 in^2

$A_{v\_min} =$ MAX(A_{v_min1} ; A_{v_min2}) = 0.07 in^2

Required Area of Reinforcement, A_{vc_Req}= MAX(A_v; A_{v_min}) = 0.07 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.3

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 in^2

Number of Stirrups, n= 1

Provided Area of Reinforcement, $A_{vc\_Prov} = A_{sb} * n * 2$ = 0.22 in^2

Check Validity= IF($A_{vc\_Prov} \geq A_{vc\_Req}$; "Valid"; "Invalid") = Valid

Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),

V_{s_limit}= $4 * \lambda * \sqrt{f'_c} * b * d / 1000$ = 41.7 kips

Factor for Maximum Spacing of Stirrups, Fac= IF($V_s \leq V_{s\_limit}$;1;0.5) = 1.0

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),

s_{max}= MIN(d/2;24) * Fac = 6.88 in

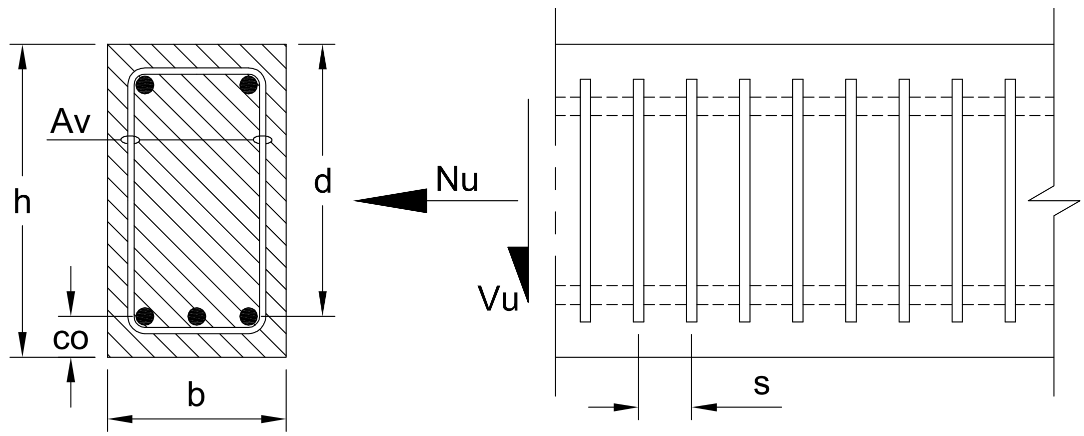
Check Validity= IF($s \leq s_{max}$;"Valid"; "Invalid") = Valid

Design Summary

Provided Area of Shear Reinforcement, A_{vc_Prov}= A_{vc_Prov} = 0.22 in^2

Spacing of Stirrups, s= s = 6.75 in

<u>Design of Shear Reinforcement for Section Subject to Shear & Axial Tension As per ACI318-11</u>



System

Width of Concrete Section, b=			10.5 in
Depth of Concrete Section, h=			18.0 in
Concrete Cover, co=			2.0 in
Effective Depth of Concrete Section, d=	$h - co$	=	16.0 in

Load

Shear Force due to Dead Load, V_D=			12.8 kips
Shear Force due to Live Load, V_L=			9.0 kips
Ultimate Shear Force, V_u=	$(1.2 * V_D) + (1.6 * V_L)$	=	29.8 kips
Axial Tension Force due to Dead Load, N_D=			-2.0 kips
Axial Tension Force due to Live Load, N_L=			-15.2 kips
Ultimate Axial Tension Force, N_u=	$(1.2 * N_D) + (1.6 * N_L)$	=	-26.7 kips

Material Properties

Concrete Strength, f'_c=	3600 psi
Yield Strength of Reinforcement, f_y=	40000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	0.85

Determine Concrete Shear Strength

Nominal Shear Strength provided by Concrete (According to Eq. 11-8 of ACI318),

$$V_c = 2 * \left(1 + \frac{N_u * 1000}{500 * h * b}\right) * \lambda * \frac{\sqrt{f'_c} * b * d}{1000} = 12.3 \text{ kips}$$

Shear Reinforcement is : IF(V_u>$\Phi * V_c$;"Required";"Not Required") = Required

Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),

$V_s = \dfrac{V_u - \Phi * V_c}{\Phi}$ = 27.4 kips

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI318),

$V_{s\_max} = 8 * \lambda * \dfrac{\sqrt{f'_c} * b * d}{1000}$ = 68.5 kips

IF(V_s>V_{s_max}; "Increase Beam Dimension"; "OK") = OK

Spacing of Provided Stirrups, s= 5.0 in

Required Area of Reinforcement, $A_v = \dfrac{V_s * s * 1000}{f_y * d}$ = 0.21 in^2

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI318),

$A_{v\_min1} = \dfrac{0.75 * \sqrt{f'_c} * b * s}{f_y}$ = 0.06 in^2

$A_{v\_min2} = \dfrac{50 * b * s}{f_y}$ = 0.07 in^2

A_{v_min}= MAX(A_{v_min1} ; A_{v_min2}) = 0.07 in^2

Required Area of Reinforcement, A_{vc_Req}= MAX(A_v; A_{v_min}) = 0.21 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.3

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 in^2

Number of Stirrups, n= 1

Provided Area of Reinforcement, A_{vc_Prov}= A_{sb} * n * 2 = 0.22 in^2

Check Validity= IF(A_{vc_Prov}≥A_{vc_Req}; "Valid"; "Invalid") = Valid

Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),

$V_{s\_limit} = 4 * \lambda * \dfrac{\sqrt{f'_c} * b * d}{1000}$ = 34.3 kips

Factor for Maximum Spacing of Stirrups, Fac= IF(V_s≤V_{s_limit};1;0.5) = 1.0

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),

s_{max}= MIN(d / 2;24) * Fac = 8.00 in

Check Validity= IF(s≤s_{max};"Valid"; "Invalid") = Valid

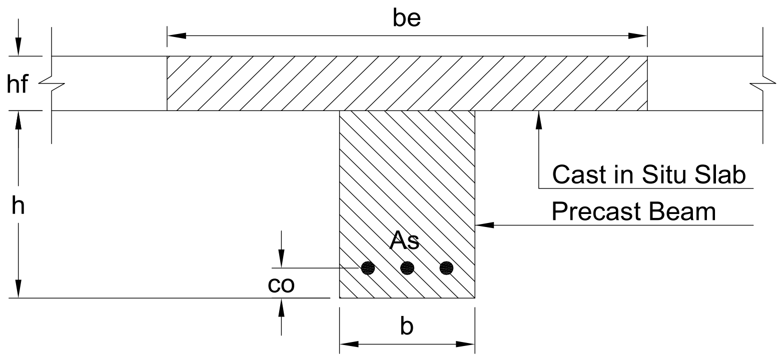
Design Summary

Provided Area of Shear Reinforcement, A_{vc_Prov}= A_{vc_Prov} = 0.22 in^2

Spacing of Stirrups, s= s = 5.00 in

Calculation of Deflection of Shored Nonprestressed Simple Support Concrete Composite Section

As per ACI318-11 Chapter 9



System

Beam Span, L=			26.0 ft
Beam Spacing, S=			8.0 ft
Width of Precast Beam, b=			12.0 in
Depth of Precast Beam, h=			20.0 in
Thickness of Cast in Situ Slab, h_f=			4.0 in
Area of Tension Reinforcement for Precast Beam, A_s=			3.00 in^2
Concrete Cover for Precast Beam, co=			2.5 in
Effective Width of Slab, b_{e1}=	L*12 / 4	=	78.0 in
Effective Width of Slab, b_{e2}=	S*12	=	96.0 in
Effective Width of Slab, b_{e3}=	16*hf + b	=	76.0 in
Effective Width of Slab, b_e=	MIN(b_{e1}; b_{e2}; b_{e3})	=	76.0 in

Material Properties

Concrete Strength of Cast in Situ Slab, f'_{c1}=	3000 psi
Concrete Strength of Precast Beam, f'_{c2}=	4000 psi
Yield Strength of Reinforcement, f_y=	40000 psi
Modulus of Elasticity of Reinforcement, E_s=	29000000 psi
Modification Factor for Lightweight Concrete, λ=	1.00
Concrete Density, w_c=	150 psi

Load

Superimposed Dead Load, SDL=			10.00 psf
Live Load, LL=			75.00 psf
Dead Load per Unit Length for Slab, w_{d1}=	$SDL*S + w_c*S*12*h_f/144$	=	480.0 lb/ft
Dead Load per Unit Length for Beam, w_{d2}=	$w_c*b*h/144$	=	250.0 lb/ft
Live Load per Unit Length, w_l=	LL*S	=	600.0 lb/ft
Percentage of Sustained Live Load, Sus=			20 %
Bending Moment of Dead Load 1, M_{D1}=	$1/1000 * w_{d1}*L^2/8$	=	40.6 kip*ft
Bending Moment of Dead Load 2, M_{D2}=	$1/1000 * w_{d2}*L^2/8$	=	21.1 kip*ft
Bending Moment of Live Load, M_L=	$1/1000 * w_l*L^2/8$	=	50.7 kip*ft
Bending Moment of Sustained Load, M_{sus}=	$M_{D1} + M_{D2} + (Sus/100) * M_L$	=	71.8 kip*ft

Calculation of Modular Ratio

For Cast in Situ Slab:

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI318),

E_{c1}= $w_c^{1.5} * 33 * \sqrt{f'_{c1}}$ = 3320561 psi

Modulus of rupture (According to Eq. 9-10 of ACI318), $f_{r1} = 7.5 * \lambda * \sqrt{f'_{c1}}$ = 411 psi

For Precast Beam:

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI318),

E_{c2}= $w_c^{1.5} * 33 * \sqrt{f'_{c2}}$ = 3834254 psi

Modulus of rupture (According to Eq. 9-10 of ACI318), $f_{r2} = 7.5 * \lambda * \sqrt{f'_{c2}}$ = 474 psi

n_c= E_{c2} / E_{c1} = 1.15

n_s= E_s / E_{c2} = 7.56

Width of Slab considering relative Concrete Strength, bs= be/n_c = 66.09 in

Calculation of Moment of Inertia for Cracked Section

For Precast Beam

Effective Depth of Section, d= 17.5 in

I_{g1}= $b*h^3/12$ = 8000 in^4

B= $b/(n_s*A_s)$ = 0.53 1/in

kd= $\dfrac{\sqrt{2*d*B+1}-1}{B}$ = 6.5 in

I_{cr1}= $\dfrac{b*kd^3}{3} + n_s*A_s*(d-kd)^2$ = 3842.8 in^4

For Composite Section

Effective Depth of Section, d= (h+hf) - co = 21.5 in

h1 = h + hf = 24.0 in

bs1 = bs - b = 54.1 in

Distance from Centroidal Axis of Gross Section to Tension Face,

$$yt = h1 - \frac{1}{2} * \frac{bs1 * hf^2 + b * h1^2}{bs1 * hf + b * h1} = 16.3 \text{ in}$$

$$I_{g2} = \frac{bs1 * hf^3}{12} + \frac{b * h1^3}{12} + bs1 * hf * \left(h + \frac{hf}{2} - yt\right)^2 + b * h1 * \left(yt - \frac{h1}{2}\right)^2 = 26468.49 \text{ in}^4$$

$$B = bs/(n_s * As) = 2.91 \text{ 1/in}$$

$$kd = \frac{\sqrt{2 * d * B + 1} - 1}{B} = 3.5 \text{ in}$$

$$I_{cr2} = \frac{bs * kd^3}{3} + n_s * As * (d - kd)^2 = 8292.9 \text{ in}^4$$

Ratio between Cracking & Gross Inertia, r= $\left(\frac{I_{g1}}{I_{g2}} + \frac{I_{cr1}}{I_{cr2}}\right)/2$ = 0.383

Cracking Moment (According to Eq. 9-9 of ACI318),

Cracking Moment for Beam Section, M_{cr1}= $\frac{f_{r2} * I_{g1}}{h/2 * 12000}$ = 31.60 kip*ft

Cracking Moment for Beam Section, M_{cr2}= $\frac{f_{r2} * I_{g2}}{yt * 12000}$ = 64.14 kip*ft

Effective Moment of Inertia for Composite Section,

$$I_{e1,2} = \left(\frac{M_{cr1}}{M_{D1} + M_{D2}}\right)^3 * I_{g1} + \left(1 - \left(\frac{M_{cr1}}{M_{D1} + M_{D2}}\right)^3\right) * I_{cr1} = 4401 \text{ in}^4$$

$$I_{ed,l} = \left(\frac{M_{cr2}}{M_{D1} + M_{D2} + M_L}\right)^3 * I_{g2} + \left(1 - \left(\frac{M_{cr2}}{M_{D1} + M_{D2} + M_L}\right)^3\right) * I_{cr2} = 11670 \text{ in}^4$$

Check Validity= IF($I_{e1.2}$<I_{g1}; "Valid"; "Invalid") = Valid

Short Term Deflection

Short Term Deflection of composite section Due to Dead Load,

$$\Delta_{i1,2} = \frac{5 * (M_{D1} + M_{D2}) * L^2 * 12^3}{48 * E_{c2} * I_{g2}/1000} = 0.074 \text{ in}$$

Deflection Due to Shrinkage

For Simple Span, Ksh= 0.125

$\rho = \dfrac{A_s * 100}{b * d}$ = 1.16 %

(According to Fig.10-3 of PCA Note on ACI318), Ash= 0.789

Time Dependant Shrinkage Strain, $\varepsilon_{sht} = 400 * 10^{-6}$ = 0.00040

Deflection Due to Shrinkage, $\Delta_{sh} = 0.64 * Ksh * Ash * \varepsilon_{sht} * L^2 * 12^2 * r / h$ = 0.047 in

Deflection Due to Creep

For No Compression Reinforcement Factor of, k_r= 0.85

Average Creep Coefficient (According to Cl.2.3.4 of ACI435), Cu= 1.67

Deflection Due to Creep, $\Delta_{cp} = Cu * \Delta_{i1,2} * k_r$ = 0.105 in

Deflection Due to Live Load

Deflection Due to Live Load , $\Delta_L = \dfrac{5 * (M_{D1} + M_{D2} + M_L) * L^2 * 12^3}{48 * E_{c2} * I_{ed,l} / 1000} - \Delta_{i1,2}$ = 0.232 in

Deflection Due to Creep Sustained Live Load, $\Delta_{cp,\,L} = Cu * \dfrac{Sus}{100} * \Delta_L * k_r$ = 0.066 in

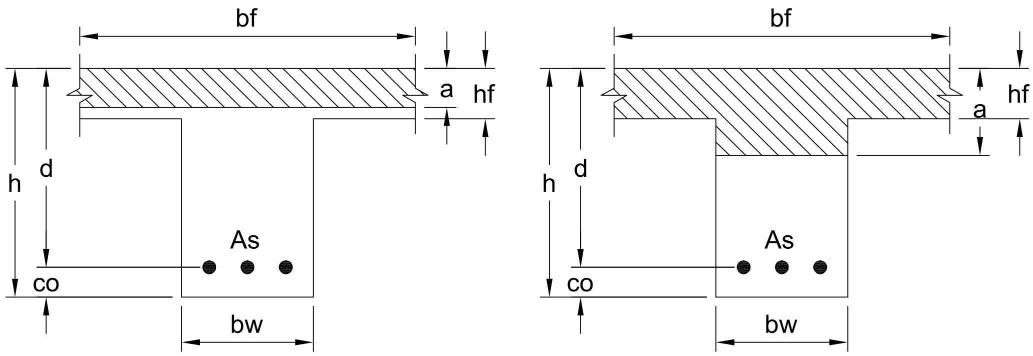
Total Long Term Deflection

Total Deflection, $\Delta_u = \Delta_{i1,2} * 3.53 + \Delta_{sh} + \Delta_{cp} + \Delta_L$ = 0.65 in

Calculation Summary

Total Deflection, $\Delta_u = \Delta_{i1,2} * 3.53 + \Delta_{sh} + \Delta_{cp} + \Delta_L$ = 0.65 in

Design of Flanged Section with Tension Reinforcement only as per ACI 318-11 Chapters 9 & 10



Design as Rectangular Section   Design as Flanged Section

System

Width of Concrete Flange, b_f=			30.0 in
Width of Concrete Web, b_w=			10.0 in
Depth of Concrete Section, h=			20.0 in
Thickness of Top Flange, h_f=			2.5 in
Concrete Cover, co=			1.0 in
Effective Depth of Concrete Section, d=	$h - co = 20.0 - 1.0$	=	19.0 in

Load

Bending Moment due to Dead Load, M_D=			72.0 kip*ft
Bending Moment due to Live Load, M_L=			196.0 kip*ft
Ultimate Bending Moment, M_U=	$(1.2 * M_D) + (1.6 * M_L)$	=	400.0 kip*ft

Material Properties

Concrete Strength, f'_c=			4000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=			0.90

Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318),

β_1= IF(f'_c≤4000;0.85;IF(f'_c≥8000;0.65;1.05-0.00005*f'_c)) = 0.85

Design as Flanged Section

Compressive Strength of Flange, C_f= $0.85 * f'_c * h_f * \frac{b_f - b_w}{1000}$ = 170.0 kips

Area of Reinforcement for Flange in Compression, A_{sf}= $\frac{C_f}{f_y} * 1000$ = 2.83 in^2

Nominal Moment for Flange, M_{nf}= $\frac{A_{sf} * f_y}{12000} * \left(d - \frac{h_f}{2}\right)$ = 251.2 kip*ft

Nominal Moment for Web, M_{nw}= $M_U / \Phi - M_{nf}$ = 193.24 kip*ft

R_{nw}= $\frac{M_{nw} * 12000}{\Phi * b_w * d^2}$ = 713.7 psi

ρ_w= $0.85 * f'_c / f_y * \left(1 - \sqrt{1 - \frac{2 * R_{nw}}{0.85 * f'_c}}\right)$ = 0.0135

Area of Reinforcement for Web in Compression, A_{sw}= $\rho_w * b_w * d$ = 2.56 in^2

Required Area of Reinforcement, A_{s_T}= $A_{sf} + A_{sw}$ = 5.39 in^2

Depth of Rectangular Stress Block for Web, a_w= $\frac{A_{sw} * f_y}{0.85 * f'_c * b_w}$ = 4.52 in

Design as Rectangular Section

R_n= $\frac{M_U * 12000}{\Phi * b_f * d^2}$ = 492.46 psi

ρ= $0.85 * \frac{f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_n}{0.85 * f'_c}}\right)$ = 0.0089

Area of Reinforcement, A_{s_R}= $\rho * b_f * d$ = 5.07 in^2

Depth of Rectangular Stress Block, a= $\frac{A_{s\_R} * f_y}{0.85 * f'_c * b_f}$ = 2.98 in

Section Type and Reinforcement

Section Design as:	IF($a>h_f$; "Flanged Sec."; "Rectangular Sec.")	=	Flanged Sec.
Area of Reinforcement, A_s=	IF($a>h_f$; A_{s_T}; A_{s_R})	=	5.39 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

A_{s_min1}=	$\frac{3*\sqrt{f'_c}*b_f*d}{fy}$	=	1.80 in^2
A_{s_min2}=	$\frac{200*b_f*d}{fy}$	=	1.90 in^2
A_{s_min}=	MAX(A_{s_min1} ; A_{s_min2})	=	1.90 in^2
Required Area of Reinforcement, A_{sc_Req}=	MAX(A_s; A_{s_min})	=	5.39 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.10
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	1.27 in^2
Number of Bars, n=			5
Vertical Reinforcement, A_{sc_Prov}=	A_{sb} * n	=	6.35 in^2
Check Validity=	IF($A_{sc\_Prov} \geq A_{sc\_Req}$; "Valid"; "Invalid")	=	Valid

Check Tension Controlled

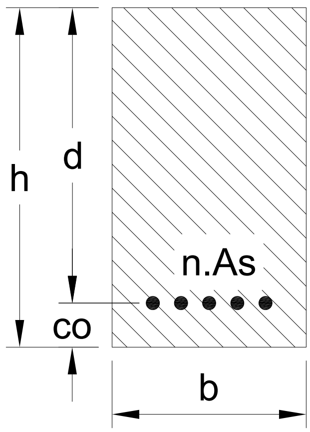
Distance from Extreme Compression Fiber to Neutral Axis,

c=	IF($a>h_f$; a_w/β_1;a/β_1)	=	5.32 in
c/d =	$c/d = 5.32/19.0$	=	0.280
IF(c/d>0.375; "Add Com. RFT"; "Tension Controlled")		=	Tension Controlled

Design Summary

Required Area of Reinforcement, A_{sc}=	A_{sc_Prov}	=	6.35 in^2

Cracking Moment Strength for Prestressed Sections as per ACI 318-11 Chapter 18



System

Width of Concrete Section, b=			12.0 in
Depth of Concrete Section, h=			24.0 in
Concrete Cover, co=			2.0 in
Effective Depth of Concrete Section, d=	$h - co = 24.0 - 2.0$	=	22.0 in
Number of Strands, n=			6.0
Area of One Strand, A_s=			0.153 in^2

Material Properties

Concrete Strength, f'_c=			5000 psi
Tensile Strength of Prestressed Steel, f_{pu}=			270000 psi
Jacking Stress, J_s=	$0.7 * f_{pu}$	=	189000 psi
Percentage of Losses, L_s=			20.00 %
Modification Factor for Lightweight Concrete, λ=			1.00
Modulus of Rupture (According to Eq. 9-10 of ACI318), f_r=	$7.5 * \lambda * \sqrt{f'_c}$	=	530 psi

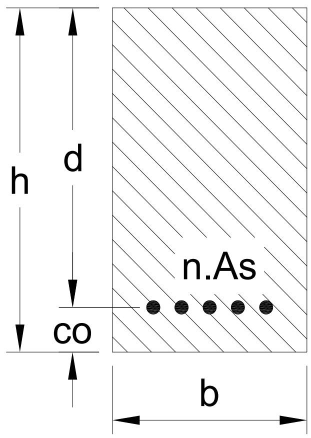
Calculation of Cracking Moment Strength

Area of Concrete, A_c=	$b * h$	=	288.0 in^2
Concrete Section Modulus, S_b=	$b * h^2 / 6$	=	1152.0 in^3
Eccentricity of Prestressing, e=	$h/2 - co$	=	10.0 in
Effective Prestress Force, P_{se}=	$(1-L_s/100) * n * A_s * J_s / 1000$	=	138.8 kips
Cracking Moment Strength, M_{cr}=	$\left(\frac{f_r}{1000} + \frac{P_{se}}{A_c}\right) * \frac{S_b}{12} + P_{se} * \frac{e}{12}$	=	212.8 kip*ft

Calculation Summary

Cracking Moment Strength, M_{cr}=	M_{cr}	=	212.8 kip*ft

Flexural Strength of Prestressed Member Using Approximate Value of f_{ps} As per ACI 318-11



System

Width of Concrete Section, b= 12.0 in

Depth of Concrete Section, h= 24.0 in

Concrete Cover, co= 2.0 in

Effective Depth of Concrete Section, d= h - co = 24.0 - 2.0 = 22.0 in

Number of Strands, n= 6

Area of One Strand, A_s= 0.153 in^2

Material Properties

Concrete Strength, f'_c= 5000 psi

Tensile Strength of Prestressed Steel, f_{pu}= 270000 psi

Yield Strength of Prestressed Steel, f_{py}= 0.9 * f_{pu} = 243000 psi

Factor for Type of Prestressing Steel (According to Cl.18.7.2 of ACI318), γ_p= 0.28

Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318),

β_1= IF(f'_c≤4000;0.85;IF(f'_c≥8000;0.65;1.05-0.00005*f'_c)) = 0.80

Calculation of Stress for Prestressed Reinforcement

Prestressed Reinforcement Ratio, ρ_p= $n * A_s / (b * d)$ = 0.00348

Prestressing Force (According to Eq. 18-1 of ACI318),

f_{ps}= $\frac{f_{pu}}{1000} * \left(1 - \frac{\gamma_p}{\beta_1} * \rho_p * \frac{f_{pu}}{f'_c}\right)$ = 252 ksi

Calculation of Nominal Moment Strength

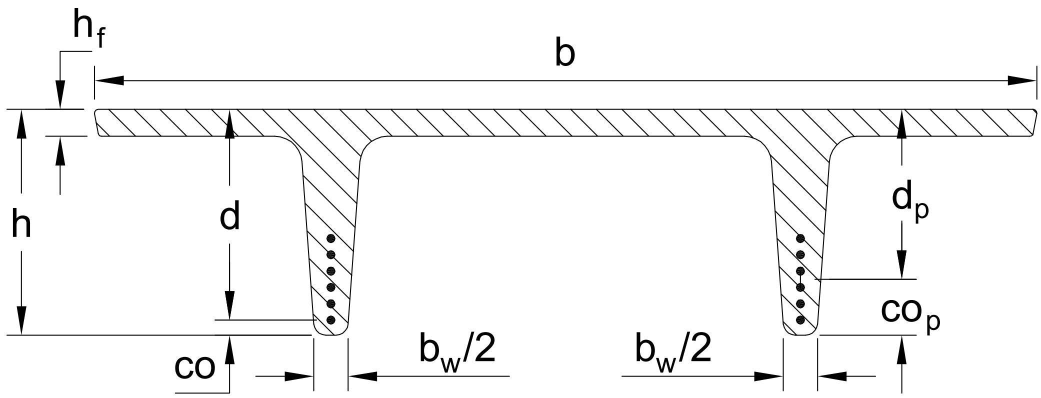
Distance of Compression Block, a= $\frac{n * A_s * f_{ps}}{0.85 * b * f'_c / 1000}$ = 4.54 in

Nominal Moment Strength, M_n= $\frac{n * A_s * f_{ps}}{12} * \left(d - \frac{a}{2}\right)$ = 380.4 kip*ft

Calculation Summary

Nominal Moment Strength, M_n= M_n = 380.4 kip*ft

Tension Controlled Limit for Prestressed Flexural Member as per ACI 318-11 Chapters 10 & 18



System

Width of Concrete Double Tee Section, b=			84.0 in
Width of Web of Concrete Double Tee Section, b_w=			15.5 in
Depth of Concrete Double Tee Section, h=			32.0 in
Thickness of Concrete Top Slab, h_f=			2.0 in
Concrete Cover, co=			2.0 in
Concrete Cover to CG of Prestressed Steel, co_p=			4.5 in
Effective Depth of Concrete Section, d=	h - co	=	30.0 in
Effective Depth of Concrete Section, d_p=	h - co_p	=	27.5 in
Number of Strands, n=			22.0
Area of One Strand, A_s=			0.153 in^2

Material Properties

Concrete Strength, f'_c=			5000 psi
Tensile Strength of Prestressed Steel, f_{pu}=			270000 psi
Yield Strength of Prestressed Steel, f_{py}=	0.9 * f_{pu}	=	243000 psi
Factor for Type of Prestressing Steel (According to Cl.18.7.2 of ACI318), γ_p=			0.28

Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318),

β_1= IF(f'_c≤4000; 0.85; IF(f'_c≥8000; 0.65; 1.05-0.00005*f'_c)) = 0.80

Calculation of Stress in Prestressed Reinforcement

ω_{pu}= $(n * A_s) * f_{pu} / (b * d_p * f'_c)$ = 0.079

Prestressing Force (According to Eq.18-1 of ACI318),

f_{ps}= $f_{pu} * \left(1 - \frac{\gamma_p}{\beta_1} * \omega_{pu}\right)$ = 262535 psi

Area of Reinforcement for Compression in Flange,

A_{pf}= $0.85 * h_f * f'_c * (b-b_w) / f_{pu}$ = 2.16 in^2

Calculation of Depth of Concrete Stress Block

a_i= $(n * A_s) * f_{ps} / (0.85 * b * f'_c)$ = 2.48 in

For $a_i > h_f$:

a_1= $(n * A_s - A_{pf}) * f_{ps} / (0.85 * b_w * f'_c)$ = 4.81 in

For $a_i \leq h_f$:

a_2= a_i = 2.48 in

a= $IF(a_i>h_f\ ;\ a_1\ ;\ a_2)$ = 4.81 in

c= a / β_1 = 6.01 in

Check Tension Controlled

c/d = c / d = 0.200

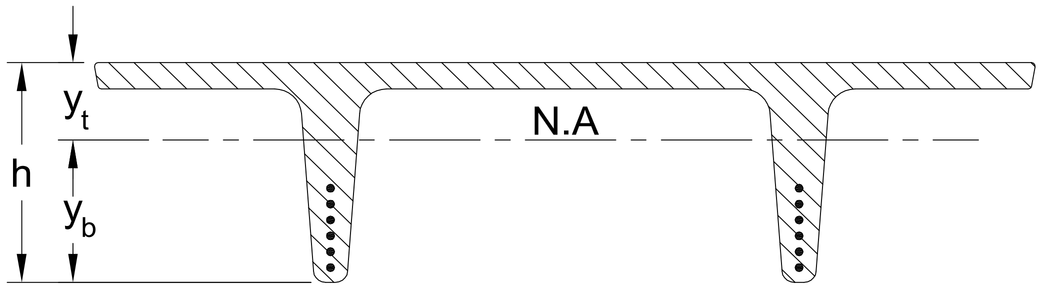
IF(c/d>0.375; "Compression Controlled"; "Tension Controlled") = Tension Controlled

Calculation Summary

Type of Section:

IF(c/d>0.375; "Compression Controlled"; "Tension Controlled") = Tension Controlled

Estimating Prestress Losses as per ACI 318-11 Chapter 18



System

Area of Concrete Section, A_c=			449 in^2
Depth of Concrete Section, h=			24 in
Concrete Cover, co=			2 in
Effective Depth of Concrete Section, d=	h - co	=	22 in
Moment of Inertia for Concrete Section, I_c=			22469 in^4
Distance from Bottom Fiber to Neutral Axis, y_b=			17.77 in
Distance from Top Fiber to Neutral Axis, y_t=	h - y_b	=	6.23 in
Number of Strands, n=			8.0
Area of One Strand, A_s=			0.153 in^2
Eccentricity of Strands, e=			9.77 in
Volume per Surface Area, V.S=			1.35 in
Average Relative Humidity, RH=			75.00 %

Load

Factored Moment due to Dead Load, M_D=	1617 kip*in
Factored Moment due to Superimposed Dead Load, M_{SD}=	691 kip*in
Factored Moment due to Live Load, M_L=	1382.00 kip*in

Material Properties

Concrete Strength, f'_{ci}= 3500 psi

Concrete Strength, f'_c= 5000 psi

Tensile Strength of Prestressed Steel, f_{pu}= 270000 psi

Yield Strength of Prestressed Steel, f_{py}= $0.9 * f_{pu}$ = 243000 psi

Jacking Stress, J_s= $0.74 * f_{pu}$ = 199800 psi

Modification Factor for Lightweight Concrete, λ= 1.00

Modulus of Rupture (According to Eq. 9-10 of ACI318), f_r= $7.5 * \lambda * \sqrt{f'_c}$ = 530 psi

Concrete Density, w_c= 150 psi

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI318),

Modulus of Elasticity for Initial Concrete, E_{ci}= $w_c^{1.5} * 33 * \sqrt{f'_{ci}}$ = 3586616 psi

Modulus of Elasticity for Concrete, E_c= $w_c^{1.5} * 33 * \sqrt{f'_c}$ = 4286826 psi

Modulus of Elasticity of Prestressed Steel, E_s= 28500000 psi

Calculation of Losses

1- Elastic Shortening of Concrete (ES)

Initial Force of Prestress, P_{pi}= $J_s * (n * A_s)/1000$ = 244.6 kips

Prestress Type= SEL("ACI/Kes" ;Type;) = Pretensioned

K_{es}= TAB("ACI/Kes" ;Kes ;Type=Type) = 1.00

K_{cir}= TAB("ACI/Kcir" ;Kcir ;Type=Type) = 0.90

f_{cir}= $K_{cir} * \left(\frac{P_{pi}}{A_c} + \frac{P_{pi}}{I_c} * e^2\right) - \frac{M_D * e}{I_c}$ = 0.722 ksi

Elastic Shortening of Concrete, ES= $K_{es} * E_s * f_{cir} / E_{ci}$ = 5.74 ksi

2- Creep of Concrete (CR)

Prestress Type= SEL("ACI/Kcr" ;Type;) = Pretensioned

Factor of, Kcr= TAB("ACI/Kcr" ;Kcr ;Type=Type) = 2.00

Creep Losses, CR= $K_{cr} * \frac{E_s}{E_c} * \left(f_{cir} - M_{SD} * \frac{e}{I_c}\right)$ = 5.61 ksi

3- Shrinkage of Concrete (SH)

Prestress Type= SEL("ACI/Ksh" ;Type;) = Pretensioned

Factor of, K_{sh}= TAB("ACI/Ksh" ;Ksh ;Type=Type) = 1.00

Shrinkage Losses, SH= $8.2 * 10^{-6} * K_{sh} * \frac{E_s}{1000} * (1 - 0.06 * V.S) * (100 - RH)$ = 5.37 ksi

4- Relaxation of Tendon (RE)

Prestress Type=	SEL("ACI/KreJ" ;Type;)	=	relaxation strand-Grade Low 270
Factor of, K_{re}=	TAB("ACI/KreJ" ;Kre ;Type=Type)	=	5000 psi
Factor of, J=	TAB("ACI/KreJ" ;J ;Type=Type)	=	0.04
Ratio of f_{pi}/f_{pu}, r=	SEL("ACI/r" ;r;)	=	0.74
Factor of, C=	TAB("ACI/r" ;C ;r=r)	=	0.95
Relaxation of Tendon, RE=	$\left(\frac{K_{re}}{1000} - J*(SH+CR+ES)\right)*C$	=	4.11 ksi

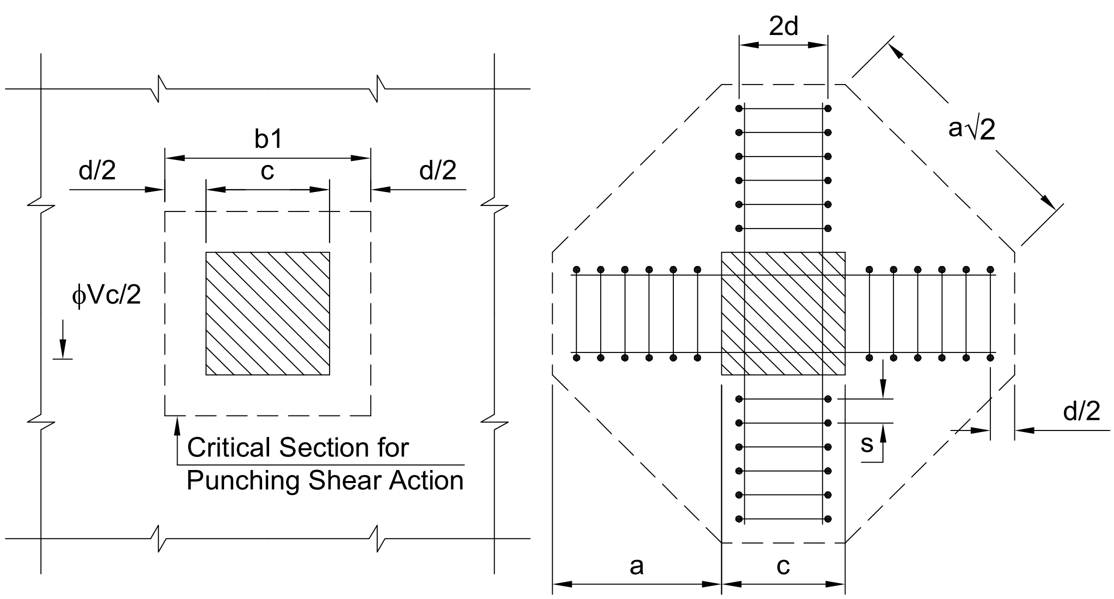
5- Total Allowance of Losses and Effective Prestress Force after all Losses

Total Allowance of Losses, L_s=	$ES+CR+SH+RE$	=	21 ksi
Effective Prestress Stress, f_{se}=	$J_s/1000-L_s$	=	179 ksi
Effective Prestress Force after All Losses, P_e=	$fse\ *(n*A_s)$	=	219 kips

Calculation Summary

Total Allowance of Losses, L_s=	L_s	=	21 ksi
Effective Prestress Force after All Losses, P_e=	P_e	=	219 kips

