AHBL

Structural Calculations

PREPARED FOR:

Panattoni Development Company
1820 Dock Street
Suite 100
Tacoma, WA 98402

PROJECT:

Panattoni - Tumwater
South Sound Commerce Center
Building A

2210856.20

PREPARED BY:

Daniel L. Booth PE SE
Principal

REVIEWED BY:

Danial L. Booth PE SE
Principal

DATE:

March 2021

Civil Engineers • Structural Engineers • Landscape Architects • Community Planners • Land Surveyors • Neighbors

Structural Calculations

For

Panattoni - Tumwater
South Sound Commerce Center
Building A

Tumwater, WA

Project # 2210856.20

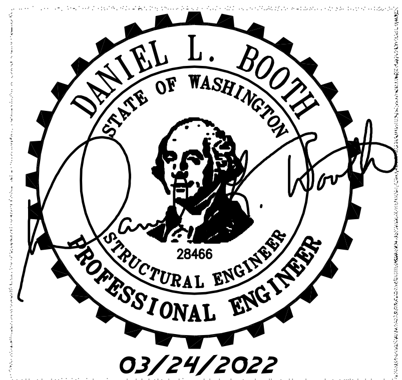
Project Principal Danial L. Booth PE SE

Design Criteria

Design Codes and Standards

Codes and Standards: Structural design and construction shall be in accordance with the applicable sections of the following codes and standards as adopted and amended by the local building authority: International Building Code, 2018 Edition.

Structural Design Criteria:



Live Load Criteria:

Roof (Min Blanket Snow)	25 psf
Slab on Grade	350 psf

Wind Load Criteria:

Ultimate Wind Speed	97mph
Risk Category	II
Wind Exposure	B
Topographic Factor	1.00

Seismic Criteria:

Risk Category	II
Seismic Importance Factor	1.00
S_s = 1.370	S_1 = 0.519
S_{ds} = 0.913	S_{d1} = 0.616
Site Class	= D
Seismic Design Category	= D
Response Modification Coeff. (R):	5
Seismic Response Coeff. (C_s):	0.183

Soil Criteria:

Based on Geotechnical Engineering Report by: Terra Associates Inc, dated January 31, 2022. Allowable Soil Bearing Capacity: 2,500psf allow 33% increase for loads from wind or seismic origin.

Project Description

The project consists of a standard tilt warehouse structure with concrete walls, steel joist and joist girders with wood purlins and plywood sheathed roof structure.

OSHPD

Latitude, Longitude: 46.9715, -122.9218

Date	2/14/2022, 11:32:25 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.377	MCE_R ground motion. (for 0.2 second period)
S_1	0.519	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.377	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.918	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.595	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.655	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	1.377	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.528	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.519	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.587	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.765	Factored deterministic acceleration value. (1.0 second)
PGAd	0.653	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.901	Mapped value of the risk coefficient at short periods
C_{R1}	0.883	Mapped value of the risk coefficient at a period of 1 s

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Code Search

Code: International Building Code 2018

Occupancy:

Occupancy Group = S Storage

Risk Category & Importance Factors:

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

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length (L)	860.0 ft	
Least width (B)	500.0 ft	
Mean Roof Ht (h)	39.5 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

Floor:

Typical Floor	40 psf
Partitions	15 psf
Lobbies & first floor corridors	100 psf
Corridors above first floor	80 psf
Balconies (1.5 times live load)	60 psf

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JOB NO. ____ SHEET NO. ____
CALCULATED BY ____ DATE ____
CHECKED BY ____ DATE ____

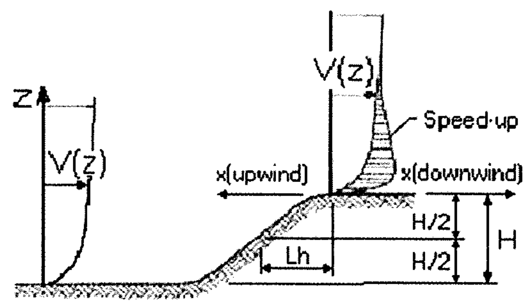
Wind Loads : ASCE 7- 16

Ultimate Wind Speed	97 mph
Nominal Wind Speed	75.1 mph
Risk Category	II
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.758
Kh case 2	0.758
Type of roof	Gable

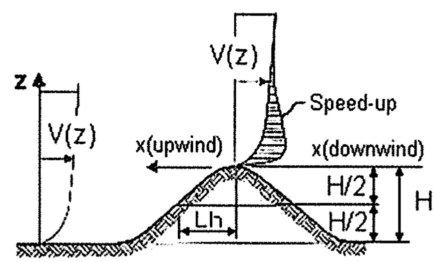
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	80.0 ft
Half Hill Length (Lh)	100.0 ft
Actual H/Lh =	0.80
Use H/Lh =	0.50
Modified Lh =	160.0 ft
From top of crest: x =	50.0 ft
Bldg up/down wind?	downwind

H/Lh= 0.50 K_1 = 0.000
x/Lh = 0.31 K_2 = 0.792
z/Lh = 0.25 K_3 = 1.000
At Mean Roof Ht:
Kzt = $(1+K_1K_2K_3)^2$ = 1.00



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h = 39.5 ft
B = 500.0 ft
/z (0.6h) = 30.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.08 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.33
ℓ =	320 ft
z_{min} =	30 ft
c =	0.30
g_Q, g_v =	3.4
L_z =	310.0 ft
Q =	0.73
I_z =	0.30
G =	0.76

Flexible or Dynamically Sensitive Structure

34 ıcy (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.45				
/α =	0.25				
Vz =	62.5				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	39.5 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
Gf =	0.000				

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JOB TITLE Panattoni Tumwater Industrial Building A

JOB NO. ______ **SHEET NO.** ______
CALCULATED BY ______ **DATE** ______
CHECKED BY ______ **DATE** ______

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

Kz = Kh (case 1) = 0.76
Base pressure (qh) = **15.5 psf**
GCpi = +/-0.18

Edge Strip (a) = 20.0 ft
End Zone (2a) = 40.0 ft
Zone 2 length = 98.8 ft

Wind Pressure Coefficients

	CASE A				CASE B		
		θ = 1.2 deg					
Surface	GCpf	w/-GCpi	w/+GCpi		GCpf	w/-GCpi	w/+GCpi
1	0.40	0.58	0.22		-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87		-0.69	-0.51	-0.87
3	-0.37	-0.19	-0.55		-0.37	-0.19	-0.55
4	-0.29	-0.11	-0.47		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.61	0.79	0.43		-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25		-1.07	-0.89	-1.25
3E	-0.53	-0.35	-0.71		-0.53	-0.35	-0.71
4E	-0.43	-0.25	-0.61		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1		9.0	3.4			-4.2	-9.8
2		-7.9	-13.5			-7.9	-13.5
3		-2.9	-8.5			-2.9	-8.5
4		-1.7	-7.3			-4.2	-9.8
5						9.0	3.4
6						-1.7	-7.3
1E		12.3	6.7			-4.7	-10.2
2E		-13.8	-19.4			-13.8	-19.4
3E		-5.4	-11.0			-5.4	-11.0
4E		-3.9	-9.5			-4.7	-10.2
5E						12.3	6.7
6E						-3.9	-9.5

Parapet

Windward parapet = 0.0 psf (GCpn = +1.5)
Leeward parapet = 0.0 psf (GCpn = -1.0)

Windward roof overhangs = 10.9 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

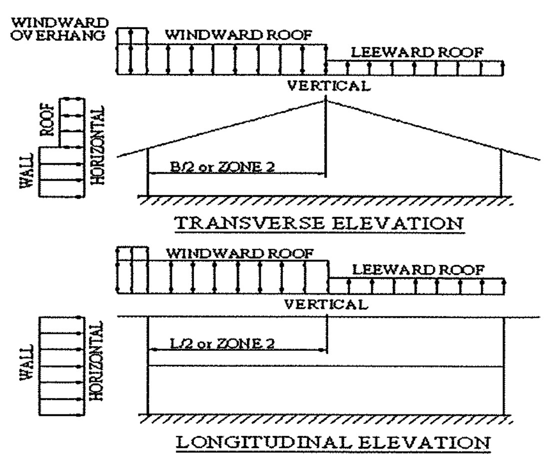
Interior Zone: Wall 10.7 psf
Roof -5.0 psf **
End Zone: Wall 16.1 psf
Roof -8.4 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 10.7 psf
End Zone: Wall 16.1 psf

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

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



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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.76	h =	39.5 ft	0.2h = 7.9 ft
Base pressure (qh) =	**15.5 psf**	0.6h =	23.7 ft	
Minimum parapet ht =	0.0 ft	GCpi =	+/-0.18	
Roof Angle (θ) =	1.2 deg	qi = qh =	15.5 psf	
Type of roof =	Gable			

Roof

	Surface Pressure (psf)								User input	
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf	10 sf	50 sf
Negative Zone 1	-29.2	-27.3	-24.7	-22.8	-20.9	-19.3	-18.3	-18.3	-29.2	-24.7
Negative Zone 1'	-16.8	-16.8	-16.8	-16.8	-16.0	-16.0	-16.0	-16.0	-16.8	-16.8
Negative Zone 2	-38.5	-36	-32.7	-30.3	-27.8	-25.8	-24.5	-24.5	-38.5	-32.7
Negative Zone 3	-52.5	-47.5	-41	-36	-31.1	-27.1	-24.5	-24.5	-52.5	-41.0
Positive All Zones	16	16	16	16	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1&1'	-26.4	-25.9	-25.3	-24.8	-20.8	-17.6	-16.0	-16.0	-26.4	-25.3
Overhang Zone 2	-35.7	-32.4	-28	-24.7	-21.4	-18.8	-17.1	-17.1	-35.7	-28.0
Overhang Zone 3	-49.7	-43.9	-36.3	-30.5	-24.7	-20.0	-17.1	-17.1	-49.7	-36.3

Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 2.8 psf)

Parapet

qp = 0.0 psf

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

wall a = 20.0 ft

Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	200 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-18.2	-16.0	-16.0	-16.0	-18.2	-16.0
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-22.3	-17.4	-16.0	-16.0	-22.3	-16.0
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	16.8	16.0	16.0	16.0	16.8	16.0

Note: GCp reduced by 10% due to roof angle <= 10 deg.