Design Shear Reinforcement for Slab which to resist Punching Stress around Interior Square Column As per ACI318-11 Chapter 11



System

Column Dimension, c=			12.0 in
Thickness of Concrete Slab, h=			7.5 in
Concrete Cover, co=			1.5 in
Effective Depth of Concrete Section, d=	$h - co = 7.5 - 1.5$	=	6.0 in
Bar Diameter of Shear Reinforcement, Dia=			0.375 in

Load

Ultimate Shear Force, V_u=			120.0 kips

Material Properties

Concrete Strength, f'_c=			4000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=			0.75
Modification Factor for Lightweight Concrete, λ=			1.00

Determine Concrete Shear Strength

b_1=	$c + d$	=	18.0 in
Perimeter of Critical Section, b_0=	$4 * b_1$	=	72.0 in

Nominal Shear Strength provided by Concrete (According to Eq. 11-33 of ACI318),

V_c=	$4 * \lambda * \sqrt{f'_c} * b_0 * d / 1000$	=	109.3 kips
Punching Shear Reinforcement is : IF(V_u>$\Phi * V_c$;"Required";"Not Required")		=	Required

Determine Area of Shear Reinforcement

Minimum Effective Depth of Slab with Shear Reinforcement (According to Cl.11.11.3 of ACI318),

d_{min}= MIN(6;16*Dia) = 6.0 in

Effective Depth of Slab : IF(d>d_{min};"Should Increase";"OK") = OK

Maximum Shear Strength of Slab with Shear Reinforcement (According to Cl.11.11.3.2 of ACI318),

V_n= $6 * \sqrt{f'_c} * b_0 * d / 1000$ = 163.9 kips

Validity : IF(V_u>Φ*V_n;"Not Valid";"Valid") = Valid

Shear Strength provided by Concrete with Shear RFT (According to Cl.11.11.3.1 of ACI318),

V_{ci}= $2 * \lambda * \sqrt{f'_c} * b_0 * d / 1000$ = 54.6 kips

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),

V_s= $\dfrac{V_u - \Phi * V_{ci}}{\Phi}$ = 105.4 kips

Spacing of Provided Bars, s= 3.0 in

Required Area of Reinforcement, A_v= $\dfrac{V_s * s * 1000}{f_y * d}$ = 0.88 in^2

Required Area of Reinforcement for each side of Column, A_{v_side}= $A_v / 4$ = 0.22 in^2

Perimeter of Critical Section where Shear Reinforcement may be terminated,

b'_0= $\dfrac{V_u * 1000}{\Phi * 2 * \lambda * \sqrt{f'_c} * d}$ = 210.8 in

Distance from Column Face where Shear Reinforcement may be terminated,

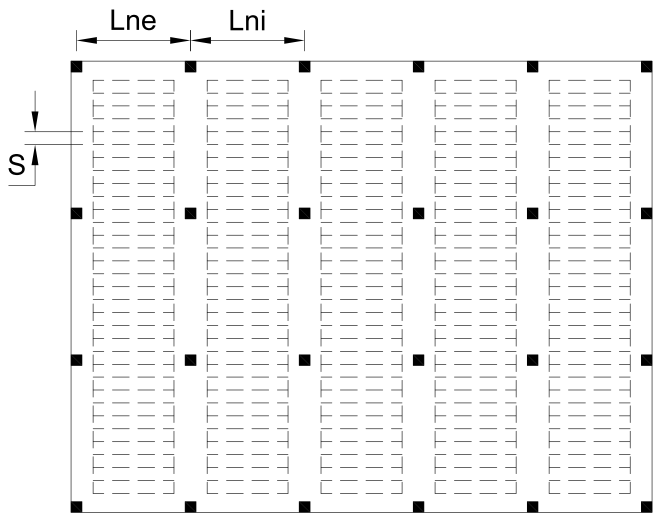
a= $\left(\dfrac{b'_0}{4} - c\right) / \sqrt{2}$ = 28.8 in

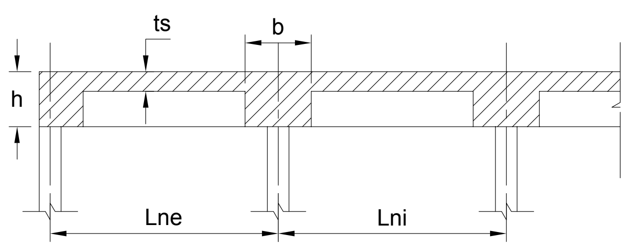
Design Summary

Required Area of Reinforcement, A_v= A_v = 0.88 in^2

Distance from Column Face where Shear Reinforcement may be terminated: a = 28.8 in

Design of One Way Joist as per ACI 318-11 Chapters 9 & 11





System

Width of Beam, b=	30.0 in
Width of Joist, b_j=	6.0 in
Spacing between Joists, s=	36.0 in
Slab Thickness, t_s=	3.5 in
Exterior Joist Span, L_{ne}=	27.5 ft
Interior Joist Span, L_{ni}=	27.0 ft
Concrete Cover, co=	1.25 in

Load

Dead Load, DL=	130 psf
Live Load, LL=	60 psf

Ultimate Load, w_u= $\frac{1.2*DL+1.6*LL}{1000}*\frac{s}{12}$ = 0.756 kip/ft

Material Properties

Concrete Strength, f'_c= 4000 psi

Yield Strength of Reinforcement, f_y= 60000 psi

Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.90

Modification Factor for Lightweight Concrete, λ= 1.00

Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3),

β_1= IF(f'_c≤4000;0.85;IF(f'_c≥8000;0.65;1.05-0.00005*f'_c)) = 0.85

Moment Distribution for Joist

1. End Span

Edge Negative Moment for Exterior Joist, $M_{nee}=\dfrac{w_u * L_{ne}^2}{24}$ = 23.8 kip*ft

Positive Moment for Exterior Joist, $M_{pe}=\dfrac{w_u * L_{ne}^2}{14}$ = 40.8 kip*ft

Negative Moment for Exterior Joist, $M_{ne}=\dfrac{w_u * ((L_{ne} + L_{ni})/2)^2}{10}$ = 56.1 kip*ft

2. Interior Spans

Negative Moment for Interior Joist, $M_{ni}=\dfrac{w_u * L_{ni}^2}{11}$ = 50.1 kip*ft

Positive Moment for Interior Joist , $M_{pi}=\dfrac{w_u * L_{ni}^2}{16}$ = 34.4 kip*ft

3. Maximum Moment

M_{max}= MAX(M_{nee}; M_{pe}; M_{ne}; M_{ni}; M_{pi}) = 56.1 kip*ft

Calculation of Required Depth for Joist

ρ_t =	$0.319 * f'_c * \beta_1 / f_y$	=	0.01808

For Reasonable Deflection Control, choose a Reinforcement Ratio (ρ) equal to about one-half (ρ_t)

Reinforcement Ratio, ρ=	$\rho_t / 2$	=	0.00904
ω=	$\rho * \frac{f_y}{f'_c}$	=	0.13560
Required Depth, d=	$\sqrt{\frac{M_{max} * 12000}{\Phi * bj * f'c * \omega * (1 - 0.59 * \omega)}}$	=	15.8 in
Required Thickness, h_{req}=	d + co	=	17.1 in
h_{min}=	MAX(L_{ne}*12+b; L_{ni}*12+b;)/18.5	=	19.5 in
Provided Thickness, h=	MAX(h_{req}; h_{min})	=	19.5 in
Effective Depth of Joist, d_j=	h-co	=	18.25 in

Calculation of Required Reinforcement for Exterior Negative Moment of End Span (A_{sc1})

R_{n1}=	$\frac{M_{nee} * 12000}{\Phi * b_j * d_j^2}$	=	159 psi
Reinforcement Ratio, ρ_1=	$0.85 * \frac{f'_c}{f_y} * \left(1 - \sqrt{1 - 2 * \frac{R_{n1}}{0.85 * f'_c}}\right)$	=	0.0027
Area of Reinforcement, A_{s1}=	$\rho_1 * b_j * d_j$	=	0.30 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

A_{s_min1}=	$\frac{3 * \sqrt{f'_c} * b_j * d_j}{f_y}$	=	0.35 in^2
A_{s_min2}=	$\frac{200 * b_j * d_j}{f_y}$	=	0.36 in^2
A_{s_min}=	MAX(A_{s_min1} ; A_{s_min2})	=	0.36 in^2
Required Area of Reinforcement, A_{sc1}=	MAX(A_{s1}; A_{s_min})	=	0.36 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.3
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.11 in^2
Number of Bars, n=			4
Vertical Reinforcement, A_{sc1_Prov}=	A_{sb} * n	=	0.44 in^2
Check Validity=	IF($A_{sc1\_Prov} \geq A_{sc1}$; "Valid"; "Invalid")	=	Valid

Calculation of Required Reinforcement for Positive Moment of End Span (A_{sc2})

$R_{n2}=$ $\dfrac{M_{pe} * 12000}{\Phi * b_j * d_j^2}$ = 272 psi

Reinforcement Ratio, $\rho_2=$ $0.85 * \dfrac{f'_c}{f_y} * \left(1 - \sqrt{1 - 2 * \dfrac{R_{n2}}{0.85 * f'_c}}\right)$ = 0.0047

Area of Reinforcement, $A_{s2}=$ $\rho_2 * b_j * d_j$ = 0.51 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$A_{s\_min1}=$ $\dfrac{3 * \sqrt{f'_c} * b_j * d_j}{f_y}$ = 0.35 in^2

$A_{s\_min2}=$ $\dfrac{200 * b_j * d_j}{f_y}$ = 0.36 in^2

$A_{s\_min}=$ MAX(A_{s_min1} ; A_{s_min2}) = 0.36 in^2

Required Area of Reinforcement, $A_{sc2}=$ MAX(A_{s2}; A_{s_min}) = 0.51 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, $A_{sb}=$ TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n= 2

Vertical Reinforcement, $A_{sc2\_Prov}=$ $A_{sb} * n$ = 0.62 in^2

Check Validity= IF($A_{sc2\_Prov} \geq A_{sc2}$; "Valid"; "Invalid") = Valid

Calculation of Required Reinforcement for Interior Negative Moment of End Span (A_{sc3})

$R_{n3}=$ $\dfrac{M_{ne} * 12000}{\Phi * b_j * d_j^2}$ = 374 psi

Reinforcement Ratio, $\rho_3=$ $0.85 * \dfrac{f'_c}{f_y} * \left(1 - \sqrt{1 - 2 * \dfrac{R_{n3}}{0.85 * f'_c}}\right)$ = 0.0066

Area of Reinforcement, $A_{s3}=$ $\rho_3 * b_j * d_j$ = 0.72 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$A_{s\_min1}=$ $\dfrac{3 * \sqrt{f'_c} * b_j * d_j}{f_y}$ = 0.35 in^2

$A_{s\_min2}=$ $\dfrac{200 * b_j * d_j}{f_y}$ = 0.36 in^2

$A_{s\_min}=$ MAX(A_{s_min1} ; A_{s_min2}) = 0.36 in^2

Required Area of Reinforcement, $A_{sc3}=$ MAX(A_{s3}; A_{s_min}) = 0.72 in^2

Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.5
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.31 in^2
Number of Bars, n=			3
Vertical Reinforcement, A_{sc3_Prov}=	$A_{sb} * n$	=	0.93 in^2
Check Validity=	IF($A_{sc3\_Prov} \geq A_{sc3}$; "Valid"; "Invalid")	=	Valid

Calculation of Required Reinforcement for Interior Negative Moment of Interior Span (A_{sc4})

R_{n4}=	$\frac{M_{ni} * 12000}{\Phi * b_j * d_j^2}$	=	334 psi
Reinforcement Ratio, ρ_4=	$0.85 * \frac{f'_c}{f_y} * \left(1 - \sqrt{1 - 2 * \frac{R_{n4}}{0.85 * f'_c}}\right)$	=	0.0059
Area of Reinforcement, A_{s4}=	$\rho_4 * b_j * d_j$	=	0.65 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

A_{s_min1}=	$\frac{3 * \sqrt{f'_c} * b_j * d_j}{f_y}$	=	0.35 in^2
A_{s_min2}=	$\frac{200 * b_j * d_j}{f_y}$	=	0.36 in^2
A_{s_min}=	MAX(A_{s_min1} ; A_{s_min2})	=	0.36 in^2
Required Area of Reinforcement, A_{sc4}=	MAX(A_{s4}; A_{s_min})	=	0.65 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.5
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.31 in^2
Number of Bars, n=			3
Vertical Reinforcement, A_{sc4_Prov}=	$A_{sb} * n$	=	0.93 in^2
Check Validity=	IF($A_{sc4\_Prov} \geq A_{sc4}$; "Valid"; "Invalid")	=	Valid

Calculation of Required Reinforcement for Interior Positive Moment of Interior Span (A_{sc5})

R_{n5}= $\dfrac{M_{pi} * 12000}{\Phi * b_j * d_j^2}$ = 230 psi

Reinforcement Ratio, ρ_5= $0.85 * \dfrac{f'_c}{f_y} * \left(1 - \sqrt{1 - 2 * \dfrac{R_{n5}}{0.85 * f'_c}}\right)$ = 0.0040

Area of Reinforcement, A_{s5}= $\rho_5 * b_j * d_j$ = 0.44 in^2

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

A_{s_min1}= $\dfrac{3 * \sqrt{f'_c} * b_j * d_j}{f_y}$ = 0.35 in^2

A_{s_min2}= $\dfrac{200 * b_j * d_j}{f_y}$ = 0.36 in^2

A_{s_min}= MAX(A_{s_min1} ; A_{s_min2}) = 0.36 in^2

Required Area of Reinforcement, A_{sc5}= MAX(A_{s5}; A_{s_min}) = 0.44 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n= 2

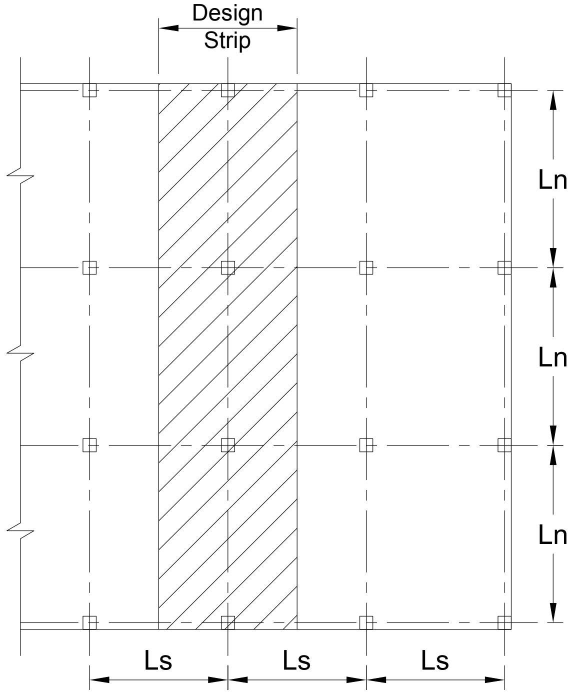
Vertical Reinforcement, A_{sc5_Prov}= $A_{sb} * n$ = 0.62 in^2

Check Validity= IF($A_{sc5\_Prov} \geq A_{sc5}$; "Valid"; "Invalid") = Valid

Design Summary

Area of Reinforcement for Exterior Negative Moment of End Span:	A_{sc1_Prov}	=	0.44 in^2
Area of Reinforcement for Positive Moment of End Span:	A_{sc2_Prov}	=	0.62 in^2
Area of Reinforcement for Interior Negative Moment of End Span:	A_{sc3_Prov}	=	0.93 in^2
Area of Reinforcement for Interior Negative Moment of Interior Span:	A_{sc4_Prov}	=	0.93 in^2
Area of Reinforcement for Interior Positive Moment of Interior Span:	A_{sc5_Prov}	=	0.62 in^2

Design Two-Way Slab without Beams Analyzed by the Direct Design Method As per ACI 318-11



System

Longer Span for Two-Way Slab, L_n=			18.00 ft
Shorter Span for Two-Way Slab, L_s=			14.00 ft
Thickness of Slab, h=			7.00 in
Concrete Cover, co=			1.25 in
Depth of Slab, d=	h-co	=	5.75 in
Square Column Dimension, b_c=			16.00 in
Width of Column Strip, b=	Ls*12 / 2	=	84 in
Width of Middle Strip, b_m=	Ls*12 - b	=	84 in

Load

Slab Self Weight, q_s=	h/12 * 150	=	87.50 psf
Partition Load, q_p=			20.00 psf
Dead Load, q_D=	$q_s + q_p$	=	107.50 psf
Live Load, q_L=			40.00 psf
Ultimate Load, q_U=	$1.2 * q_D + 1.6 * q_L$	=	193.00 psf

Material Properties

Concrete Strength, f'_c=	3000 psi
Yield Strength of Reinforcement, f_y=	60000 psi

Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.90

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

ρ_{min}= IF(f_y≤50000;0.002;IF(f_y≥77143;0.0014;0.0018)) = 0.0018

Total Static Moment of Slab

Total Factored Static Moment Per Span (According to Eq. 13-4 of ACI318),

$$M_0 = \frac{q_U * L_s}{8*1000} * \left(L_n - \frac{b_c}{12}\right)^2 = 93.82 \text{ kip*ft}$$

Flexural Reinforcement Required for Negative Moment of Column Strip

$$R_{n1} = \frac{M_0 * 12000 * 0.53}{\Phi * b * d^2} = 239 \text{ psi}$$

$$\text{Ratio of RFT, } \rho_1 = \frac{0.85 * f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_{n1}}{0.85 * f'_c}}\right) = 0.00419$$

Area of Steel, A_{s1_Req}=	MAX(ρ_1;ρ_{min}) * b * d	=	2.02 in²
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.6
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.44 in^2
Number of Bars, n=			5
Vertical Reinforcement, A_{s1_Prov}=	A_{sb} * n	=	2.20 in^2
Check Validity=	IF(A_{s1_Prov}≥A_{s1_Req}; "Valid"; "Invalid")	=	Valid

Flexural Reinforcement Required for Positive Moment of Column Strip

$$R_{n2} = \frac{M_0 * 12000 * 0.31}{\Phi * b * d^2} = 140 \text{ psi}$$

$$\text{Ratio of RFT, } \rho_2 = \frac{0.85 * f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_{n2}}{0.85 * f'_c}}\right) = 0.00240$$

Area of Steel, A_{s2_Req}=	MAX(ρ_2;ρ_{min}) * b * d	=	1.16 in²
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.6
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.44 in^2
Number of Bars, n=			3
Vertical Reinforcement, A_{s2_Prov}=	A_{sb} * n	=	1.32 in^2
Check Validity=	IF(A_{s2_Prov}≥A_{s2_Req}; "Valid"; "Invalid")	=	Valid

Flexural Reinforcement Required for Negative Moment of Middle Strip

$R_{n3}=$ $\frac{M_0 * 12000 * 0.17}{\Phi * b * d^2}$ = 77 psi

Ratio of RFT, $\rho_3=$ $\frac{0.85 * f'c}{fy} * \left(1 - \sqrt{1 - \frac{2 * R_{n3}}{0.85 * f'c}}\right)$ = 0.00130

Area of Steel, $A_{s3\_Req}=$ MAX(ρ_3;ρ_{min}) * b * d = 0.87 in²

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.6

Provided Reinforcement, $A_{sb}=$ TAB("ACI/Bar"; Asb; Bar=Bar) = 0.44 in^2

Number of Bars, n= 2

Vertical Reinforcement, $A_{s3\_Prov}=$ A_{sb} * n = 0.88 in^2

Check Validity= IF($A_{s3\_Prov} \geq A_{s3\_Req}$; "Valid"; "Invalid") = Valid

Flexural Reinforcement Required for Positive Moment of Middle Strip

$R_{n4}=$ $\frac{M_0 * 12000 * 0.21}{\Phi * b * d^2}$ = 95 psi

Ratio of RFT, $\rho_4=$ $\frac{0.85 * f'c}{fy} * \left(1 - \sqrt{1 - \frac{2 * R_{n4}}{0.85 * f'c}}\right)$ = 0.00161

Area of Steel, $A_{s4\_Req}=$ MAX(ρ_4;ρ_{min}) * b * d = 0.87 in²

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.6

Provided Reinforcement, $A_{sb}=$ TAB("ACI/Bar"; Asb; Bar=Bar) = 0.44 in^2

Number of Bars, n= 2

Vertical Reinforcement, $A_{s4\_Prov}=$ A_{sb} * n = 0.88 in^2

Check Validity= IF($A_{s4\_Prov} \geq A_{s4\_Req}$; "Valid"; "Invalid") = Valid

Design Summary

Area of Reinforcement Required for Negative Moment of Middle Strip,

$A_{s1\_Prov}=$ A_{s1_Prov} = 2.20 in^2

Area of Reinforcement Required for Negative Moment of Middle Strip,

$A_{s2\_Prov}=$ A_{s2_Prov} = 1.32 in^2

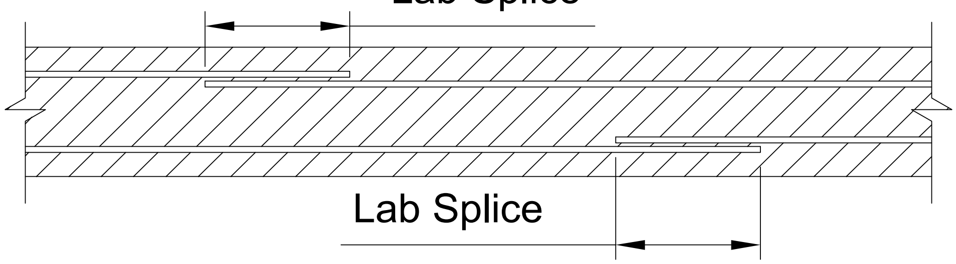
Area of Reinforcement Required for Negative Moment of Middle Strip,

$A_{s3\_Prov}=$ A_{s3_Prov} = 0.88 in^2

Area of Reinforcement Required for Positive Moment of Middle Strip,

$A_{s4\_Prov}=$ A_{s4_Prov} = 0.88 in^2

Calculating Development Length of Bars in Tension as per ACI 318-11 Chapter 12



Material Properties

Concrete Strength, f'_c= 4000 psi

Yield Strength of Reinforcement, f_y= 60000 psi

Modification Factor for Lightweight Concrete, λ= 0.75

Factor of Development Length Based on RFT Location (According to Cl.12.2.4 of ACI318),

ψ_t= 1.30

Factor of Development Length Based on RFT Coating (According to Cl.12.2.4 of ACI318),

ψ_e= 1.50

Maximum Modifying Factor, ψ_{te}= MIN(ψ_t*ψ_e; 1.7) = 1.70

Identification of, Bar= SEL("ACI/Bar" ;Bar;) = No.7

Diameter of Bars, d_b= TAB("ACI/Bar" ;Dia ;Bar=Bar) = 0.88 in

Calculation of Development Length

1. Class A Splice

Development Length for Bars No.6 and Smaller (According to Cl.12.2.2 of ACI318),

$$L_{d\_A1} = \left(\frac{3 * f_y * \psi_{te}}{50 * \lambda * \sqrt{f'_c}}\right) * d_b = 114 \text{ in}$$

Development Length for Bars No.7 and Greater (According to Cl.12.2.2 of ACI318),

$$L_{d\_A2} = \left(\frac{3 * f_y * \psi_{te}}{40 * \lambda * \sqrt{f'_c}}\right) * d_b = 142 \text{ in}$$

L_{d_A}= IF($d_b \leq 0.75$; L_{d_A1} ; L_{d_A2}) = 142 in

2. Class B Splice

Development Length for Bars No.6 and Smaller (According to Cl.12.2.2 of ACI 318),

$$L_{d\_B1} = \left(\frac{3 * f_y * \psi_{te}}{50 * \lambda * \sqrt{f'_c}}\right) * 1.3 * d_b = 148 \text{ in}$$

Development Length for Bars No.7 and Greater (According to Cl.12.2.2 of ACI318),

$L_{d\_B2}=$ $\left(\dfrac{3*f_y*\psi_{te}}{40*\lambda*\sqrt{f'_c}}\right)*1.3*d_b$ = 184 in

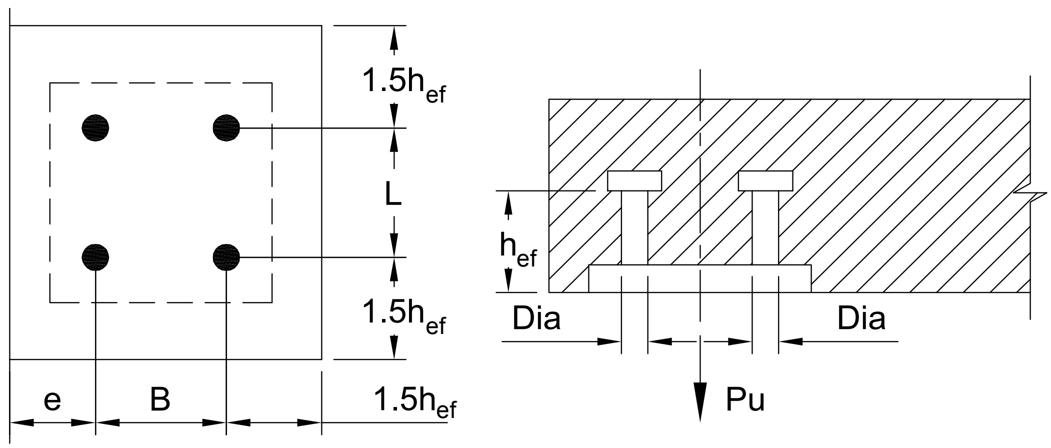
$L_{d\_B}=$ $IF(d_b\leq 0.75 ; L_{d\_B1} ; L_{d\_B2})$ = 184 in

Calculation Summary

Development Length for Class A, $L_{d\_A}=$ L_{d_A} = 142 in

Development Length for Class B, $L_{d\_B}=$ L_{d_B} = 184 in

<u>Group of Headed Studs in Tension Near an Edge as per ACI 318-11 Appendix D</u>



System

Spacing between Bolts along x-x, B=	6.00 in
Spacing between Bolts along y-y, L=	6.00 in
Distance to Edge from Nearst bolt, e=	3.00 in

Load

Ultimate Load, P_u=	14000 lb
Number of Anchors, n=	4

Material Properties

Concrete Strength, f'_c=	4000 psi
Tensile Strenth of Anchor Bolt Grade, f_{uta}=	60000 psi
Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), Φ_1=	0.75
Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), Φ_2=	0.70
Modification Factor for Lightweight Concrete, λ=	1.00

Determine Anchor Diameter

Required Effective Area of Anchor Bolt (According to Eq.D.2 of ACI318),

A_{se_Req}= $\dfrac{P_u}{\Phi_1 * n * f_{uta}}$ = 0.078 in^2

Provided Anchor Bolt, Dia= SEL("ACI/Anchor"; Dia;) = 0.500 in

Provided Area of Anchor Bolt, A_{se_Prov}= TAB("ACI/Anchor"; Ase; Dia=Dia) = 0.142 in^2

Check Validity= IF($A_{se\_Prov} \geq A_{se\_Req}$; "Valid"; "Increase Dia") = Valid

Determine Embedment Length

Assume that, h_{ef_Prov}= 4.50 in

Projected Area of Failure Surface for Anchors (According to Cl.D.5.2.1 of ACI318),

A_{nc}= $(1.5 * h_{ef\_Prov} + L + e) * (1.5 * 2 * h_{ef\_Prov} + B)$ = 307 in^2

Projected Area of Failure Surface for Single Anchor (According to Cl.D.5.2.1 of ACI318),

A_{nco}= $9 * h_{ef\_Prov}^{2}$ = 182 in^2

Check Validity= IF(A_{nc}<n*A_{nco}; "Valid"; "Increase hef") = Valid

Factor (According to Cl.D.5.2.4 of ACI318), $\psi_{ec,N}$= 1.00

Factor (According to Cl.D.5.2.5 of ACI318), $\psi_{ed,N}$= $0.7 + \frac{0.3 * e}{1.5 * h_{ef\_Prov}}$ = 0.83

Factor (According to Cl.D.5.2.6 of ACI318), $\psi_{c,N}$= 1.00

Factor (According to Cl.D.5.2.6 of ACI318), $\psi_{cp,N}$= 1.00

Basic Strength of Concrete Breakout (According to Eq.D-6 of ACI318),

N_b= $24 * \lambda * \sqrt{f'c} * h_{ef\_Prov}^{1.5}$ = 14490 lb

Nominal Strength of Concrete Breakout (According to Eq.D-5 of ACI318),

N_{cbg}= $\frac{A_{nc}}{A_{nco}} * \psi_{ec,N} * \psi_{ed,N} * \psi_{c,N} * \psi_{cp,N} * N_b$ = 20287 lb

Check Validition= IF(P_u< Φ_2*N_{cbg}; "Valid"; "Increase h_{ef}") = Valid

Calculation of Reqired Head Size

Factor (According to Cl.D.5.3.6 of ACI318), $\psi_{c,P}$= 1.00

Required Head Size for Anchor Bolt (According to Eq.D-15 of ACI318),

A_{brg}= $\frac{Pu / n}{\Phi_2 * \psi_{c,P} * 8 * f'c}$ = 0.156 in^2

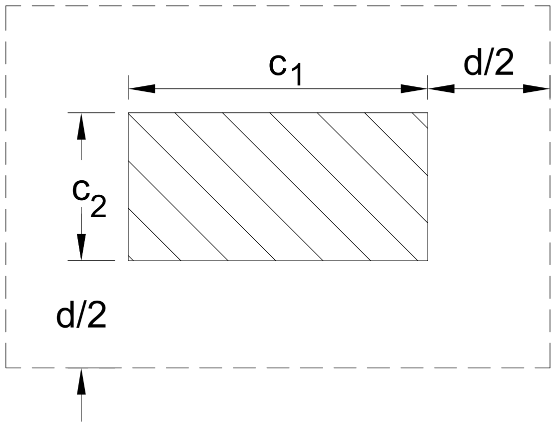
Design Summary

Diameter of Anchor Bolt, Dia= Dia = 0.500 in

Embedment Length of Anchor Bolt, h_{ef}= h_{ef_Prov} = 4.50 in

Head Size of Anchor Bolt, A_{brg}= A_{brg} = 0.156 in^2

Shear Strength of Slab at Column Support as per ACI 318-11 Chapter 11



System

Width of Column, c_1=			48.0 in
Length of Column, c_2=			8.0 in
Thickness of Slab, t=			10.0 in
Concrete Cover, co=			3.5 in
Effective Depth of Slab, d=	t - co	=	6.5 in

Load

Ultimate Shear Force, V_u= 20 kips

Material Properties

Concrete Strength, f'_c=	4000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Check Slab Thickness

Perimeter of Critical Section for Two-Way Shear, b_0=	$2*(c_1+d)+2*(c_2+d)$	=	138 in
Column Type=	SEL("ACI/Alfa S";Type;)	=	Interior
Alfa Constant, α_s=	TAB("ACI/AlfaS"; Alfa; Type=Type)	=	40.00
Ratio of Long to Short Column Dimensions, β=	$MAX(c_1;c_2)/MIN(c_1;c_2)$	=	6.00

Concrete Shear Strength (According to Eq. 11-31 of ACI318),

V_{c1}= $(2+4/\beta)*\lambda*\sqrt{f'_c}*b_0*d/1000$ = 151.3 kips

Concrete Shear Strength (According to Eq. 11-32 of ACI318),

V_{c2}= $(\alpha_s*d/b_0+2)*\lambda*\sqrt{f'c}*b_0*d/1000$ = 220.3 kips

Concrete Shear Strength (According to Eq. 11-33 of ACI318),

V_{c3}= $4 * \lambda * \sqrt{f'_c} * b_0 * d / 1000$ = 226.9 kips

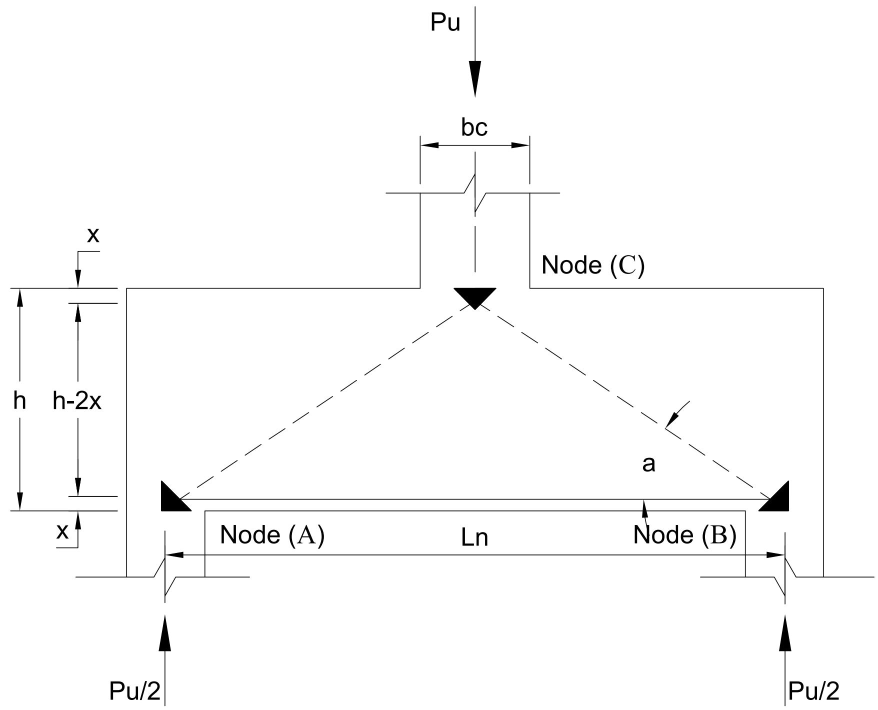
Nominal Concrete Shear Strength, $\Phi V_c = \Phi * MIN(V_{c1}; V_{c2}; V_{c3})$ = 113.5 kips

Validation = IF($\Phi V_c > V_u$; "O.K."; "Increase Depth") = O.K.

Calculation Summary

Thickness of Slab, t= t = 10 in

Design of Simple Span Deep Beam by the Strut-and-Tie Model as per ACI318 Appendix A



System

Width of Deep Beam, b=			7.0 in
Height of Deep Beam, h=			60.0 in
Concrete Cover, co=			1.25 in
Depth of Deep Beam, d=	h - co	=	58.75 in
End Distance of Truss Model, x=			5.0 in
Span of Deep Beam, L_n=			13.3 ft
Column Width, b_c=			20.0 in

Load

Dead Load for Column, P_D=			173.35 kips
Live Load for Column, P_L=			270.0 kips
Service Load for Column, P=	$1.0*P_D+1.0*P_L$	=	443.4 kips
Ultimate Load for Column, P_u=	$1.2*P_D+1.6*P_L$	=	640.0 kips

Material Properties

Concrete Strength, f'_c= 4000 psi

Yield Strength of Reinforcement, f_y= 60000 psi

Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.75

Modification Factor for Lightweight Concrete, λ= 1.00

Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= $1.4*\lambda$ = 1.40

Check Deep Beam Requirments

Check on Height of Deep Beam Requirements (According to Cl.11.7.1 of ACI318),

R = IF(12*Ln/h<4; "Deep Beam Design"; "Normal Beam Design") = Deep Beam Design

Estimation of Truss Model

Length of Diagonal Strut, L1= $\sqrt{\left(\frac{L_n*12}{2}\right)^2+(h-2*x)^2}$ = 94.17 in

The Force in Diagonal Strut, Fs= $\frac{P_u}{2}*\frac{L1}{h-2*x}$ = 602.69 kips

The Force in Horizontal Tie, Ft= $\frac{P_u}{2}*\frac{0.5*L_n*12}{h-2*x}$ = 510.72 kips

Angle Between Diagonal Strut and Horizontal Tie, α= $\operatorname{atan}\left(\frac{h-2*x}{0.5*L_n*12}\right)$ = 32.07 ^{o}

Check Validity (According to Cl.A.2.5 of ACI318)= IF(α>25; "Valid"; "Invalid") = Valid

Calculation of Effective Concrete Strength

(According to Cl.3.2.2(a) of ACI318) Factor of, β_s= 0.75

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce1}= $0.85*\beta_s*f'_c$ = 2550 psi

Calculation of Effective Concrete Strength for Nodal Zones

For Nodal Zone C Bounded by Three Struts (C-C-C Nodal Zone)

(According to Cl.A.5.2.1 of ACI318) Factor of, β_n= 1.00

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce2}= $0.85*\beta_n*f'_c$ = 3400 psi

For Nodal Zone A&B Bounded by Three Struts (C-C-T Nodal Zone)

(According to Cl.A.5.2.2 of ACI318) Factor of, β_n= 0.80

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce3}= $0.85*\beta_n*f'_c$ = 2720 psi

Minimum Effective Concrete Strength, f_{ce}= MIN(f_{ce1}; f_{ce2}; f_{ce3};) = 2550 psi

Check Strength at Node C

The Length of The Horizontal Face of Nodal Zone C,

Lhc= $\dfrac{P_u * 1000}{\Phi * b_c * f_{ce}}$ = 16.73 in

The Length of Other Faces of Nodal Zone C,

Lc= $Lhc * \dfrac{Fs}{P_u}$ = 15.75 in

Check Strength at Node A&B

The Length of The Horizontal Face of Nodal Zone A,

Lha= $\dfrac{Ft * 1000}{\Phi * b_c * f_{ce}}$ = 13.35 in

Width of Node at Support A,

La= $\dfrac{0.5 * P_u * 1000}{\Phi * b_c * f_{ce}}$ = 8.37 in

Calculation VL. and HZ. Reinforcement to Resist Splitting Diagonal Struts

1. Vertical Reinforcement

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.4

Provided Reinforcement, A_{sbv}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in^2

Number of Bars, n_v= 4

Vertical Reinforcement, A_{sv}= $A_{sbv} * n_v$ = 0.80 in^2

Provided Spacing between Bars, s= 11.00 in

Vertical Reinforcement (According to Eq.A4 of ACI318),

VL= $\dfrac{A_{sv}}{b_c * s} * \sin(90 - \alpha)$ = 0.00308

2. Horizontal Reinforcement

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, A_{sbh}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n_h= 2

Vertical Reinforcement, A_{sh}= $A_{sbh} * n_h$ = 0.62 in^2

Provided Spacing between Bars, s= 11.00 in

Horizontal Reinforcement (According to Eq.A4 of ACI318),

HZ= $\dfrac{A_{sh}}{b_c * s} * \sin(\alpha)$ = 0.00150

Check Validity= IF(VL+HZ>0.003; "Valid"; "Invalid") = Valid

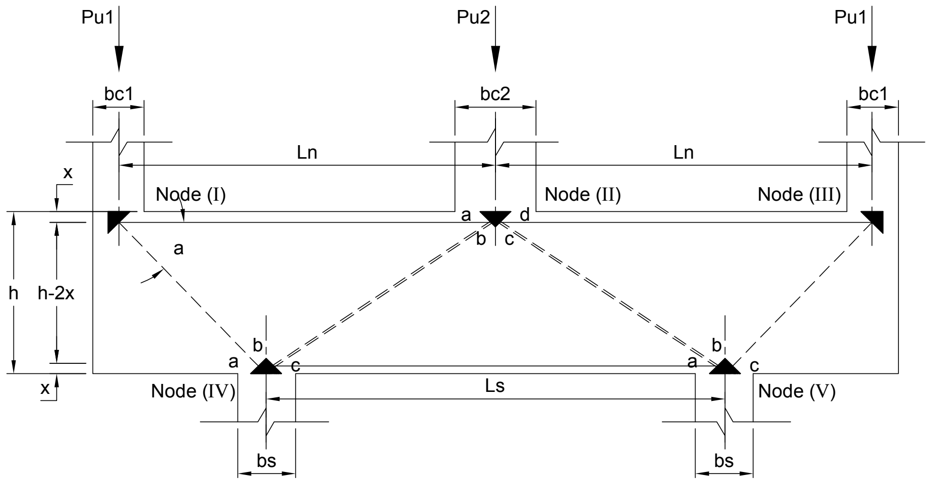
Calculation of Tension Reinforcement for Tie Connecting Node A&B

Required Reinforcement Area, A_{sreq}=	$\dfrac{Ft*1000}{\Phi*fy}$	=	11.35 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.8
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.79 in^2
Number of Bars, n=			16
Total Provided Area, A_{sprov}=	$n*A_{sb}$	=	12.64 in^2
Check Validity=	IF(Asprov>Asreq; "Valid"; "Invalid")	=	Valid

Design Summary

Provided Vertical Reinforcement, A_{sv}=	A_{sv}	=	0.80 in^2
Provided Horizontal Reinforcement, A_{sh}=	A_{sh}	=	0.62 in^2
Provided Tension Reinforcement, A_{sprov}=	A_{sprov}	=	12.64 in^2

Design of Continuous Deep Beam by the Strut-and-Tie Model as per ACI318 Appendix A



System

Width of Deep Beam, b=			24.0 in
Height of Deep Beam, h=			144.0 in
Concrete Cover, co=			1.25 in
Depth of Deep Beam, d=	$h - co$	=	142.75 in
Upper End Distance of Truss Model, x_1=			6.0 in
Lower End Distance of Truss Model, x_2=			9.0 in
Span of Deep Beam, L_n=			24.0 ft
Exterior Planted Column Width, b_{c1}=			24.0 in
Interior Planted Column Width, b_{c2}=			56.0 in
Distance between Supports of Deep Beam, L_s=			24.0 ft
Support Column Width, b_s=			48.0 in
Support Column Depth, d_s=			24.0 in

Load

Dead Load for Exterior Column, P_{D1}=			100.0 kips
Live Load for Exterior Column, P_{L1}=			237.5 kips
Ultimate Load for Exterior Column, P_{u1}=	$1.2*P_{D1} + 1.6*P_{L1}$	=	500.0 kips
Dead Load for Interior Column, P_{D2}=			750.0 kips
Live Load for Interior Column, P_{L2}=			1000.0 kips
Ultimate Load for Interior Column, P_{u2}=	$1.2*P_{D2} + 1.6*P_{L2}$	=	2500.0 kips
Support Column Ultimate Load, P_u=	$P_{u1} + P_{u2}/2$	=	1750.0 kips

Material Properties

Concrete Strength, f'_c= 4000 psi

Yield Strength of Reinforcement, f_y= 60000 psi

Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.75

Modification Factor for Lightweight Concrete, λ= 1.00

Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= $1.4 * \lambda$ = 1.40

Check Deep Beam Requirements

Check on Height of Deep Beam Requirements (According to Cl.11.7.1 of ACI318),

R = IF(12*Ln/h<4; "Deep Beam Design"; "Normal Beam Design") = Deep Beam Design

Calculation of Effective Concrete Strength

(According to Cl.A.3.2 of ACI318) Factor of, β_s= 1.00

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce1}= $0.85 * \beta_s * f'_c$ = 3400 psi

Calculation of Effective Concrete Strength for Nodal Zones

For Nodal Zone IV Bounded by Three Struts (C-C-C Nodal Zone)

(According to Cl.A.5.2.1 of ACI318) Factor of, β_n= 1.00

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce2}= $0.85 * \beta_n * f'_c$ = 3400 psi

For Nodal Zone A&B Bounded by Three Struts (C-C-T Nodal Zone)

(According to Cl.A.5.2.2 of ACI318) Factor of, β_n= 0.80

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

f_{ce3}= $0.85 * \beta_n * f'_c$ = 2720 psi

Minimum Effective Concrete Strength, f_{ce}= MIN(f_{ce1}; f_{ce2}; f_{ce3};) = 2720 psi

Calculation of Forces in Struts

For Node IV Will Carry Exterior Column Load Strut, Fa= P_{u1} = 500.00 kips

For Node IV Other Struts B and C, Fbc= $0.5*(P_u - Fa)$ = 625.00 kips

Check Width of Struts at Node IV

Width of Strut a, Wsa= $\dfrac{Fa*1000}{\Phi * f_{ce} * b}$ = 10.21 in

Width of Strut b&c, Wsbc= $\dfrac{Fbc*1000}{\Phi * f_{ce} * b}$ = 12.77 in

Total Width of Struts, Ws= $Wsa + Wsbc * 2$ = 35.75 in

Check Validity= IF(Ws<b_s; "Valid"; "Invalid") = Valid

Check Width of Struts at Node I

Width of Strut, WsI= $\dfrac{P_{u1}*1000}{\Phi*f_{ce}*b}$ = 10.21 in

Check Validity= IF(WsI<b_{c1}; "Valid"; "Invalid") = Valid

Check Width of Struts at Node II

Width of Strut, WsII= $\dfrac{P_{u2}*1000}{\Phi*f_{ce}*b}$ = 51.06 in

Check Validity= IF(WsII<b_{c2}; "Valid"; "Invalid") = Valid

Calculation of Force in Strut I-IVa and Tie I-IIa

Horizontal Projection of Strut I-IVa, Lhiiva= $\dfrac{(L_n*2-L_s)*12}{2}-Wsbc$ = 131.23 in

Vertical Projection of Strut I-IVa, Lviiva= $h-(x_1+x_2)$ = 129.00 in

Horizontal Force in Strut I-IVa and Tie I-IIa, Fiiva= $P_{u1}*\dfrac{Lhiiva}{Lviiva}$ = 508.64 kips

Length of Strut I-IVa, Liiva= $\sqrt{Lhiiva^2+Lviiva^2}$ = 184.02 in

Compression Force in Strut I-IVa at Node I, Fi= $\dfrac{P_{u1}*Liiva}{h-(x_1+x_2)}$ = 713.26 kips

Check Validity= IF(Fi<f_{ce}; "Valid"; "Invalid") = Valid

Calculation of Width of Strut IIa-IVb

Horizontal Projection of Strut IIa-IVb, Lhiiaivb= $\dfrac{(Ln*2-Ls)*12}{2}-\dfrac{WsII*3}{8}$ = 124.9 in

Vertical Projection of Strut IIa-IVb, Lviiaivb= $h-(x_1+x_2*2)$ = 120.0 in

Vertical Force in Strut IIa-IVb, Fiiaivb= $Fiiva*\dfrac{Lhiiaivb}{Lviiaivb}$ = 529.4 kips

Calculation of Width of Strut IIa-IVc

Horizontal Projection of Strut IIa-IVb, Lhiiaivb= $\dfrac{(L_n*2-L_s)*12}{2}-\dfrac{WsII*3}{8}$ = 124.9 in

Vertical Projection of Strut IIa-IVb, Lviiaivb= $h-(x_1+x_2*2)$ = 120.0 in

Vertical Force in Strut IIa-IVb, Fiiaivb= $Fiiva*\dfrac{Lhiiaivb}{Lviiaivb}$ = 529.4 kips

Calculation of Width of Strut IIb-IVc

Horizontal Projection of Strut IIa-IVb, Lhiiaivb= $\frac{(L_n * 2 - L_s) * 12}{2} - \frac{WsII * 7}{50}$ = 136.9 in

Vertical Projection of Strut IIa-IVb, Lviiaivb= $h - (x_1 + x_2 * 2)$ = 120.0 in

Vertical Force in Strut IIa-IVb, Fiiaivb= $Fi * \frac{Lhiiaivb}{Lviiaivb}$ = 813.7 kips

Calculation of Width of Tie IVc-Va

Force in Tie IVc-Va, Fivcva= $\frac{Fiiaivb * 1000}{\Phi * f_{ce} * b}$ = 16.62 in

Calculation VL. and HZ. Reinforcement to Resist Splitting of Diagonal Struts

1. Vertical Reinforcement

Angle of Strut, α= 46.60 ^{o}

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, A_{sbv}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n_v= 2

Vertical Reinforcement, A_{sv}= $A_{sbv} * n_v$ = 0.62 in^2

Provided Spacing between Bars, s= 10.00 in

Vertical Reinforcement (According to Eq.A4 of ACI318),

VL= $\frac{A_{sv}}{b * s} * \sin(90 - \alpha)$ = 0.00177

2. Horizontal Reinforcement

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.5

Provided Reinforcement, A_{sbh}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, n_h= 2

Vertical Reinforcement, A_{sh}= $A_{sbh} * n_h$ = 0.62 in^2

Provided Spacing between Bars, s= 10.00 in

Horizontal Reinforcement (According to Eq.A4 of ACI318),

HZ= $\frac{A_{sh}}{b * s} * \sin(\alpha)$ = 0.00188

Check Validity= IF(VL+HZ>0.003; "Valid"; "Invalid") = Valid

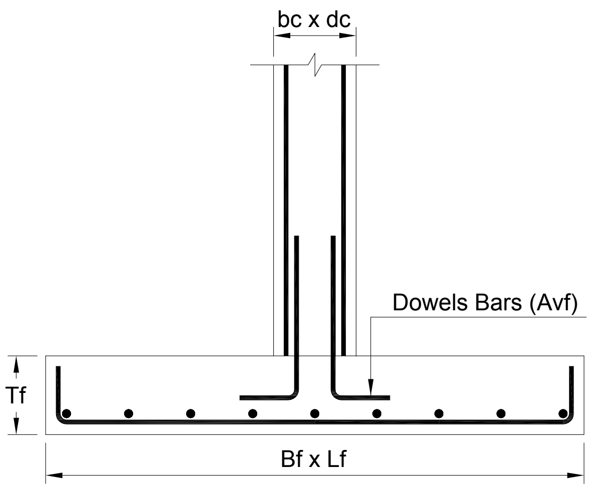
Calculation of Tension Reinforcement for Tie Connecting Joint I-IIa

Required Reinforcement Area, A_{sreq}=	$\dfrac{Fiiva*1000}{\Phi*fy}$	=	11.30 in^2
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.9
Number of Bars, n=			12.00
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	1.00 in^2
Total Provided Area, A_{sprov}=	$n*Asb$	=	12.00 in^2
Check Validity=	IF(Asprov>Asreq; "Valid"; "Invalid")	=	Valid

Design Summary

Provided Vertical Reinforcement, A_{sv}=	A_{sv}	=	0.62 in^2
Provided Horizontal Reinforcement, A_{sh}=	A_{sh}	=	0.62 in^2
Provided Tension Reinforcement, A_{sprov}=	A_{sprov}	=	12.00 in^2

Design for Transfer of Horizontal Force at Base of Column where The Footing Surface is not Intentionally Roughened as per ACI 318-11 Chapter 12



System

Column Width, b_c=	12.0 in
Column Depth, d_c=	12.0 in
Footing Width, B_f=	9.0 ft
Footing Length, L_f=	9.0 ft
Footing Thickness, T_f=	22.0 in

Load

Ultimate Horizontal Force at the Base of Column, V_u=	84.0 kips

Material Properties

Concrete Strength, f'_c=		4000 psi
Yield Strength of Reinforcement, f_y=		60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=		0.75
Modification Factor for Lightweight Concrete, λ=		1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= 0.6* λ	=	0.60

Check on Maximum Shear Transfer Permitted

Nominal Shear Force (According to Cl.11.6.5 of ACI318),

ΦV_{n1}=	$\Phi*(0.2*f'_c/1000*b_c*d_c)$	=	86.4 kips
ΦV_{n2}=	$\Phi*(800*b_c*d_c)/1000$	=	86.4 kips
Minimum Nominal Shear, ΦV_n=	MIN(ΦV_{n1}; ΦV_{n2};)	=	86.4 kips
Check Validity=	IF(Vu<ΦV_n; "Valid"; "Increase Dimension")	=	Valid

Required Area of Reinforcement (According to Eq.11-25 of ACI318),

A_{vf}= $\frac{Vu*1000}{\Phi*fy*\mu}$ = 3.11 in^2

Provided Shear Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.8

Diameter of Bars, Dia= TAB("ACI/Bar"; Dia; Bar=Bar) = 1.0000 in

Number of Bars, n= 4

Provided Area of Reinforcement, A_s= $n*\frac{\pi*Dia^2}{4}$ = 3.14 in^2

Check Validity= IF(A_s>A_{vf}; "Valid"; "Increase RFT") = Valid

Check on Development Length of Tensile Reinforcement with Column

Clear Cover to Center of Bars, c= 3.25 in

Center to Center Bar Spacing, S= 4.50 in

Factor of, cb= MIN(c+Dia/2; S/2) = 2.25 in

(According to Cl.12.2.3 of ACI318) Factor of, Ktr= 0.00

(According to Cl.12.2.4 of ACI318) Factor of, Ψ_t= 1.00

(According to Cl.12.2.4 of ACI318) Factor of, Ψ_e= 1.00

(According to Cl.12.2.4 of ACI318) Factor of, Ψ_s= 1.00

Development Length within Column

Development Length (According to Eq.12-1 of ACI318),

L_{d1}= $\frac{3}{40}*\frac{fy}{\lambda*\sqrt{f'c}}*\frac{\Psi_t*\Psi_e*\Psi_s}{(cb+Ktr)/Dia}*Dia$ = 31.6 in

Development Length within Footing

Development Length (According to Cl.12.5.2 of ACI318),

L_{d2}= $\frac{0.02*\Psi_e*fy}{\lambda*\sqrt{f'c}}*Dia$ = 19.0 in

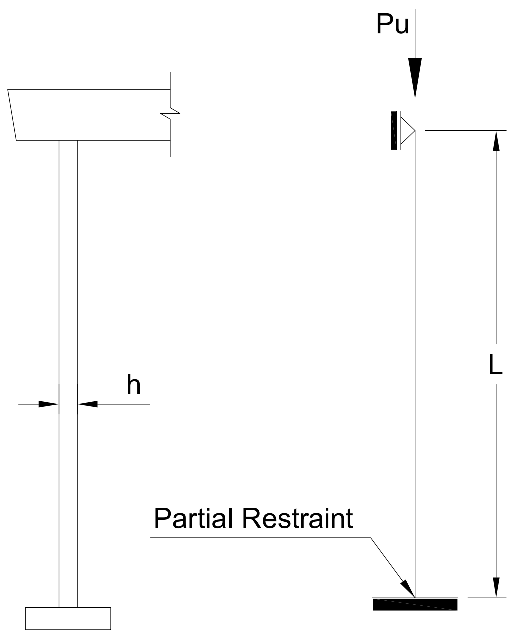
Design Summary

Provided Area of Reinforcement, A_s= A_s = 3.14 in^2

Development Length within Column, L_{d1}= L_{d1} = 31.6 in

Development Length within Footing, L_{d2}= L_{d2} = 19.0 in

Design of Bearing Wall by Empirical Method as per ACI 318-11 Chapters 10 & 14



System

Height of Wall, L= 15 ft

Spacing of Wall Panels, s_p= 8 ft

Width of Stem for Bearing Wall, b_w= 7 in

Load

Service Dead Load, P_D= 28 kips

Service Live Load, P_L= 14 kips

Ultimate Load, P_u= $1.2*P_D+1.6*P_L$ = 56 kips

Material Properties

Concrete Strength, f'_c= 4000 psi

Bearing Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.65

Modification Factor for Lightweight Concrete, λ= 1.00

Determine Wall Thickness

Assume Wall Thickness, h= 7.5 in

Minimum Wall Thickness, h_{min}= MAX(L*12/25 ; 4) = 7.2 in

Check Validation = IF(h> h_{min}; "O.K."; "Increase Thickness") = O.K.

Check Concrete Bearing Strength

Loaded Area, A_1=	$h * b_w$	=	52.50 in^2

Nominal Concrete Bearing Capacity (According to Cl.10.14.1 of ACI318),

ΦV_b=	$\Phi * 0.85 * f'_c * A_1 / 1000$	=	116 kips
Check Validation =	IF($\Phi V_b > P_u$; "Valid."; "Invalid")	=	Valid.

Calculate Design Strength of Wall

Effective Width of Wall, w=	MIN(b_w+4*h ; s_p*12)	=	37 in

Wall Resistant Type,

Type-1 : Restrained Rotation - One or Both Ends (T/B/Both)

Type-2 : Unrestrained Rotation at Both Ends

Type-3 : For Walls not Braced Against Lateral Translation

Type=	SEL("ACI/K" ;Type;)	=	Type-1
Effective Length Factor, K=	TAB("ACI/K" ;K ;Type=Type)	=	0.80

Nominal Strength of Wall (According to Eq.14-1 of ACI 318),

$$\Phi P_n = 0.55 * \Phi * f'_c * w * h * \frac{1 - \left(\frac{K * L * 12}{32 * h}\right)^2}{1000} = 254 \text{ kips}$$

Check Validity=	IF($\Phi P_n > P_u$;"Valid" ;"Invalid")	=	Valid

Determine Single Layer of Reinforcement

Vertical Area of Reinforcement for Wall (According to Cl.14.3.2 of ACI318),

A_{sv}=	0.0012 * 12 * h	=	0.108 in^2 /ft
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.4
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.20 in^2
Bar Spacing, s=			18 in
A_{sv_Prov}=	A_{sb}*12/s	=	0.13 in^2
Check Validity=	IF(A_{sv_Prov}>A_{sv}; "Valid"; "Invalid")	=	Valid

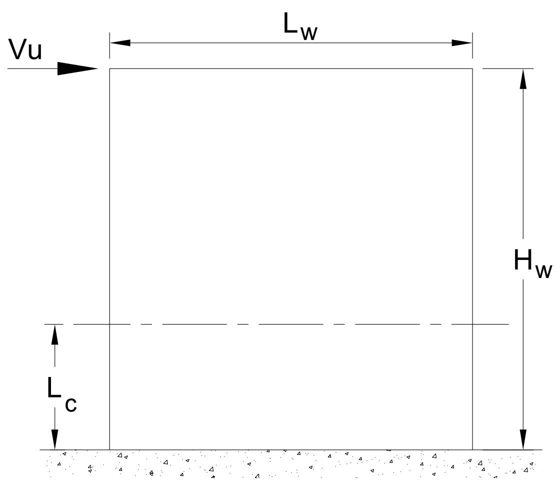
Horizontal Area of Reinforcement for Wall (According to Cl.14.3.3 of ACI318),

A_{sh}=	0.0020 * 12 * h	=	0.180 in^2 /ft
Provided Reinforcement, Bar=	SEL("ACI/Bar"; Bar;)	=	No.4
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.20 in^2
Bar Spacing, s=			12 in
A_{sh_Prov}=	A_{sb}*12/s	=	0.20 in^2
Check Validity=	IF(A_{sh_Prov}>A_{sh}; "Valid"; "Invalid")	=	Valid

Design Summary

Wall Thickness, h=	h	=	8 in
Length of Footing, L=	L	=	15 ft
Vertical Area of Reinforcement for Wall, A_{sv_Prov}=	A_{sv_Prov}	=	0.13 in^2
Horizontal Area of Reinforcement for Wall, A_{sh_Prov}=	A_{sh_Prov}	=	0.20 in^2

Shear Design of Wall as per ACI 318-11 Chapter 11



System

Height of Wall, h_w=			12.0 ft
Width of Wall, L_w=			8.0 ft
Over All Wall Depth, d=	$0.8 * L_w$	=	6.4 ft
Thickness of Wall, h=			8.0 in
Concrete Cover, co=			2.0 in
Effective Depth of Wall Section, d_c=	$h - co = 8.0 - 2.0$	=	6.0 in

Load

Ultimate Bending Moment, M_u=	19200 kip*ft
Ultimate Shear Force, V_u=	200 kips
Ultimate Normal Force, N_u=	0 kips

Material Properties

Concrete Strength, f'_c=	3000 psi
Yield Strength of Reinforcement, f_y=	60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Check Shear Reinforcement Requirement

Maximum Shear Strength of Wall (According to Cl.11.9.3 of ACI318),

ΦV_n= $\Phi * 10 * \sqrt{f'_c} * h * d * 12 / 1000$ = 252 kips

Check Validity= IF($V_u \le \Phi V_n$;"Valid" ;"Invalid") = Valid

Critical Section of Shear Force, L_c= MIN(L_w/2;h_w/2) = 4.00 ft

Concrete Shear Strength (According to Eq.11-27 of ACI318),

$$V_{c1} = 3.3 * \lambda * \sqrt{f'_c} * \frac{h * d * 12}{1000} + \frac{N_u * d}{4 * Lw} = 111 \text{ kips}$$

Concrete Shear Strength (According to Eq.11-28 of ACI318),

$$V_{c2} = \left(0.6 * \lambda * \sqrt{f'_c} + \frac{L_w * 12 * \left(1.25 * \lambda * \sqrt{f'_c} + \frac{0.2 * N_u}{L_w * 12 * h} \right)}{M_u / V_u - L_w * 12 / 2} \right) * \frac{h * d * 12}{1000} = 104 \text{ kips}$$

Concrete Shear Strength, V_c= MIN(V_{c1}; V_{c2}) = 104 kips

Shear Reinforcement: IF($V_u < \Phi * V_c / 2$;"Not Required" ;"Required") = Required

Determine Horizontal Shear Reinforcement

Identification of, Bar= SEL("ACI/Bar" ;Bar;) = No.4

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in^2

Number of Bars, n= 2

Area of Shear Reinforcement, A_h= n * A_{sb} = 0.40 in^2

Spacing between Bars (According to Eq.11-29 of ACI318),

$$s_{hi} = \frac{\Phi * fy / 1000 * d * 12 * A_h}{V_u - \Phi * V_c} = 11.3 \text{ in}$$

Provided Reinforcement Spacing, s_h= 10 in

Check Validity= IF($s_h \le s_{hi}$; "Valid"; "Invalid") = Valid

Ratio of Horizontal Shear Reinforcement (According to Cl.11.9.9.2 of ACI318),

$$\rho_{hi} = \frac{A_h}{h * s_h} = 0.005$$

ρ_h= MAX(ρ_{hi} ; 0.0025) = 0.005

Calculation of Vertical Shear Reinforcement

Ratio of Vertical Shear Reinforcement (According to Eq.11-30 of ACI318),

ρ_{vi}=	$0.0025+0.5*\left(2.5-\frac{h_w}{L_w}\right)*(\rho_h-0.0025)$	=	0.0037
ρ_v=	MAX(ρ_{vi} ; 0.0025)	=	0.0037
Identification of, Bar=	SEL("ACI/Bar" ;Bar;)	=	No.4
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.20 in^2
Number of Bars, n=			2
Area of Shear Reinforcement, A_v=	$n * A_{sb}$	=	0.40 in^2

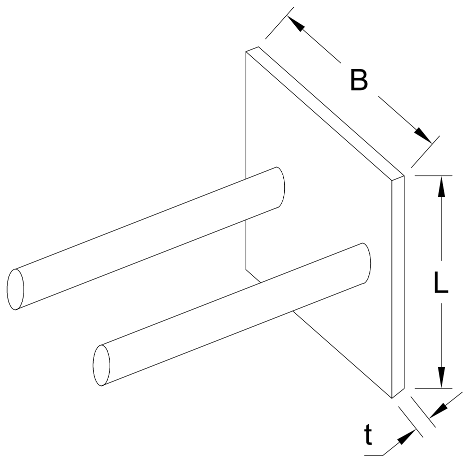
Spacing between Bars (According to Eq.11-29 of ACI318),

s_{vi}=	$\frac{A_v}{\rho_v*h}=\frac{0.40}{0.0037*8.0}$	=	13.5 in
Provided Reinforcement Spacing, s_v=			13 in
Check Validity=	IF($s_v \le s_{vi}$; "Valid"; "Invalid")	=	Valid

Design Summary

Horizontal Shear Reinforcement, A_h=	A_h	=	0.40 in^2
Spacing Between Horizontal Shear Reinforcement, s_h=	s_h	=	10 in
Vertical Shear Reinforcement, A_v=	A_v	=	0.40 in^2
Spacing Between Horizontal Shear Reinforcement, s_v=	s_v	=	13 in

Design of Shear Friction as per ACI 318-11 Chapter 11



System

Width of Steel Plate, B=			2.00 in
Length of Steel Plate, L=			4.00 in
Thickness of Steel Plate, t=			0.25 in
Identification of, Bar=	SEL("ACI/Bar" ;Bar;)	=	No.3
Diameter of Bars, d_b=	TAB("ACI/Bar" ;Dia ;Bar=Bar)	=	0.38 in
Number of Bars, n=			2

Load

Ultimate Shear Force, V_u=	3570 lb

Material Properties

Concrete Strength, f'_c=			4000 psi
Yield Strength of Reinforcement, f_y=			60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=			0.75
Modification Factor for Lightweight Concrete, λ=			0.75
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ=	$0.7 * \lambda$	=	0.525

Calculation of Required Reinforcement Area

Area of Shear Friction Reinforcement (According to Eq.11-25 of ACI318),

$A_{vf}=$ $\dfrac{V_u}{\Phi * fy * \mu}$ = $0.151\ in^2$

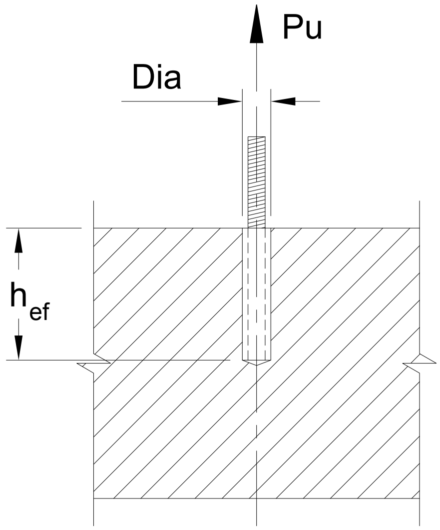
Provided Area, $A_{act}=$ $n * \dfrac{\pi * d_b^2}{4}$ = $0.23\ in^2$

Check Validity= IF(A_{act}>$A_{vf;}$ "Valid" ; "Invalid") = Valid

Design Summary

Provided Area of Reinforcement, $A_{act}=$ A_{act} = $0.23\ in^2$

Design a Single Adhesive Anchor in Tension Away from Edges as per ACI 318-11 Appendix D



System

Diameter of Adhesive Anchor Bolt, Dia=	SEL("ACI/Anchor"; Dia;)	=	0.500 in
Area of Adhesive Anchor Bolt, A_{se_N}=	TAB("ACI/Anchor"; Ase; Dia=Dia)	=	0.142 in^2
Effective Embedment Length, h_{ef}=			4.0 in

Material Properties

Concrete Strength, f'_c=	4000 psi
Characteristic Bond Stress in Cracked Concrete, τ_{cr}=	300 psi
Characteristic Bond Stress in Uncracked Concrete, τ_{uncr}=	1000 psi
Tensile Strength of Anchor Bolt Grade, f_{uta}=	58000 psi
Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), Φ_1=	0.75
Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), Φ_2=	0.45
Modification Factor for Lightweight Concrete, λ=	1.00

Determine The Steel Strength of Adhesive Anchor

The Steel Strength of Anchor Bolt (According to Cl.D.4.1.1 of ACI318),

ΦN_{sa}=	$\Phi_1 * A_{se\_N} * f_{uta}$	=	6177 lb

Determine The Bond Strength of Adhesive Anchor

(According to Eq.D-21 of ACI318), $c_{Na} = 10 * Dia * \sqrt{\tau_{uncr} / 1100}$ = 4.77 in

(According to Eq.D-20 of ACI318), $A_{Nao} = (2 * c_{Na})^2$ = 91.0 in^2

(According to Cl.D.5.5.1 of ACI318), $A_{Na} = A_{Nao}$ = 91.0 in^2

The Basic Bond Strength (According to Eq.D-22 of ACI318),

N_{ba}= $\lambda * \tau_{cr} * \pi * Dia * h_{ef}$ = 1885 lb

Factor (According to Cl.D.5.5.3 of ACI318), $\psi_{ed,Na}$= 1.00

Factor (According to Cl.D.5.5.5 of ACI318), $\psi_{c,Na}$= 1.00

The Basic Bond Strength for A Single Anchor (According to Eq.D-3 of ACI318),

ΦN_a= $\Phi_2 * (A_{Na}/A_{Nao}) * \psi_{ed,Na} * \psi_{c,Na} * N_{ba}$ = 848 lb

Determine The Concrete Breakout Strength

(According to Eq.D-6 of ACI318), κ_c= 17.0

Basic Strength of Concrete Breakout (According to Eq.D-6 of ACI318),

N_a= $\kappa_c * \lambda * \sqrt{f'_c} * h_{ef}^{1.5}$ = 8601 lb

Factor (According to Cl.D.5.2.6 of ACI318), $\psi_{cp,Na}$= 1.00

The Strength of Concrete Breakout (According to Eq.D-3 of ACI318),

ΦN_{cb}= $\Phi_2 * (A_{Na}/A_{Nao}) * \psi_{ed,Na} * \psi_{c,Na} * \psi_{cp,Na} * N_a$ = 3870 lb

Determine The Tension Force Carried by Adhesive Anchor Bolt

The Tension Force Carried by Adhesive Anchor, T_u= MIN(ΦN_{sa};ΦN_a;ΦN_{cb}) = 848 lb

Design Summary

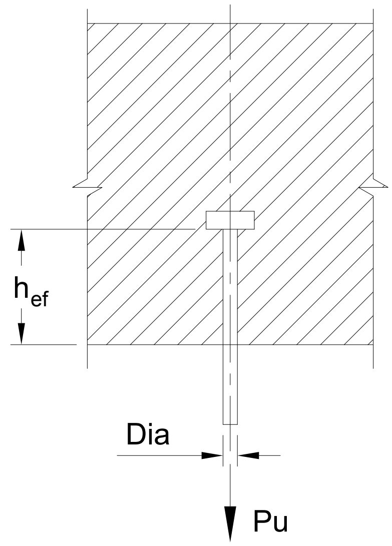
The Steel Strength of Adhesive Anchor Bolt, ΦN_{sa}= ΦN_{sa} = 6177 lb

The Bond Strength of Adhesive Anchor Bolt, ΦN_a= ΦN_a = 848 lb

The Concrete Breakout Strength of Adhesive Anchor Bolt, ΦN_{cb}= ΦN_{cb} = 3870 lb

The Tension Force Carried by Adhesive Anchor, T_u= T_u = 848 lb

Design a Single Headed Anchor Bolt in Tension Away from Edges as per ACI 318-11 Appendix D



Load

Ultimate Load, P_u= 7000 lb

Material Properties

Concrete Strength, f'_c= 4000 psi

Tensile Strength of Anchor Bolt Grade, f_{uta}= 58000 psi

Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), Φ_1= 0.75

Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), Φ_2= 0.70

Modification Factor for Lightweight Concrete, λ= 1.00

Determine Anchor Diameter

Required Effective Area of Anchor Bolt (According to Eq.D.3 of ACI318),

$$A_{se\_Req} = \frac{P_u}{\Phi_1 * 1.0 * f_{uta}} = 0.161\ in^2$$

Provided Anchor Bolt, Dia= SEL("ACI/Anchor"; Dia;) = 0.625 in

Provided Area of Anchor Bolt, A_{se_Prov}= TAB("ACI/Anchor"; Ase; Dia=Dia) = 0.226 in^2

Check Validity= IF($A_{se\_Prov} \geq A_{se\_Req}$; "Valid"; "Increase Dia") = Valid

Determine Emended Length

Factor (According to Cl.D.5.2.6 of ACI318), $\psi_{c,N}$= 1.00

Effective Embedment Length (According to Cl.D.5.2.1 of ACI318),

$$h_{ef\_Req} = \left(\frac{P_u}{\Phi_2 * \psi_{c,N} * 24 * \lambda * \sqrt{f'_c}}\right)^{2/3} = 3.51 \text{ in}$$

Provided Embedment Length, h_{ef_Prov}= 4.00 in

Check Validity= IF($h_{ef\_Prov} \geq h_{ef\_Req}$; "Valid"; "Increase h_{ef}") = Valid

Determine Head Size

Factor (According to Cl.D.5.3.6 of ACI318), $\psi_{c,P}$= 1.00

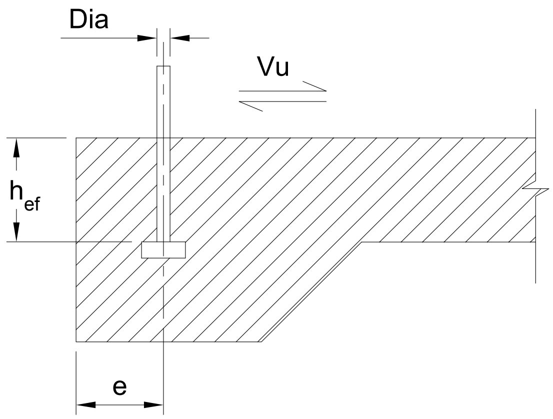
Required Head Size for Anchor Bolt (According to Eq.D-15 of ACI318),

$$A_{brg} = \frac{P_u}{\Phi_2 * \psi_{c,P} * 8 * f'c} = 0.313 \text{ in}^2$$

Design Summary

Diameter of Anchor Bolt, Dia=	Dia	=	0.625 in
Embedment Length of Anchor Bolt, h_{ef}=	h_{ef_Prov}	=	4.00 in
Head Size of Anchor Bolt, A_{brg}=	A_{brg}	=	0.313 in^2