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Seismic Loads: IBC 2018 Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class : D

Ss (0.2 sec) = 137.00 %g
S1 (1.0 sec) = 51.90 %g
A site specific ground motion analysis is required for seismically isolated structures or with damping systems, see ASCE7 11.4.8

Fa =	1.000	Sms =	1.370	S_{DS} =	0.913	Design Category =	D
Fv =	1.781	Sm1 =	0.924	S_{D1} =	0.616	Design Category =	D

Seismic Design Category = **D**

Redundancy Coefficient ρ = 1.00 Code exception must be met for ρ to equal 1.0
Number of Stories: 1
Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Special reinforced concrete shear walls**
System Structural Height Limit: **160 ft**
Actual Structural Height (hn) = 44.7 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 5
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 5
S_{DS} = 0.913
S_{D1} = 0.616

Seismic Load Effect (E) = Eh +/-Ev = $\rho Q_E \pm 0.2S_{DS} D$ = Qe +/- 0.183D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E \pm 0.2S_{DS} D$ = 2Qe +/- 0.183D   D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.020   Cu = 1.40
Approx fundamental period (Ta) = $C_T h_n^{\wedge}$ = 0.346 sec   x= 0.75   Tmax = CuTa = 0.484
User calculated fundamental period (T) = sec   Use T = 0.346
Long Period Transition Period (TL) = ASCE7 map = 16

Seismic response coef. (Cs) = SdsI/R = 0.183   ASCE7 11.4.8 exception 2 equations used
but not less than Cs = 0.044SdsI = 0.040
USE Cs = 0.183
Design Base Shear V = 0.183W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

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2018 IBC SEISMIC LOADS / ASCE 7-16...EQUIVALENT *STATIC FORCE PROCEDURE*

All References to ASCE 7 unless noted otherwise

BUILDING STRUCTURAL SYSTEM: Table 12.2-1

A1. Special Reinforced Concrete Shear Walls (PC & CIP)

RISK CATEGORY	II	Section 1.5		
GEOTECH : SITE CLASS	D	Section 11.4		
BUILDING HEIGHT (ft)	39.5		Diaphragm= F	R=Rigid F=Flex
SHORT PERIOD SPECTRAL Ss	1.377	Chapter 22	http://earthquake.usgs.gov/research/hazmaps/design/	
1 SECOND PERIOD SPECTRAL	0.519	Chapter 22		

DESIGN CATEGORY = D TABLE 11.6-1 or 11.6-2 **HEIGHT LIMIT CATG. D/E/F=** 160/160/100

Fa=	**1.00**	Table 11.4-1				
Fv=	**1.80**	Table 11.4-2		DESIGN BASE SHEAR % g		
I =	**1.00**	Table 11.5-1		Cs	**0.184**	Section 12.8 Equation 12.8-2
R =	**5.00**	Table 12-2-1		MAX-1	**0.395**	Section 12.8 Equation 12.8-3
T =	**0.32**	Section 12.8.2.1		MIN - 1	**0.010**	Section 12.8 Equation 12.8-5
S_{MS}=	**1.38**	S_{DS}=	0.9180	MIN -2	**0.010**	Section 12.8 Equation 12.8-6
S_{M1}	**0.93**	S_{DI}=	0.6228	MAX-2	**T<TL**	Section 12.8 Equation 12.8-4
ρ =	**1.00**	Section 12.3.4		Q_E = V =	**0.184**	x W
Ω_o =	2.50	Table 12-2-1				
Ω =	**2.00**	Table 12.2-1 NOTE G (except Cantilever Column Systems)				

MINIMUM DESIGN LATERAL FORCES

BASED ON STRENGTH DESIGN

$E_h=\rho Q_E$ =	0.184 x W	$E = \rho Q_E$	EQ 12.4-3
E_v =	0.184 x DEAD LOAD	Ev = +/- 0.2xSDSxD	EQ 12.4-4

E_{mh}=	0.367 x W	EQ 12.4-7

EQ16-52/53

FOR ASD (0.7 x Q_E)

ρE =	0.129 x W

1605.3.1

Project Panattoni Tumwater Industrial Building A Project No. 2210856.20

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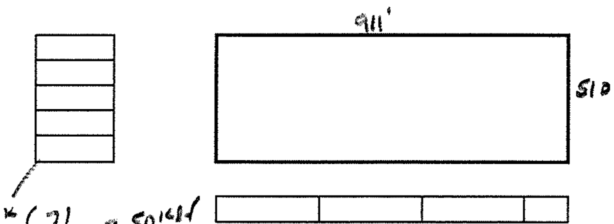
2018 IBC / ASCE 7-16 VERTICAL DISTRIBUTION OF SEISMIC FORCES
EQUIVALENT LATERAL FORCE METHOD

LEVEL	DL PSF	TRIB AREA ft X ft	DL (WALL) PSF	E/W TRIB. LENGTH FT	N/S TRIB. LENGTH FT	E/W -- WX K	N/S -- WX	Hs ft	Hx ft	E/W W X Hx^K	N/S WxHx^K	E/W %	N/S %
1	13.60	480,820	119	1,822	1,460	10745	9910	**38.8**	38.8	416922	384497	1.000	1.000
2	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
3	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
4	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
5	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
6	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
7	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
8	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
9	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
10	0	0	0	0	0	0	0	**0.0**	38.8	0	0		
						SUM	SUM			SUM	SUM		
WEIGHT OF BUILDING						10745	9910			416922	384497		
Eh 18.4 % OF DEAD LOAD						KIP 1973	KIP 1819			K= 1			
										T= 0.32			

HORIZONTAL FORCE AT EACH LEVEL					
Eh		0.7*Eh ALLOW.			
E/W	N/S	E/W	N/S		
1972.9	**1819.4**	**1381.0**	**1273.6**	Kip	1st FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	2nd FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	3rd FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	4th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	5th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	6th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	7th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	8th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	9th FLOOR
0.0	**0.0**	**0.0**	**0.0**	Kip	10th FLOOR



$\frac{1819^k(.7)}{510} = 2.50^{klf}$ (ASD)

$\frac{1973^k \times 0.7}{911'} = 1.516klf$ (ASD)

$CF_{EW} = \frac{1516plf(911')^2/8}{510} = 308.4^k = 440.6^k$ (ASD) (ULT)

use L8x6x3/4 (50ksi)

$\phi P_n = .9(50ksi)(9.99in^2) = \underline{449.5^k}$ o.k.

Chord Splice

Use $1\frac{3}{4}$"x6" (50ksi) $\phi P_n = .9(50ksi)(1.75")(6") = 472.5^k$ o.k.

Weld Length $= \frac{440.6}{1.392(5)} = 63.3$ use (2) sides 32" of 5/16 fillet

Reduce Chord to L8x6x9/16 (50ksi) @ 231.5' from end wall (Grid ⑤ & ⑬)

$CF_{EW} = \frac{\frac{1973^k(.7)}{2}(231.5') - 1.516klf(231.5')^2/2}{510'} = 233.8^k = 334.0^k$ (ASD) (ULT) <

$\phi P_n = .9(50ksi)(7.61in^2) = 342^k$

Splice - use $1\frac{3}{8}$"x6" $\phi P_n = .9(50ksi)(1.375)(6) = 371.3^k$ o.k

Weld $= 334.0^k / 1.392(5) = 48"$ use (2) sides 24" long 5/16" weld

Chord force at Grid (2.5) & 15.5

$CF_{EW} = \frac{1973^k(.7)(91.5') - 1.516klf(91.5')^2/2}{510} = 111.4^k = 159.2^k$ (ASD) (ULT)

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Civil Engineers

Structural Engineers

Landscape Architects

Community Planners

Land Surveyors

Neighbors

$CF_{N-S} = \frac{2.50 klf(510)^2}{8(911)} = 89.2K$ (ASD) 127.5^K (ULT)

Use L5x5x 5/16 (50ksi)

$\phi P_n = .9(50ksi)(3.07 in^2) = 138.2^K$ O.K.

Chord Splice

Use 1"x3" (50ksi) PL $\phi P_n = .9(50)(1)(3) = 135^K$ O.K.

Weld Length $= \frac{127.5K}{1.392(5)} = 18.3''$ Use (2) sides 24" of 5/16" fillet weld

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253.383.2572 FAX

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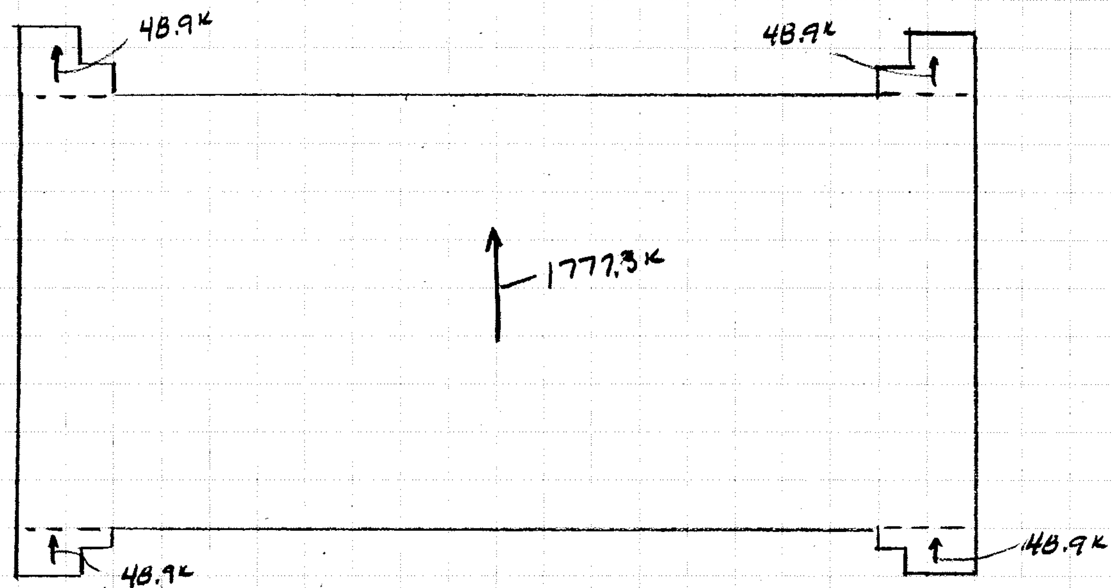
464,610 SF	1456	1777.3
~~435,499 SF~~	~~1404 ft~~	~~1667.9K~~
4052.5	91.5	48.9K
~~3410~~	~~79 ft~~	~~42.06K~~



Maximum Diaphragm Shear = $\frac{1777.3^K/2}{510'} \times 0.7 = 1,220$ plf (ASD)

Type (A) Shtg/Nailing — Use 15/32" APA RATED Sheathing w/ (2) lines 10d NAILS AT 2½" o.c. AT PNL Edges and Boundaries and 10d @ 12" o.c. field. Use 4x framing AT PNL Edges.

Capacity = 2580 plf/2 = 1290 plf O.K.

AT first 91.5' in from Endwall

$\frac{1777.3^K/2 - 1.95^{K/ft}(83.5)}{510} \times .7 = 1050$ plf (ASD

Type (B) Shtg/Nailing — Use 15/32 APA RATED Shtg w/ (2) Lines 10d NAILS AT 2½" o.c. AT PNL Edges & Boundaries 10d @ 12" o.c. field. Use 3x framing AT PNL Edges Cap = 2300/2 = 1190

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AT 190.5' ft from Endwall

$$\frac{1777.3^k/2 - 1.951^k(190.5')}{510} \times .7 = 710\ plf$$

Type © Shtg/Nailing — Use 15/32 APA Rated Shtg w/ (1) line of 10d Nails at 2½" o.c. at Pnl Edges & Boundaries 10d @ 12" o.c. field. Use 3x framing at Pnl Edges

Capacity = 1440 plf / 2 = 720 plf o.k.

AT 250.5 ft from Endwall

$$\frac{1777.3^k/2 - 1.951^k(250.5')}{510'} \times 0.7 = 550\ plf$$

Type Ⓓ Shtg/Nailing — Use 15/32 APA Rated Shtg w/ (1) line of 10d Nails at 3" o.c. at Pnl Edges & Boundaries. 10d @ 12" o.c. field. Use 2x framing at Pnl Edges

Capacity = 1137 plf / 2 = 569 plf o.k.