Design a Single Headed Anchor Bolt in Shear Near an Edge as per ACI 318-11 Appendix D



System

Edge Distance, e= 1.75 in

Load

Ultimate Load, V_u= 700 lb

Material Properties

Concrete Strength, f'_c= 4000 psi

Tensile Strength of Anchor Bolt Grade, f_{uta}= 58000 psi

Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), Φ_1= 0.65

Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), Φ_2= 0.70

Modification Factor for Lightweight Concrete, λ= 1.00

Determine Anchor Diameter

Required Effective Area of Anchor Bolt (According to Eq.D.29 of ACI318),

$$A_{se\_Req} = \frac{V_u}{\Phi_1 * 1.0 * 0.6 * f_{uta}} = 0.031 \text{ in}^2$$

Provided Anchor Bolt, Dia= SEL("ACI/Anchor"; Dia;) = 0.500 in

Provided Area of Anchor Bolt, A_{se_Prov}= TAB("ACI/Anchor"; Ase; Dia=Dia) = 0.142 in^2

Check Validity= IF($A_{se\_Prov} \geq A_{se\_Req}$; "Valid"; "Increase Dia") = Valid

Calculation of Embedment Strength

Assume that, h_{ef_Prov}=			7.00 in
Ratio A_{vc}/A_{vco}, A'=			1.00
Factor (According to Cl.D.6 of ACI318), $\psi_{ed,V}$=			1.00
Factor (According to Cl.D.6.2.7 of ACI318), $\psi_{c,V}$=			1.00

Length of Load Bearing of Anchor Bolt (According to Cl.D.6.2.2 of ACI318),

l_e= MIN(h_{ef_Prov}; 8*Dia;) = 4.00 in

Basic Strength of Concrete Breakout (According to Eq.D-33 of ACI318),

$$V_{b1} = 7 * \left(\frac{l_e}{Dia}\right)^{0.2} * \sqrt{Dia} * \lambda * \sqrt{f'_c} * e^{1.5} = 1098 \text{ lb}$$

Basic Strength of Concrete Breakout (According to Eq.D-34 of ACI318),

$$V_{b2} = 9 * \lambda * \sqrt{f'_c} * e^{1.5} = 1318 \text{ lb}$$

Basic Strength of Concrete Breakout, V_b= MIN(V_{b1} ; V_{b2}) = 1098 lb

Nominal Strength of Concrete Breakout (According to Eq.D-30 of ACI318),

ΦV_{cb}= $A' * \Phi_2 * \psi_{ed,V} * \psi_{c,V} * V_b$ = 769 lb

Check Validation= IF($V_u \leq \Phi V_{cb}$; "Valid"; "Invalid") = Valid

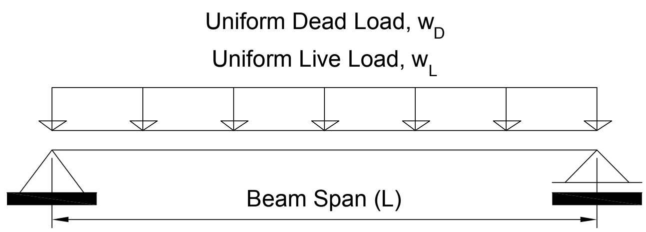
Design Summary

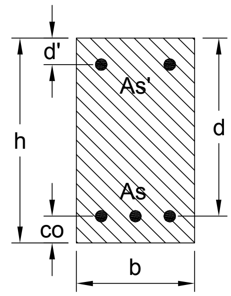
Diameter of Anchor Bolt, Dia= Dia = 0.500 in

Embedment Length of Anchor Bolt, h_{ef}= h_{ef_Prov} = 7.00 in

Calculation of Deflection for Simple Support Concrete Beam Under Uniform Loads

As per ACI 318-11 Chapter 9





System

Width of Concrete Section, b=			12.0 in
Depth of Concrete Section, h=			22.0 in
Concrete Cover, co=			2.5 in
Effective Depth of Concrete Section, d=	h - co = 22.0 - 2.5	=	19.5 in
Depth of Compression Reinforcement, d'=			2.5 in
Area of Tension Reinforcement, A_s=			1.80 in^2
Area of Compression Reinforcement, A_s'=			0.6 in^2
ρ=	As / (b * d)	=	0.0077
ρ'=	As' / (b * d)	=	0.0026
Distance from Centroidal Axis of Gross Section, y_t=	h / 2	=	11.0 in
Beam Span, L=			25.0 ft

Load

Uniform Dead Load, w_D=			0.395 kip/ft
Uniform Live Load, w_L=			0.300 kip/ft
Percentage of Sustained Live Load, Sus=			50 %
Moment due to Dead Load, M_D=	w_D * L² / 8	=	30.9 kip*ft
Moment due to Live Load, M_L=	w_L * L² / 8	=	23.4 kip*ft
Sustained Moment, M_{sus}=	M_D + (Sus/100)*M_L	=	42.6 kip*ft

Material Properties

Concrete Strength, f'_c=	3000 psi
Yield Strength of Reinforcement, f_y=	40000 psi
Modulus of Elasticity of Reinforcement, E_s=	29000000 psi
Modification Factor for Lightweight Concrete, λ=	1.00
Concrete Density, w_c=	150 psi

Properties of Cracked Section

Modulus of Rupture (According to Eq. 9-10 of ACI318), $f_r = 7.5 * \lambda * \sqrt{f'_c}$ = 411 psi

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI318),

$E_c = w_c^{1.5} * 33 * \sqrt{f'_c}$ = 3320561 psi

$n_s = E_s / E_c$ = 8.7

$I_g = b * h^3 / 12$ = 10648 in^4

$B = b / (n_s * A_s)$ = 0.77 in

$r = \dfrac{(n_s - 1) * As'}{n_s * A_s}$ = 0.295

$kd = \dfrac{\sqrt{2 * d * B * \left(1 + r * \frac{d'}{d}\right) + (1 + r)^2} - (1 + r)}{B}$ = 5.76 in

$I_{cr} = \dfrac{b * kd^3}{3} + n_s * As * (d - kd)^2 + (n_s - 1) * As' * (kd - d')^2$ = 3770 in^4

Cracking Moment (According to Eq. 9-9 of ACI318),

$M_{cr} = f_r * I_g / (y_t * 12000)$ = 33.2 kip*ft

Properties of Effective Section

Effective Moment of Inertia (According to Eq 9-8 of ACI318):

$I_{e\_Dead1} = \left(\dfrac{M_{cr}}{M_D}\right)^3 * I_g + \left(1 - \left(\dfrac{M_{cr}}{M_D}\right)^3\right) * I_{cr}$ = 12301 in^4

$I_{e\_Dead} = MIN(I_g ; I_{e\_Dead1})$ = 10648 in^4

$I_{e\_Sus1} = \left(\dfrac{M_{cr}}{M_{sus}}\right)^3 * I_g + \left(1 - \left(\dfrac{M_{cr}}{M_{sus}}\right)^3\right) * I_{cr}$ = 7026 in^4

$I_{e\_Sus} = MIN(I_g ; I_{e\_Sus1})$ = 7026 in^4

$I_{e\_All1} = \left(\dfrac{M_{cr}}{M_D + M_L}\right)^3 * I_g + \left(1 - \left(\dfrac{M_{cr}}{M_D + M_L}\right)^3\right) * I_{cr}$ = 5342 in^4

$I_{e\_All} = MIN(I_g ; I_{e\_All1})$ = 5342 in^4

Short Term Deflection

Δ_{i_Dead}= $\dfrac{5*M_D*L^2*12^3}{48*E_c*I_{e\_Dead}/1000}$ = 0.098 in

Δ_{i_Sus}= $\dfrac{5*M_{sus}*L^2*12^3}{48*E_c*I_{e\_Sus}/1000}$ = 0.205 in

Δ_{i_All}= $\dfrac{5*(M_D+M_L)*L^2*12^3}{48*E_c*I_{e\_All}/1000}$ = 0.344 in

Δ_{i_Live}= Δ_{i_All} - Δ_{i_Dead} = 0.246 in

Long Term Deflection

Duration of Sustained loads, Dur: SEL("ACI/Sustained";Dur;) = 5 Years or more

Time-Dependent Factor for Sustained Loads (According to Cl. 9.5.2.5 of ACI318):

ξ= TAB("ACI/Sustained";x;Dur=Dur;) = 2.00

Multiplier Factor for Long-Term Deflection (According to Eq. 9-11 of ACI318),

λ_Δ= ξ/ (1 + 50*ρ') = 1.77

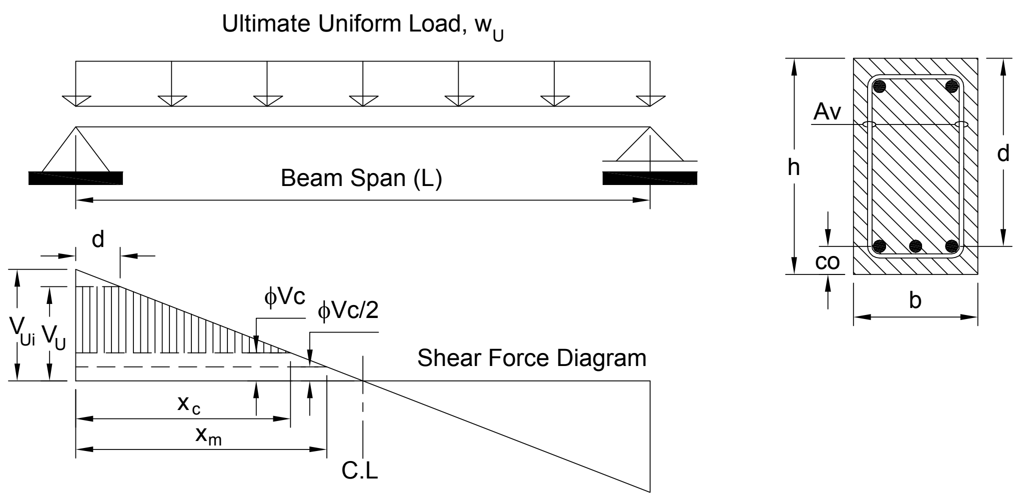
Creep and Shrinkage Deflection, Δ_{cp_sh} = λ_Δ * Δ_{i_Sus} = 0.36 in

Δ_{total}= Δ_{cp_sh} + Δ_{i_Live} = 0.61 in

Calculation Summary

Long Term Deflection, Δ_{total}= Δ_{cp_sh} + Δ_{i_Live} = 0.61 in

Design of Shear Reinforcement for Section Subject to Shear & Flexure as per ACI318-11 Chapter 11



System

Width of Concrete Section, b=			13.0 in
Depth of Concrete Section, h=			22.5 in
Concrete Cover, co=			2.5 in
Effective Depth of Concrete Section, d=	$h - co = 22.5 - 2.5$	=	20.0 in
Beam Span, L=			30.0 ft

Load

Ultimate Uniform Load, w_u=			4.5 kip/ft
Ultimate Shear Force at Support, V_{ui}=	w_u*L/2	=	67.5 kips
Ultimate Shear Force at Distance [d] from Support, V_u=	$V_{ui} - w_u * (d/12)$	=	60.0 kips

Material Properties

Concrete Strength, f'_c=	3000 psi
Yield Strength of Reinforcement, f_y=	40000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00
Concrete Density, w_c=	150 psi

Determine Concrete Shear Strength

Nominal Shear Strength provided by Concrete (According to Eq. 11-3 of ACI318),

V_c= $$2 * \lambda * \sqrt{f'_c} * \frac{b * d}{1000}$$ = 28.5 kips

Shear Reinforcement is : IF(V_u>$\Phi * V_c$;"Required";"Not Required") = Required

Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),

$V_s = \dfrac{V_u - \Phi * V_c}{\Phi}$ = 51.5 kips

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI318),

$V_{s\_max} = 8 * \sqrt{f'_c} * \dfrac{b * d}{1000}$ = 113.9 kips

IF(V_s>V_{s_max}; "Increase Beam Dimension"; "OK") = OK

Spacing of Provided Stirrups, s= 6.0 in

Required Area of Reinforcement, $A_v = \dfrac{V_s * s * 1000}{f_y * d}$ = 0.39 in^2

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI318),

$A_{v\_min1} = \dfrac{0.75 * \sqrt{f'_c} * b * s}{f_y}$ = 0.08 in^2

$A_{v\_min2} = \dfrac{50 * b * s}{f_y}$ = 0.10 in^2

A_{v_min}= MAX(A_{v_min1} ; A_{v_min2}) = 0.10 in^2

Required Area of Reinforcement, A_{vc_Req}= MAX(A_v; A_{v_min}) = 0.39 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.4

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in^2

Number of Stirrups, n= 1

Provided Area of Reinforcement, A_{vc_Prov}= A_{sb} * n * 2 = 0.40 in^2

Check Validity= IF(A_{vc_Prov}≥A_{vc_Req}; "Valid"; "Invalid") = Valid

Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),

$V_{s\_limit} = 4 * \lambda * \sqrt{f'_c} * \dfrac{b * d}{1000}$ = 57.0 kips

Factor for Maximum Spacing of Stirrups, Fac= IF(V_s≤V_{s_limit};1;0.5) = 1.0

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),

s_{max}= MIN(d/2;24) * Fac = 10.00 in

Check Validity= IF(s≤s_{max};"Valid"; "Invalid") = Valid

Determine Distribution Distance of Shear Reinforcement

Distance from Support beyond which Minimum Shear Reinforcement is Required,

$$x_c = \frac{V_{ui} - \Phi * Vc}{w_u} = 10.3 \text{ ft}$$

Distance from Support beyond which Concrete can carry Shear Force,

$$x_m = \frac{V_{ui} - \Phi * Vc/2}{w_u} = 12.6 \text{ ft}$$

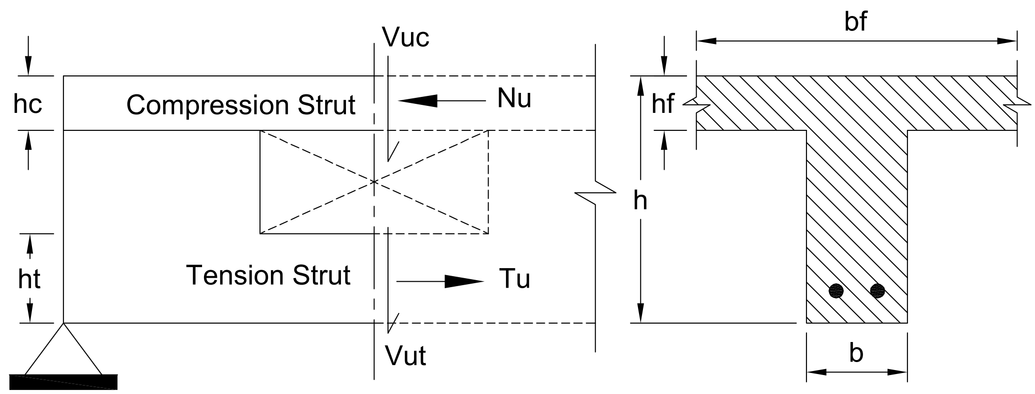
Design Summary

Provided Area of Shear Reinforcement, $A_{vc\_Prov} = A_{vc\_Prov}$ = 0.40 in^2

Distance from Support beyond which Minimum Shear Reinforcement, $x_c = x_c$ = 10.3 ft

Distance from Support beyond which Concrete can carry Shear Force, $x_m = x_m$ = 12.6 ft

Check Shear Reinforcement at Opening as per ACI 318-11 Chapter 11



System

Width of Beam, b=	4.3 in
Width of Top Flange, b_f=	48.0 in
Height of Beam, h=	26.0 in
Height of Compression Strut, h_c=	4.0 in
Height of Tension Strut, h_t=	12.0 in

Load

Ultimate Shear Force at Center of Opening, V_u=	7.2 kips
Ultimate Axial Tension Force in Tension Strut, T_u=	-10.8 kips
Ultimate Axial Compression Force in Compression Strut, N_u=	60.0 kips
Ultimate Shear Force in Tension Strut, V_{ut}=	6.0 kips
Ultimate Shear Force in Compression Strut, V_{uc}=	5.4 kips

Material Properties

Concrete Strength for Beam, f'_{c1}=	6000 psi
Concrete Strength for Topping, f'_{c2}=	3000 psi
Yield Strength of Reinforcement, f_y=	60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Calculation of Required Shear Reinforcement

Required Shear RFT, A_{v_Req}= $\dfrac{V_u}{\Phi * fy / 1000}$ = 0.16 in^2

Identification of, Bar= SEL("ACI/Bar" ;Bar;) = No.3

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 in^2

Number of Bars, n= 2

Provided Shear Reinforcement, A_{v_Prov}= $n * A_{sb}$ = 0.22 in^2

Check Validity= IF($A_{v\_Prov} \geq A_{v\_Req}$; "Valid"; "Invalid") = Valid

Calculation of Shear Reinforcement of Tensile Strut

Strut Depth, d_t= $0.8 * h_t$ = 9.6 in

Concrete Shear Strength for Tensile Strut (According to Eq.11-8 of ACI318),

V_{ct}= $2 * \left(1 + \dfrac{T_u * 1000}{500 * b * h_t}\right) * \lambda * \dfrac{\sqrt{f'_{c1}}}{1000} * b * d_t$ = 3.72 kips

Shear Reinforcement for Ten. Strut=IF($V_{ut} \leq \Phi * V_{ct}$;"Not Required";"Required") = Required

Spacing between Stirrups (According to Cl.11.4.5.1 of ACI318),

s= $0.75 * h_t$ = 9.00 in

Required RFT Area, A_{vt_Req}= $\dfrac{(V_{ut} - \Phi * V_{ct}) * s}{\Phi * f_y * d_t / 1000}$ = 0.07 in^2

Identification of, Bar= SEL("ACI/Bar" ;Bar;) = No.3

Provided Reinforcement, A_{sb}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 in^2

Number of Bars, n= 1

Provided Shear Reinforcement, A_{vt_Prov}= $n * A_{sb}$ = 0.11 in^2

Check Validity= IF($A_{vt\_Prov} \geq A_{vt\_Req}$; "Valid"; "Invalid") = Valid

Calculation of Shear Reinforcement of Compression Strut

Strut Depth, d_c= $0.8 * h_c$ = 3.2 in

Concrete Shear Strength for Tensile Strut (According to Eq.11-8 of ACI318),

V_{cc}= $2 * \left(1 + \dfrac{N_u * 1000}{2000 * b_f * h_c}\right) * \lambda * \dfrac{\sqrt{f'_{c2}}}{1000} * b_f * d_c$ = 19.46 kips

Shear Reinforcement for Comp. Strut=IF($V_{uc} \leq \Phi * V_{cc}$;"Not Required";"Required") = Not Required

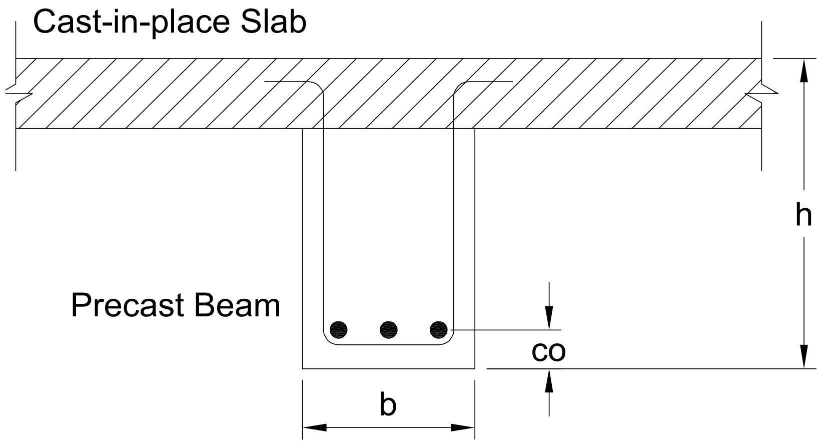
Spacing between Stirrups (According to Cl.11.4.5.1 of ACI318),

s=	$0.75 * h_c$	=	3.00 in
Required RFT Area, A_{vc_Req}=	$\frac{(V_{uc} - \Phi * V_{cc}) * s}{\Phi * f_y * d_c / 1000}$	=	-0.19 in^2
Identification of, Bar=	SEL("ACI/Bar" ;Bar;)	=	No.3
Provided Reinforcement, A_{sb}=	TAB("ACI/Bar"; Asb; Bar=Bar)	=	0.11 in^2
Number of Bars, n=			1
Provided Shear Reinforcement, A_{vc_Prov}=	$n * A_{sb}$	=	0.11 in^2
Check Validity=	IF($A_{vc\_Prov} \geq A_{vc\_Req}$; "Valid"; "Invalid")	=	Valid

Design Summary

Shear Reinforcement for Beam, A_{v_Prov}=	A_{v_Prov}	=	0.22 in
Shear Reinforcement for Ten. Strut, A_{vt_Prov}=	IF($V_{ut} \leq \Phi * V_{ct}$;"Zero" ; A_{vt_Prov})	=	0.11 in^2
Shear Reinforcement for Com. Strut, A_{vc_Prov}=	IF($V_{uc} \leq \Phi * V_{cc}$;"Zero" ; A_{vc_Prov})	=	Zero

Design of Horizontal Shear for Composite Slab and Precast Beam as per ACI 318-11 Chapters 11 & 17



System

Width of Beam, b=			10.0 in
Height of Beam, h=			20.5 in
Concrete Cover, co=			1.5 in
Depth of Beam, d=	h - co	=	19.0 in
Span of Simple Beam, L=			30.0 ft
Identification of, Bar=	SEL("ACI/Bar" ;Bar;)	=	No.5
Diameter of Bars, d_b=	TAB("ACI/Bar" ;Dia ;Bar=Bar)	=	0.63 in
Number of Bars, n=			2

Load

Service Dead Load, W_D=			315 lb/ft
Service Live Load, W_L=			3370 lb/ft
Ultimate Load, W_u=	$1.2 * W_D + 1.6 * W_L$	=	5770 lb/ft

Material Properties

Concrete Strength, f'_c=		3000 psi
Yield Strength of Reinforcement, f_y=		60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=		0.75
Modification Factor for Lightweight Concrete, λ=		1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), μ= 1.0* λ	=	1.00

Calculation of Horizontal Shear Reinforcement

Ultimate Shear Force at Distance (d) from Support

$V_u = \dfrac{\left(W_u * \dfrac{L}{2}\right) - \left(W_u * \dfrac{d}{12}\right)}{1000}$ = 77.4 kips

Horizontal Shear Strength (According to Cl.17.5.3 of ACI318),

$\Phi V_{nh} = \dfrac{\Phi * 500 * b * d}{1000}$ = 71.3 kips

Horizontal Shear Reinforcement= IF($V_u \le \Phi V_{nh}$; "Not Required"; "Required") = Required

Horizontal Shear Force Pre one foot, $v_{uh} = \dfrac{V_u}{d * b}$ = 0.407 ksi

Required RFT Area for Shear Friction, $A_{vf} = \dfrac{v_{uh} * b * 12}{\Phi * fy * \mu / 1000}$ = 1.09 in^2 /ft

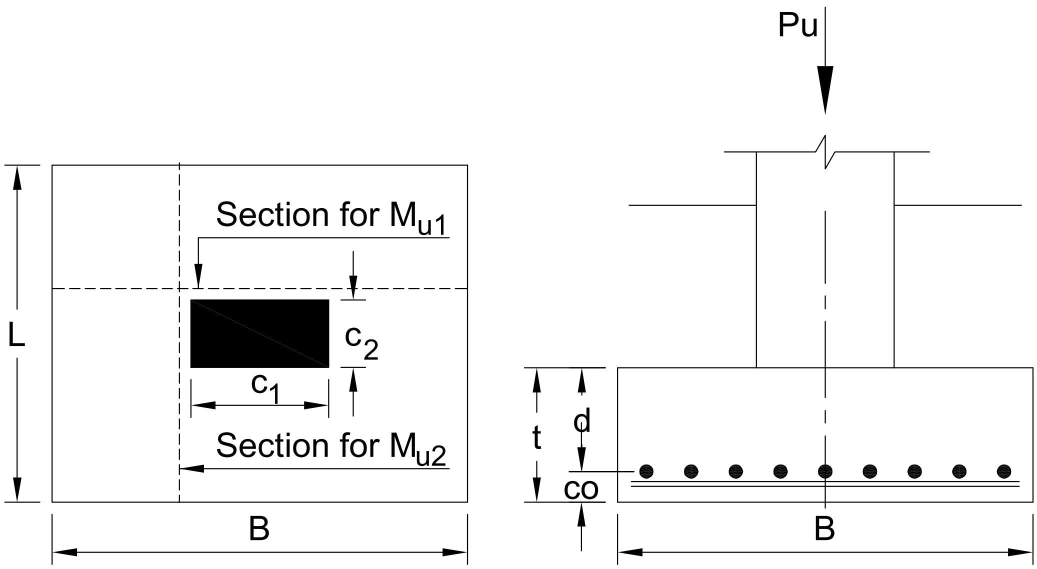
Spacing Between Links, $s = \dfrac{\pi * n * 12 * d_b^{\,2}}{A_{vf} * 4}$ = 6.9 in

Design Summary

Spacing Between Links, s= s = 6.9 in

Chapter 2: Foundation Design

Design of Reinforcement for Shallow Foundation as per ACI 318-11 Chapter 15



System

Width of Column, c_1=			30.0 in
Length of Column, c_2=			12.0 in
Width of Footing, B=			13 ft
Length of Footing, L=			13 ft
Area of Footing, A_f=	B * L	=	169 ft^2
Depth of Footing, t=			30.5 in
Concrete Cover, co=			2.5 in
Effective Depth of Footing, d=	t-co	=	28.0 in

Load

Service Dead Load, P_D=			350 kips
Service Live Load, P_L=			275 kips
Ultimate Load, P_u=	$1.2*P_D+1.6*P_L$	=	860 kips
Ultimate Pressure, q_s=	P_u / A_f	=	5.09 ksf

Material Properties

Concrete Strength, f'_c= 3000 psi

Yield Strength of Reinforcement, f_y= 60000 psi

Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ= 0.90

Determine Area of Reinforcement Distributed in Footing Width (B)

$$M_{u1} = q_s * B * \left(0.5 * \left(L - \frac{c_2}{12}\right)\right)^2 / 2 = 1191 \text{ kip*ft}$$

$$R_{n1} = \frac{M_{u1} * 12000}{\Phi * B * 12 * d^2} = 130 \text{ psi}$$

$$\rho_1 = \frac{0.85 * f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_{n1}}{0.85 * f'_c}}\right) = 0.0022$$

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

ρ_{min}= IF(f_y≤50000;0.002;IF(f_y≥77143;0.0014;0.0018)) = 0.0018

Required Area of Reinforcement, A_{s1_Req}= MAX(ρ_{min}; ρ_1) * B * d * 12 = 9.61 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.8

Provided Reinforcement, A_{sb1}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.79 in^2

Number of Bars, n_1= 13

Provided Area of Reinforcement, A_{s1_Prov}= $n_1 * A_{sb1}$ = 10.27 in^2

Check Validity= IF(A_{s1_Prov}≥A_{s1_Req}; "Valid"; "Invalid") = Valid

Determine Area of Reinforcement Distributed in Footing Length (L)

$M_{u2} =$ $q_s * L * \left(0.5 * \left(B - \frac{c_1}{12}\right)\right)^2 / 2$ = 912 kip*ft

$R_{n2} =$ $\frac{M_{u2} * 12000}{\Phi * L * 12 * d^2}$ = 99 psi

$\rho_2 =$ $\frac{0.85 * f'_c}{f_y} * \left(1 - \sqrt{1 - \frac{2 * R_{n2}}{0.85 * f'_c}}\right)$ = 0.0017

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

$\rho_{min} =$ IF($f_y \le 50000$;0.002;IF($f_y \ge 77143$;0.0014;0.0018)) = 0.0018

Required Area of Reinforcement, A_{s2_Req} = MAX(ρ_{min}; ρ_2) * L * d * 12 = 7.86 in^2

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar;) = No.8

Provided Reinforcement, A_{sb2}= TAB("ACI/Bar"; Asb; Bar=Bar) = 0.79 in^2

Number of Bars, n_2= 11

Provided Area of Reinforcement, $A_{s2\_Prov} = n_2 * A_{sb2}$ = 8.69 in^2

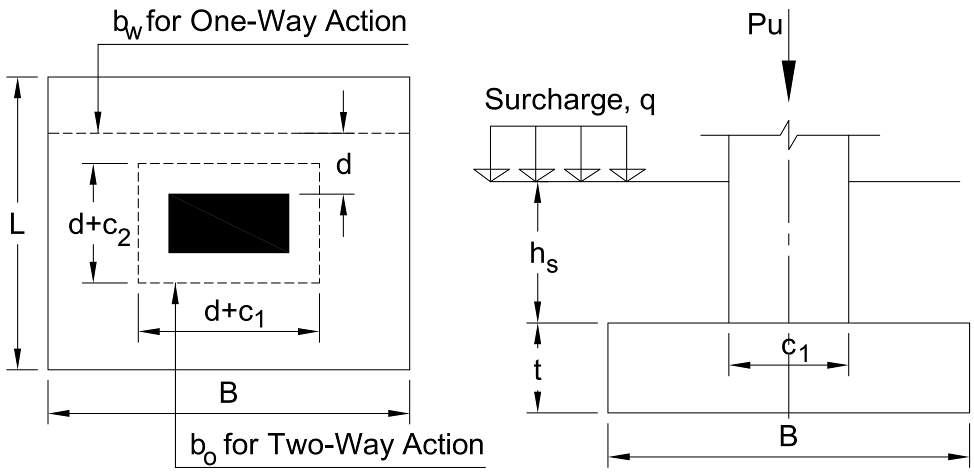
Check Validity= IF($A_{s2\_Prov} \ge A_{s2\_Req}$; "Valid"; "Invalid") = Valid

Design Summary

Area of Reinforcement Distributed in Footing Width, A_{s1}= A_{s1_Prov} = 10.27 in^2

Area of Reinforcement Distributed in Footing Length, A_{s2}= A_{s2_Prov} = 8.69 in^2

Design for Depth of Shallow Foundation as per ACI 318-11 Chapters 11 & 15



System

Width of Column, c_1=			30.0 in
Length of Column, c_2=			12.0 in
Concrete Cover, co=			5.0 in
Height of Soil above Footing, h_s=			5 ft

Load

Service Dead Load, P_D=			350 kips
Service Live Load, P_L=			275 kips
Ultimate Load, P_u=	$1.2 * P_D + 1.6 * P_L$	=	860 kips
Service Surcharge, q=			0.1 ksf
Allowable Soil Pressure at Bottom of Footing, P_a=			4.5 ksf
Average Weight of Soil and Concrete above Footing Base, w=			130.0 pcf

Material Properties

Concrete Strength, f'_c=	3000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Calculation of Base Area

Net Allowable Soil Pressure, P_{na}=	$P_a - q - \frac{w * h_s}{1000}$	=	3.75 ksf
Required Area of Footing, A_f=	$\frac{P_D + P_L}{P_{na}}$	=	166.7 ft^2
Assume Width of Footing, B=			13 ft
Assume Length of Footing, L=			13 ft
Check Validity=	IF(A_f > L*B; "Invalid"; "Valid")	=	Valid
Ultimate Pressure, q_s=	P_u / (B * L)	=	5.09 ksf

Calculation of Required Thickness for One-Way Action

Assume that Thickness of Footing, t=			33 in
Depth of Footing, d=	t - co	=	28 in
Critical Area of One-Way Shear, A_{1B}=	$B * \left(\frac{L - c_2/12}{2} - \frac{d}{12}\right)$	=	47.67 ft^2
Critical Area of One-Way Shear, A_{1L}=	$L * \left(\frac{B - c_1/12}{2} - \frac{d}{12}\right)$	=	37.92 ft^2
Critical Area of One-Way Shear, A_1=	MAX(A_{1B} ; A_{1L})	=	47.67 ft^2
Width of Critical Section for One-Way Shear, b_w=	IF(A_{1B}>A_{1L} ; B ; L)	=	13 ft
Ultimate Shear force at Critical Area Section, V_{u1}=	q_s * A_1	=	243 kips
Nominal Concrete Shear Strength, ΦV_c=	$\Phi * 2 * \lambda * \sqrt{f'_c} * \frac{b_w * 12 * d}{1000}$	=	359 kips
Check Validation =	IF(ΦV_c > V_{u1}; "O.K."; "Increase Depth")	=	O.K.

Calculation of Required Thickness for Two-Way Action

Critical Area of Two-Way Shear, A_2=	$B * L - \left(\frac{(c_1 + d) * (c_2 + d)}{144}\right)$	=	152.89 ft^2
Ultimate Shear force at Critical Area Section, V_{u2}=	q_s * A_2	=	778.2 kips
Perimeter of Critical Section for Two-Way Shear, b_0=	2*(c_1+d) + 2*(c_2+d)	=	196.0 in
Column Type=	SEL("ACI/Alfa S";Type;)	=	Interior
Alfa Constant, α_s=	TAB("ACI/AlfaS"; Alfa; Type=Type)	=	40.00
Ratio of Long to Short Column Dimensions, β=	MAX(c_1;c_2)/MIN(c_1;c_2)	=	2.50

Concrete Shear Strength (According to Eq. 11-31 of ACI318),

$V_{c1}=$ $\left(2+\frac{4}{\beta}\right)*\lambda*\sqrt{f'_c}*\frac{b_0*d}{1000}$ = 1082 kips

Concrete Shear Strength (According to Eq. 11-32 of ACI318),

$V_{c2}=$ $\left(\alpha_s*\frac{d}{b_0}+2\right)*\lambda*\sqrt{f'c}*\frac{b_0*d}{1000}$ = 2319 kips

Concrete Shear Strength (According to Eq. 11-33 of ACI318),

$V_{c3}=$ $4*\lambda*\sqrt{f'_c}*\frac{b_0*d}{1000}$ = 1202 kips

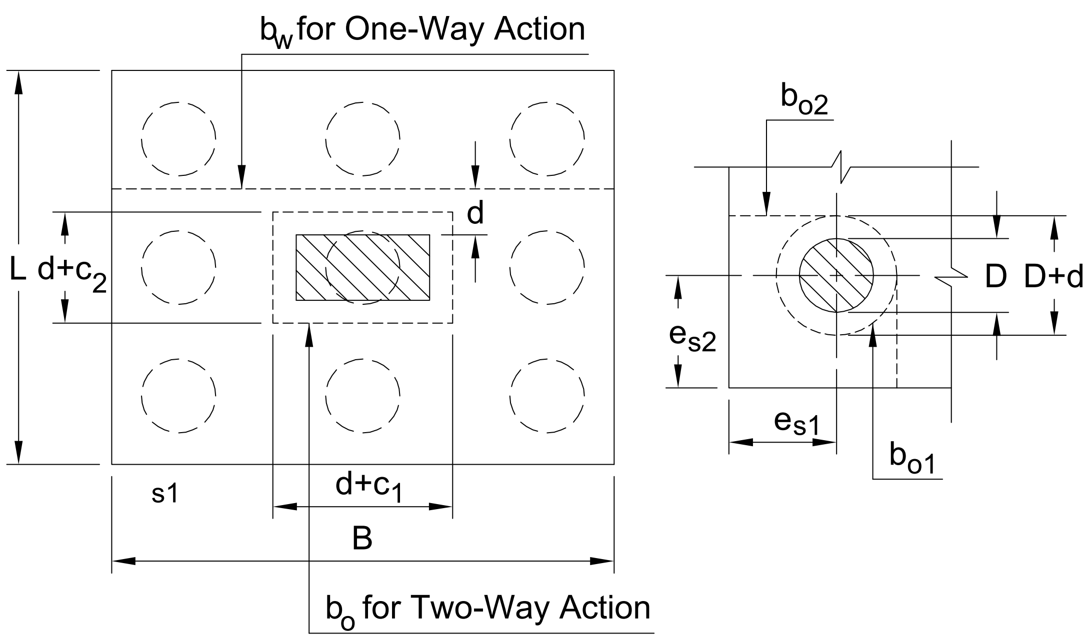
Nominal Concrete Shear Strength, $\Phi V_c = \Phi$ * MIN(V_{c1}; V_{c2}; V_{c3}) = 812 kips

Check Validation = IF($\Phi V_c > V_{u2}$; "O.K."; "Increase Depth") = O.K.

Calculation Summary

Width of Footing, B=	B	=	13 ft
Length of Footing, L=	L	=	13 ft
Thickness of Footing, t=	t	=	33 in

Design Depth for Pile Cap as per ACI 318-11 Chapter 11



System

Width of Column, c_1=	16.0 in
Length of Column, c_2=	16.0 in
Pile Diameter, D=	12.0 in
Edge Distance for Corner Pile, e_{s1}=	15 in
Edge Distance for Corner Pile, e_{s2}=	15 in
Width of Pile Cap, B=	8.5 ft
Length of Pile Cap, L=	8.5 ft
Concrete Cover, co=	7.0 in

Load

Pile Service Dead Load, P_D=			20 kips
Pile Service Live Load, P_L=			10 kips
Ultimate Pile Load, P_u=	$1.2 * P_D + 1.6 * P_L$	=	40 kips

Material Properties

Concrete Strength, f'_c=	4000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ=	0.75
Modification Factor for Lightweight Concrete, λ=	1.00

Calculation of Required Thickness due to One-Way Shear

Assume that Thickness of Pile Cap, t= 22 in

Depth of Pile Cap, d= t - co = 15 in

Width of Critical Section for One-Way Shear, b_w= MIN(B ; L) = 8.5 ft

Number of Piles fall within Critical Section for One-Way Action, n_{r1}= 3

Ultimate Shear force at Critical Area Section, V_{u1}= $P_u * n_{r1}$ = 120 kips

Nominal Concrete Shear Strength, $\Phi V_c = \Phi * 2 * \lambda * \sqrt{f'_c} * \dfrac{b_w * 12 * d}{1000}$ = 145 kips

Check Validation = IF($\Phi V_c > V_{u1}$; "O.K."; "Increase Depth") = O.K.

Calculation of Required Thickness due to Two-Way Shear for Group Piles

Perimeter of Critical Section for Two-Way Shear, b_0= $2*(c_1+d) + 2*(c_2+d)$ = 124.0 in

Number of Piles fall within Critical Section for Two-Way Action, n_{r2}= 8

Ultimate Shear force at Critical Area Section, V_{u2}= $P_u * n_{r2}$ = 320 kips

Column Type= SEL("ACI/Alfa S";Type;) = Interior

Alfa Constant, α_s= TAB("ACI/AlfaS"; Alfa; Type=Type) = 40.00

Ratio of Long to Short Column Dimensions, β= MAX(c_1;c_2)/MIN(c_1;c_2) = 1.00

Concrete Shear Strength (According to Eq. 11-31 of ACI318),

V_{c1}= $\left(2 + \dfrac{4}{\beta}\right) * \lambda * \sqrt{f'_c} * \dfrac{b_0 * d}{1000}$ = 706 kips

Concrete Shear Strength (According to Eq. 11-32 of ACI318),

V_{c2}= $\left(\alpha_s * \dfrac{d}{b_0} + 2\right) * \lambda * \sqrt{f'c} * \dfrac{b_0 * d}{1000}$ = 804 kips

Concrete Shear Strength (According to Eq. 11-33 of ACI318),

V_{c3}= $4 * \lambda * \sqrt{f'_c} * \dfrac{b_0 * d}{1000}$ = 471 kips

Nominal Concrete Shear Strength, $\Phi V_c = \Phi$ * MIN(V_{c1}; V_{c2}; V_{c3}) = 353 kips

Check Validation = IF($\Phi V_c > V_{u2}$; "O.K."; "Increase Depth") = O.K.

Calculation of Required Thickness due to Two-Way Shear for Single Corner Pile

Perimeter of Critical Section for Two-Way Shear, b_{01}= $\pi * (D + d)$ = 84.8 in

Perimeter of Critical Section for Two-Way Shear, b_{02}= $\pi * (D+d)/4 + e_{s1} + e_{s2}$ = 51.2 in

Perimeter of Critical Section for Two-Way Shear, b_0= $MIN(b_{01} ; b_{02})$ = 51.2 in

Perimeter Ultimate Shear force at Critical Section, $V_{u3}= P_u * 1.0$ = 40 kips

Column Type= SEL("ACI/Alfa S";Type;) = Corner

Alfa Constant, α_s= TAB("ACI/AlfaS"; Alfa; Type=Type) = 20.00

Ratio of Long to Short Column Dimensions, $\beta= MAX(c_1;c_2)/MIN(c_1;c_2)$ = 1.00

Concrete Shear Strength (According to Eq. 11-31 of ACI318),

V_{c1}= $\left(2+\frac{4}{\beta}\right)*\lambda*\sqrt{f'_c}*\frac{b_0*d}{1000}$ = 291 kips

Concrete Shear Strength (According to Eq. 11-32 of ACI318),

V_{c2}= $\left(\alpha_s*\frac{d}{b_0}+2\right)*\lambda*\sqrt{f'c}*\frac{b_0*d}{1000}$ = 382 kips

Concrete Shear Strength (According to Eq. 11-33 of ACI318),

V_{c3}= $4*\lambda*\sqrt{f'_c}*\frac{b_0*d}{1000}$ = 194 kips

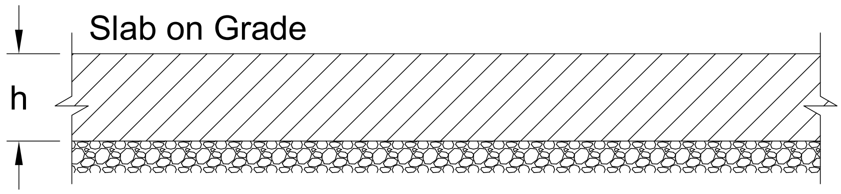
Nominal Concrete Shear Strength, $\Phi V_c= \Phi * MIN(V_{c1}; V_{c2}; V_{c3})$ = 146 kips

Check Validation = IF($\Phi V_c > V_{u3}$; "O.K."; "Increase Depth") = O.K.

Calculation Summary

Thickness of Pile Cap, t= t = 22 in

Design of Slab on Grade Due to Wheel Load as per ACI 360-10 Appendix 1



System

Thickness of Slab on Grade, h=	10.0 in
Spacing between Wheels, s=	40.0 in
Contact Area per Wheel, A_c=	50.0 in^2
Effective Contact Area per Wheel (According to Cl.A1.2 of ACI360), A_{c_eff}=	61.5 in^2

Load

Wheel Axle Load, P=	30 kips

Material Properties

Concrete Strength, f'_c=	4000 psi
Subgrade Modulus, K=	100 lb/in^3
Safety Factor, FS=	2.00

Checking Slab Thickness

Modulus of Rupture of Concrete, f_r=	$9 * \sqrt{f'_c}$	=	569.2 psi
Concrete Working Stress, f_{t_all}=	f_r / FS	=	284.6 psi
Slab Stress per 1000 lb Axle Load, f_t=	f_{t_all} / (P/1.0)	=	9.5 psi
Required Slab Thickness (According to Fig.A1.1 of ACI360),			
h_{min}=			9.92 in
Check Validation=	IF(h≥h_{min}; "Valid"; "Invalid")	=	Valid

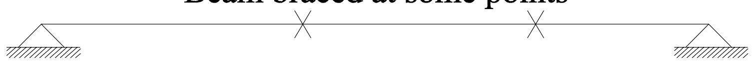
Design Summary

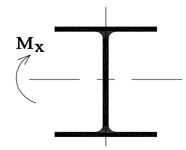
Thickness of Slab on Grade, h=	h	=	10.0 in

Chapter 3: Steel Design

Design of W-Shapes Subjected to Bending Moment about Strong Axis and Braced at Some Points

Beam braced at some points





Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
Fy=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length and Cb

Total length, L=	35.00 ft
Unsupported length, L_b=	17.50 ft
From Table 3-1 (AISC), C_b=	1.50

Design Moments and Uniform Live Load

Ultimate moment, M_u=	200.00 kip*ft
Ultimate moment due to live load case, M_L=	140.00 kip*ft
Ultimate shear, Q_u=	30.00 kips

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W21X48
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	20.60 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.35 in
Flange width, b_f=	TAB("AISC/W";$b_{f;}$NAME=sec.)	=	8.14 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.43 in
Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	107.00 in^3
Elastic sec. modulus, S_x=	TAB("AISC/W";S_x;NAME=sec.)	=	93.00 in^3
Inertia about x-axis, I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	959.00 in^4
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	1.66 in
r_{ts}=	TAB("AISC/W";r_{ts};NAME=sec.)	=	2.05 in

(r_y is radius of gyration about y-axis and r_{ts} : is effective radius of gyration for the L.T.B.)

Torsional constant, J= TAB("AISC/W";J;NAME=sec.) = 0.80 in^4

h_o= TAB("AISC/W";h_o;NAME=sec.) = 20.20 in

(h_o : is the distance between C.L. of flanges)

AISC Specification Eqn. (F2-1):

Yielding Moment, Mp= $Z_x*F_y*1/12$ = 446 kip*ft

Element Classification

(1) Web:

h/t_w, λ_w= TAB("AISC/W";h/t_w;NAME=sec.) = 53.60

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web_Class= IF($\lambda_w \le 3.76*\sqrt{(E/F_y)}$;"Compact"; "Non-Compact") = Compact

(2) Comp. flange:

$b_f/2t_f$, λ_f= TAB("AISC/W";bf/2tf;NAME=sec.) = 9.47

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

λ_{pf}= $0.38*\sqrt{(E/F_y)}$ = 9

λ_{rf}= $1.00*\sqrt{(E/F_y)}$ = 24

Fl_Class= IF($\lambda_f \le \lambda_{pf}$;"Compact";IF($\lambda_f > \lambda_{rf}$;"Slender";"Non-Compact")) = Non-Compact

The available strength provided by AISC Specification Sections F3.1 and F3.2, the nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{n1a}= $M_p-0.7*F_y*S_x*1/12$ = 175 kip*ft

M_{n1}= IF(Fl_Class="Compact"; M_p; $(M_p- M_{n1a}*(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}))$) = 441 kip*ft

Lateral Torsional Buckling (LTB)

The limiting lengths Lp and Lr are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:

L_p= $1.76*r_y*\sqrt{(E/F_y)}/12$ = 5.86 ft

L_{r1}= $\sqrt{\frac{J*1.0}{S_x*h_o}}$ = 0.02

L_{r2}= $\sqrt{1+\sqrt{6.76*\left(\frac{0.7*F_y*S_x*h_o}{E*J*1.0}\right)^2}}$ = 2.89

L_r= $1.95/12*r_{ts}*\frac{E}{0.7*F_y}*L_{r1}*L_{r2}$ = 15.95 ft

Case=	IF(L_b>L_r;"ELTB";IF($L_b \le L_p$; "No LTB";"InLTB"))	=	ELTB

("ELTB" refers to elastic LTB. and "InLTB" refers to Inelastic LTB.)

According to the AISC Spec. Eqn. F2-2:

M_1=	MIN(M_p; C_b*(M_p-(M_p-0.7*1/12*F_y*S_x)*(L_b-L_p)/(L_r-L_p)))	=	367 kip*ft

According to the AISC Spec. Eqn. F2-4:

F_{cr}=	C_b*π^2*E/((L_b+0.01)*12/r_{ts})^2	=	40.87 ksi
$F_{cr,mod}$=	√(1+0.078*J*1.0/(S_x*h_o)*(L_b*12/r_{ts})^2)	=	1.16 ksi

According to the AISC Spec. Eqns. F2-3:

M_2=	MIN(M_p;F_{cr}*S_x/12*$F_{cr,mod}$)	=	367 kip*ft

According to the AISC Spec. Eqn. F2-2:

M_{n2}=	IF (Case="No LTB";M_p;IF(Case="InLTB";M_1;M_2))	=	367 kip*ft

Check The Available Flexure Strength

ΦM_n=	0.90*MIN(M_p;M_{n1};M_{n2})	=	330 kip*ft
Safety=	IF($\Phi M_n \ge M_u$;"Safe";"Unsafe")	=	Safe
Moment_ratio=	M_u/ΦM_n	=	0.61

Check Shear Strength

h/t_w, λ_w=	TAB("AISC/W";h/t_w;NAME=sec.)	=	53.60
λ_{w0}=	2.24*√(E/F_y)	=	54
λ_{w1}=	1.10*√(5*E/F_y)	=	59
λ_{w2}=	1.37*√(5*E/F_y)	=	74

Except for very few sections, which are listed in the User Note, AISC Specification Section G2.1(a) is applicable to the I-shaped beams published in the AISC Manual for Fy ƒ- 50 ksi. Cv is calculated exactly according to Eqns. G2-2, G2-3, G2-4, and G2-5

C_{va}=	1.51*5*E/(F_y*λ_w^2)	=	1.52
C_v=	IF($\lambda_w \le \lambda_{w0}$;1;IF(($\lambda_w$>$\lambda_{w1}$ AND $\lambda_w \le \lambda_{w2}$);$\lambda_{w1}$/$l_w$;$C_{va}$))	=	1.00

From AISC Specification Section G2.1b,

A_w=	d*t_w	=	7 in^2

From AISC Specification Section G2.1, the available shear strength is:

V_n=	0.6*F_y*A_w*C_v	=	210 kips
Φ_v=			1.00
$\Phi_v V_n$=	Φ_v*V_n	=	210 kips
Shear_safety=	IF(Φ_v*V_n>Q_u;"Safe";"Unsafe")	=	Safe

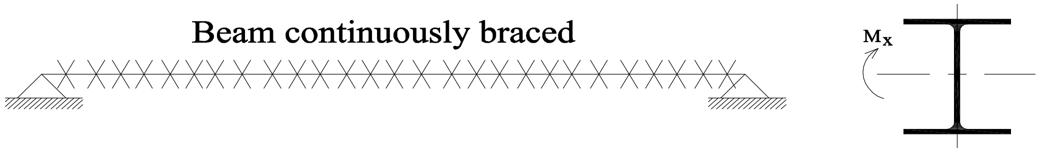
Check Deflection

Δ_{all}=	12*L/360	=	1.17 in
W_{eq} (LL), W_L=	$\frac{8*M_L}{L^2}$	=	0.91 kip/ft
Δ_{act}=	$12^3 * \frac{5*W_L*L^4}{384*E*I_x}$	=	1.10 in
Deflection safety (D_s):			
D_s=	IF(Δ_{all}≥Δ_{act};"Safe";"Unsafe")	=	Safe

Design Summary

ΦM_n=	0.90*MIN(M_p;M_{n1};M_{n2})	=	330 kip*ft
Safety=	IF(ΦM_n≥M_u;"Safe";"Unsafe")	=	Safe
Moment_ratio=	$M_u/\Phi M_n$	=	0.61
$\Phi_v V_n$=	Φ_v*V_n	=	210 kips
Shear_safety=	IF(Φ_v*V_n>Q_u;"Safe";"Unsafe")	=	Safe
Δ_{all}=	12*L/360	=	1.17 in
Δ_{act}=	$12*\frac{5*W_L*L^4}{384*E*I_x}$	=	0.01 in
D_s=	IF(Δ_{all}≥Δ_{act};"Safe";"Unsafe")	=	Safe

Design of W-Shapes Subjected to Moment about Strong Axis and Continuously Braced



Materials

Grade:	SEL("Material/ASTM";NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length and Cb

Total length, L= 35.00 ft

Design Moments and Uniform Live Load

Ultimate moment, M_u=	200.00 kip*ft
Ultimate moment due to live load case, M_L=	140.00 kip*ft
Ultimate shear, Q_u=	30.00 kips

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W21X48
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	20.60 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.35 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	8.14 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.43 in
Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	107.00 in^3
Elastic sec. modulus, S_x=	TAB("AISC/W";S_x;NAME=sec.)	=	93.00 in^3
Inertia about x-axis, I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	959.00 in^4
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	1.66 in
r_{ts}=	TAB("AISC/W";r_{ts};NAME=sec.)	=	2.05 in

(r_y is radius of gyration about y-axis and r_{ts} : is effective radius of gyration for the L.T.B.)

Torsional constant, J=	TAB("AISC/W";J;NAME=sec.)	=	0.80 in^4
h_o=	TAB("AISC/W";h_o;NAME=sec.)	=	20.20 in

(h_o is the distance between C.L. of flanges)

AISC Specification Eqn. (F2-1):

Yielding Moment, M_p= Z_x*F_y*1/12 = 446 kip*ft

Element Classification

(1) Web:

h/t_w, λ_w= TAB("AISC/W";h/t_w;NAME=sec.) = 53.60

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web_Class= IF($\lambda_w \leq 3.76*\sqrt{(E/F_y)}$;"Compact"; "Non-Compact") = Compact

(2) Comp. flange:

$b_f/2t_f$, λ_f= TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 9.47

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

λ_{pf}= $0.38*\sqrt{(E/F_y)}$ = 9

λ_{rf}= $1.00*\sqrt{(E/F_y)}$ = 24

Fl_Class= IF($\lambda_f \leq \lambda_{pf}$;"Compact";IF($\lambda_f > \lambda_{rf}$;"Slender";"Non-Compact")) = Non-Compact

Because the beam is continuously braced, and therefore not subjected to lateral-torsional buckling, the available strength is governed by AISC Specification Sections F3.1 and F3.2. The nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{n1a}= M_p-0.7*F_y*S_x*1/12 = 174.75 kip*ft

M_{n1}= IF(Fl_Class="Compact"; M_p; (M_p-(M_{n1a})*($\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}$))) = 441 kip*ft

Check The Available Flexure Strength

ΦM_n= 0.90*MIN(M_p;M_{n1}) = 397 kip*ft

Safety= IF($\Phi M_n \geq M_u$;"Safe";"Unsafe") = Safe

Moment_ratio= $M_u/\Phi M_n$ = 0.50

Check Shear Strength

h/t_w, λ_w= TAB("AISC/W";h/tw;NAME=sec.) = 53.6

λ_{w0}= $2.24*\sqrt{(E/Fy)}$ = 53.9

λ_{w1}= $1.10*\sqrt{(5*E/F_y)}$ = 59.2

λ_{w2}= $1.37*\sqrt{(5*E/F_y)}$ = 73.8

Except for very few sections, which are listed in the User Note, AISC Specification Section G2.1(a) is applicable to the I-shaped beams published in the AISC Manual for F_y = 50 ksi. C_v is calculated exactly according to Eqns. G2-2, G2-3, G2-4, and G2-5

C_v= IF($\lambda_w \leq \lambda_{w0}$;1;IF(($\lambda_w > \lambda_{w1}$ AND $\lambda_w \leq \lambda_{w2}$);$\lambda_{w1}/\lambda_w$;1.51*5*E/($F_y$*$\lambda_w^2$)))= 1.00

From AISC Specification Section G2.1b,

A_w= $d*t_w$ = 7 in^2

From AISC Specification Section G2.1, the available shear strength is:

V_n= $0.6*F_y*A_w*C_v$ = 210 kips

Φ_v= 1.00

$\Phi_v V_n$= Φ_v*V_n = 210 kips

Shear_safety= IF(Φ_v*V_n>Qu;"Safe";"Unsafe") = Safe

Check Deflection

Δ_{all}= 12*L/360 = 1.17 in

W_{eq} (LL), W_L= $\dfrac{8*M_L}{L^2}$ = 0.91 kip/ft

Δ_{act}= $12^3 * \dfrac{5*W_L*L^4}{384*E*I_x}$ = 1.10 in

Deflection safety (D_s):

Ds= IF($\Delta_{all}\geq\Delta_{act}$;"Safe";"Unsafe, increase section") = Safe

Design Summary

ΦM_n= 0.90*MIN(M_p;M_{n1}) = 397 kip*ft

Safety= IF($\Phi M_n \geq M_u$;"Safe";"Unsafe") = Safe

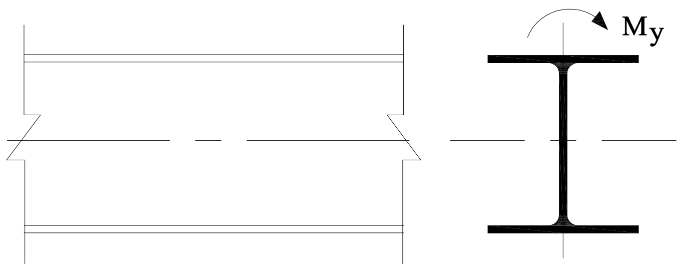
Moment_ratio= $M_u/\Phi M_n$ = 0.50

Δ_{all}= 12*L/360 = 1.17 in

Δ_{act}= $12* \dfrac{5*W_L*L^4}{384*E*I_x}$ = 0.01 in

Ds= IF($\Delta_{all}\geq\Delta_{act}$;"Safe";"Unsafe, increase section") = Safe

Design of W-shapes Subjected to Moment about Minor Axis



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length

Total length, L=	35.00 ft

Design Moments and Uniform Live Load

Ultimate moment in minor-axis, M_{yu}=	266.00 kip*ft
Ultimate shear force, Q_{xu}=	20.00 kips

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W24X131
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	24.50 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.60 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	12.90 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.96 in
Plastic sec. modulus, Z_y=	TAB("AISC/W";Z_y;NAME=sec.)	=	81.50 in^3
Elastic sec. modulus, S_y=	TAB("AISC/W";S_y;NAME=sec.)	=	53.00 in^3
AISC Specification Eqn. (F6-1):			
Yielding Moment, M_p=	MIN (Z_y*F_y*1/12;1.6/12*S_y*F_y)	=	340 kip*ft

Element Classification

Flanges:

$b_f/2t_f$, λ_f= TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 6.7

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

λ_{pf}= $0.38*\sqrt{(E/F_y)}$ = 9.2

λ_{rf}= $1.0*\sqrt{(E/F_y)}$ = 24.1

Fl_Class= IF($\lambda_f \leq \lambda_{pf}$;"Compact";"Non-Compact") = Compact

Because the beam bent about the minor-axis, the available strength is governed by AISC Specification Sections F6.1 and F6.2. The nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{n1}= IF(Fl_Class="Compact"; M_p; $(M_p-(M_p-0.7*F_y*S_y*1/12)*(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}))$) = 340 kip*ft

Check The Available Flexure Strength

ΦM_n= $0.90*MIN(M_p;M_{n1})$ = 306 kip*ft

Safety= IF($\Phi M_n \geq M_{yu}$;"Safe";"Unsafe") = Safe

Moment ratio= $M_{yu}/\Phi M_n$ = 0.87

Check Shear Strength Section G (AISC Spec.)

Calculate A_w. (Multiply by 2 for both shear resisting elements.)

A_w= $2*b_f*t_f$ = 24.77 in^2

Calculate Cv: (Eqns. G2-2, G2-3, G2-4 and G2-5)

$b_f/2t_f$, λ_f= TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 6.70

k_v= 1.20

ψ_{f1}= $1.1*\sqrt{(k_v*E/F_y)}$ = 29.0

ψ_{f2}= $1.37*\sqrt{(k_v*E/F_y)}$ = 36.1

C_v= IF(λf≤ψf1;1; IF(λf>ψf1 AND λf≤ψf2;ψf1/λf;1.51*kv*E/(Fy*λf^2))) = 1.0

From AISC Specification Section G2.1, the available shear strength is:

Φ_v= 0.90

Nominal shear strength, V_n= $0.6*F_y*A_w*C_v$ = 743 kips

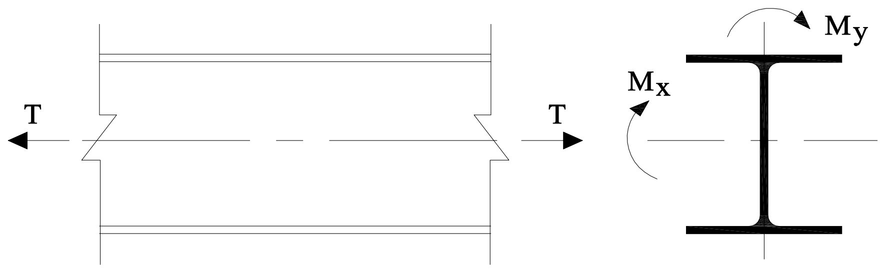
Design shear, ΦV_n= Φ_v*V_n = 669 kips

Shear_safety= IF($V_n>Q_{xu}$; "Safe"; "Unsafe") = Safe

Design Summary

ΦM_n=	0.90*MIN(M_p;M_{n1})	=	306 kip*ft
Safety=	IF($\Phi M_n \geq M_{yu}$;"Safe";"Unsafe")	=	Safe
Moment ratio=	$M_{yu}/\Phi M_n$	=	0.87
Design shear, ΦV_n=	Φ_v *V_n	=	669 kips
Shear_safety=	IF(V_n>Q_{xu}; "Safe"; "Unsafe")	=	Safe

Design of W-Shapes Subjected to Tension Force and Bending Moments



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length

Unsupported length, L_b=	30.00 ft
kL_{in}=	14.00 ft
kL_{out}=	30.00 ft

(kL_{in} and kL_{out} are the strong and weak/torsional unbraced lengths, respectively)

From Table 3-1 (AISC), C_{b1}= 1.14

Given Straining Actions

Dead Load:

T_D=	29.0 kips
M_{xD}=	32.0 kip*ft
M_{yD}=	11.3 kip*ft

Live Load:

T_L=	87.0 kips
M_{xL}=	96.0 kip*ft
M_{yL}=	33.8 kip*ft

Ultimate Tension Force and Bending Moments

T_u=	$1.2*T_D+1.6*T_L$	=	174.0 kips
M_{ux}=	$1.2*M_{xD}+1.6*M_{xL}$	=	192.0 kip*ft
M_{uy}=	$1.2*M_{yD}+1.6*M_{yL}$	=	67.6 kip*ft

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W14X82
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	14.30 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.51 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	10.10 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.85 in
Gross Area, A=	TAB("AISC/W";A;NAME=sec.)	=	24.00 in^2
I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	881.00 in^4
I_y=	TAB("AISC/W";I_y;NAME=sec.)	=	148.00 in^4

(I_x and I_y are the moment of inertia about x-and y-axes, respectively)

Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	139.00 in^3
Elastic sec. modulus, S_x=	TAB("AISC/W";S_x;NAME=sec.)	=	123.00 in^3
Plastic sec. modulus, Z_y=	TAB("AISC/W";Z_y;NAME=sec.)	=	44.80 in^3
Elastic sec. modulus, S_y=	TAB("AISC/W";S_y;NAME=sec.)	=	29.30 in^3
r_x=	TAB("AISC/W";r_x;NAME=sec.)	=	6.05 in
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	2.48 in

(r_x and r_y are the radius of gyration about x- and y-axis, respectively)

Torsional constant, J=	TAB("AISC/W";J;NAME=sec.)	=	5.07 in^4
r_{ts}=	TAB("AISC/W";r_{ts};NAME=sec.)	=	2.85 in
h_o=	TAB("AISC/W";h_o;NAME=sec.)	=	13.40 in

(r_{ts} is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)

AISC Specification Eqn. (F6-1), the yielding moment in minor axis (M_{py}):

M_{py}= MIN (Z_y*F_y*1/12;1.6/12*S_y*F_y) = 187 kip*ft

Slenderness Check (According to section E2)

For members designed on the basis of compression, the slenderness ratio KL/r should not exceed 300.

$$\lambda_x = \frac{kL_{in}}{r_x} * 12 = 27.8$$

$$\lambda_y = \frac{kL_{out}}{r_y} * 12 = 145.2$$

Then, the governed slenderness (λ_{max}):

λ_{max}=	MAX(λ_x;λ_y)	=	145.2
Slenderness_check=	IF($\lambda_{max} \leq 300$; "Safe"; "Unsafe")	=	Safe

Nominal Tensile Strength

From AISC Specification Section D2(a), the nominal tensile strength due to tensile yielding on the gross section is:

T_n= F_y*A = 1200.0 kips

Nominal Flexural Strength about x-x Axis

Yielding: from AISC specification section F2.1, the nominal flexural strength due to yielding (plastic moment) is:

M_{px}= Z_x*F_y*1/12 = 579 kip*ft

Lateral torsional buckling (LTB): the limiting lengths L_p and L_r are determined according to the AISC spec. eqns. F2-5 and F2-6, as follows:

L_p= 1.76*r_y*√(E/F_y)/12 = 8.76 ft

$$L_{r1}=\sqrt{\frac{J*1.0}{S_x*h_o}} = 0.06$$

$$L_{r2}=\sqrt{1+\sqrt{6.76*\left(\frac{0.7*F_y*S_x*h_o}{E*J*1.0}\right)^2}} = 1.42$$

$$L_r=1.95/12*r_{ts}*\frac{E}{0.7*F_y}*L_{r1}*L_{r2} = 32.69\text{ ft}$$

Case= IF(L_b>L_r;"ELTB";IF(L_b≤L_p; "No LTB";"InLTB")) = InLTB

("ELTB" refers to elastic lateral torsional buckling and "InLTB" refers to inelastic lateral torsional buckling).

The lateral torsional buckling modification factor, C_b:

$$T_{ey}=\frac{\pi^2*E*I_y}{(L_b*12)^2} = 326.9\text{ kips}$$

$$C_b=C_{b1}*\sqrt{1+\frac{T_u}{T_{ey}}} = 1.41$$

According to the AISC Spec. Eqn. F2-2:

M_{1a}= M_{px}-0.7*1/12*F_y*S_x = 220 kip*ft

M_1= MIN(M_{px}; C_b*(M_{px}-M_{1a}*(L_b-L_p)/(L_r-L_p))) = 541 kip*ft

According to the AISC Spec. Eqn. F2-4:

F_{cr}= $\dfrac{C_b * \pi^2 * E}{\left(\dfrac{(L_b + 0.01) * 12}{r_{ts}}\right)^2}$ = 25.28 ksi

$F_{cr,mod}$= $\sqrt{1 + \dfrac{0.078 * J * 1.0}{S_x * h_o} * \left(\dfrac{L_b * 12}{r_{ts}}\right)^2}$ = 2.20 ksi

According to the AISC Spec. Eqns. F2-3:

M_2= MIN(M_{px};F_{cr}*S_x/12*$F_{cr,mod}$) = 570 kip*ft

According to the AISC Spec. Eqn. F2-2:

M_{nx2}= IF (Case="No LTB";M_{px};IF(Case="InLTB";M_1;M_2)) = 541 kip*ft

Element Classification

(1) Web:

h/tw, λ_w= TAB("AISC/W";h/t_w;NAME=sec.) = 22.40

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web_Class= IF($\lambda_w \leq 3.76 * \sqrt{(E/F_y)}$;"Compact"; "Non-Compact") = Compact

(2) Comp. flange:

$b_f/2t_f$, λ_f= TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 5.92

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

λ_{pf}= 0.38*$\sqrt{(E/F_y)}$ = 9

λ_{rf}= 1.00*$\sqrt{(E/F_y)}$ = 24

Fl_Class= IF($\lambda_f \leq \lambda_{pf}$;"Compact";IF($\lambda_f > \lambda_{rf}$;"Slender";"Non-Compact")) = Compact

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{nx1a}= M_{px}-0.7*F_y*S_x*1/12 = 220 kip*ft

M_{nx1}= IF(Fl_Class="Compact"; M_{px}; M_{px}-M_{nx1a}*($\dfrac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}$)) = 579 kip*ft

M_{ny1a}= M_{py}-0.7*F_y*S_y*1/12 = 102 kip*ft

M_{ny1}= IF(Fl_Class="Compact"; M_{py}; M_{py}- M_{ny1a}*($\dfrac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}$)) = 187 kip*ft

Design Flexure Moment in Major/Minor Axes

M_{nx}=	$MIN(M_{px};M_{nx1};M_{nx2})$	=	541 kip*ft
M_{ny}=	$MIN(M_{py};M_{ny1})$	=	187 kip*ft

Calculate The Available Flexural and Axial Strengths

Φ_b=			0.90
Φ_t=			0.90
T_c=	$\Phi_t * T_n$	=	1080 kips
M_{cx}=	$\Phi_b * M_{nx}$	=	487 kip*ft
M_{cy}=	$\Phi_b * M_{ny}$	=	168 kip*ft

Interaction of Tension and Flexure

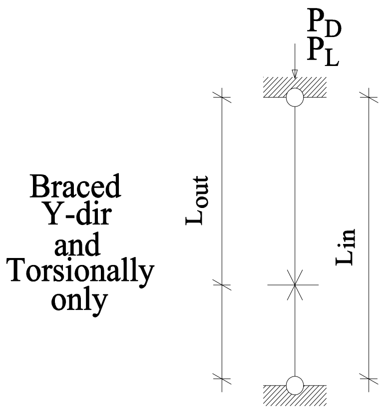
Check limit for AISC Specification Equation H1-1a.

Tension_ratio, t=	$\frac{T_u}{T_c}$	=	0.16
Moment_ratio, m=	$\frac{M_{ux}}{M_{cx}}+\frac{M_{uy}}{M_{cy}}$	=	0.80
Safety_ratio, r=	IF(t ≥0.2; t+8/9*m; t/2+m)	=	0.88
Safety=	IF(r≤1;"Safe";"Unsafe")	=	Safe

Design Summary

T_u=	$1.2*T_D+1.6*T_L$	=	174.0 kips
M_{ux}=	$1.2*M_{xD}+1.6*M_{xL}$	=	192.0 kip*ft
M_{uy}=	$1.2*M_{yD}+1.6*M_{yL}$	=	67.6 kip*ft
T_c=	$\Phi_t * T_n$	=	1080 kips
M_{cx}=	$\Phi_b * M_{nx}$	=	487 kip*ft
M_{cy}=	$\Phi_b * M_{ny}$	=	168 kip*ft
Slenderness_check=	IF(λ_{max}≤300; "Safe"; "Unsafe")	=	Safe
Safety_ratio, r=	IF(t ≥0.2; t+8/9*m; t/2+m)	=	0.88
Safety=	IF(r≤1;"Safe";"Unsafe")	=	Safe

Design of W-Shapes Subjected to Axial Compression



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50.00
E=			29000 ksi

Buckling Lengths

kL_{in}=	30.00 ft
kL_{out}=	15.00 ft

(kL_{in} and kL_{out} are unbraced lengths for the strong- and weak-axes)

Axial Loads

Dead load, P_D=	140 kips
Live load, P_L=	420 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, P_u=	$1.2{*}P_D{+}1.6{*}P_L$	=	840 kips

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W14X90
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	14.00 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.44 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	14.50 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.71 in
Area, A=	TAB("AISC/W";A;NAME=sec.)	=	26.50 in^2
r_x=	TAB("AISC/W";r_x;NAME=sec.)	=	6.14 in
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	3.70 in

(r_x and r_y are the radius of gyration about x- and y-axes, respectively)

Element Classification (According to Table B4-1)

(1) Web:

h/t_w, λ_w= TAB("AISC/W";h/t_w;NAME=sec.) = 25.90

According to AISC Specification Table B4.1 Case 10, the limiting width-to-thickness ratio for non-compact web is:

Web_Class= IF($\lambda_w \leq 1.49 * \sqrt{E/F_y}$);"Non-Compact";"Slender") = Non-Compact

(2) Flanges:

$b_f/2t_f$, λ_f= TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 10.20

According to AISC Specification Table B4.1 Case 4, the limiting width-to-thickness ratio for non-compact flange is:

k_c= MIN(MAX($4/\sqrt{\lambda_w}$);0.35);0.76) = 0.76

λ_{rf}= $0.64*\sqrt{k_c*E/F_y}$) = 13

Fl_Class= IF($\lambda_f \leq \lambda_{rf}$;"Non-Compact";"Slender") = Non-Compact

Slenderness Check (According to Section E2)

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

λ_x= $\dfrac{kL_{in}}{r_x}*12$ = 58.6

λ_y= $\dfrac{kL_{out}}{r_y}*12$ = 48.6

Then, the governed slenderness (λ_{max}):

λ_{max}= MAX(λ_x;λ_y) = 58.6

Critical Stresses

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

-Calculate the elastic critical buckling stress, F_e:

F_e= $\dfrac{\pi^2 * E}{\lambda_{max}^2}$ = 83.3 ksi

-Calculate the flexural buckling stress, F_{cr} (Eqns. E3-2 and E3-3):

λ_1= $4.71*\sqrt{E/F_y}$) = 113

F_{cr}= IF($\lambda_{max} \leq \lambda_1$; $0.658^{(F_y/F_e)} * F_y$;$0.877*F_e$) = 38.9 ksi

Nominal Compressive Strength (Eqn. E3-1)

P_n=	F_{cr}*A	=	1031 kips
Φ_v=			0.90
$\Phi_v P_n$=	Φ_v*P_n	=	928 kips

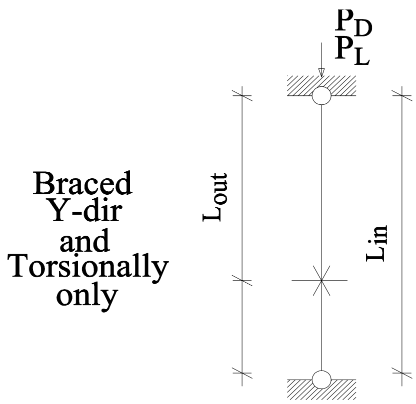
Compressive stress_safety (S_s):

S_s=	IF(Φ_v*P_n>Pu;"Safe";"Unsafe")	=	Safe
Stress_ratio=	P_u/(Φ_v*P_n)	=	0.91

Design Summary

Ultimate load, P_u=	1.2*P_D+1.6*P_L	=	840 kips
Design load, $\Phi_v P_n$=	Φ_v*P_n	=	928 kips
Stress_ratio=	P_u/(Φ_v*P_n)	=	0.91
S_s=	IF(Φ_v*P_n>Pu;"Safe";"Unsafe")	=	Safe

Design of WT-Shapes Subjected to Compression Axial Force



Materials

Grade:	SEL("Material/ASTM";NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi
G=			11200 ksi

Buckling Lengths

kL_{in}=	20.00 ft
kL_{out}=	20.00 ft
k_zL=	20.00 ft

(kL_{in} and kL_{out} are unbraced lengths for the strong- and weak- axes, respectively; k_zL is the torsional unbraced length)

Axial Loads

Axial dead load, P_D=			6 kips
Axial Live load, P_L=			18 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, P_u=	$1.2*P_D+1.6*P_L$	=	36.0 kips

Section Details

sec.:	SEL("AISC/WT"; NAME;)	=	WT7X15
Depth, d=	TAB("AISC/WT";d;NAME=sec.)	=	6.92 in
Stem thickness, t_w=	TAB("AISC/WT";t_w;NAME=sec.)	=	0.270 in
Flange width, b_f=	TAB("AISC/WT";b_f;NAME=sec.)	=	6.73 in
Flange thickness, t_f=	TAB("AISC/WT";t_f;NAME=sec.)	=	0.385 in
Gross area, A=	TAB("AISC/WT";A;NAME=sec.)	=	4.4 in^2

I_x=	TAB("AISC/WT";I_x;NAME=sec.)	=	19 in^4
I_y=	TAB("AISC/WT";I_y;NAME=sec.)	=	10 in^4

(I_x and I_y are the moment of inertia about x-and y-axes, respectively)

r_x=	TAB("AISC/WT";r_x;NAME=sec.)	=	2.07 in
r_y=	TAB("AISC/WT";r_y;NAME=sec.)	=	1.49 in

(r_x and r_y are the radius of gyration about x- and y-axis, respectively)

Torsion constant, J=	TAB("AISC/WT";J;NAME=sec.)	=	0.19 in^4
Q_s=	TAB("AISC/WT";Q_s;NAME=sec.)	=	0.61
y=	TAB("AISC/WT";y;NAME=sec.)	=	1.58 in

(Q_s is a reduction factor for unstiffned elements and y is the distance to the N.A.)