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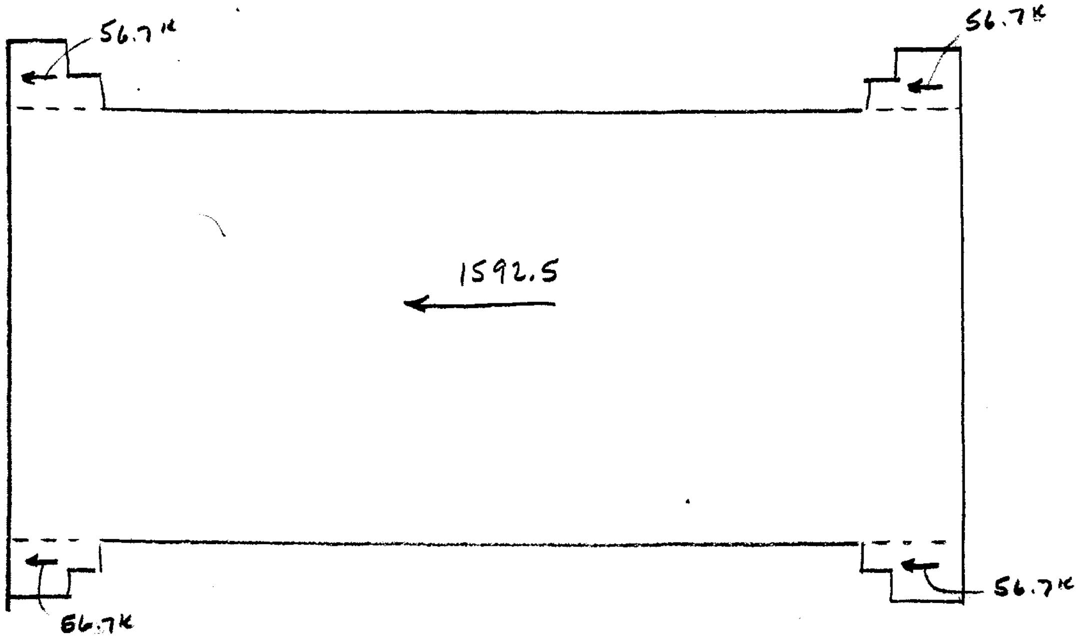
464,610 ~~435,499 sf~~ 1020 ~~1000'~~ 1592.45

4052.5 ~~3440 SF~~ 111 56.74



Maximum Diaphragm Shear = $\frac{1592.5^k/2}{911} \times 0.7 = 612$ plf

Use Typ (C) Nailing Capacity = 720 plf O.K.

AT 30' from side wall

$\frac{(1592.5^k/2 - 3.1225\,k/ft\,(30'))}{911}.7 = 539.8$ plf

Use Type (D) Nailing Capacity = 569 plf O.K.

If this does not meet with your understanding, please contact us in writing within seven days. THANK YOU.

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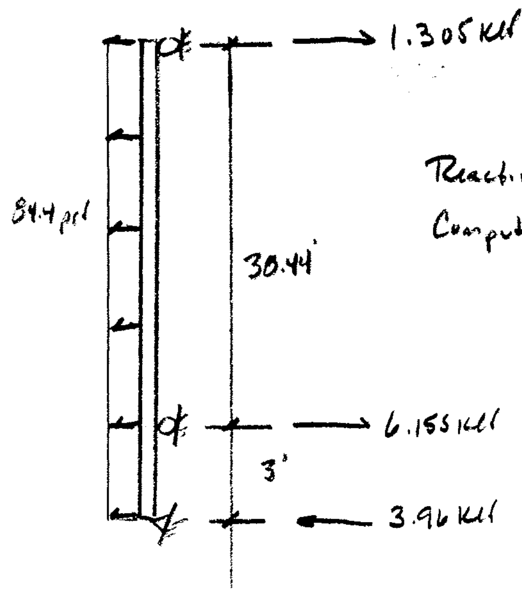
Wall Anchorage

Typ. Pnl

$F_p = 0.8(S_{DS}) I_E W_p = 0.8(0.913)(1.0) W_p = .73 W_p$

$F_p = .73(150)(9.25/12) = 84.4$ psf



Reactions per Computer Analysis

Anchorage force = 1.305 klf Use **1.4 klf**

Joists @ 10'-0" o.c. : 1.4 klf (10') = 14K

Girders D & J = 1.4 klf (120'/2) = 84K

Girders E & H = 1.4 klf ((60'+50')/2) = 77K

Girders F & G = 1.4 klf (100'/2) = 70K

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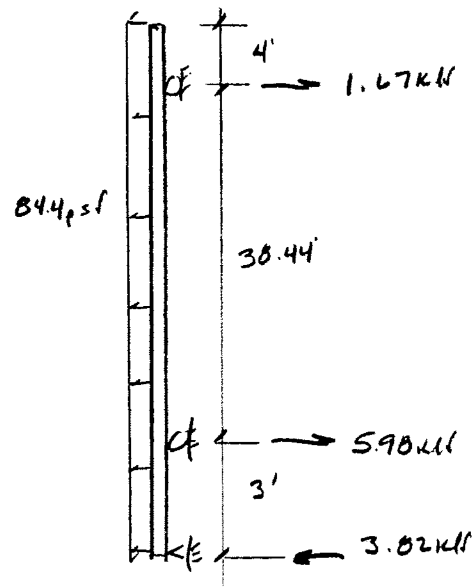
Parapet Panel



Anchorage force = 1.67 klf   Use 1.7 klf

Girders C & K = 1.4klf(60'/2) + 1.7klf(60'/2) = 93k

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Sub-diaphragm

End wall Sub diaphragm

- Max Shear = $\frac{1.7 klf (60'/2)}{63.5'}$ = 803 plf

End Bay of diaphragm is type A Nailing

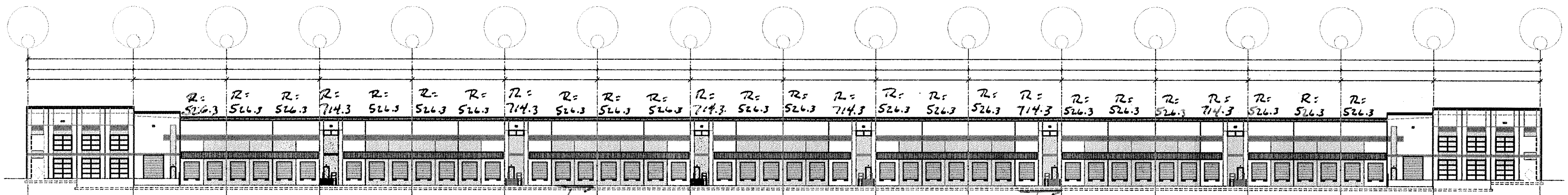
Capacity = 1290 plf > 803 plf o.k.

- Node Sub diaphragm

Shear = $\frac{1.7 klf (63.5'/2)}{55'}$ = 981.4 plf

Use Type B Nailing in Nodes

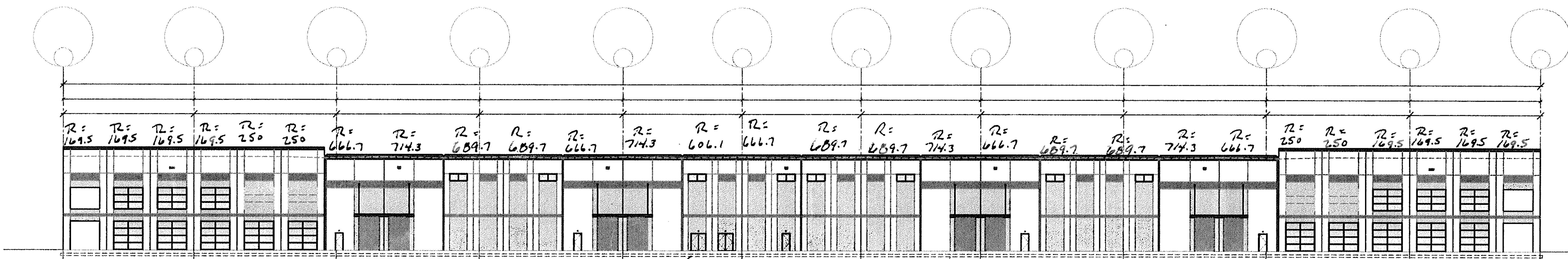
Capacity = 1190 plf > 981.4 plf o.k.



$\Sigma R = 14{,}811.8$

Dock pnl Shear $= \frac{526.3}{14{,}811.8}\left(\frac{1819^k}{2}\right) = 32.3^k$

Pnl Shear $= \frac{714.3}{14{,}811.8}\left(\frac{1819^k}{2}\right) = 43.8^k$



Entry Node Pnl

$V = \frac{169.5}{13{,}291}\left(\frac{1973}{2}\right) = 12.6^K$

Man-Door

$V = \frac{666.7}{13{,}291}(1973/2) = 49.5^K$

$\Sigma R = 13{,}291$

DBL Door

$V = \frac{606.1}{13{,}291}(1973/2) = 45.0^K$

Man-door w/ Window

$V = \frac{666.7}{13{,}291}(1973/2) = 49.5^K$

High Window

$V = \frac{689.7}{13{,}291}\frac{1973}{2} = 51.2^K$

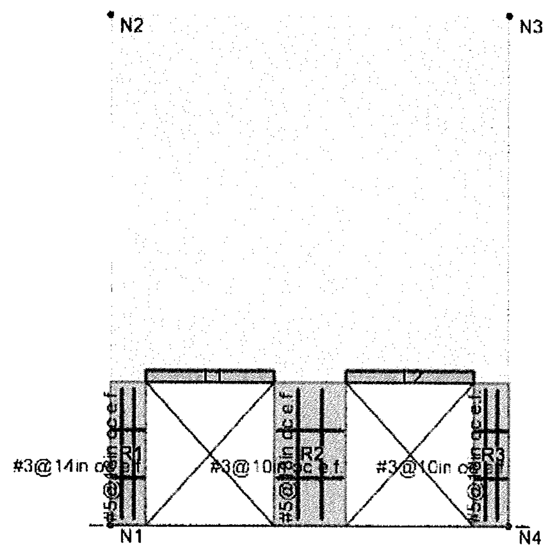
Solid Pnl

$V = \frac{714.3}{13{,}291}(1973/2) = 53.0^K$

Curtain Openings

$V = \frac{250}{13{,}291}\left(\frac{1973}{2}\right) = 18.55^K$

Detail Report: WP1 **Enveloped Results**



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

Geometry:

Total Height (ft):	35.67	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	28	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

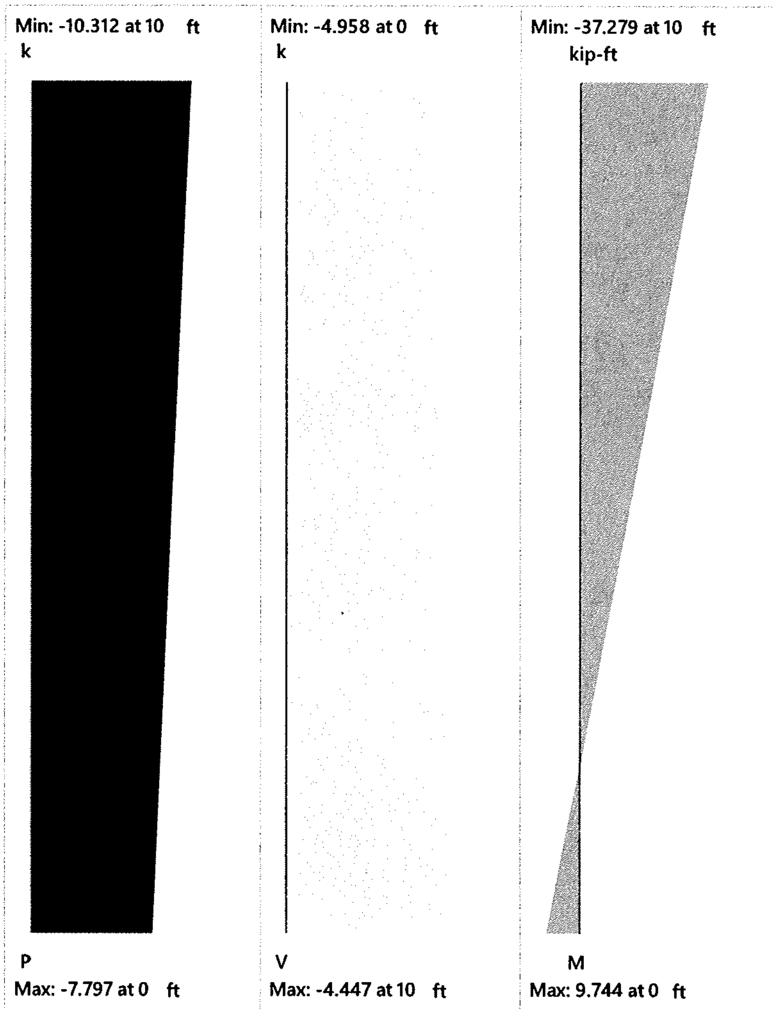
Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.7061**	PASS
R1	1			0.4861	PASS
R2	1			0.7061	PASS
R3	1			0.4094	PASS
UC Shear In-Plane				**0.4685**	PASS
R1	1	0 k	29.2624 k	0.152	PASS
R2	1	0 k	80.8685 k	0.2692	PASS
R3	1	0 k	45.4814 k	0.4685	PASS
Delta Max In-Plane					
R1	1	0.053 in			
R2	1	0.0537 in			
R3	1	0.0532 in			