Element Classification

(1) Flanges:

$b_f/2t_f$, λ_f=	$b_f/(2*t_f)$	=	8.74

Determine the flange limiting slenderness ratio, λrf, from AISC Specification Table B4.1a case 2:

λ_{rf}=	$0.56*\sqrt{(E/F_y)}$	=	13.5
Fl_Class=	IF($\lambda_f \leq \lambda_{rf}$;"Non-Compact";"Slender")	=	Non-Compact

(2) Web:

d/tw, λ_w=	d/t_w	=	25.6

Determine the slender web limit from AISC Specification Table B4.1a case 4:

λ_{rw}=	$0.75*\sqrt{(E/F_y)}$	=	18.06
Web_Class=	IF($\lambda_w \leq \lambda_{rw}$;"Non-compact"; "Slender")	=	Slender
Q=	Q_s	=	0.610

Slenderness check:

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

λ_x=	$\frac{kL_{in}}{r_x}*12$	=	115.9
λ_y=	$\frac{kL_{out}}{r_y}*12$	=	161.1
λ_{max}=	MAX(λ_x;λ_y)	=	161.1

X-X Axis Critical Elastic Flexural Buckling Stress:

$$F_{ex} = \frac{\pi^2 * E}{\lambda_x^2} = 21.3 \text{ ksi}$$

Critical Elastic Torsional and Flexural-Torsional Buckling Stress:

$$F_{ey} = \frac{\pi^2 * E}{\lambda_y^2} = 11.0 \text{ ksi}$$

Torsional Parameters

The shear center for a T-shaped section is located on the axis of symmetry at the mid-depth of the flange.

$x_o =$ 0.0 in

$y_o = y - t_f/2 = 1.39$ in

According to the AISC Specification Eqn. E4-11:

$$r_o = \sqrt{(x_o^2 + y_o^2 + \frac{I_x + I_y}{A})} = 2.92 \text{ in}$$

According to the AISC Specification Eqn. E4-10:

$$H = 1 - \frac{x_o^2 + y_o^2}{r_o^2} = 0.77 \text{ in}$$

According to the AISC Specification Eqn. E4-9:

$$F_{ez} = \left(\frac{\pi^2 * E * C_w}{(k_z * L)^2} + GJ \right) \frac{1}{A * r_o^2}$$

Omit term with C_w per User Note at end of AISC Specification Section E4.

$$F_{ez} = \frac{G * J}{A * r_o^2} = 56.72 \text{ ksi}$$

According to the AISC Specification Eqn. E4-5:

$$F_{e2} = \frac{F_{ey} + F_{ez}}{2 * H} * \left(1 - \sqrt{1 - \frac{4 * F_{ey} * F_{ez} * H}{(F_{ey} + F_{ez})^2}} \right) = 10.5 \text{ ksi}$$

Governed Critical Elastic Buckling Stress

$F_e = \text{MIN}(F_{ex}; F_{ey}; F_{e2}) = 10.5$ ksi

Buckling Stress for The Section

Determine whether AISC Specification Equation E7-2 or E7-3 applies.

F_{er}=	$0.44*Q*F_y$	=	13.4 ksi
F_{cr}=	IF($F_e \geq F_{er}$; $Q*0.658^{(F_y*Q/F_e)}*F_y$;$0.877*F_e$)	=	9.2 ksi

Nominal Compressive Strength

P_n=	$F_{cr}*A$	=	40.5 kips
Φ_v=			0.90
$\Phi_v P_n$=	Φ_v*P_n	=	36.5 kips

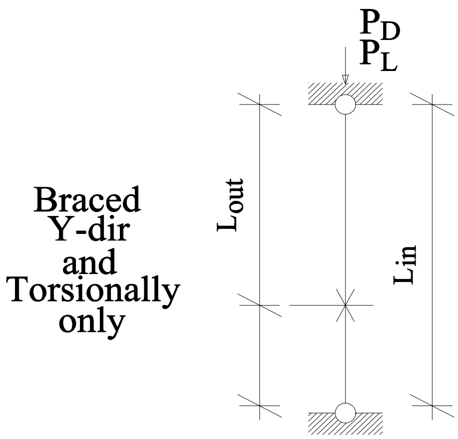
Compressive stress safety (S_s):

S_s=	IF(Φ_v*P_n>Pu;"Safe";"Unsafe")	=	Safe
Stress_ratio=	$\dfrac{P_u}{\Phi_v*P_n}$	=	0.99

Design Summary

Ultimate load, P_u=	$1.2*P_D+1.6*P_L$	=	36.0 kips
Design load, $\Phi_v P_n$=	Φ_v*P_n	=	36.5 kips
Stress_ratio=	$\dfrac{P_u}{\Phi_v*P_n}$	=	0.99
S_s=	IF(Φ_v*P_n>P_u;"Safe";"Unsafe")	=	Safe

Design of Built-Up W-Shapes with Slender Elements Subjected to Compression Axial Force



Materials

Grade:	SEL("Material/ASTM";NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi
G=			11200 ksi

Buckling Lengths

kL_{in}=	30.00 ft
kL_{out}=	15.00 ft
k_zL=	15.00 ft

(kL_{in} and kL_{out} are unbraced lengths for the strong- and weak- axes, respectively; k_zL is the torsional unbraced length)

Axial Loads

Dead load, P_D=		140 kips
Live load, P_L=		200 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, P_u=1.2*P_D+1.6*P_L	=	488 kips

Section Details

Web height, h=	15.0 in
Web th., t_w=	0.25 in
Flange width, b_f=	8.00 in
Flange th., t_f=	1.00 in

Built-Up Section Properties (ignoring fillet welds):

Area, A= $h*t_w+2*b_f*t_f$ = 19.75 in^2

I_x= $2*(b_f*t_f)*(t_f/2+h/2)^2 + \frac{t_w*h^3}{12} + \frac{b_f*(t_f)^3*2}{12}$ = 1096 in^4

I_y= $\frac{2*b_f^3*t_f}{12} + \frac{h*t_w^3}{12}$ = 85.35 in^4

r_x= $\sqrt{\frac{I_x}{A}}$ = 7.45 in

r_y= $\sqrt{\frac{I_y}{A}}$ = 2.08 in

Slenderness Check

For members designed on the basis of compression, the slenderness ratio *KL*/*r* preferably should not exceed 200.

λ_x= $\frac{kL_{in}}{r_x}*12$ = 48.3

λ_y= $\frac{kL_{out}}{r_y}*12$ = 86.5

Then, the governed slenderness (λ_{max}):

λ_{max}= $MAX(\lambda_x;\lambda_y)$ = 86.5

Elastic Flexural Buckling Stress

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly As follows:

F_{e1}= $\frac{\pi^2*E}{\lambda_{max}^2}$ = 38.3 ksi

Elastic Critical Torsional Buckling Stress

From the User Note in AISC *Specification* Section E4,

h_o= $h+t_f$ = 16 in

C_w= $\frac{I_y*h_o^2}{4}$ = 5462 in^6

From AISC Design Guide 9, Equation 3.4,

J= $\frac{2*b_f*t_f^3+h*t_w^3}{3}$ = 5.41 in^4

According to AISC Specification (Eqn. E4-4),

F_{e2}= $\left(\dfrac{\pi^2 * E * C_w}{(kzL * 12)^2} + G*J\right) * \dfrac{1}{I_x + I_y}$ = 92.1 ksi

Elastic Governed Stress

F_e= MIN(F_{e1}; F_{e2}) = 38.3 ksi

Element Classification

(1) Flanges: Check for slender flanges using AISC *Specification* Table B4.1a, then determine Q_s, the unstiffened element (flange) reduction factor using AISC *Specification* Section E7.1.

Calculate k_c using AISC *Specification* Table B4.1b note [a].

k_c= MIN(MAX(4/√(h/t_w);0.35);0.76) = 0.52

$b_f/2t_f$, λ_f= $b_f/(2*t_f)$ = 4.00

Determine the flange limiting slenderness ratio, λrf, from AISC *Specification* Table B4.1a case 2

λ_{rf}= 0.64*√(k_c*E/F_y) = 11.11

Fl_Class= IF($\lambda_f \le \lambda_{rf}$;"Non-Compact";"Slender") = Non-Compact

Calculate Qs, according to the AISC Specification Eqns. E7-4, E7-5 and E7-6

λ_{rf1}= 1.17*√(k_c*E/F_y) = 20.32

Q_{s1}= 0.9*E*k_c/(F_y*λ_f^2) = 16.97

Q_{s2}= 1.415- 0.65*λ_f*√(F_y/(E*k_c)) = 1.27

Qs= IF($\lambda_f \le \lambda_{rf}$; 1; IF($\lambda_f > \lambda_{rf1}$; Q_{s1}; Q_{s2})) = 1.00

Web: Check for a slender web, then determine Qa, the stiffened element (web) reduction factor using AISC Specification, Section E7.2.

h/t_w, λ_w= h/t_w = 60.00

Determine the slender web limit from AISC *Specification* Table B4.1a case 5

λ_{rw}= 1.49*√(E/F_y) = 35.88

Web_Class= IF($\lambda_w \le \lambda_{rw}$;"Non-compact"; "Slender") = Slender

$$Q_a = \frac{A_e}{A_g}$$

where A_e is the effective area based on the reduced effective width (b_e). For AISC *Specification* Equation E7-17, take f as F_{cr} with F_{cr} calculated based on Q = 1.0. Select between AISC *Specification* Equations E7-2 and E7-3 based on KL/r_y.

λ_1= $4.71 * \sqrt{\dfrac{E}{F_y}}$ = 113

Calculate the flexural buckling stress, F_{cr}:

F_{cr1}=	IF($\lambda_{max} \le \lambda_1$; $0.658^{(F_y/F_e)} * F_y$;0.877*F_e)	=	29.0 ksi
b_e=	MIN(1.92*t_w*$\sqrt{\frac{E}{F_{cr1}}}$ *(1- $\frac{0.34}{\lambda_w}$ * $\sqrt{\frac{E}{F_{cr1}}}$); h)	=	12.5 in

(Note that b_e should be less than the web height)

A_e=	b_e*t_w+2*b_f*t_f	=	19.1 in^2
Q_a=	$\frac{A_e}{A}$	=	0.967
Q=	Q_s*Q_a	=	0.967

Flexural Buckling Strength

Determine another time whether AISC *Specification* Equation E7-2 or E7-3 applies.

λ_2=	4.71*√(E/(Q*F_y))	=	115
F_{cr2}=	IF($\lambda_{max} \le \lambda_1$; Q*$0.658^{(F_y*Q/F_e)} * F_y$;0.877*$F_e$)	=	28.5 ksi

Nominal Compressive Strength

P_n=	F_{cr2}*A	=	563 kips
Φ_v=			0.90
$\Phi_v P_n$=	Φ_v*P_n	=	507 kips

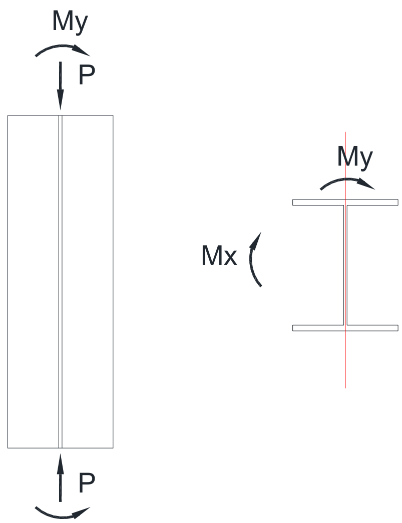
Compressive stress safety (S_s):

S_s=	IF(Φ_v*P_n>P_u;"Safe";"Unsafe")	=	Safe
Stress_ratio=	$\frac{P_u}{\Phi_v * P_n}$	=	0.96

Design Summary

Ultimate load, P_u=	1.2*P_D+1.6*P_L	=	488 kips
$\Phi_v P_n$=	Φ_v*P_n	=	507 kips
Stress_ratio=	$\frac{P_u}{\Phi_v * P_n}$	=	0.96
S_s=	IF(Φ_v*P_n>P_u;"Safe";"Unsafe")	=	Safe

Design of W-Shapes Subjected to Compression Force and Bending Moment



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
Fy=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length and Cb

Unsupported length, L_b= 14.00 ft

kL_{in}= 14.00 ft

kL_{out}= 14.00 ft

(kL_{in} and kL_{out} are strong- and weak/torsional- axes unbraced length, respectively)

From Table 3-1 (AISC), C_b= 1.00

Ultimate Compression Force and Bending Moments

(obtained from a second-order analysis that includes P- δ effects)

P_u= 200.0 kips

M_{ux}= 50.0 kip*ft

M_{uy}= 50.0 kip*ft

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W14X68
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	14.00 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.41 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	10.00 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.72 in
Gross Area, A=	TAB("AISC/W";A;NAME=sec.)	=	20.00 in^2
I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	722.00 in^4
I_y=	TAB("AISC/W";I_y;NAME=sec.)	=	121.00 in^4

(I_x and I_y are the moment of inertia about x-and y-axes, respectively)

Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	115.00 in^3
Elastic sec. modulus, S_x=	TAB("AISC/W";S_x;NAME=sec.)	=	103.00 in^3
Plastic sec. modulus, Z_y=	TAB("AISC/W";Z_y;NAME=sec.)	=	36.90 in^3
Elastic sec. modulus, S_y=	TAB("AISC/W";S_y;NAME=sec.)	=	24.20 in^3
Radius of gyration about x-axis, r_x=	TAB("AISC/W";r_x;NAME=sec.)	=	6.01 in
Radius of gyration about y-axis, r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	2.46 in
Torsional constant, J=	TAB("AISC/W";J;NAME=sec.)	=	3.01 in^4
r_{ts}=	TAB("AISC/W";r_{ts};NAME=sec.)	=	2.80 in
h_o=	TAB("AISC/W";h_o;NAME=sec.)	=	13.30 in

(r_{ts} is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)

AISC Specification Eqn. (F2-1):

Yielding Moment in major axis, M_{px}=	Z_x*F_y*1/12	=	479 kip*ft

AISC Specification Eqn. (F6-1):

Yielding Moment in minor axis, M_{py}=	MIN (Z_y*F_y*1/12;1.6/12*S_y*F_y)	=	154 kip*ft

Element Classification

(1) Web:

h/tw, λ_w=	TAB("AISC/W";h/t_w;NAME=sec.)	=	27.50

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web_Class=	IF($\lambda_w \leq 3.76 * \sqrt{(E/F_y)}$);"Compact"; "Non-Compact")	=	Compact

(2) Comp. flange:

$b_f/2t_f$, λ_f=	TAB("AISC/W";$b_f/2t_f$;NAME=sec.)	=	6.97

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

λ_{pf}= $0.38*\sqrt{\frac{E}{F_y}}$ = 9

λ_{rf}= $1.00*\sqrt{\frac{E}{F_y}}$ = 24

Fl_Class= IF($\lambda_f \le \lambda_{pf}$;"Compact";IF($\lambda_f > \lambda_{rf}$;"Slender";"Non-Compact")) = Compact

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{px1}= M_{px}-0.7*F_y*S_x*1/12 = 179 kip*ft

M_{nx1}= IF(Fl_Class="Compact"; M_{px}; (M_{px}-(M_{px1})*($\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}$))) = 479 kip*ft

M_{py1}= M_{py}-0.7*F_y*S_y*1/12 = 83 kip*ft

M_{ny1}= IF(Fl_Class="Compact"; M_{py}; (M_{py}-(M_{py1})*($\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}$))) = 154 kip*ft

Slenderness Check: (According to Section E2)

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

λ_x= $\frac{kL_{in}}{r_x}*12$ = 28.0

λ_y= $\frac{kL_{out}}{r_y}*12$ = 68.3

Then, the governed slenderness (λ_{max}):

λ_{max}= MAX(λ_x;λ_y) = 68.3

Critical Stresses

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

-Calculate the elastic critical buckling stress, F_e:

F_e= $\frac{\pi^2*E}{\lambda_{max}^2}$ = 61.4 ksi

-Calculate the flexural buckling stress, F_{cr} (Eqns. E3-2 and E3-3):

$\lambda_1=$ $4.71*\sqrt{\frac{E}{F_y}}$ = 113

$F_{cr}=$ $IF(\lambda_{max}\leq\lambda_1;\ 0.658^{(Fy/Fe)}*F_y;0.877*F_e)$ = 35.6 ksi

Design Compressive Strength

(Eqn. E3-1)

$P_n=$ $F_{cr}*A$ = 712.0 kips

$\Phi_c=$ 0.90

$\Phi_c P_n=$ Φ_c*P_n = 640.8 kips

Lateral Torsional Buckling (LTB)

The limiting lengths Lp and Lr are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:

$L_p=$ $1.76*r_y*\sqrt{\frac{E}{F_y}}/12$ = 8.69 ft

$L_{r1}=$ $\sqrt{\frac{J*1.0}{S_x*h_o}}$ = 0.05

$L_{r2}=$ $\sqrt{1+\sqrt{6.76*\left(\frac{0.7*F_y*S_x*h_o}{E*J*1.0}\right)^2}}$ = 1.56

$L_r=$ $1.95/12*r_{ts}*\frac{E}{0.7*F_y}*L_{r1}*L_{r2}$ = 29.41 ft

Case= IF(L_b>L_r;"ELTB";IF($L_b\leq L_p$; "No LTB";"InLTB")) = InLTB

("ELTB" refers to elastic LTB. and "InLTB" refers to Inelastic LTB.)

According to the AISC Spec. Eqn. F2-2:

$M_1=$ $MIN(M_{px};\ C_b*(M_{px}-(M_{px}-0.7*1/12*F_y*S_x)*(L_b-L_p)/(L_r-L_p)))$ = 433 kip*ft

According to the AISC Spec. Eqn. F2-4:

$F_{cr}=$ $C_b*\pi^2*\frac{E}{((L_b+0.01)*12/r_{ts})^2}$ = 79.4 ksi

$F_{cr,mod}=$ $\sqrt{1+0.078*J*\frac{1.0}{S_x*h_o}*\left(L_b*\frac{12}{r_{ts}}\right)^2}$ = 1.3 ksi

According to the AISC Spec. Eqns. F2-3:

$M_2=$ $MIN(M_{px};F_{cr}*S_x/12*F_{cr,mod})$ = 479 kip*ft

According to the AISC Spec. Eqn. F2-2:

M_{nx2}= IF (Case="No L.T.B.";M_{px};IF(Case="InLTB";M_1;M_2)) = 433 kip*ft

Design Flexure Moments in Major/Minor Axes

Φ_b= 0.90

M_{nx}= MIN(M_{px};M_{nx1};M_{nx2}) = 433 kip*ft

M_{ny}= MIN(M_{py};M_{ny1}) = 154 kip*ft

Calculate the Available Flexural and Axial Strengths

$$F_{ca} = \frac{\Phi_c * P_n}{A} = 32.04 \text{ ksi}$$

$$F_{bcx} = 12 * \frac{\Phi_b * M_{nx}}{S_x} = 45.40 \text{ ksi}$$

$$F_{bcy} = 12 * \frac{\Phi_b * M_{ny}}{S_y} = 68.73 \text{ ksi}$$

Calculate the Actual Flexural and Axial Stresses

$$f_{ra} = \frac{P_u}{A} = 10.00 \text{ ksi}$$

$$f_{rbx} = 12 * \frac{M_{ux}}{S_x} = 5.83 \text{ ksi}$$

$$f_{rby} = 12 * \frac{M_{uy}}{S_y} = 24.79 \text{ ksi}$$

Check the Combined Stress Ratio

(AISC Specification Section H2)

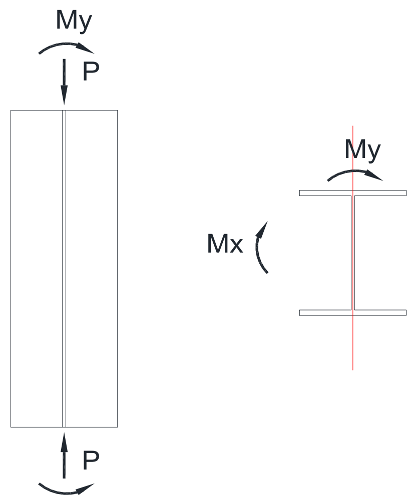
$$\text{Stress\_ratio} = \frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{bcx}} + \frac{f_{rby}}{F_{bcy}} = 0.80$$

Safety= IF(Stress_ratio≤1;"Safe";"Unsafe") = Safe

Design Summary

f_{ra}=	$\dfrac{P_u}{A}$	=	10.0 ksi
F_{ca}=	$\dfrac{\Phi_c * P_n}{A}$	=	32.0 ksi
f_{rbx}=	$12 * \dfrac{M_{ux}}{S_x}$	=	5.8 ksi
F_{bcx}=	$12 * \dfrac{\Phi_b * M_{nx}}{S_x}$	=	45.4 ksi
f_{rby}=	$12 * \dfrac{M_{uy}}{S_y}$	=	24.8 ksi
Fbcy=	$12 * \dfrac{\Phi_b * M_{ny}}{S_y}$	=	68.7 ksi
Stress_ratio=	$\dfrac{f_{ra}}{F_{ca}} + \dfrac{f_{rbx}}{F_{bcx}} + \dfrac{f_{rby}}{F_{bcy}}$	=	0.80
Safety=	IF(Stress_ratio≤1;"Safe";"Unsafe")	=	Safe

Design of W-Shapes Subjected to Axial Compression and Moments including the Second Order Effect



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
E=			29000 ksi

Beam Length and C_b (The member is not subjected to sidesway)

Unsupported length, L_b=	14.00 ft
Strong axis unbraced length, kL_{in}=	14.00 ft
Weak axis and torsional unbraced length, kL_{out}=	14.00 ft
From Table 3-1 (AISC), C_b=	1.14

Given Straining Actions (Not including second-order effects)

P_D=	5.0 kips
P_L=	15.0 kips
M_{xD}=	15.0 kip*ft
M_{xL}=	45.0 kip*ft
M_{yD}=	2.0 kip*ft
M_{yL}=	6.0 kip*ft

From Chapter 2 of ASCE/SEI 7, the required strength (not considering second-order effects) is:

P_u=	$1.2{*}P_D{+}1.6{*}P_L$	=	30.0 kips
M_{ux1}=	$1.2{*}M_{xD}{+}1.6{*}M_{xL}$	=	90.0 kip*ft
M_{uy1}=	$1.2{*}M_{yD}{+}1.6{*}M_{yL}$	=	12.0 kip*ft

Section Details

sec.:	SEL("AISC/W";NAME;)	=	W10X33
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	9.73 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.29 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	7.96 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.44 in
Gross Area, A=	TAB("AISC/W";A;NAME=sec.)	=	9.71 in^2
I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	171.00 in^4
I_y=	TAB("AISC/W";I_y;NAME=sec.)	=	36.60 in^4

(I_x and I_y are the moment of inertia about x-and y-axes, respectively)

Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	38.80 in^3
Elastic sec. modulus, S_x=	TAB("AISC/W";S_x;NAME=sec.)	=	35.00 in^3
Plastic sec. modulus, Z_y=	TAB("AISC/W";Z_y;NAME=sec.)	=	14.00 in^3
Elastic sec. modulus, S_y=	TAB("AISC/W";S_y;NAME=sec.)	=	9.20 in^3
r_x=	TAB("AISC/W";r_x;NAME=sec.)	=	4.19 in
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	1.94 in

(r_x and r_y are the radius of gyration about x- and y- axis, respectively)

Torsional constant, J=	TAB("AISC/W";J;NAME=sec.)	=	0.58 in^4
r_{ts}=	TAB("AISC/W";r_{ts};NAME=sec.)	=	2.20 in
h_o=	TAB("AISC/W";h_o;NAME=sec.)	=	9.30 in

(r_{ts} is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)

AISC Specification Eqn. (F2-1), yielding Moment in major axis (M_{px}):

M_{px}=	Z_x*F_y*1/12	=	162 kip*ft

AISC Specification Eqn. (F6-1), yielding Moment in minor axis (M_{py}):

M_{py}=	MIN (Z_y*F_y*1/12;1.6/12*S_y*F_y)	=	58 kip*ft

Required Flexural Strength (including second-order amplification)

Use the approximate method of second-order analysis procedure from AISC Specification Appendix 8.

Because the member is not subject to sidesway, only P-δ amplifiers need to be added.

$B_1 = \dfrac{C_m}{1 - \alpha * P_r / P_{e1}} \geq 1$ (Spec. Eq. A-8-3)

The x-x axis flexural magnifier is,

$C_{mx}=$ 1.00

$P_{e1}= \dfrac{\pi^2 * E * I_x}{(kL_{in} * 12)^2}$ = 1734 kips

$\alpha=$ 1.00

$B_{1x}= \dfrac{C_{mx}}{1 - \alpha * P_u / P_{e1}}$ = 1.02

$M_{ux}=$ $B_{1x} * M_{ux1}$ = 91.8 kip*ft

The Y-Y axis flexural magnifier is,

$P_{e2}= \dfrac{\pi^2 * E * I_y}{(kL_{out} * 12)^2}$ = 371.2 kips

$C_{my}=$ 1.00

$B_{1y}= \dfrac{C_{my}}{1 - \alpha * P_u / P_{e2}}$ = 1.09

$M_{uy}=$ $B_{1y} * M_{uy1}$ = 13.1 kip*ft

Element Classification

(1) Web:

h/t_w, $\lambda_w=$ TAB("AISC/W";h/t_w;NAME=sec.) = 27.10

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web_Class= IF($\lambda_w \leq 3.76 * \sqrt{(E/F_y)}$;"Compact"; "Non-Compact") = Compact

(2) Comp. flange:

$b_f/2t_f$, $\lambda_f=$ TAB("AISC/W";$b_f/2t_f$;NAME=sec.) = 9.15

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

$\lambda_{pf}=$ $0.38 * \sqrt{(E/F_y)}$ = 9

$\lambda_{rf}=$ $1.00 * \sqrt{(E/F_y)}$ = 24

Fl_Class= IF($\lambda_f \leq \lambda_{pf}$;"Compact";IF($\lambda_f > \lambda_{rf}$;"Slender";"Non-Compact")) = Non-Compact

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{nx1a}= M_{px}-0.7*F_y*S_x*1/12 = 60 kip*ft

M_{nx1}= IF(Fl_Class="Compact"; M_{px}; (M_{px}-M_{nx1a}*$\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)$))) = 161 kip*ft

M_{ny1a}= M_{py}-0.7*F_y*S_y*1/12 = 31 kip*ft

M_{ny1}= IF(Fl_Class="Compact"; M_{py}; (M_{py}-M_{ny1a}*$\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)$))) = 58 kip*ft

Slenderness Check (According to section E2)

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

λ_x= $\frac{kL_{in}}{r_x}*12$ = 40.1

λ_y= $\frac{kL_{out}}{r_y}*12$ = 86.6

Then, the governed slenderness (λ_{max}):

λ_{max}= MAX(λ_x;λ_y) = 86.6

Critical Stresses

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

-Calculate the elastic critical buckling stress, F_e:

F_e= $\frac{\pi^2*E}{\lambda_{max}^2}$ = 38.2 ksi

-Calculate the flexural buckling stress, F_{cr} (Eqns. E3-2 and E3-3):

λ_1= 4.71*$\sqrt{(E/F_y)}$ = 113

F_{cr}= IF($\lambda_{max}\le\lambda_1$; $(0.658)^{\left(\frac{F_y}{F_e}\right)}*F_y$;0.877*$F_e$) = 28.9 ksi

Design Compressive Strength (Eqn. E3-1)

P_n=	F_{cr}*A	=	281 kips
Φ_c=			0.90
$\Phi_c P_n$=	Φ_c*P_n	=	253 kips

Lateral Torsional Buckling (LTB)

The limiting lengths Lp and Lr are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:

L_p=	$1.76*r_y*\sqrt{(E/F_y)}/12$	=	6.85 ft
L_{r1}=	$\sqrt{\frac{J*1.0}{S_x*h_o}}$	=	0.04
L_{r2}=	$\sqrt{1+\sqrt{6.76*\left(\frac{0.7*F_y*S_x*h_o}{E*J*1.0}\right)^2}}$	=	1.66
L_r=	$1.95/12*r_{ts}*\frac{E}{0.7*F_y}*L_{r1}*L_{r2}$	=	19.67 ft
Case=	IF(L_b>L_r;"ELTB";IF(L_b≤L_p; "No LTB";"InLTB"))	=	InLTB

("ELTB" refers to elastic LTB and "InLTB" refers to inelastic LTB)

According to the AISC Spec. Eqn. F2-2:

M_{1a}=	M_{px}-0.7*1/12*F_y*S_x	=	59.9 kip*ft
M_1=	MIN(M_{px}; C_b*(M_{px}- M_{1a}*(L_b-L_p)/(L_r-L_p)))	=	147 kip*ft

According to the AISC Spec. Eqn. F2-4:

F_{cr}=	$C_b*\pi^2*E/((L_b+0.01)*12/r_{ts})^2$	=	55.87 ksi
$F_{cr,mod}$=	$\sqrt{(1+0.078*J*1.0/(S_x*h_o)*(L_b*12/r_{ts})^2)}$	=	1.35

According to the AISC Spec. Eqns. F2-3:

M_2=	MIN(M_{px};F_{cr}*S_x/12*$F_{cr,mod}$)	=	162 kip*ft

According to the AISC Spec. Eqn. F2-2:

M_{nx2}=	IF (Case="No LTB";M_{px};IF(Case="InLTB";M_1;M_2))	=	147.0 kip*ft

Design Flexure Moment in Major/Minor Axes

Φ_b=			0.90
M_{nx}=	MIN(M_{px};M_{nx1};M_{nx2})	=	147.0 kip*ft
M_{ny}=	MIN(M_{py};M_{ny1})	=	58.0 kip*ft

Calculate The Available Flexural and Axial Strengths

P_c=	Φ_c* P_n	=	252.90 kips
M_{cx}=	Φ_b* M_{nx}	=	132.3 kip*ft
M_{cy}=	Φ_b* M_{ny}	=	52.20 kip*ft

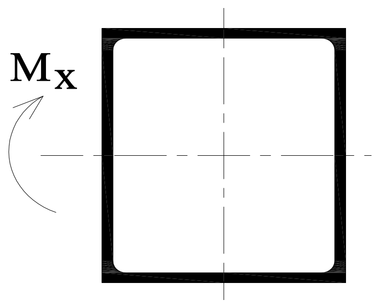
Check The Combined Stress Ratio (AISC Specification Section H1-1a and H1-1b)

Axial ratio, p=	$\frac{P_u}{P_c}$	=	0.12
Moments ratio, m=	$\frac{M_{ux}}{M_{cx}}+\frac{M_{uy}}{M_{cy}}$	=	0.94
Stress_ratio=	IF(p≥0.2; (p+8/9*m);(p/2+m))	=	1.00 in
Safety=	IF(Stress_ratio≤1;"Safe";"Unsafe")	=	Safe

Design Summary

P_c=	Φ_{c*} P_n	=	252.9 kips
M_{cx}=	Φ_b* M_{nx}	=	132.3 kip*ft
M_{cy}=	Φ_b* M_{ny}	=	52.2 kip*ft
Stress_ratio=	IF(p≥0.2; (p+8/9*m);(p/2+m))	=	1.00
Safety=	IF(Stress_ratio≤1;"Safe";"Unsafe")	=	Safe

Design of HSS-Shapes Subjected to Moment about Strong Axis



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A500
Fy=	TAB("Material/ASTM";F_y;NAME=Grade)	=	46 ksi
E=			29000 ksi

Beam Length and C_b

Total length, L=	50.00 ft

Design Moments and Uniform Live Load

Ultimate moment, M_u=	150.00 kip*ft
Ultimate moment due to live load case, M_L=	80.00 kip*ft
Ultimate Shear force, Q_u=	66 kips

Section Details

sec.:	SEL("AISC/HSS";NAME;)	=	HSS18X6X3/8
depth, H_t=	TAB("AISC/HSS";H_t;NAME=sec.)	=	18.00 in
HSS th., t_{des}=	TAB("AISC/HSS";t_{des};NAME=sec.)	=	0.349 in
HSS width, b=	TAB("AISC/HSS";B;NAME=sec.)	=	6.00 in
Plastic sec. modulus, Z_x=	TAB("AISC/HSS";Z_x;NAME=sec.)	=	86.40 in^3
Elastic sec. modulus, S_x=	TAB("AISC/HSS";S_x;NAME=sec.)	=	66.90 in^3
Inertia about x-axis, I_x=	TAB("AISC/HSS";I_x;NAME=sec.)	=	602.00 in^4
Yielding Moment, M_p=	Z_x*F_y*1/12	=	331 kip*ft

Element Classification

(1) Web:

h/t_{des}, λ_w=	TAB("AISC/HSS";h/t_{des};NAME=sec.)	=	48.70

Determine the limiting ratio for a compact HSS web in flexure from AISC Specification Table B4.1b Case 19.

λ_{wr}=	$2.42*\sqrt{(E/F_y)}$	=	60.8
Web_Class=	IF($\lambda_w \le \lambda_{wr}$;"Compact"; "Non-Compact")	=	Compact

(2) Comp. flange:

b/t_{des}, λ_f=	TAB("AISC/HSS";b/t_{des};NAME=sec.)	=	14.20

Determine the limiting ratio for a slender HSS flange in flexure from AISC Specification Table B4.1b Case 17.

λ_{fr}=	$1.12*\sqrt{(E/F_y)}$	=	28.1
Fl_Class=	IF($\lambda_f \le \lambda_{fr}$;"Compact";"Non-Compact")	=	Compact
M_{n1a}=	$M_p - F_y * S_x * 1/12$	=	74.55 kip*ft
λ_{fa}=	$3.57*\lambda_f*\sqrt{\frac{F_y}{E}}-4$	=	-1.98
M_{n1b}=	MIN(M_p ; $M_p - M_{n1a} * \lambda_{fa}$)	=	331.0 kip*ft
M_{n1}=	IF(Fl_Class="Compact"; M_p; M_{n1b})	=	331.0 kip*ft

(Note that For HSS with noncompact flanges and compact webs, AISC Specification Section F7.2(b) applies)

Check The Available Flexure Strength

ΦM_n=	0.90*MIN(M_p;M_{n1})	=	298 kip*ft
Safety=	IF($\Phi M_n \ge M_u$;"Safe";"Unsafe")	=	Safe
Moment raio=	$M_u/\Phi M_n$	=	0.50

Check Shear Strength

From AISC Specification Section G5, if the exact radius is unknown, h shall be taken as the corresponding outside dimension minus three times the design thickness.

h=	$H_t - 3*t_{des}$	=	17 in
λ_w=	TAB("AISC/HSS";h/t_{des};NAME=sec.)	=	48.70

For rectangular HSS in shear, use AISC Specification Section G2.1 with Aw = 2ht (per AISC Specification Section G5) and kv = 5.

k_v=			5
λ_{w1}=	$1.1*\sqrt{(k_v*E/F_y)}$	=	62
λ_{w2}=	$1.37*\sqrt{(k_v*E/F_y)}$	=	77
C_{va}=	$1.51*5*E/(F_y*\lambda_w^2)$	=	2.0
C_v=	IF($\lambda_w \le \lambda_{w1}$;1;IF(($\lambda_w > \lambda_{w1}$ AND $\lambda_w \le \lambda_{w2}$);$\lambda_{w1}/\lambda_w$;$C_{va}$))	=	1
A_w=	$2*h*t_{des}$	=	12 in^2

Nominal shear strength (V_n):

V_n=	$0.6*F_y*A_w*C_v$	=	331.20 kips

From AISC Specification Section G1, the available shear strength is:

Φ_v=			0.90
$\Phi_v V_n$=	Φ_v*V_n	=	298.08 kips
Shear_safety=	IF($\Phi_v*V_n>Q_u$;"Safe";"Unsafe")	=	Safe

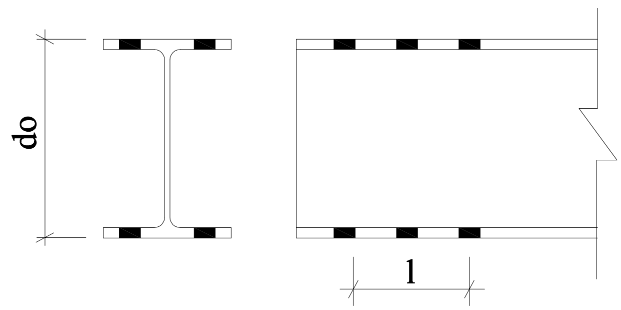
Check Deflection

Δ_{all}=	L/240	=	0.21 ft
W_{eq} (LL), W_L=	$\frac{8*M_L}{L^2}$	=	0.26 kip/ft
Δ_{act}=	$12^2 * \frac{5*W_L*L^4}{384*E*Ix}$	=	0.175 ft
Deflection safety, D_s=	IF($\Delta_{all} \ge \Delta_{act}$;"Safe";"Increase section")	=	Safe

Design Summary

ΦM_n=	$0.90*MIN(M_p;M_{n1})$	=	298 kip*ft
Safety=	IF($\Phi M_n \ge M_u$;"Safe";"Unsafe")	=	Safe
Moment raio=	$M_u/\Phi M_n$	=	0.50
$\Phi_v V_n$=	Φ_v*V_n	=	298 kips
Shear_safety=	IF($\Phi_v*V_n>Q_u$;"Safe";"Unsafe")	=	Safe
Δ_{act}=	$\frac{5*WL*L^4}{384*E*Ix}$	=	0.00 ft
Δ_{all}=	L/240	=	0.21 ft
Deflection safety, D_s=	IF($\Delta_{all} \ge \Delta_{act}$;"Safe";"Increase section")	=	Safe

Design of W-Shapes Subjected to Tension Force in a Bolted Connection



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
F_u=	TAB("Material/ASTM";F_u;NAME=Grade)	=	65 ksi

Buckling Lengths

Member length, L= 25.00 ft

Axial Loads

Axial dead load, P_D=			30 kips
Axial Live load, P_L=			90 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, P_u=	$1.2*P_D+1.6*P_L$	=	180 kips

Section and Connection Details

sec.:	SEL("AISC/W";NAME;)	=	W12X106
depth, d_o=	TAB("AISC/W";d;NAME=sec.)	=	12.90 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.61 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	12.20 in
Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.99 in
Gross Area, A_g=	TAB("AISC/W";A;NAME=sec.)	=	31.20 in^2
r_x=	TAB("AISC/W";r_x;NAME=sec.)	=	5.47 in
r_y=	TAB("AISC/W";r_y;NAME=sec.)	=	3.11 in

(r_x and r_y are the radius of gyration about x- and y- axis)

Rounded depth, d_{WT}=	TAB("AISC/W";d_{WT};NAME=sec.)	=	13.00 in
WT=	TAB("AISC/WT";NAME;d_{ro}=d_{WT}/2)	=	WT6X53
y =	TAB("AISC/WT"; y;NAME=WT)	=	1.190 in
Bolt_diameter:	SEL("AISC/Bolt";Size;)	=	d3/4
d_b=	TAB("AISC/Bolt";dia;Size=Bolt_diameter)	=	0.750 in
hole diameter, d_h=	d_b+1/16	=	0.813 in
Connection length, l =			9.00 in

Check Tensile Yielding

From AISC Manual Table 5-1, the tensile yielding strength is:

Φ_{t1} =			0.90
P_{n1}=	Φ_{t1}* F_y * A_g	=	1404.0 kips
Yield_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe

Check Tensile Rupture

Calculate the shear lag factor, U, as the larger of the values from AISC specification section D3, Table D3.1 case 2 and case 7. From AISC Specification Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$U_1 = \frac{2 * b_f * t_f}{A_g} = 0.774$$

Case 2: Check as two WT-shapes per AISC Specification Commentary Figure C-D3.1

$$U_2 = 1 - \frac{y}{l} = 0.868$$

Case 7:

U_3=	IF($b_f \ge 2/3*d_o$; 0.90; 0.85)	=	0.900
U =	MAX(U_1;U_2;U_3)	=	0.900

Effective Net Area

Calculate A_n using AISC Specification Section B4.3.

$$A_n = A_g - 4 * (d_h + 1/16) * t_f = 27.73\ in^2$$

Calculate A_e using AISC Specification Section D3

$$A_e = A_n * U = 24.96\ in^2$$

Available Tensile Rupture Strength

P_2=	$F_u * A_e$	=	1622.4 kips
Φ_{t2}=			0.75
P_{n2}=	$P_2 * \Phi_{t2}$	=	1216.8 kips
Rupture_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe

Slenderness Check (According to section D1)

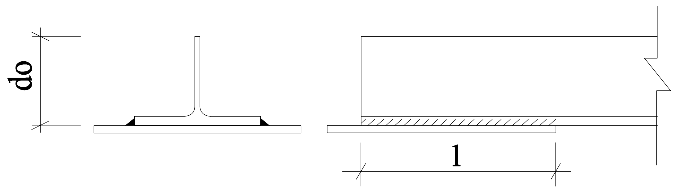
For members designed on the basis of compression, the slenderness ratio KL/r should not exceed 300.

λ_{max}=	$\frac{L}{r_y}*12$	=	96.5
Slenderness_limit=	IF($\lambda_{max} \le 300$; "Safe"; "Unsafe")	=	Safe

Design Summary

Ultimate load, P_u=	$1.2*P_D+1.6*P_L$	=	180.0 kips
P_{n1}=	$\Phi_{t1} * F_y * A_g$	=	1404.0 kips
Yield_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe
P_{n2}=	$P_2 * \Phi_{t2}$	=	1216.8 kips
Rupture_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe
Slenderness_limit=	IF($\lambda_{max} \le 300$; "Safe"; "Unsafe")	=	Safe

Design of WT-Shapes Subjected to Tension Force in Welded Connections



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
F_u=	TAB("Material/ASTM";F_u;NAME=Grade)	=	65 ksi

Buckling Length

Member length, L= 30.00 ft

Axial Loads

Axial dead load, P_D= 40 kips

Axial Live load, P_L= 120 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, P_u= $1.2*P_D+1.6*P_L$ = 240 kips

Section and Connection Details

sec.:	SEL("AISC/WT";NAME;)	=	WT6X20
depth, d_o=	TAB("AISC/WT";d;NAME=sec.)	=	5.97 in
Web th., t_w=	TAB("AISC/WT";t_w;NAME=sec.)	=	0.29 in
Flange width, b_f=	TAB("AISC/WT";b_f;NAME=sec.)	=	8.01 in
Flange th., t_f=	TAB("AISC/WT";t_f;NAME=sec.)	=	0.515 in
Gross Area, A_g=	TAB("AISC/WT";A;NAME=sec.)	=	5.84 in^2
r_x=	TAB("AISC/WT";r_x;NAME=sec.)	=	1.57 in
r_y=	TAB("AISC/WT";r_y;NAME=sec.)	=	1.94 in

(r_x and r_y are the radius of gyration about x- and y- axis)

Distance to centroid, y = TAB("AISC/WT"; y;NAME=sec.) = 1.09 in

Connection length, l = 16.00 in

Check Tensile Yielding

From AISC *Manual* Table 5-1, the tensile yielding strength is:

Φ_{t1} = 0.90

P_{n1}= $\Phi_{t1} * F_y * A_g$ = 262.8 kips

Yield_safety= IF($P_u \le P_{n1}$;"Safe";"Unsafe") = Safe

Check Tensile Rupture

Calculate the shear lag factor, U, as the larger of the values from AISC Specification Section D3, Table D3.1 case 2 and case 7. From AISC Specification Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

U_1= $\frac{b_f * t_f}{A_g}$ = 0.706

Case 2: Check as two WT-shapes per AISC Specification Commentary Figure C-D3.1

U_2= $1 - \frac{y}{l}$ = 0.932

Case 7:

U_3= IF($b_f \ge 2/3 * d_o$; 0.90; 0.85) = 0.900

U = MAX(U_1;U_2;U_3) = 0.932

Effective Net Area

Calculate A_n using AISC Specification Section B4.3.

A_n= A_g = 5.8 in^2

Calculate A_e using AISC Specification Section D3

A_e= $A_n * U$ = 5.4 in^2

Available Tensile Rupture Strength

P_2= $F_u * A_e$ = 351.0 kips

Φ_{t2}= 0.75

P_{n2}= $P_2 * \Phi_{t2}$ = 263.3 kips

Rupture_safety= IF($P_u \le P_{n1}$;"Safe";"Unsafe") = Safe

Slenderness Check (According to section D1)

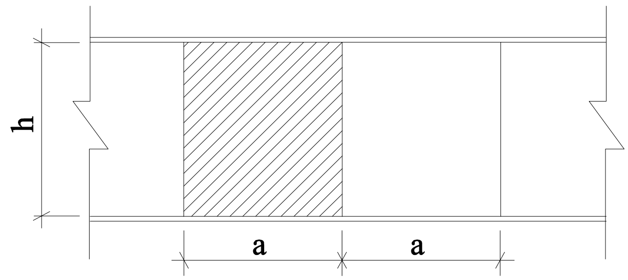
For members designed on the basis of compression, the slenderness ratio KL/r should not exceed 300.

r_{min}=	MIN(r_x; r_y)	=	1.57 in
λ_{max}=	$\frac{L}{r_{min}}$*12	=	229.3
Slenderness_limit=	IF($\lambda_{max} \le 300$; "Safe"; "Unsafe")	=	Safe

Design Summary

Ultimate load, P_u=	1.2*P_D+1.6*P_L	=	240.0 kips
P_{n1}=	Φ_{t1}* F_y * A_g	=	262.8 kips
Yield_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe
P_{n2}=	P_2 * Φ_{t2}	=	263.3 kips
Rupture_safety=	IF($P_u \le P_{n1}$;"Safe";"Unsafe")	=	Safe
Slenderness_limit=	IF($\lambda_{max} \le 300$; "Safe"; "Unsafe")	=	Safe

Design of Interior Panel of Built-Up Girder with Transverse Stiffeners Subjected to Shear Force



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi
E=			29000 ksi

Loads

The required shear strength at the start of this panel from the end , V_u:

V_u= 120 kips

Section Details

Depth, d=	36 in
Web thickness, t_w=	0.3125 in
Clear distance between stiffeners, a=	99 in
Upper flange width, b_{fc}=	12 in
Upper flange thickness, t_{fc}=	1.50 in
Lower flange width, b_{ft}=	12.00 in
Lower flange thickness, t_{ft}=	1.50 in

Shear Strength for this Panel

h=	$d-t_{fc}-t_{ft}$	=	33.00 in
ψ=	a/h	=	3.00
k_{v1}=	$5+(5/\psi^2)$	=	5.56

Based on AISC Specification Section G2.1, k_v=5 when a/h>3 or a/h>$[260/(h/tw)]^2$

λ_w=	h/tw	=	106
ψ_1=	$(260/\lambda_w)^2$	=	6.02
Use k_v=	IF(ψ>3 AND ψ>ψ_1;5;k_{v1})	=	5.56
λ_{w1}=	$1.10*\sqrt{(k_v*E/F_y)}$	=	74
λ_{w2}=	$1.37*\sqrt{(k_v*E/F_y)}$	=	92

Calculate C_v according to Eqns. G2-2, G2-3, G2-4 and G2-5:

C_{v1}= IF($\lambda_w \le \lambda_{w1}$;1;IF($\lambda_w > \lambda_{w1}$ AND $\lambda_w \le \lambda_{w2}$;$\lambda_{w1}/\lambda_w$;1.51*$k_v$*E/($F_y$*$\lambda_w^2$)))= 0.60

Check the additional limits from AISC Specification Section G3.1 for the use of tension field action:

A_w= $d*t_w$ = 11.3 in^2

η= $\dfrac{A_w*2}{b_{fc}*t_{fc}+b_{ft}*t_{ft}}$ = 0.63

ν_{fc}= $\dfrac{h}{b_{fc}}$ = 2.75

ν_{ft}= $\dfrac{h}{b_{ft}}$ = 2.75

Check_TF= IF(ψ>MIN(3;ψ_1) AND η>2.5 AND ν_{fc}>6 AND ν_{ft}>6;"1";"2") = 2

Case= IF(Check_TF="1";"NP";"P") = P

"P": permitted and "NP": not permitted

Calculate C_v and V_n, according to Section G3-2a and b:

C_{v2}= $IF(\lambda_w \le \lambda_{w1};1;(C_{v1}+\dfrac{1-C_{v1}}{1.15*\sqrt{1+\left(\frac{a}{h}\right)^2}}))$ = 0.71

Nominal shear strength:

V_n= IF(Case="PERMITTED";0.6*F_y*A_w*C_{v2};0.6*F_y*A_w*C_{v1}) = 146 kips

Φ_v= 0.90

$\Phi_v V_n$= Φ_v*V_n = 131 kips

Shear_safety= IF(Φ_v*V_n>V_u;"Safe";"Unsafe") = Safe

Force_ratio= Vu/$\Phi_v V_n$ = 0.92

Design Summary

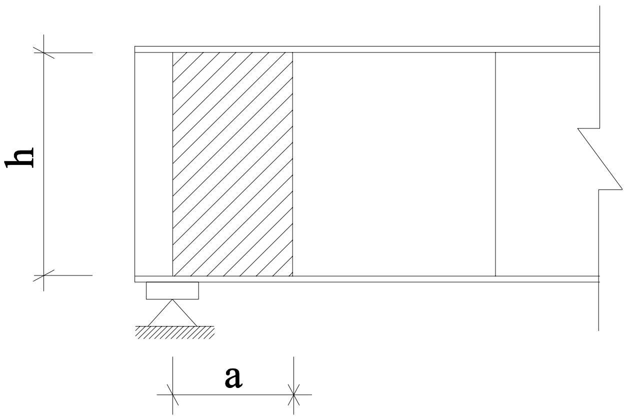
V_u= 120 kips

$\Phi_v V_n$= Φ_v*V_n = 131 kips

Force_ratio= Vu/$\Phi_v V_n$ = 0.92

Shear_safety= IF(Φ_v*V_n>V_u;"Safe";"Unsafe") = Safe

Design of End Panel of Built-Up Girder with Transverse Stiffeners Subjected to Shear Force



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi
E=			29000 ksi

Loads

Reaction at support, R_v= 154.0 kips

Section Details

depth, d=	36 in
Web thickness, t_w=	0.3125 in
clear distance between stiffeners, a=	60.00 in
Upper flange width, b_{fc}=	12 in
Upper flange thickness, t_{fc}=	1.50 in
Lower flange width, b_{ft}=	12.00 in
Lower flange thickness, t_{ft}=	1.50 in

Shear Strength for End Panel

h=	$d-t_{fc}-t_{ft}$	=	33.00 in
ψ=	a/h	=	1.82
k_{v1}=	$5+(5/\psi^2)$	=	6.51

Based on AISC Specification Section G2.1, kv=5 when a/h>3 or a/h>$[260/(h/tw)]^2$

λ_w=	h/tw	=	106
ψ_1=	$(260/\lambda_w)^2$	=	6.02
Therefore, use k_v=	IF(ψ>3 AND ψ>ψ_1;5;k_{v1})	=	6.51

Tension field action is not allowed because the panel is an end panel.

λ_{w1}=	$1.10*\sqrt{(k_v*E/F_y)}$	=	80
λ_{w2}=	$1.37*\sqrt{(k_v*E/F_y)}$	=	99

Calculate C_v according to Eqns. G2-2, G2-3, G2-4 and G2-5:

C_v=	IF($\lambda_w \leq \lambda_{w1}$;1;IF($\lambda_w > \lambda_{w1}$ AND $\lambda_w \leq \lambda_{w2}$;$\lambda_{w1}/\lambda_w$;$1.51*k_v*E/(F_y*\lambda_w^2)$)))	=	0.705
A_w=	$d*t_w$	=	11.3 in^2

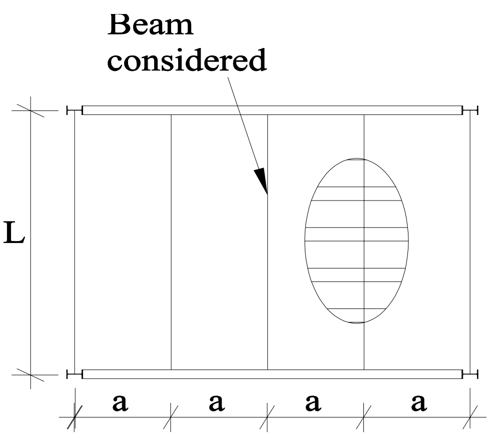
Calculate Vn using Eqn. G2-1:

Nominal shear strength, V_n=	$0.6*F_y*A_w*C_v$	=	172 kips
Φ_v=			0.90
$\Phi_v V_n$=	Φ_v*V_n	=	155 kips
Shear_safety=	IF($\Phi_v*V_n>R_v$;"Safe";"Unsafe")	=	Safe
Force_ratio=	$R_v/\Phi_v V_n$	=	0.99

Design Summary

Reaction at support, R_v=			154.0 kips
$\Phi_v V_n$=	Φ_v*V_n	=	154.8 kips
Force_ratio=	$Rv/\Phi_v V_n$	=	0.99

Design of Composite Beam Subjected to Bending about its Major Axis



Materials

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
F_u=	TAB("Material/ASTM";F_u;NAME=Grade)	=	65 ksi
E_s=			29000 ksi

Beam Length

Total length, L=	45.00 ft
Beam spacing, a=	10.0 ft

Concrete Details

f'_c=	4.0 ksi
Total thickness, t_s=	7.5 in
concrete weight, γ_c=	145.0 lb/ft^3

Loads

Superimposed (HVAC, ceiling, floor covering, etc.), w_{sd}=	10.0 lb/ft^2
Live load for construction (temporary loads during concrete placement), w_{Lc}=	25.0 lb/ft^2
Live load non-reducible, w_{LL}=	100.0 lb/ft^2

Section Details

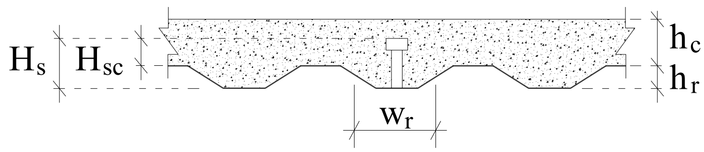
sec.:	SEL("AISC/W";NAME;)	=	W21X50
Weight, w_{sec}=	TAB("AISC/W";W;NAME=sec.)	=	50.00 lb/ft
depth, d=	TAB("AISC/W";d;NAME=sec.)	=	20.80 in
Web th., t_w=	TAB("AISC/W";t_w;NAME=sec.)	=	0.38 in
Flange width, b_f=	TAB("AISC/W";b_f;NAME=sec.)	=	6.53 in

Flange th., t_f=	TAB("AISC/W";t_f;NAME=sec.)	=	0.54 in
Gross area, A_s=	TAB("AISC/W";A;NAME=sec.)	=	14.70 in^2
Plastic sec. modulus, Z_x=	TAB("AISC/W";Z_x;NAME=sec.)	=	110.00 in^3
Inertia about x-axis, I_x=	TAB("AISC/W";I_x;NAME=sec.)	=	984.00 in^4

AISC Specification Eqn. (F2-1):

Yielding Moment, M_p=	Z_x*F_y*1/12	=	458 kip*ft

Metal Deck and Stud Connector Details (in accordance with metal deck manufacturer's data)



Rib height, h_r=	3.0 in
Average rib width, w_r=	6.0 in
weight of slab, w_s=	75.0 lb/ft^2
Diameter of stud, d_{sa}=	0.750 in
Height of stud (minimum is $4d_{sa}$), H_s=	4.50 in
Extension of the stud above the deck (minimum is 1.5 in), H_{sc}=	1.50 in

Composite Deck and Anchor Requirements

Check composite deck and anchor requirements stipulated in AISC Specification Sections I1.3, I3.2c and I8.

- Conc_strength_check =	IF((f'_c ≤ 10 AND f'c ≥ 3) ; "o.k."; "change f'_c")	=	o.k.
- h_r _check=	IF(h_r ≤ 3; "o.k."; "decrease h_r")	=	o.k.
- w_r _check=	IF(w_r ≥2; "o.k."; "increase w_r")	=	o.k.
- d_{sa}_check=	IF(d_{sa} ≤0.75; "o.k."; "decrease d_{sa}")	=	o.k.
- t_f_check=	IF(tf ≥ d_{sa}/2.5; "o.k."; "decrease d_{sa}")	=	o.k.
-H_{sc}_check=	IF((H_s≥ (h_r+1.5) AND H_s < t_s-0.5); "o.k."; "unsafe")	=	o.k.
-H_s_check=	IF(H_s ≥4*d_{sa}; "o.k."; "increase H_s")	=	o.k.
- h_c_check=	IF((t_s-h_r) ≥ 2; "o.k."; "increase h_c")	=	o.k.

Design for Pre-Composite Condition

- Construction Pre-composite Loads:

w_{D1}= $0.001*(w_s*a+w_{sec})$ = 0.80 kip/ft

w_{L1}= $0.001*(w_{Lc}*a)$ = 0.25 kip/ft

- Construction Pre-composite flexural strength, from Chapter 2 of ASCE/SEI 7, the required flexural strength is:

w_{u1}= $1.2*w_{D1}+1.6*w_{L1}$ = 1.36 kip/ft

M_{u1}= $\frac{w_{u1}*L^2}{8}$ = 344 kip*ft

Assume that attachment of the deck perpendicular to the beam provides adequate bracing to the compression flange during construction, thus the beam can develop its full plastic moment capacity. The design flexural strength is determined as follows, from AISC Specification Equation F2-1:

Φ_b= 0.90

M_{n1}= $\Phi_b * M_p$ = 412 kip*ft

Flexural_safety1= IF($M_{n1} \geq M_{u1}$; "Safe"; "Unsafe") = Safe

- Pre-composite deflection:

Δ_{nc}= $\frac{5*\frac{w_{D1}}{12}*(L*12)^4}{384*E_s*I_x}$ = 2.59 in

Δ_{recom}= $\frac{L*12}{360}$ = 1.50 in

If pre-composite deflection exceeds the recommended limit. One possible solution is to increase the member size. A second solution is to induce camber into the member. So, the user in this step has to determine a solution in case of exceeding the recommended limit.

Camber= 2.00 in

deflection_check= IF((Δ_{nc} - Camber)< Δ_{recom}; "Safe"; "Unsafe") = Safe

Design for Composite Condition

- Required Flexural Strength:

w_{D2}= $0.001*((w_s+w_{sd})*a+w_{sec})$ = 0.90 kip/ft

w_{L2}= $0.001*(w_{LL}*a)$ = 1.00 kip/ft

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

w_{u2}= $1.2*w_{D2}+1.6*w_{L2}$ = 2.68 kip/ft

M_{u2}= $\frac{w_{u2}*L^2}{8}$ = 678 kip*ft

Determine The Effective Slab Width, b_e

The effective width of the concrete slab is the sum of the effective widths to each side of the beam centerline as determined by the minimum value of the three widths set forth in AISC Specification Section I3.1a:

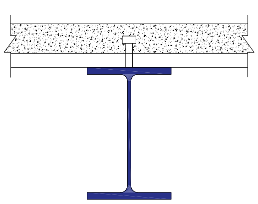
b_{e1}=	$\frac{L}{8}*2$	=	11.25 ft
b_{e2}=	$\frac{a}{2}*2$	=	10.00 ft
b_e=	MIN(b_{e1}; b_{e2})	=	10.00 ft

Available Flexural Strength

According to AISC Specification Section I3.2a, the nominal flexural strength shall be determined from the plastic stress distribution on the composite section when h / tw ≤ √(3.76 E / Fy)

λ_w=	$\frac{d}{t_w}$	=	54.74
Web_Class=	IF($\lambda_w \le 3.76*\sqrt{(E_s/F_y)}$);"Compact"; "Non-Compact")	=	Compact

According to AISC Specification Commentary Section I3.2a, the number and strength of steel headed stud anchors will govern the compressive force, C, for a partially composite beam. The composite percentage is based on the minimum of the limit states of concrete crushing and steel yielding as follows:



- Concrete crushing:

A_c= Area of concrete slab within effective width. Assume that the deck profile is 50% void and 50% concrete fill.

A_c=	$(b_e* 12)*(t_s-h_r) + 0.5 * (b_e*12*h_r)$	=	720.00 in^2
C_c=	$0.85*f'_c *A_c$	=	2448 kips

- Steel yielding:

C_s=	A_s*F_y	=	735 kips

- Shear transfer:

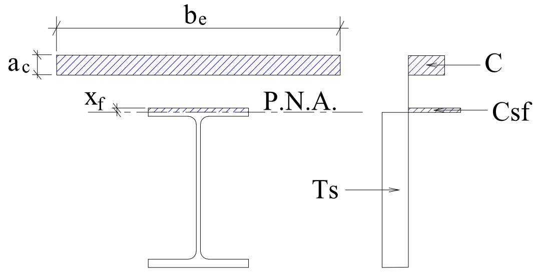
60% is used as a trial percentage of composite action as follows:

$C_1 = \Sigma\, Q_n$

C_1=	0.6 * MIN(C_c;C_s)	=	441 kips

Location of The Plastic Neutral Axis

The plastic neutral axis (PNA) is located by determining the axis above and below which the sum of horizontal forces is equal. This concept assumes the trial PNA location is within the top flange of the beam.



P.N.A. inside flange

Σ F $_{\text{above PNA}}$ = Σ F $_{\text{below PNA}}$

$C_1 + C_{sf1} = T_{s1} * F_y$

$C_{sf1} = x_{f1} * b_f * F_y$

$T_{s1} = (A_s - x_{f1} * b_f) * F_y$

x_{f1}=	$\dfrac{A_s * F_y - C_1}{2 * b_f * F_y}$	=	0.45 in
x_f=	IF($x_{f1} \leq t_f$; x_{f1}; t_f)	=	0.45 in
C_{sf} =	$x_f * b_f * F_y$	=	147 kips
T_s=	$(A_s - x_f * b_f) * F_y$	=	588 kips
C=	$T_s - C_{sf}$	=	441 kips

Check the percentage of partial composite action:

α =	C / MIN(C_c; C_s)	=	0.60
Check_case=	IF($\alpha < 0.50$; "Conservative"; "o.k.")	=	o.k.

Determine the nominal moment resistance of the composite section following the procedure in Specification Commentary Section I3.2a: (calculating the sum. of moments about the P.N.A.)

$M_2 = \Sigma$ F*Y

a_c=	$\dfrac{C}{0.85 * f'_c * b_e * 12}$	=	1.08 in
M_{n2}=	1/12*(C*(t_s+x_f - a_c/2)+C_{sf}*(x_f/2)+T_s*(d/2-x_f))	=	763 kip*ft
Flexural_safety2=	IF($M_{n2} \geq M_{u2}$; "Safe"; "Unsafe")	=	Safe

Composite deflection: (AISC specs. I3.1 and Eqn. C-13-1)

Y_{ENA}=	$(A_s*d/2+(C/F_y)*(d+(t_s-a_c/2)))/(A_s+C/F_y)$	=	16.91 in
I_{Lb}=	$I_x+A_s*(Y_{ENA}-d/2)^2+(C/F_y)*(d+(t_s-a_c)-Y_{ENA})^2$	=	2545 in^3
Δ_c=	$\dfrac{5*\dfrac{w_{L2}}{12}*(L*12)^4}{384*E_s*I_{Lb}}$	=	1.25 in
$\Delta_{\mathbf{recom}}$=	$\dfrac{L*12}{360}$	=	1.50 in
Deflection_check2=	IF($\Delta_c<\Delta_{recom}$; "Safe"; "Unsafe")	=	Safe

Summary

Pre-composite condition:

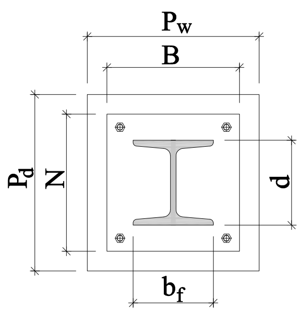
M_{u1}=	$\dfrac{w_{u1}*L^2}{8}$	=	344 kip*ft
M_{n1}=	Φ_b*M_p	=	412 kip*ft
Flexural_safety1=	IF($M_{n1}\geq M_{u1}$; "Safe"; "Unsafe")	=	Safe
Δ_{nc}=	$\dfrac{5*\dfrac{w_{D1}}{12}*(L*12)^4}{384*E_s*I_x}$	=	2.59 in
$\Delta_{\mathbf{recom}}$=	$\dfrac{L*12}{360}$	=	1.50 in
Deflection_check=	IF((Δ_{nc} - Camber)< Δ_{recom}; "Safe"; "Unsafe")	=	Safe

Composite condition:

M_{u2}=	$\dfrac{w_{u2}*L^2}{8}$	=	678 kip*ft
M_{n2}=	$1/12*(C*(t_s+x_f-a_c/2)+C_{sf}*(x_f/2)+T_s*(d/2-x_f))$	=	763 kip*ft
Flexural_safety2=	IF($M_{n2}\geq M_{u2}$; "Safe"; "Unsafe")	=	Safe
Δ_{nc}=	$\dfrac{5*\dfrac{w_{D1}}{12}*(L*12)^4}{384*E_s*I_x}$	=	2.59 in
Deflection_check=	IF((Δ_{nc} - Camber)< Δ_{recom}; "Safe"; "Unsafe")	=	Safe

Chapter 4: Connection Design

Design of Base Plate Bearing on Concrete Subjected to Concentric Loading



Loads

Dead Load, P_D=			115.0 kips
Live Load, P_L=			345.0 kips

Material Properties

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
Yield stress, f_{yp}=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi

Column and Pedestal Data

Concrete strength for pedestal (f'_c):

f'_c=			3 ksi
Sec.:	SEL("AISC/W"; NAME;)	=	W12X96
Pedestal depth, P_d=			24 in
Pedestal width, P_w=			24 in
Depth of column, d=	TAB("AISC/W";d; NAME=Sec.)	=	12.7 in
Flange of column, b_f=	TAB("AISC/W";b_f;NAME=Sec.)	=	12.2 in

The Required Strength

(Chapter 2 of ASCE/SEI 7)

P_u=	$1.2*P_D+1.6*P_L$	=	690 kips

Preliminary Base Plate Dimensions

Φ_c=			0.65
A_{1req}=	$\dfrac{P_u}{\Phi_c * 0.85 * f'_c}$	=	416 in^2
N1=	d+2*3	=	18.7 in
N=	MAX(N1;√(A_{1req})+0.5*(0.95*d-0.8*b_f))	=	22 in

B1=	b_f+2*3	=	18.2 in
B=	IF((d-b_f)<1;MAX(B1;A_{1req}/N;N);MAX(B1;A_{1req}/N))	=	22 in

Concrete Bearing Strength

Pedestal area, A_2=	P_d* P_w	=	576 in^2
Base plate area, A_1=	N*B	=	484 in^2
Check plate area=	IF(A_1>A_{1req};"o.k.";"Unsafe")	=	o.k.
P_b=	MIN(0.85*f'c*A_1*√(A_2/A_1) ; 1.7*f'c*A_1)	=	1346 kips
Concrete bearing strength ($\Phi_c P_p$):			
$\Phi_c P_p$=	Φ_c*P_b	=	875 kips
Check safety, check=	IF($\Phi_c P_p$>P_u;"o.k.";"Unsafe")	=	o.k.

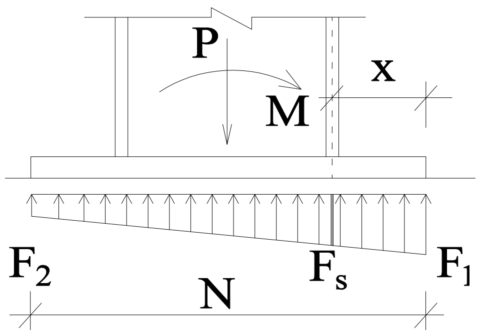
Base Plate Thickness

m=	$\frac{N-0.95*d}{2}$	=	4.97 in
n=	$\frac{B-0.8*b_f}{2}$	=	6.12 in
n'=	$\frac{\sqrt{d*b_f}}{4}$	=	3.11 in
X=	$\left(4*d*\frac{b_f}{(d+b_f)^2}\right)*\left(\frac{P_u}{\Phi_c*P_b}\right)$	=	0.79
λ=	MIN(2*√(X)/(1+√(1-X)) ; 1.00)	=	1.00
l=	MAX(m ; n; λ*n')	=	6.12 in
f_{pu}=	$\frac{P_u}{N*B}$	=	1.43 ksi
t_{min}=	$l*\sqrt{\frac{2*f_{pu}}{0.9*f_{yp}}}$	=	1.82 in
t=	TAB("Material/plate_th";t_fr;t_in>tmin)	=	2.00 in

Summary: Use Plate with The Following Dimensions

Plate length=	N	=	22 in
Plate width=	B	=	22 in
Plate thickness=	t	=	2.00 in

Design of Base Plate Bearing on Concrete Subjected to Small Eccentricity, e ≤ N/6



Loads

Dead Load, P_D=			50 kips
Live Load, P_L=			90 kips
Moment from D.L., M_D=			100 kip*in
Moment from L.L., M_L=			180 kip*in

Base Plate Material Properties

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
Yield stress, f_{yp}=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi

Column, Base Plate and Pedestal Dimensions

Concrete strength for pedestal (f'_c):

f'_c=			3 ksi
Sec.:	SEL("AISC/W"; NAME;)	=	W10X112
Pedestal depth, P_d=			17 in
Pedestal width, P_w=			14 in
Base plate depth, N=			17 in
Base plate width, B=			14 in
Depth of column, d=	TAB("AISC/W";d; NAME=Sec.)	=	11.4 in
Flange of column, b_f=	TAB("AISC/W";b_f;NAME=Sec.)	=	10.4 in

Check Eccentricity Size

M_t=	M_D+M_L	=	280 kip*in
P_t=	P_D+P_L	=	140 kips
e=	M_t/P_t	=	2.00 in
Check_e=	IF(e≤ N/6;"O.K.";"not O.K.")	=	O.K.