Delta Max Out-of-Plane

Wall Reinforcement

Region Design		Result
Region R1 (In-Plane)	0.4861	PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

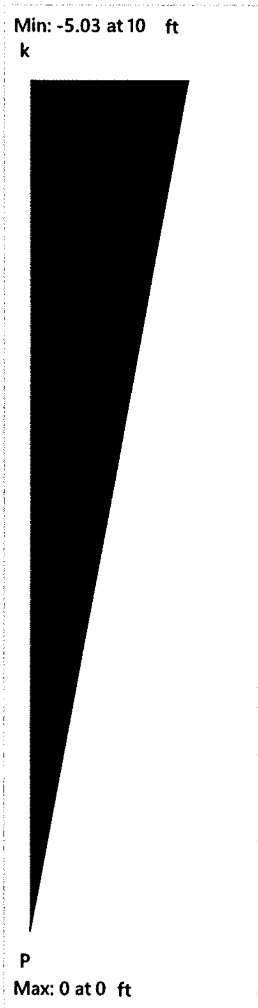
Geometry

Total Height (ft):	10
Total Length (ft):	2.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

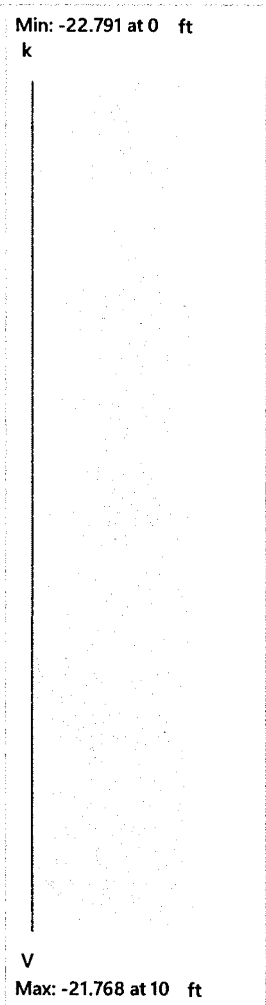
Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.4861	PASS
Shear Details	1	-4.4471 k	29.2624 k	0.152	PASS
Deflection Details	1		0.053 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

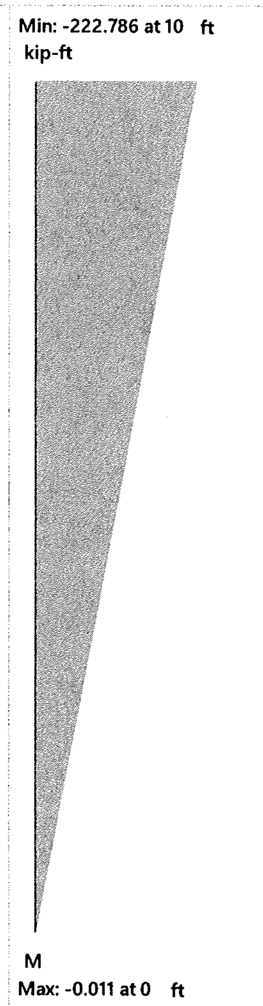
Region R2 (In-Plane) **0.7061** **PASS**



Axial Diagram



Shear Diagram



Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	10
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

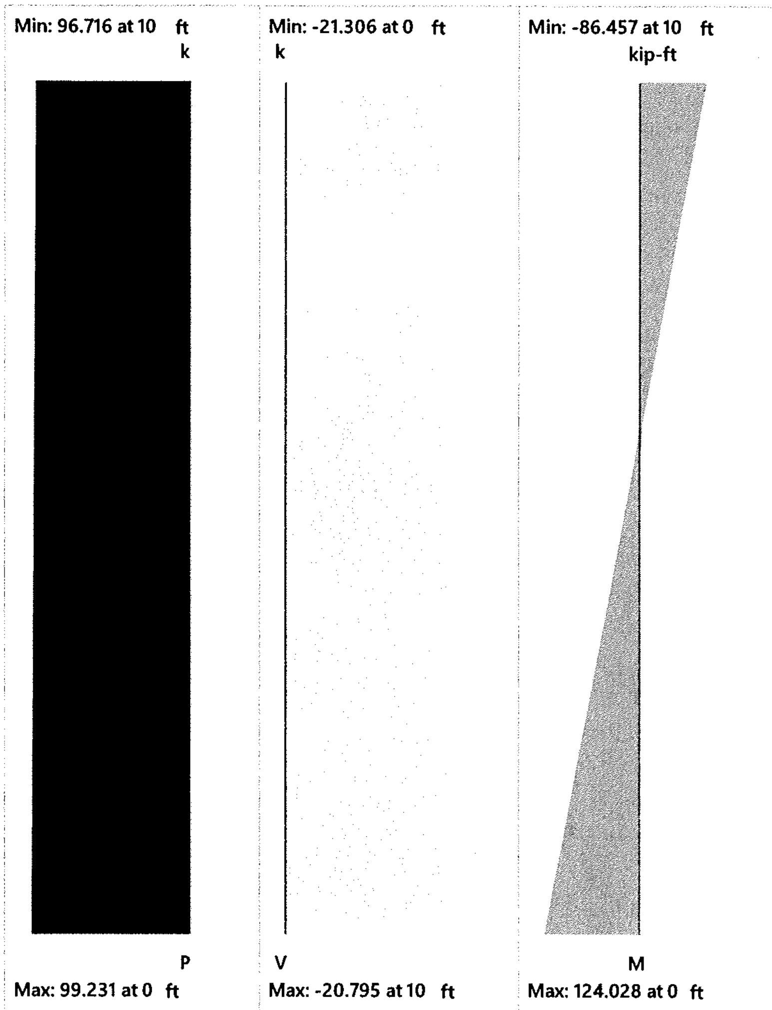
Geometry

Total Height (ft):	10
Total Length (ft):	5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.7061	PASS
Shear Details	1	-21.7684 k	80.8685 k	0.2692	PASS
Deflection Details	1		0.0537 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R3 (In-Plane) 0.4685 PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	10
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

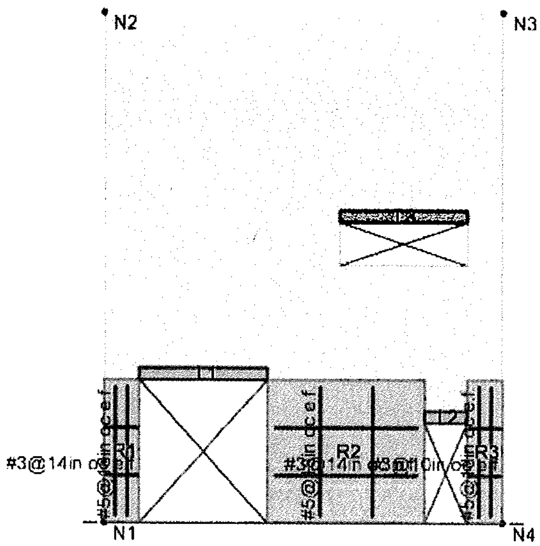
Total Height (ft):	10
Total Length (ft):	2.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.4094	PASS
Shear Details	1	-21.3063 k	45.4814 k	0.4685	PASS
Deflection Details	1		0.0532 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

Geometry:

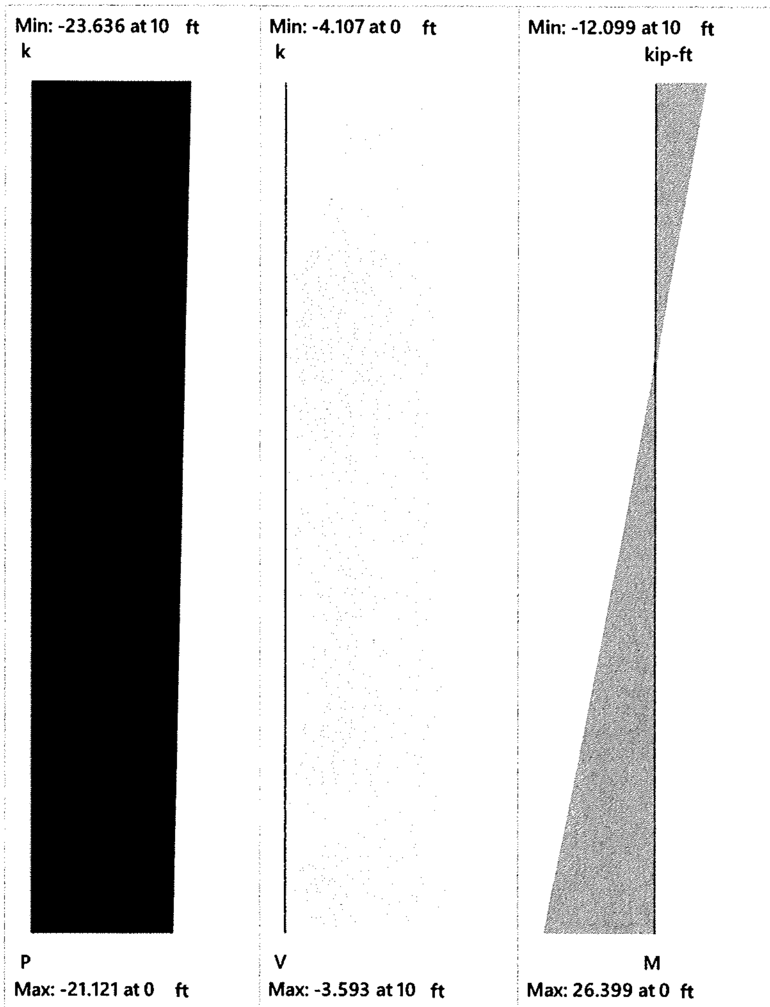
Total Height (ft):	35.67	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	28	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.4818**	PASS
R1	1			0.3567	PASS
R2	1			0.1289	PASS
R3	1			0.4818	PASS
UC Shear In-Plane				**0.5793**	PASS
R1	1	0 k	29.9867 k	0.137	PASS
R2	1	0 k	245.9262 k	0.1162	PASS
R3	1	0 k	49.7518 k	0.5793	PASS
Delta Max In-Plane					
R1	1	0.0179 in			
R2	1	0.0176 in			
R3	1	0.0155 in			

Delta Max Out-of-Plane	
Wall Reinforcement	
Region Design	Result

Region R1 (In-Plane)	0.3567	PASS



Axial Diagram | Shear Diagram | Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