(Note that: if this check was not O.K., this template will give a wrong solution)

The Ultimate Load and Moment

(Chapter 2 of ASCE/SEI 7)

P_u=	$1.2*P_D+1.6*P_L$	=	204 kips
M_u=	$1.2*M_D+1.6*M_L$	=	408 kip*in

The Maximum Bearing Stress, F_b

Φ_c=			0.60
A_1=	N*B	=	238.00 in^2
A_2=	P_d*P_w	=	238.00 in^2
F_{b1}=	$0.85*\Phi_c*f'_c*\sqrt{(A_2/A_1)}$	=	1.53 ksi
F_{b2}=	$1.7*f'_c$	=	5.10 ksi
F_b=	$MIN(F_{b1};F_{b2})$	=	1.53 ksi
F_1=	IF(Check_e="O.K."; $\frac{P_u}{A_1}+\frac{M_u*N/2}{(B*N^3)/12}$; " ")	=	1.46 ksi
F_2=	$\frac{P_u}{A_1}-\frac{M_u*\frac{N}{2}}{\frac{B*N^3}{12}}$	=	0.25 ksi
Check_F=	IF(F_1<F_b;"Safe";"Unsafe")	=	Safe

Base Plate Thickness:

The critical section is at distance (x) from the plate edge, where:

x=	$\frac{N-0.95*d}{2}$	=	3.09 in

The distance from the base center to this section (x_1) and the stress at this section (F_s) can be calculated as:

x_1=	$\frac{N}{2}-x$	=	5.41 in
F_s=	IF(Check_e="O.K."; $\frac{P_u}{A_1}+\frac{M_u*x_1}{(B*N^3)/12}$; " ")	=	1.24 ksi

The factored moment for a 1-in strip at this section (M_s) can be calculated as follows:

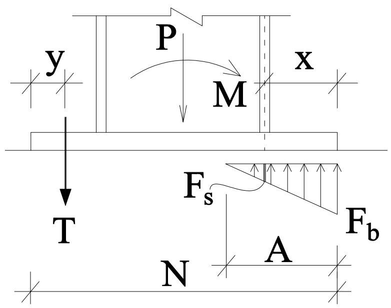
$$M_s = \frac{F_s * x^2}{2} + (F_1 - F_s) * \frac{x^2 * 0.67}{2} = 6.62 \text{ kip*in}$$

$$t_{p,min} = \sqrt{\frac{4 * M_s}{0.9 * f_{yp}}} = 0.90 \text{ in}$$

Summary: Use Plate with The Following Minimum Dimensions

Plate length=	N	=	17 in
Plate width=	B	=	14 in
Plate thickness=	$t_{p,min}$	=	0.90 in

Design of Base Plate Bearing on Concrete Subjected to Large Eccentricity, $e \geq N/2$



Loads

Dead Load, P_D=			21 kips
Live Load, P_L=			39 kips
Moment from D.L., M_D=			171 kip*in
Moment from L.L., M_L=			309 kip*in

Base Plate Material Properties

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
Yield stress, f_{yp}=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi

Column, Base Plate and Pedestal Dimensions

Concrete strength for pedestal (f'_c):

f'_c=			3 ksi
Sec.:	SEL("AISC/W"; NAME;)	=	W8X35
Pedestal depth, P_d=			28 in
Pedestal width, P_w=			28 in
Base plate depth, N=			14 in
Base plate width, B=			14 in
Distance to bolt, y=			1.50 in
Depth of column, d=	TAB("AISC/W";d; NAME=Sec.)	=	8.1 in
Flange of column, b_f=	TAB("AISC/W";b_f;NAME=Sec.)	=	8.0 in

Check Eccentricity Size

M_t=	M_D+M_L	=	480 kip*in
P_t=	P_D+P_L	=	60 kips
e=	M_t/P_t	=	8.00 in
Check_e=	IF(e> N/2;"O.K.";"not O.K.")	=	O.K.

(Note that: if this check was not O.K., this template will give a wrong solution)

The Ultimate Load and Moment

(Chapter 2 of ASCE/SEI 7)

P_u=	$1.2*P_D+1.6*P_L$	=	88 kips
M_u=	$1.2*M_D+1.6*M_L$	=	700 kip*in

The Maximum Bearing Stress, F_b

Φ_c=			0.60
A_1=	N*B	=	196.00 in^2
A_2=	P_d*P_w	=	784.00 in^2
F_{b1}=	$0.85*\Phi_c*f'_c*\sqrt{(A_2/A_1)}$	=	3.06 ksi
F_{b2}=	$1.7*f'_c$	=	5.10 ksi
F_b=	$MIN(F_{b1};F_{b2})$	=	3.06 ksi
N'=	N-y	=	12.50 in
F'=	$\frac{F_b*B*N'}{2}$	=	267.8 ksi

Distance from the base to bolt centers (A') is:

A'=	$\frac{N}{2}-y$	=	5.50 in
M_1=	$P_u*A'+M_u$	=	1184 kip*in
A_1=	$\dfrac{F'+\sqrt{(F')^2-4*\frac{F_b*B}{6}*M_1}}{\frac{F_b*B}{3}}$	=	32.4 in
A_2=	$\dfrac{F'-\sqrt{(F')^2-4*\frac{F_b*B}{6}*M1}}{\frac{F_b*B}{3}}$	=	5.1 in
A=	$MIN(A_1 ; A_2)$	=	5.1 in
Check_A=	IF(A<N';"O.K.";"not O.K.")	=	O.K.

(If this check was not O.K., increase plate dimensions to find a solution or check inputs)

Tension in Bolts

T=	$\frac{F_b*A*B}{2}-P_u$	=	21.2 kips

Base Plate Thickness

The critical section is at distance (x) from the plate edge, where:

$$x = \frac{N - 0.95 * d}{2} = 3.15 \text{ in}$$

The distance from the zero stress point to this section (x_1) and the stress at this section (F_s) can be calculated as:

$x_1 =$ $A - x$ = 1.95 in

$F_s =$ $x_1/A*F_b$ = 1.17 ksi

The factored moment for a 1-in strip at this section (M_{s1}) can be determined from the bearing stress distribution as follows:

$$M_{s1} = \frac{F_s * x^2}{2} + (F_b - F_s)* \frac{x^2 * 0.67}{2} = 12.09 \text{ kip*in}$$

Also, The moment based on the critical section on the anchor bolt side is determined as follows (it is assumed that the critical plate width is based on the load spreading out at 45 degrees:

$M_{s2} =$ $T/2*(x-y)/(2*(x-y))$ = 5.30 kip*in

So, the critical moment is the maximum from M_{s1} and M_{s2} as follows:

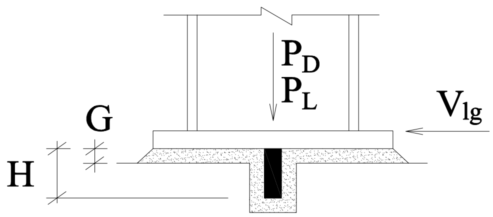
$M_s =$ $MAX(M_{s1};M_{s2})$ = 12.09 kip*in

$$t_{p,min} = \sqrt{\frac{4 * M_s}{0.9 * f_{yp}}} = 1.22 \text{ in}$$

Summary: Use Plate with The following Minimum Dimensions

Plate length=	N	=	14 in
Plate width=	B	=	14 in
Plate thickness=	$t_{p,min}$	=	1.22 in

Design of a Shear Lug for Base Plates Subjected to Axial and Shear Loads



Loads

Dead Load, P_D=			120 kips
Live Load, P_L=			150 kips
Shear Load, V_w=			55 kips

Base Plate Material Properties

Grade:	SEL("Material/ASTM"; NAME;)	=	A36
Yield stress, f_{yp}=	TAB("Material/ASTM";F_y;NAME=Grade)	=	36 ksi

Column, Base Plate and Pedestal Dimensions

Concrete strength for pedestal (f'_c):

f'_c=	3 ksi
Base plate depth, N=	14 in
Base plate width, B=	14 in
The coefficient of friction, μ=	0.55
Shear lug width, W=	8 in
Grout depth, G=	1.0 in

The Portion of The Shear which can be Transferred by Friction Equal to μ:

V_{lgu}= $1.3*V_w - \mu*(0.9*P_D)$ = 12.1 kips

The Required Bearing Area

Φ_c= 0.60

A_{lgu}= $\dfrac{V_{lgu}}{0.85 * \Phi_c * f'_c}$ = 7.9 in^2

The Height of The Bearing Portion

H= A_{lgu}/W = 0.99 in

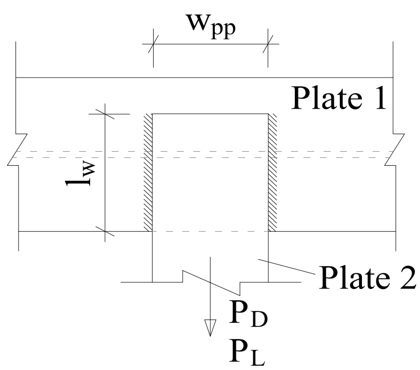
Base Plate Thickness

M_{lgu}=	$\frac{V_{lgu}}{W} * \frac{H+G}{2}$	=	1.5 kip*in
t_{lg}=	$\sqrt{\frac{4 * M_{lgu}}{0.9 * f_{yp}}}$	=	0.43 in

Summary: Use Shear Lug with the Following Minimum Dimensions

Depth=	H+G	=	2 in
Width=	W	=	8 in
Thickness=	t_{lg}	=	0.43 in

Design of Fillet Weld Subjected to Longitudinal Shear Force



Details of The Connected Plates

Grade:	SEL("Material/ASTM"; NAME;)	=	A992
F_y=	TAB("Material/ASTM";F_y;NAME=Grade)	=	50 ksi
Thickness of the first plate, t_{p1}=			0.2500 in
Thickness of the second plate, t_{p2}=			0.3750 in
Width of the perpendicular part, w_{pp}=			18.0 in

Loads

Force:	SEL ("AISC/force"; type;)	=	Tension
Dead load, P_D=			33.0 kips
Live load, P_L=			100.0 kips

From Chapter 2 of ASCE/SEI 7, the required strength is:

P_u=	$1.2*P_D+1.6*P_L$	=	199.6 kips

Preliminary Welding Details

Electrode classification number, F_{EXX}=	70 ksi
Length of weld on each side, l_w=	27.00 in
Thickness of weld, t_w=	0.1875 in

Design of Weld

- Check the maximum and minimum Weld Size (AISC Specification Section J2.2b)

Minimum thickness of the connected parts (t_{pmin}):

t_{pmin}=	MIN(t_{p1}; t_{p2})	=	0.2500 in
$t_{w,max}$=	IF(t_{pmin}<1/4; t_{pmin}; (t_{pmin}-1/16))	=	0.1875 in
$t_{w,min1}$=	IF(t_{pmin}≤1/4; 1/8; IF((t_{pmin}>1/4 AND t_{pmin}≤1/2); 3/16; 0))	=	0.1250 in
$t_{w,min2}$=	IF((t_{pmin}>1/2 AND t_{pmin}≤3/4); 1/4; 5/16)	=	0.3125 in
$t_{w,min}$=	IF($t_{w,min1}$=0;$t_{w,min2}$; MIN($t_{w,min1}$;$t_{w,min2}$))	=	0.1250 in
Check_1=	IF((t_w ≥$t_{w,min}$ AND tw ≤ $t_{w,max}$);"O.K."; "Increase t_w")	=	O.K.

- *Minimum required length:*

$$l_{w,min} = \frac{P_u}{0.60 * F_{EXX} * t_w / \sqrt{2} * 0.75 * 2} = 23.9 \text{ in}$$

- *Check length for perpendicular plate width:*

Check1=	IF(F="C"; "O.K."; IF($l_{w,min}$≥w_{pp}; "O.K."; "increase l_w"))	=	O.K.

- *Calculate the effective weld length:*

λ_w=	l_w/t_w	=	144.0 in
β_w=	MIN(1.2-0.002*λ_w;1)	=	0.91 in
$l_{w,eff}$=	β_w* l_w	=	24.57 in

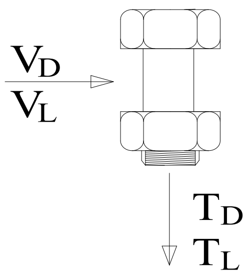
- *Recheck the weld at its reduced strength:*

ΦR_n=	0.75 * 2 * $l_{w,eff}$ * t_w /√(2) * 0.6*F_{EXX}	=	205.2 kips
Check_2=	IF(ΦR_n>P_u;"Safe"; "Unsafe")	=	Safe

Design Summary

Electrode=	F_{EXX}	=	70.0 ksi
Size=	t_w	=	0.1875 in
length=	l_w	=	27.00 in
P_u=	1.2*P_D+1.6*P_L	=	199.6 kips
ΦR_n=	0.75 * 2 * $l_{w,eff}$ * t_w /√(2) * 0.6*F_{EXX}	=	205.2 kips
Check_1=	IF((t_w ≥$t_{w,min}$ AND t_w ≤ $t_{w,max}$);"O.K."; "Increase t_w")	=	O.K.
Check_2=	IF(ΦR_n>P_u;"Safe"; "Increase weld size/length")	=	Safe

Design of Bolts in Bearing Type Connection Subjected to Combined Tension and Shear Forces



Details of Bolt

Grade:	SEL("AISC/ASTM_bolts"; Name;)	=	A325
F_u=	TAB("AISC/ASTM_bolts";F_u;Name=Grade)	=	120 ksi
Bolt:	SEL("AISC/bolt"; Size;)	=	d3/4
Bolt diameter, d_b=	TAB("AISC/bolt";dia;Size=Bolt)	=	0.7500 in
Area of bolt, A_b=	TAB("AISC/bolt";Area;Size=Bolt)	=	0.4418 in^4

Loads

Dead:

Tension force, T_D=			3.50 kips
Shear force, V_D=			1.3 kips

Live:

Tension force, T_L=			12.0 kips
Shear force, V_L=			4.0 kips

From Chapter 2 of ASCE/SEI 7, the required strength is:

T_u=	$1.2*T_D+1.6*T_L$	=	23.4 kips
V_u=	$1.2*V_D+1.6*V_L$	=	8.0 kips

Check for Shear

Φ= 0.75

From AISC Specification Table J3.2,

The available shear strength (F_{nv}):

F_{nv}=	$0.40*F_u$	=	48.0 ksi

The available shear stress (f_{rv}):

f_{rv}=	V_u / A_b	=	18.1 ksi
Check_Shear=	IF(Φ*F_{nv}≥f_{rv};"Safe";"Increase d_b")	=	Safe

Check for Tension Accompanied by Shear

From AISC Specification Table J3.2,

The available tensile strength (F_{nt}):

F_{nt}=	$0.75*F_u$	=	90.0 ksi

The available tensile strength of a bolt subject to combined tension and shear is as follows, from AISC Spec. Eqn. J3.3a,

F_{nt}' =	$1.30*F_{nt} - \frac{F_{nt}}{\Phi*F_{nv}}*f_{rv}$	=	71.8 ksi

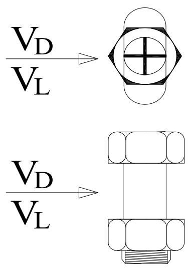
The available tension force (R_n):

R_n=	A_b * MIN(F_{nt}' ; F_{nt})	=	31.7 kips
Check_Tension=	IF($\Phi*R_n \geq T_u$;"Safe";"Increase d_b")	=	Safe

Design Summary

Size=	d_b	=	0.7500 in
F_u=	TAB("AISC/ASTM_bolts";F_u;Name=Grade)	=	120 ksi
f_{rv}=	V_u / A_b	=	18.1 ksi
F_{nv}=	$0.40*F_u$	=	48.0 ksi
Check_Shear=	IF($\Phi*F_{nv} \geq f_{rv}$;"Safe";"Increase d_b")	=	Safe
T_u=	$1.2*T_D+1.6*T_L$	=	23.4 kips
R_n=	A_b * MIN(F_{nt}' ; F_{nt})	=	31.7 kips
Check_Tension=	IF($\Phi*R_n \geq T_u$;"Safe";"Increase d_b")	=	Safe

Design of Slip Critical Connection with Short-Slotted Holes Subjected to Shear Force



Details of Bolt (short slots transverse to the load)

Grade:	SEL("AISC/ASTM_bolts"; Name;)	=	A325
F_u=	TAB("AISC/ASTM_bolts";F_u;Name=Grade)	=	120 ksi
Bolt:	SEL("AISC/J3.1"; Size;)	=	d3/4 A325
Bolt diameter, d_b=	TAB("AISC/J3.1";dia;Size=Bolt)	=	0.7500 in
Area of bolt, A_b=	TAB("AISC/J3.1";Area;Size=Bolt)	=	0.4418 in^4
Class:	SEL("AISC/Class";class;)	=	Class_A
T_b=	TAB("AISC/J3.1";T_b;Size=Bolt)	=	28 kips
μ=	TAB("AISC/Class2"; mio; cat=Class_sym)	=	0.35

(T_b is the minimum bolt pretension and μ is the mean slip coefficient)

D_u=	1.13
Number of slip planes, n_s=	2
hole factor, h_f=	1.00

Loads

Shear force due to dead load, V_D=	3.00 kips
Shear force due to live load, V_L=	7.00 kips

From Chapter 2 of ASCE/SEI 7, the required strength is:

V_u=	$1.2*V_D+1.6*V_L$	=	14.8 kips

Check for Slip Resistance

Φ=			1.00
The design slip resistance, ΦR_n=	$\Phi * \mu * D_u * h_f * T_b * n_s$	=	22.15 kips
Check_slip=	IF($\Phi R_n \geq V_u$;"Safe"; "Unsafe")	=	Safe

Check The Limit State of Bolt Shear

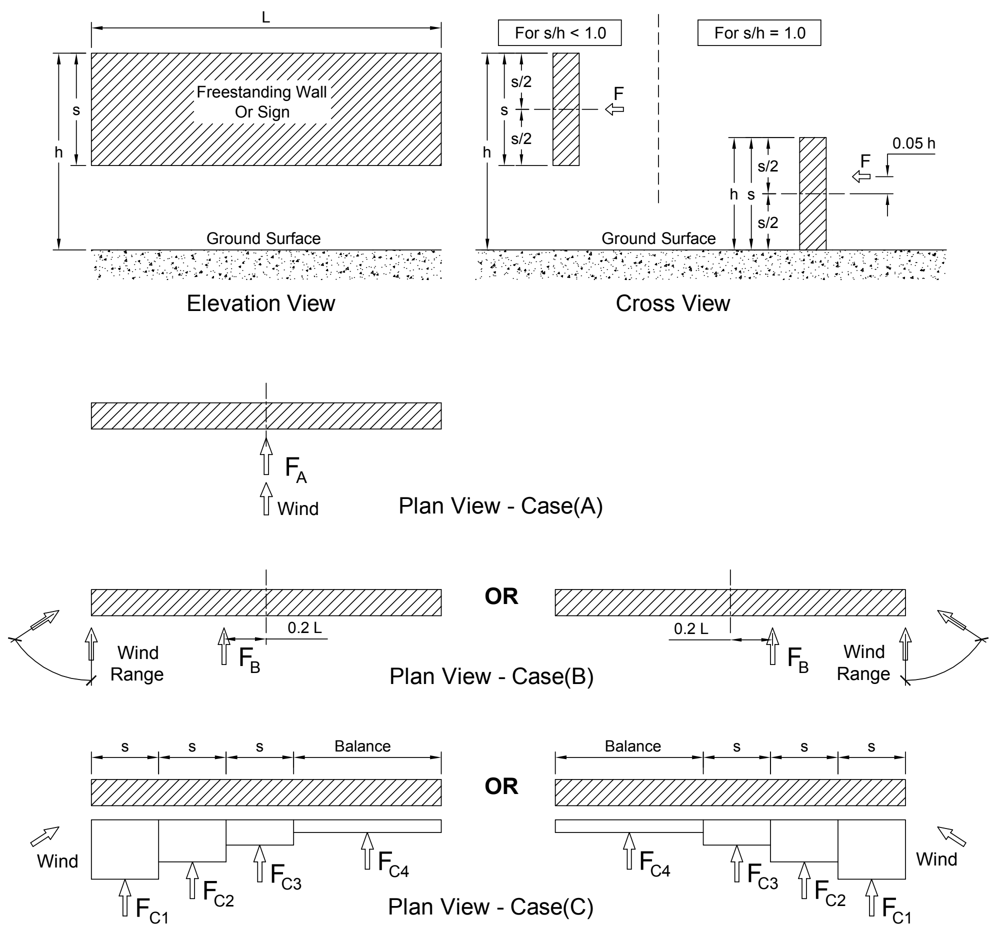
Φ_2=			0.75
From AISC Specification Table J3.2,			
The available shear strength, F_{nv}=	0.40*F_u	=	48.0 ksi
The actual shear stress, f_{rv}=	V_u / (A_b)	=	33.5 ksi
Check_Shear=	IF(Φ_2*F_{nv}≥f_{rv};"Safe";"Increase d_b")	=	Safe

Design Summary

Size=	d_b	=	0.7500 in
F_u=	TAB("AISC/ASTM_bolts";F_u;Name=Grade)	=	120 ksi
V_u=	1.2*V_D+1.6*V_L	=	14.8 kips
The design slip resistance, ΦR_n=	Φ * μ * D_u * h_f * T_b * n_s	=	22.15 kips
Check_slip=	IF(ΦR_n ≥ V_u;"Safe"; "Unsafe")	=	Safe
The available shear stress, f_{rv}=	V_u / A_b	=	33.5 ksi
The available shear strength, F_{nv}=	0.40*F_u	=	48.00 ksi
Check_Shear=	IF(Φ_2*F_{nv}≥f_{rv};"Safe";"Increase d_b")	=	Safe

Chapter 5: Design Loads

Calculation of Wind load for Solid Freestanding Walls & Signs as per ASCE/SEI 7-10 Chapters 26 & 29



Dimensions of Solid Freestanding Walls or Solid Freestanding Signs

Horizontal Dimension of Wall or Sign, L=	75.0 ft
Mean top height of Wall or Sign, h=	10.0 ft
Vertical Dimension of Wall or Sign, s=	10.0 ft

Wind load Parameters

Basic wind speed, V=		105.00 mph
Wind Directionality Factor (According to Table 26.6-1 of ASCE/SEI 7),		
K_d=		0.85
The Exposure Category (According to Cl. 26.7.3 of ASCE/SEI 7):		
Category: SEL("ASCE/Exp"; Category;)	=	B
Gust-Effect Factor (According to Cl. 26.9.1 of ASCE/SEI 7), G=		0.85
Topographic Factor (According to Cl. 26.8.2 of ASCE/SEI 7), K_{zt}=		1.00

Velocity Pressure

According to (Table 26.9-1 of ASCE/SEI 7),

α= TAB("ASCE/Exp"; Alph; Category=Category) = 7.00

According to (Table 26.9-1 of ASCE/SEI 7),

z_g= TAB("ASCE/Exp"; zg; Category=Category) = 1200.00

The Velocity Pressure Exposure Coefficient (According to Table 29.3-1 of ASCE/SEI 7):

K_h= $IF(h<15; 2.01 * (15/z_g)^{(2/\alpha)}; 2.01*(h/z_g)^{(2/\alpha)})$ = 0.57

Velocity Pressure (According to Cl. 29.3.2 of ASCE/SEI 7),

q_h= $0.00256 * K_h * K_{zt} * K_d * V^2$ = 13.67 psf

The gross area of the solid freestanding wall or sign, A_s= L * s = 750 ft²

Ratio of solid area to gross area, ε = 1.00

Reduction Factor for Openings, RF= $1 - (1 - \varepsilon)^{1.5}$ = 1.00

Wind Forces - Case A

Force Coefficient (According to Fig. 29.4-1 of ASCE/SEI 7), C_{fA}= 1.33

Design wind force (According to Cl. 29.4.1 of ASCE/SEI 7),

F_A= $q_h * G * C_{fA} * A_s * RF / 1000$ = 11.6 kips

Wind Forces - Case B

Force Coefficient (According to Fig. 29.4-1 of ASCE/SEI 7), C_{fB}= 1.33

Design wind force (According to Cl. 29.4.1 of ASCE/SEI 7),

F_B= $q_h \times G \times C_{fB} \times A_s \times RF / 1000$ = 11.6 kips

Wind Forces - Case C

Region-1 from (0 to s or less)

Force Coefficient for Region-1 (According to Fig. 29.4-1 of ASCE/SEI 7),

C_{fC1}= 3.48

Effective Area for Region-1, A_{sC1}= IF(L/s>1;s*s;L*s;) = 100 ft²

Design Wind Force for Region-1 (According to Cl. 29.4.1 of ASCE/SEI 7).

F_{C1}= RF/1000*MAX(16;q_h*G*C_{fC1}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC1} = 3.2 kips

Region-2 from (s to 2s) Validation: IF(L/s>1;"Valid";"Invalid";) = Valid

Force Coefficient for Region-2 (According to Fig. 29.4-1 of ASCE/SEI 7),

C_{fC2}= 2.28

Effective Area for Region-2, A_{sC2}= IF(L/s>2;s*s;IF(L/s<1;0;(L-s)*s;);) = 100 ft²

Design Wind Force for Region-2 (According to Cl. 29.4.1 of ASCE/SEI 7).

F_{C2}= RF/1000*MAX(16;q_h*G*C_{fC2}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC2} = 2.1 kips

Region-3 from (2s to 3s) Validation: IF(L/s>2;"Valid";"Invalid";) = Valid

Force Coefficient for Region-3 (According to Fig. 29.4-1 of ASCE/SEI 7),

C_{fC3}= 1.68

Effective Area for Region-3, A_{sC3}= IF(L/s>3;s*s;IF(L/s<2;0;(L-2*s)*s;);) = 100 ft²

Design Wind Force for Region-3 (According to Cl. 29.4.1 of ASCE/SEI 7).

F_{C3}= RF/1000*MAX(16;q_h*G*C_{fC3}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC3} = 1.6 kips

Region-4 from (3s to L) Validation: IF(L/s>3;"Valid";"Invalid";) = Valid

Force Coefficient for Region-4 (According to Fig. 29.4-1 of ASCE/SEI 7),

C_{fC4}= 1.05

Effective Area for Region-4, A_{sC4}= IF(L/s>3;(L-3*s)*s;0;) = 450 ft²

Design Wind Force for Region-4 (According to Cl. 29.4.1 of ASCE/SEI 7).

F_{C4}= RF/1000*MAX(16;q_h*G*C_{fC4}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC4} = 7.2 kips

Calculation Summary

F_A=	F_A	=	11.6 kips
F_B=	F_B	=	11.6 kips
F_{C1}=	F_{C1}	=	3.2 kips
F_{C2}=	F_{C2}	=	2.1 kips
F_{C3}=	F_{C3}	=	1.6 kips
F_{C4}=	F_{C4}	=	7.2 kips

Calculation of Snow Loads for Flat Roof as per ASCE/SEI 7-10 Chapter 7

Parameters of Snow Load

Ground Snow Loads (According to Table 7-1 of ASCE/SEI 7), p_g= 18.00 psf

Snow Density (According to Eq. 7.7-1 of ASCE/SEI 7),

γ= MIN(0.13 * p_g + 14; 30) = 16.34 lb/ft^3

Terrain Category (According to Table 7-2 of ASCE/SEI 7),

TER_CAT= SEL("ASCE/Ter_Cat"; ID;) = B

Exposure of Roof (According to Table 7-2 of ASCE/SEI 7),

EX_RF= SEL("ASCE/EX_RF"; ID;) = Fully Exposed

Exposure Factor (According to Table 7-2 of ASCE/SEI 7),

C_e= 0.90

Thermal Factor (According to Table 7-3 of ASCE/SEI 7),

C_t= SEL("ASCE/Ct"; ID;) = 1.00

Risk Category (According to Table 1.5-1 of ASCE/SEI 7),

RI_CAT= SEL("ASCE/Risk_Cat"; ID;) = II

Importance Factor (According to Table 1.5-2 of ASCE/SEI 7),

I_s= TAB("ASCE/Is"; Is; RI_CAT=RI_CAT;) = 1.00

Snow Load for Flat Roof

Min Snow Load (According to Cl. 7.3.4 of ASCE/SEI 7),

p_m= IF(p_g>20; 20*I_s; p_g*I_s) = 18.00 psf

Flat Roof Snow Load (According to Cl. 7.3 of ASCE/SEI 7),

p_f= 0.7 * C_e * C_t *I_s * p_g = 11.34 psf

Calculation Summary

Min Snow Load, p_m= p_m = 18.00 psf

Flat Roof Snow Load, p_f= p_f = 11.34 psf

Calculation of Snow Loads for Sloped Roof as per ASCE/SEI 7-10 Chapter 7

Parameters of Snow Load

Ground Snow Loads (According to Table 7-1 of ASCE/SEI 7), p_g=			40.00 pf
Density of Snow (According to Eq. 7.7-1 of ASCE/SEI 7),			
γ=	MIN(0.13 * p_g + 14; 30)	=	19.20 lb/ft^3
Terrain Category (According to Table 7-2 of ASCE/SEI 7),			
TER_CAT=	SEL("ASCE/Ter_Cat"; ID;)	=	B
Exposure of Roof (According to Table 7-2 of ASCE/SEI 7),			
EX_RF=	SEL("ASCE/EX_RF"; ID;)	=	Fully Exposed
Exposure Factor (According to Table 7-2 of ASCE/SEI 7),			
C_e=			0.70
Thermal Factor (According to Table 7-3 of ASCE/SEI 7),			
C_t=	SEL("ASCE/Ct"; ID;)	=	1.10
Risk Category (According to Table 1.5-1 of ASCE/SEI 7),			
RI_CAT=	SEL("ASCE/Risk_Cat"; ID;)	=	II
Importance Factor (According to Table 1.5-2 of ASCE/SEI 7),			
I_s=	TAB("ASCE/Is"; Is; RI_CAT=RI_CAT;)	=	1.00

Snow Load for Flat Roof

Min Snow Load (According to Cl. 7.3.4 of ASCE/SEI 7),			
p_m=	IF(p_g>20; 20*I_s; p_g*I_s)	=	20.00 psf
Flat Roof Snow Load (According to Cl. 7.3 of ASCE/SEI 7),			
p_f=	0.7 * C_e * C_t *I_s * p_g	=	21.56 psf

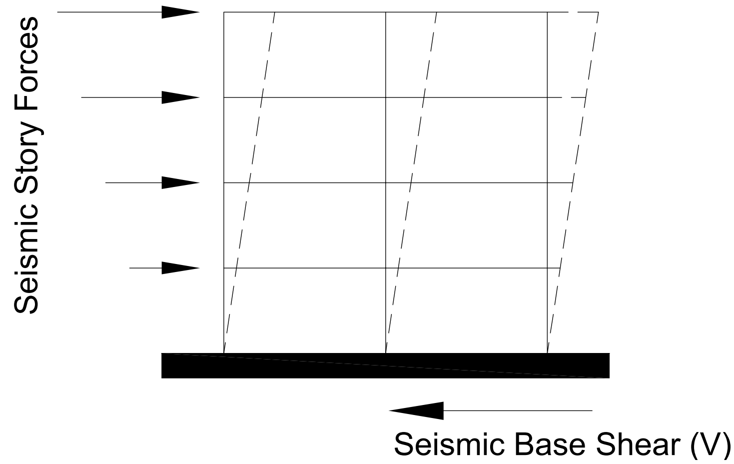
Snow Load for Sloped Roof

α=			20.0 ^{0}
Thermal Resistance Value R=			30.0 ^{0} Fh ft^2/Btu
Roof Slope Factor (According to Fig. (7-2b) of ASCE/SEL 7,			
C_s=			1.00
Sloped Roof Snow Load (According to Cl.7.4 of ASCE/SEL 7),			
p_s=	p_f * C_s	=	21.56 in

Calculation Summary

Min Snow Load, p_m=	p_m	=	20.00 psf
Flat Roof Snow Load, p_f=	p_f	=	21.56 psf
Sloped Roof Snow Load, p_s=	p_s	=	21.56 in

Calculation of Seismic Base Shear as per ASCE/SEI 7-10 Chapter 11



Site Parameters

(According to Cl.11.4.2 of ASCE/SEI 7),

Site Class:	SEL("ASCE/Site_Cl"; ID;)	=	A

Mapped Acceleration Parameters

(According to Cl.11.4.1 of ASCE/SE1 7),

At Short Period, S_s=			1.50
At 1 Second Period, S_1=			0.50
Site Coefficient at Short Period, F_a=			1.00
Site Coefficient at 1 Second Period, F_v=			1.30

Spectral Response Acceleration Parameters

(According to Cl.11.4.3 of ASCE/SE1 7),

Spectral Response Acceleration at Short Period, S_{MS}=	$F_a * S_s$	=	1.50
Spectral Response Acceleration at 1 Second Period, S_{M1}=	$F_v * S_1$	=	0.65

Design Spectral Acceleration Parameters

(According to Cl.11.4.4 of ASCE/SE1 7),

Design Spectral Acceleration Parameter at Short Period, S_{DS}=	$2/3 * S_{MS}$	=	1.00
Design Spectral Acceleration Parameter at 1 Second period, S_{D1}=	$2/3 * S_{M1}$	=	0.43

Risk Category

Risk Category (According to Table 1.5-1 of ASCE/SEI 7),

RI_CAT=	SEL("ASCE/Risk_Cat"; ID;)	=	I

Importance Factor (According to Table 1.5-12 of ASCE/SEI 7),

I_e=	TAB("ASCE/Ie"; Ie; RI_CAT=RI_CAT)	=	1.00

Fundamental Period:

(According to Cl.12.8.2 of ASCE/SEI 7),

Type of Structure, STR= SEL("ACI/Ct&X" ;STR;) = All other structural systems

Building Period Parameter, C_t= TAB("ACI/Ct&X";Ct;STR=STR;) = 0.020

Building Period Parameter, x= TAB("ACI/Ct&X";X;STR=STR;) = 0.750

Structure Height, h_n= 66.00 ft

Approximate Fundamental Period, T_a= $C_t * h_n^{x}$ = 0.46 sec

Building fundamental period, T= T_a = 0.46 sec

Long-period transition period, T_L= 2.00

Seismic Response Coefficient

(According to Cl.12.8.1.1 of ASCE/SEI 7)

Response Modification Coefficient (According to Table 12.2-1), R= 3.25

Calculated Seismic Response Coefficient, $C_{s.c}$= $\dfrac{S_{DS}}{R / I_e}$ = 0.3077

Maximum Seismic Response Coefficient, $C_{s.max1}$= $\dfrac{S_{D1}}{T*(R/I_e)}$ = 0.2876

Maximum Seismic Response Coefficient, $C_{s.max2}$= $\dfrac{S_{D1} * T_L}{T^2 \times (R/I_e)}$ = 1.2505

Maximum Seismic Response Coefficient, $C_{s.max}$= IF(T>T_L; $C_{s.max2}$; $C_{s.max1}$) = 0.2876

Minimum Seismic Response Coefficient, $C_{s.min}$= $0.044 * S_{DS} * I_e$ = 0.0440

Seismic Response Coefficient C_s= IF($C_{s.c} \geq C_{s.min}$;MIN($C_{s.c}$; $C_{s.max}$);$C_{s.min}$) = 0.2876

Seismic Base Shear

(According to Cl.12.8.1 of ASCE/SEI 7),

Effective Seismic Weight of Structure, W= 1305.00 kips

Seismic Base Shear V= $C_s * W$ = 375.32 kips

Calculation Summary

Seismic Base Shear V= $C_s * W$ = 375.32 kips