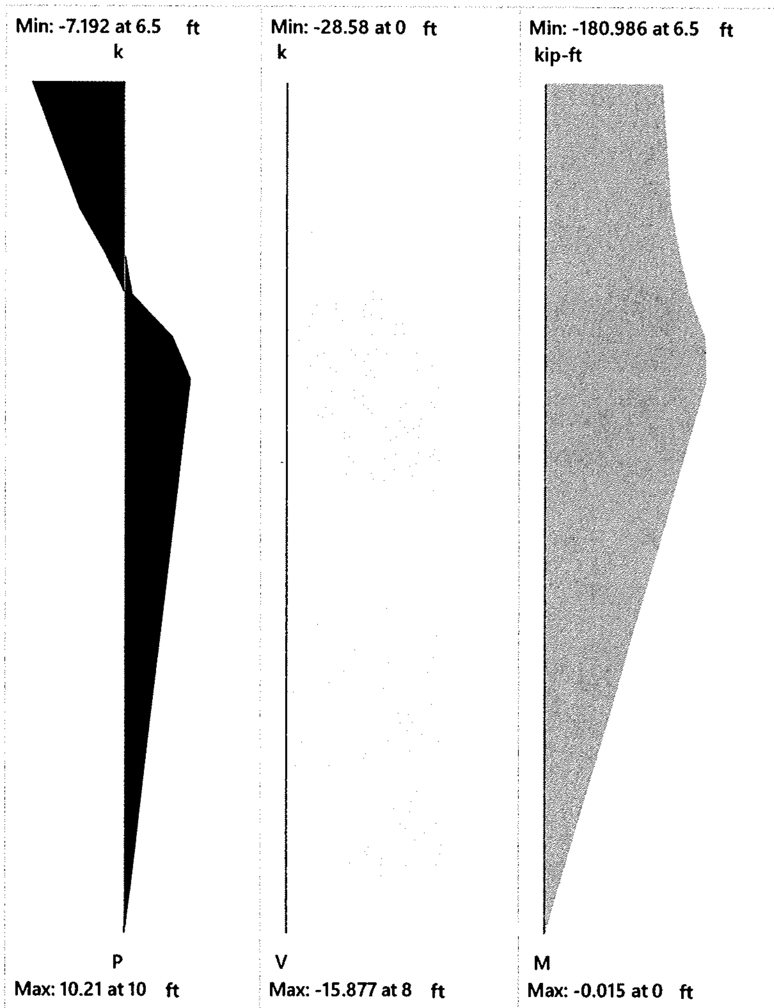
Geometry

Total Height (ft):	10
Total Length (ft):	2.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.3567	PASS
Shear Details	1	-4.1073 k	29.9867 k	0.137	PASS
Deflection Details	1		0.0179 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R2 (In-Plane) **0.1289** **PASS**



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

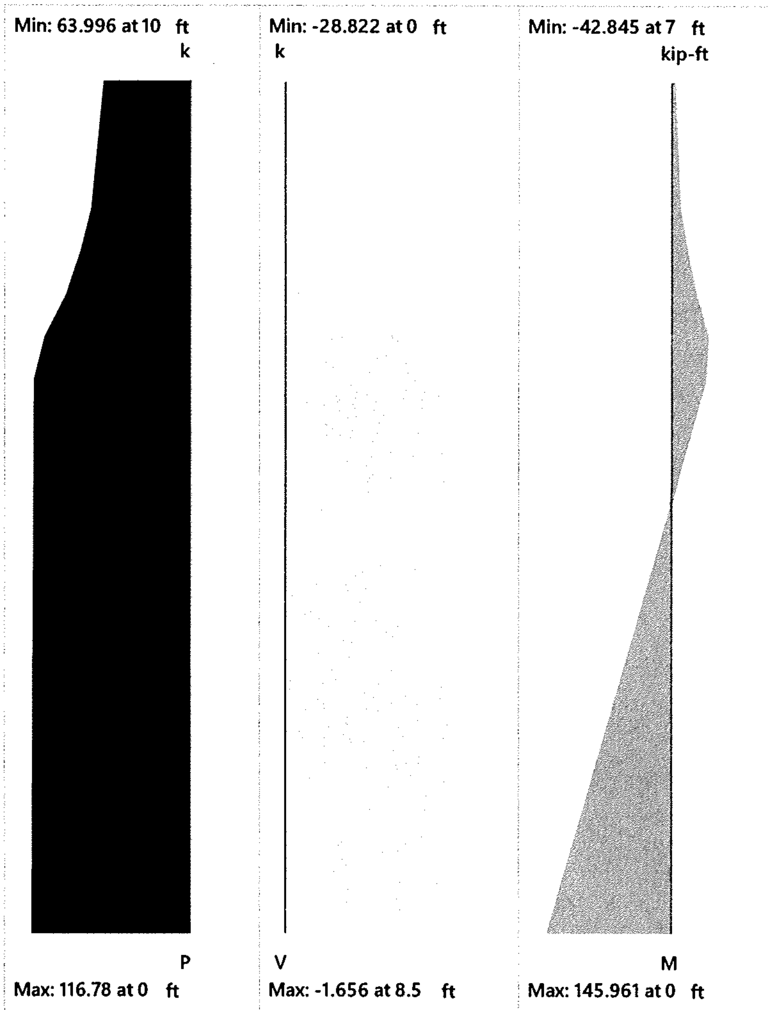
Geometry

Total Height (ft):	10
Total Length (ft):	11
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.1289	PASS
Shear Details	1	-28.5803 k	245.9262 k	0.1162	PASS
Deflection Details	1		0.0176 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R3 (In-Plane) 0.5793 PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	10
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

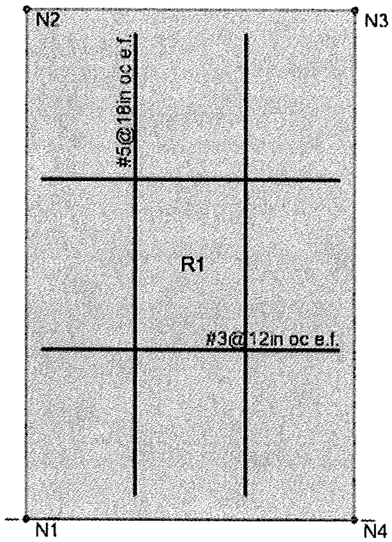
Geometry

Total Height (ft):	10
Total Length (ft):	2.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.4818	PASS
Shear Details	1	-28.8225 k	49.7518 k	0.5793	PASS
Deflection Details	1		0.0155 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1 | **Enveloped Results**



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

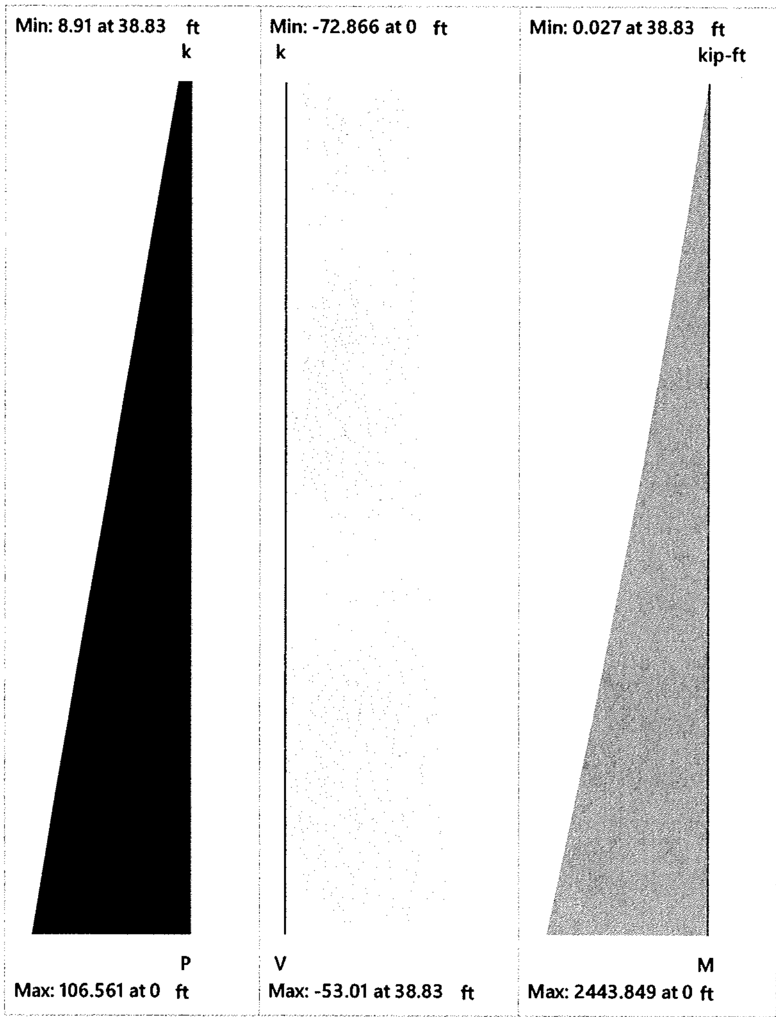
Geometry:

Total Height (ft):	38.83	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	25	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				0.364	PASS
R1	1			0.364	PASS
UC Shear In-Plane				0.164	PASS
R1	1	0 k	444.3177 k	0.164	PASS
Delta Max In-Plane					

Region Design		Result
Region R1 (In-Plane)	0.364	PASS



Axial Diagram　Shear Diagram　Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	12
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

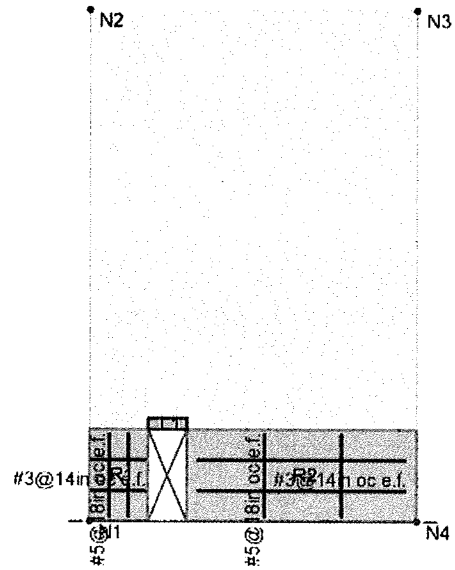
Total Height (ft):	38.83
Total Length (ft):	25
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.364	PASS
Shear Details	1	-72.866 k	444.3177 k	0.164	PASS
Deflection Details	1		0.0845 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

Geometry:

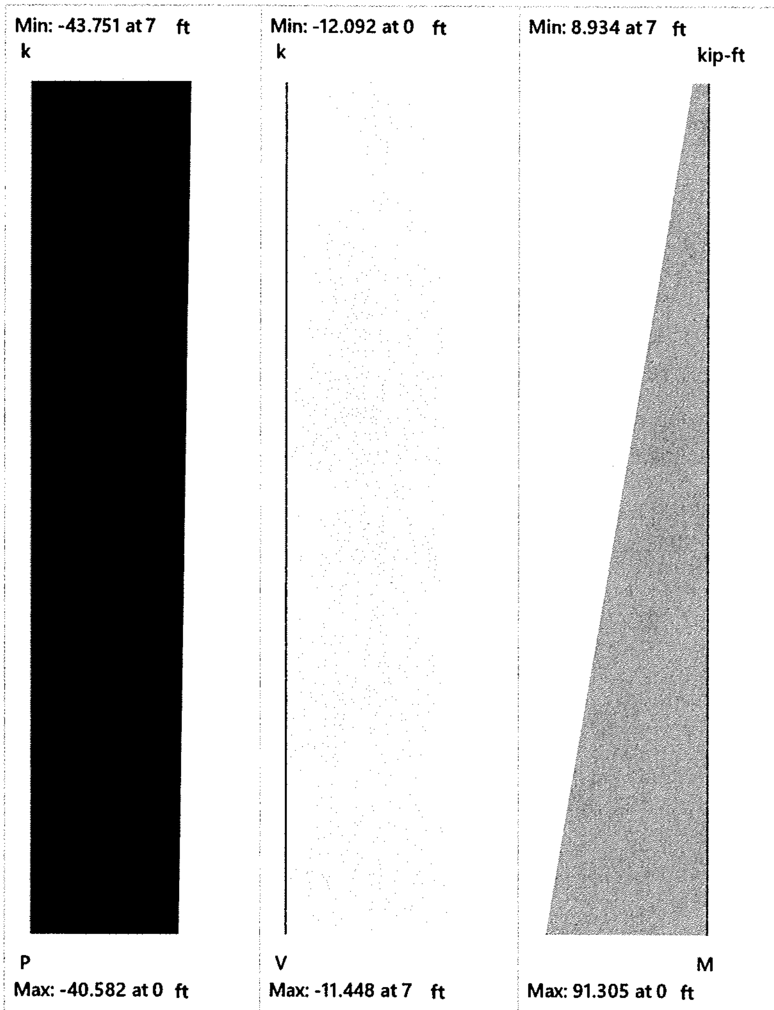
Total Height (ft):	38.83	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	25	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.4401**	**PASS**
R1	1			0.4401	PASS
R2	1			0.1039	PASS
UC Shear In-Plane				**0.1958**	**PASS**
R1	1	0 k	61.7416 k	0.1958	PASS
R2	1	0 k	323.9756 k	0.1755	PASS
Delta Max In-Plane					
R1	1	0.0079 in			
R2	1	0.0062 in			

Region Design

		Result
Region R1 (In-Plane)	0.4401	PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

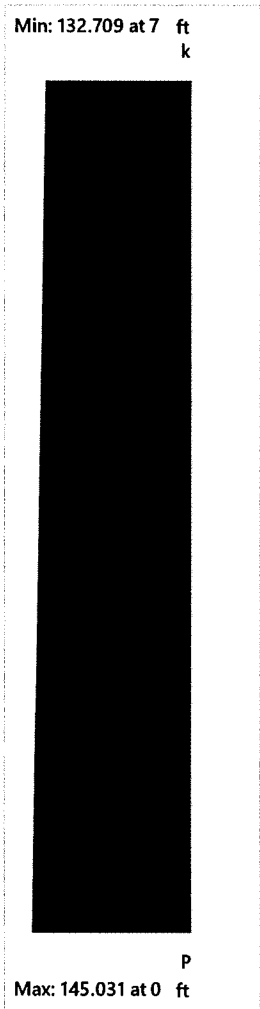
Geometry

Total Height (ft):	7
Total Length (ft):	4.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

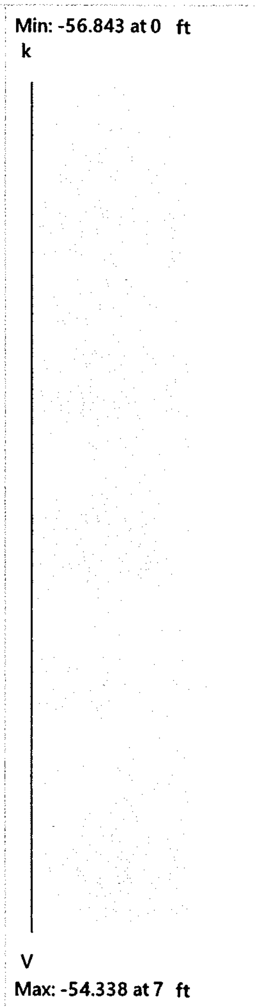
Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.4401	PASS
Shear Details	1	-12.092 k	61.7416 k	0.1958	PASS
Deflection Details	1		0.0079 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

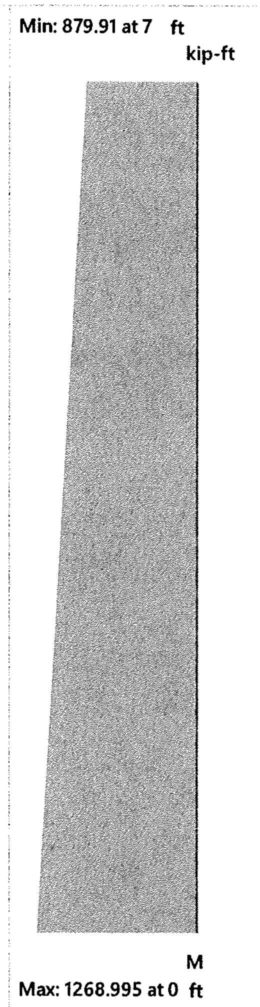
Region R2 (In-Plane) **0.1755** **PASS**



Axial Diagram



Shear Diagram



Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

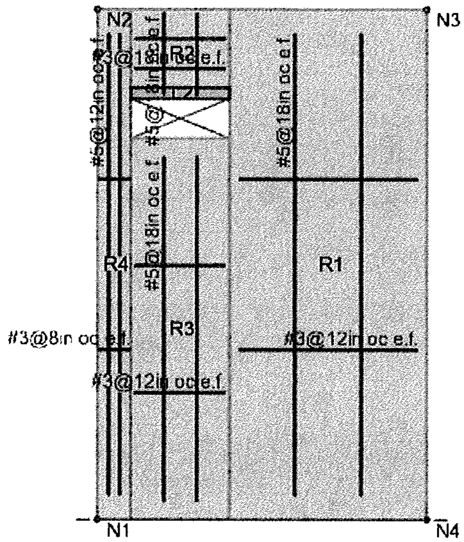
Total Height (ft):	7
Total Length (ft):	17.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.1039	PASS
Shear Details	1	-56.8435 k	323.9756 k	0.1755	PASS
Deflection Details	1		0.0062 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

Geometry:

Total Height (ft):	38.83	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	25	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

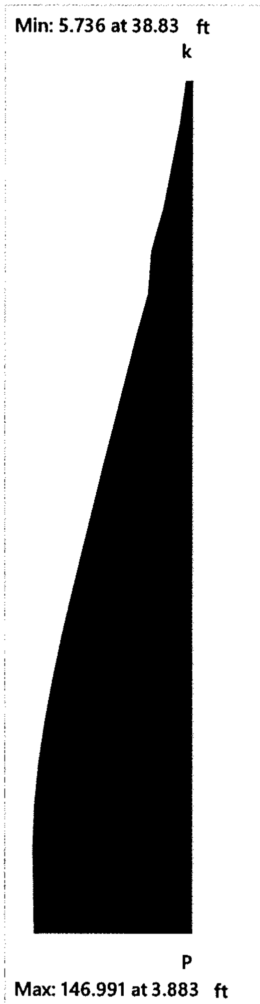
Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.3964**	PASS
R1	1			0.1185	PASS
R2	1			0.0523	PASS
R3	1			0.0615	PASS
R4	1			0.3964	PASS
UC Shear In-Plane				**0.3584**	PASS
R1	1	0 k	324.4047 k	0.1918	PASS
R2	1	0 k	156.3167 k	0.1535	PASS
R3	1	0 k	176.1971 k	0.1229	PASS
R4	1	0 k	56.3106 k	0.3584	PASS
Delta Max In-Plane					

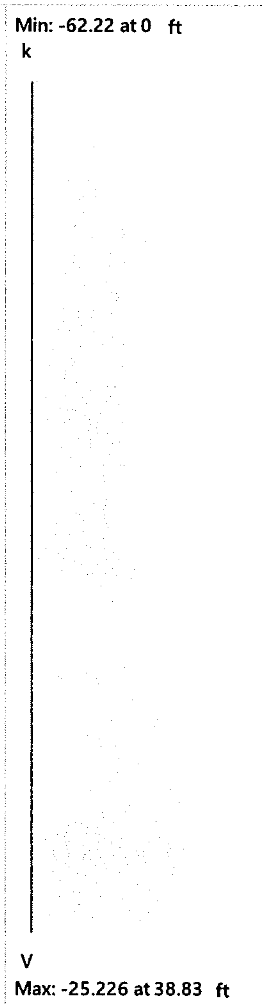
Wall Reinforcement

Region Design

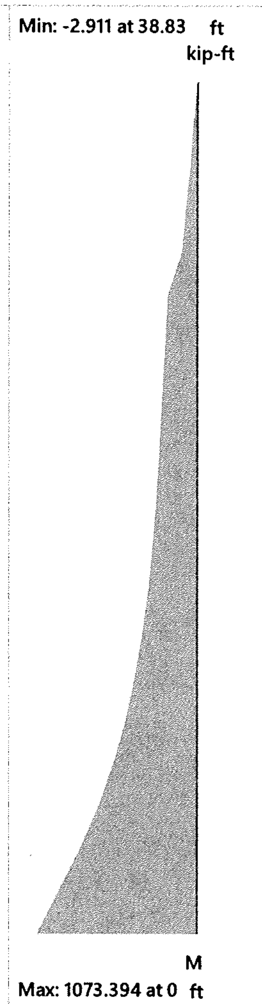
		Result
Region R1 (In-Plane)	0.1918	PASS



Axial Diagram



Shear Diagram



Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	12
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

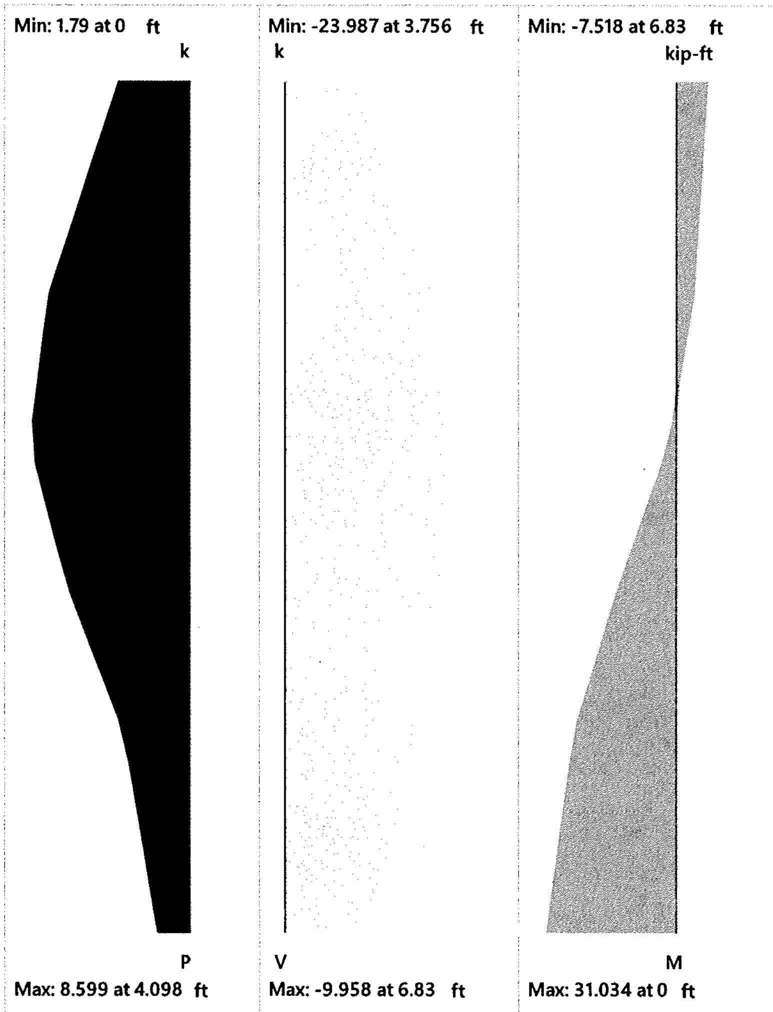
Geometry

Total Height (ft):	38.83
Total Length (ft):	15
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.1185	PASS
Shear Details	1	-62.2196 k	324.4047 k	0.1918	PASS
Deflection Details	1		0.0783 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R2 (In-Plane) 0.1535 PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	18
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

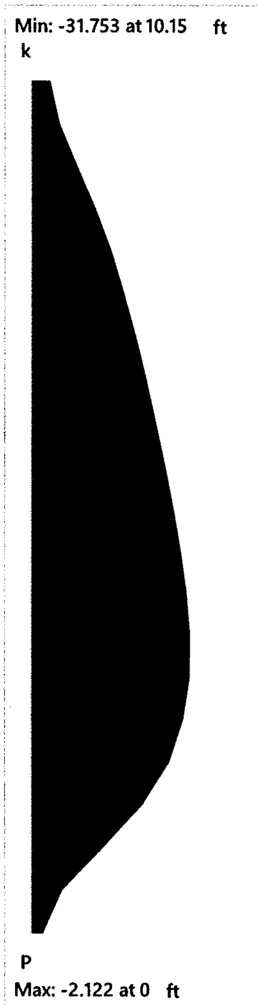
Geometry

Total Height (ft):	6.83
Total Length (ft):	7.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

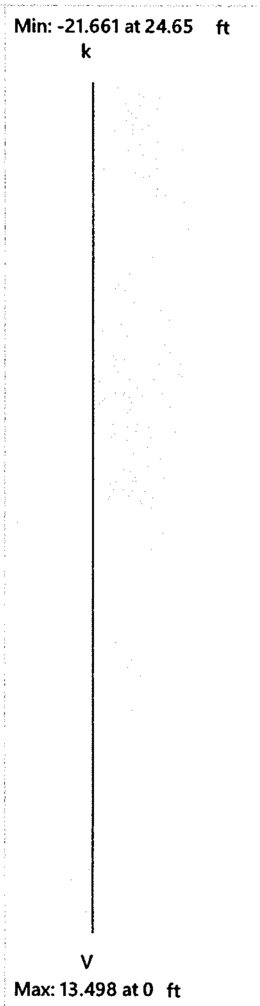
Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.0523	PASS
Shear Details	1	-23.987 k	156.3167 k	0.1535	PASS
Deflection Details	1		0.0164 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

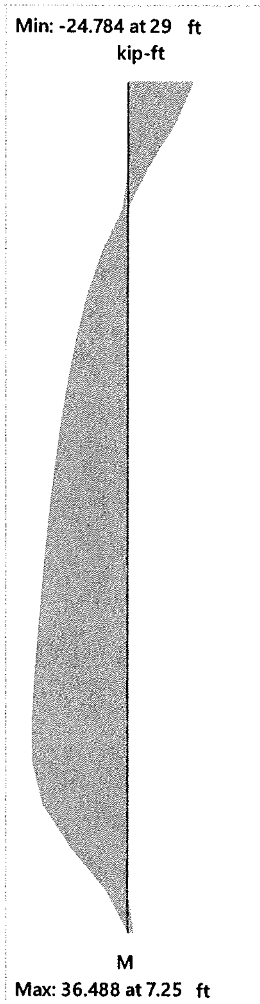
Region R3 (In-Plane) 0.1229 PASS



Axial Diagram



Shear Diagram



Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	12
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

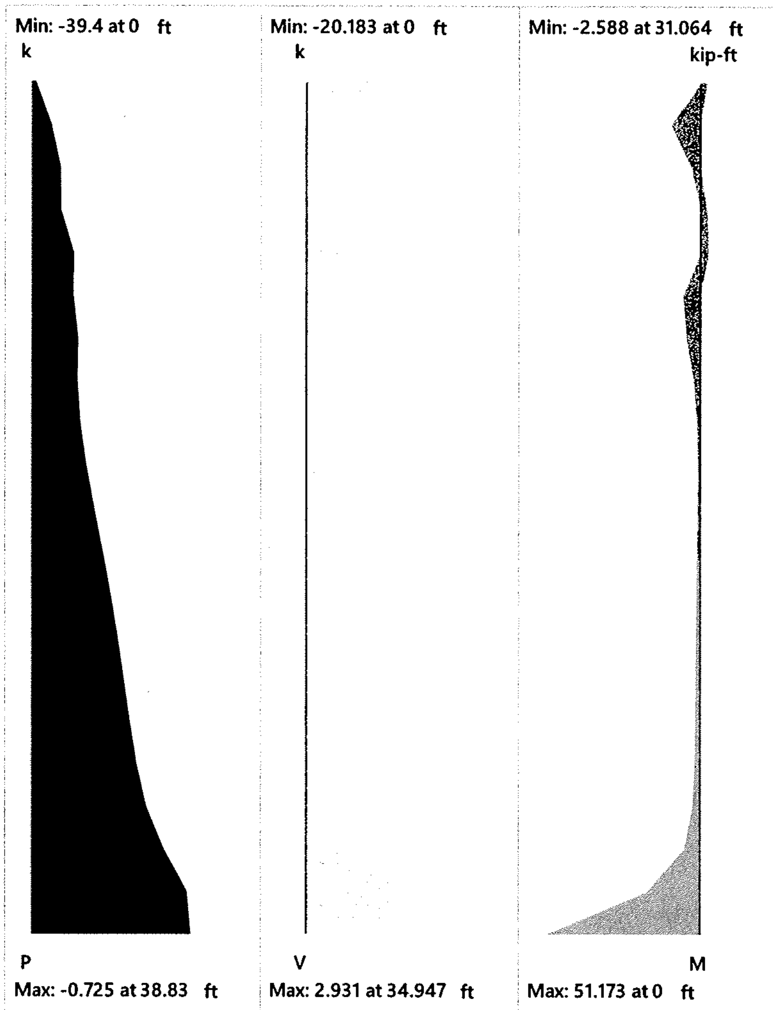
Geometry

Total Height (ft):	29
Total Length (ft):	7.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.0615	PASS
Shear Details	1	-21.6609 k	176.1971 k	0.1229	PASS
Deflection Details	1		0.0532 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R4 (In-Plane) 0.3964 PASS



Axial Diagram   Shear Diagram   Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	12
Horz Bar Spac (in):	8
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

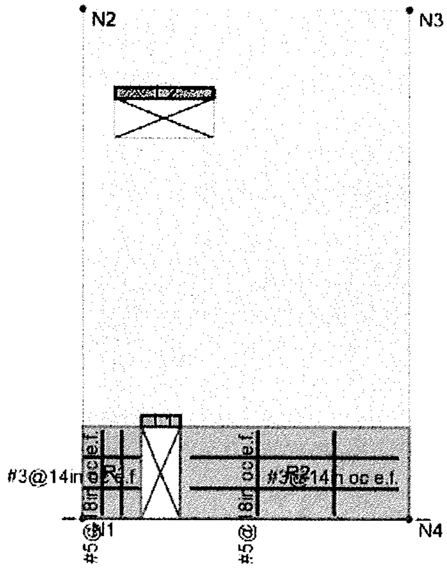
Total Height (ft):	38.83
Total Length (ft):	2.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.3964	PASS
Shear Details	1	-20.1829 k	56.3106 k	0.3584	PASS
Deflection Details	1		0.0809 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft^3):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

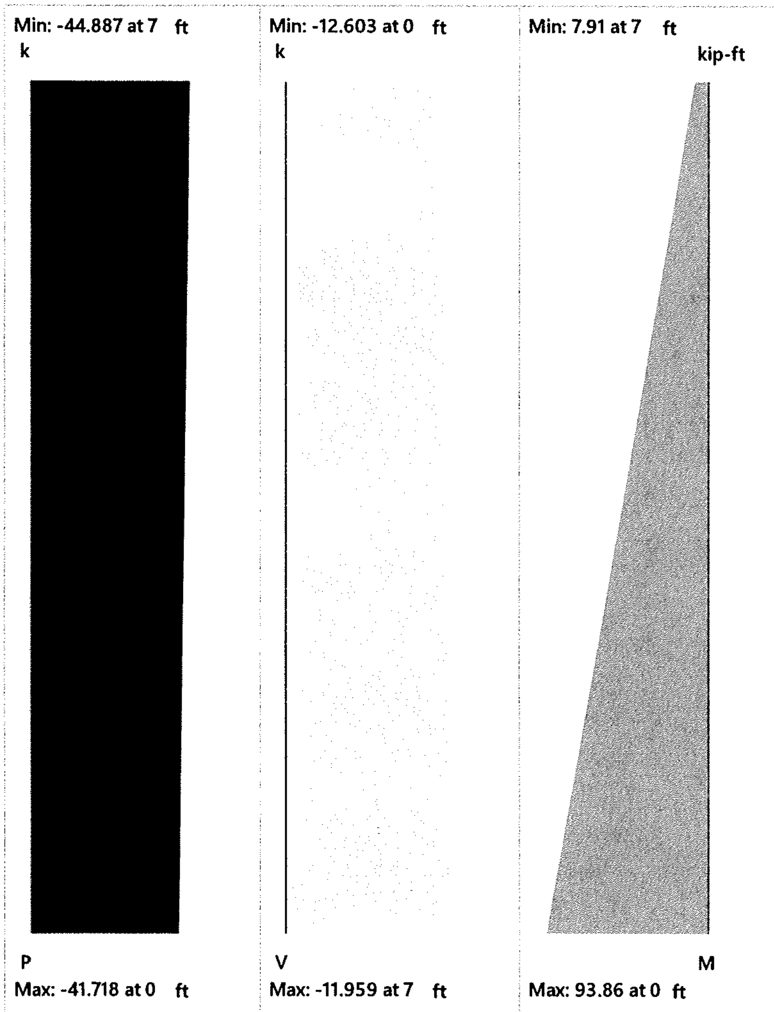
Geometry:

Total Height (ft):	38.83	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	25	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.4516**	**PASS**
R1	1			0.4516	PASS
R2	1			0.1031	PASS
UC Shear In-Plane				**0.2033**	**PASS**
R1	1	0 k	61.9896 k	0.2033	PASS
R2	1	0 k	321.42 k	0.1738	PASS
Delta Max In-Plane					
R1	1	0.0079 in			
R2	1	0.0062 in			

Region Design		Result
Region R1 (In-Plane)	0.4516	PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

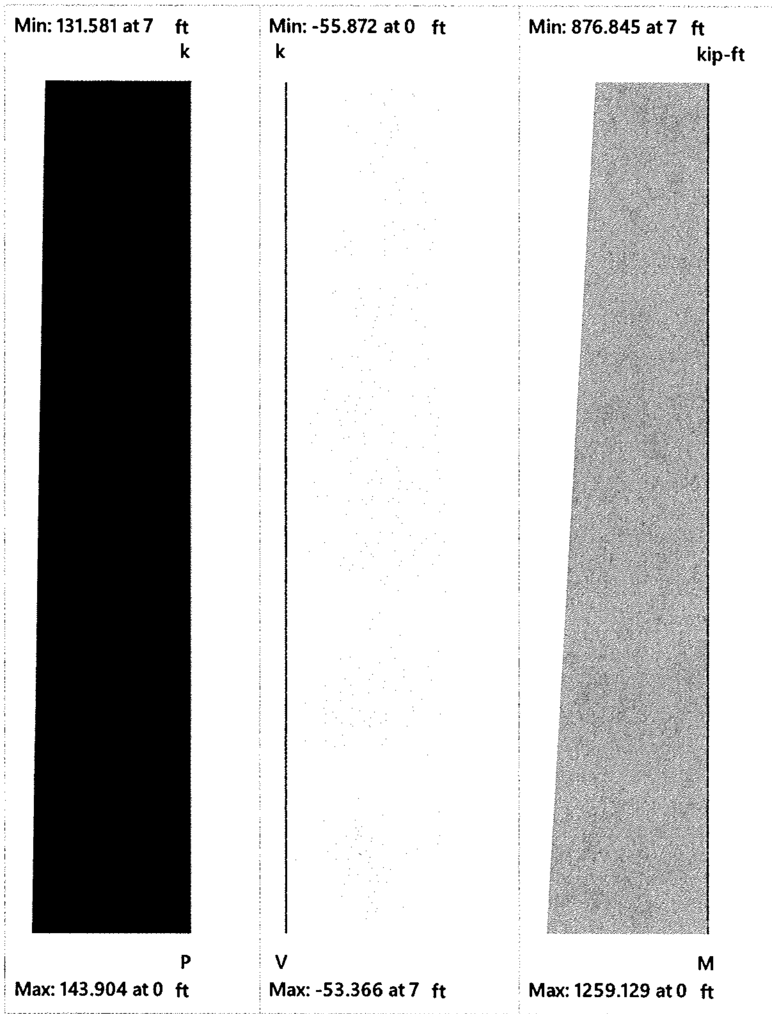
Geometry

Total Height (ft):	7
Total Length (ft):	4.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.4516	PASS
Shear Details	1	-12.6034 k	61.9896 k	0.2033	PASS
Deflection Details	1		0.0079 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R2 (In-Plane) 0.1738 PASS



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

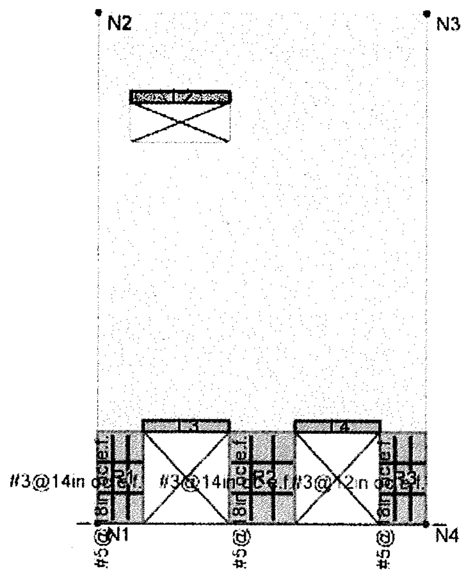
Total Height (ft):	7
Total Length (ft):	17.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.1031	PASS
Shear Details	1	-55.8717 k	321.42 k	0.1738	PASS
Deflection Details	1		0.0062 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

Geometry:

Total Height (ft):	38.83	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	25	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.402**	PASS
R1	1			0.402	PASS
R2	1			0.1387	PASS
R3	1			0.3951	PASS
UC Shear In-Plane				**0.5894**	PASS
R1	1	0 k	51.4881 k	0.2587	PASS
R2	1	0 k	78.0934 k	0.0755	PASS
R3	1	0 k	72.3018 k	0.5894	PASS
Delta Max In-Plane					
R1	1	0.0152 in			
R2	1	0.0164 in			
R3	1	0.0134 in			

Delta Max Out-of-Plane

Wall Reinforcement

Region Design		Result
Region R1 (In-Plane)	0.402	PASS

Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

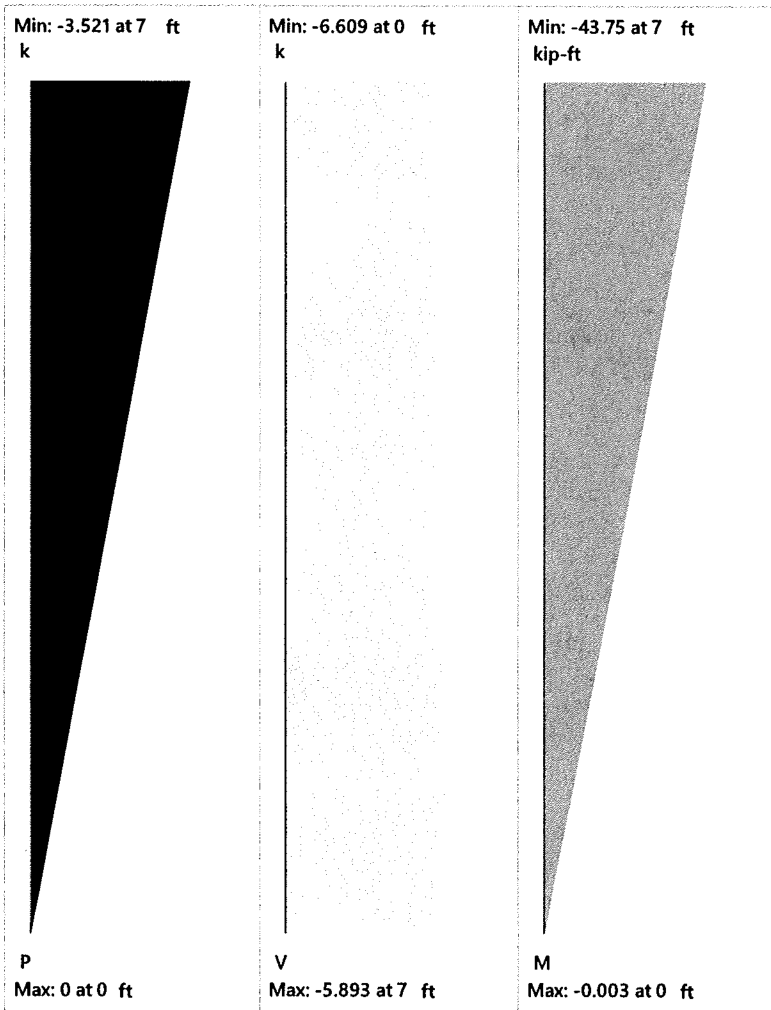
Geometry

Total Height (ft):	7
Total Length (ft):	3.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.402	PASS
Shear Details	1	-13.3189 k	51.4881 k	0.2587	PASS
Deflection Details	1		0.0152 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R2 (In-Plane) 0.1387 PASS



Axial Diagram   Shear Diagram   Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	14
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

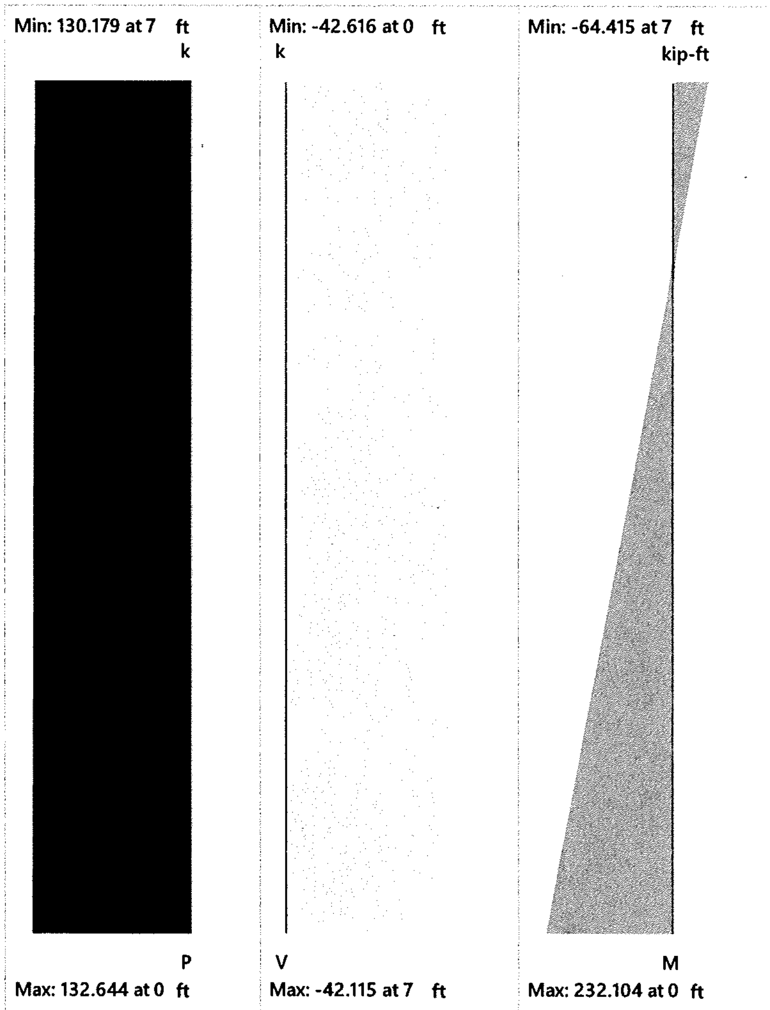
Geometry

Total Height (ft):	7
Total Length (ft):	5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.1387	PASS
Shear Details	1	-5.8927 k	78.0934 k	0.0755	PASS
Deflection Details	1		0.0164 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R3 (In-Plane) **0.5894** **PASS**



Axial Diagram Shear Diagram Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	12
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

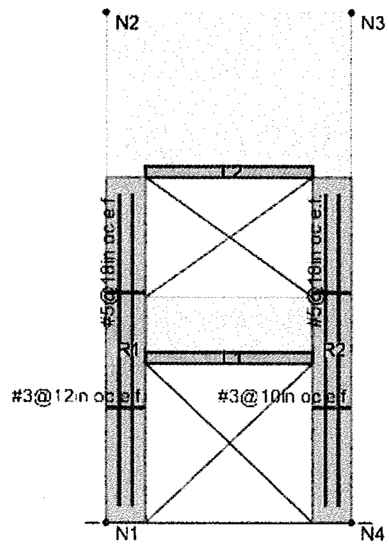
Total Height (ft):	7
Total Length (ft):	3.5
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.3951	PASS
Shear Details	1	-42.6157 k	72.3018 k	0.5894	PASS
Deflection Details	1		0.0134 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1

Enveloped Results



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft^3):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

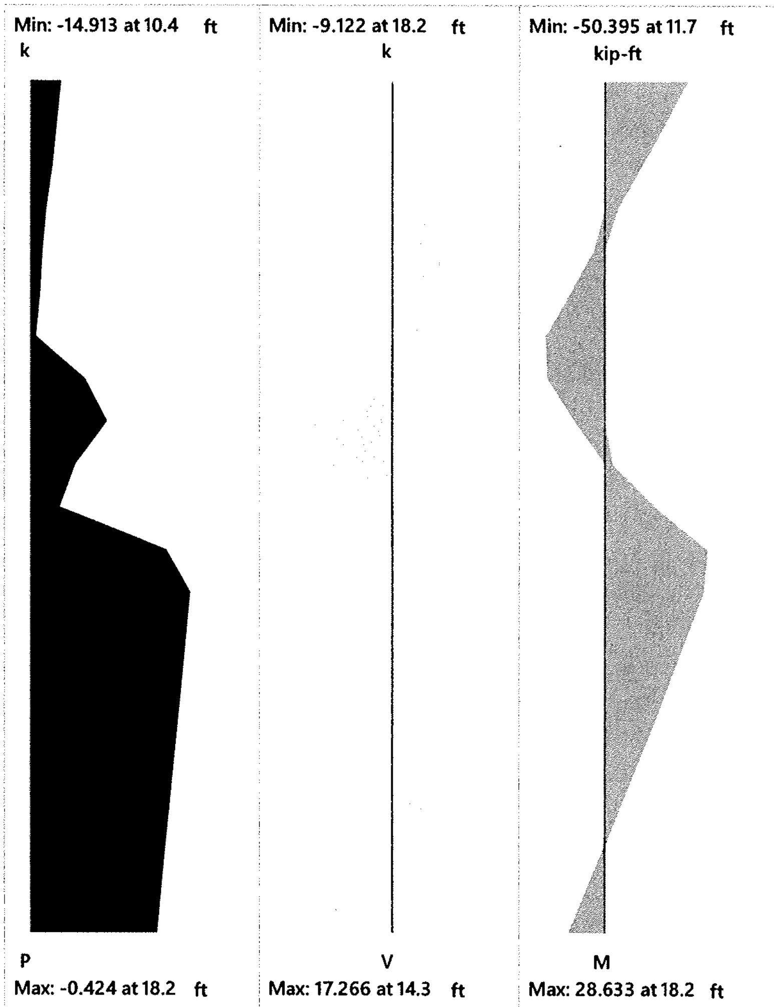
Geometry:

Total Height (ft):	38.44	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	18.5	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				**0.5404**	**PASS**
R1	1			0.5404	PASS
R2	1			0.2355	PASS
UC Shear In-Plane				**0.2856**	**PASS**
R1	1	17.2664 k	70.4788 k	0.245	PASS
R2	1	0 k	52.8837 k	0.2856	PASS
Delta Max In-Plane					
R1	1	0.1005 in			
R2	1	0.1009 in			

Region Design		Result
Region R1 (In-Plane)	0.5404	PASS



Axial Diagram   Shear Diagram   Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	12
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

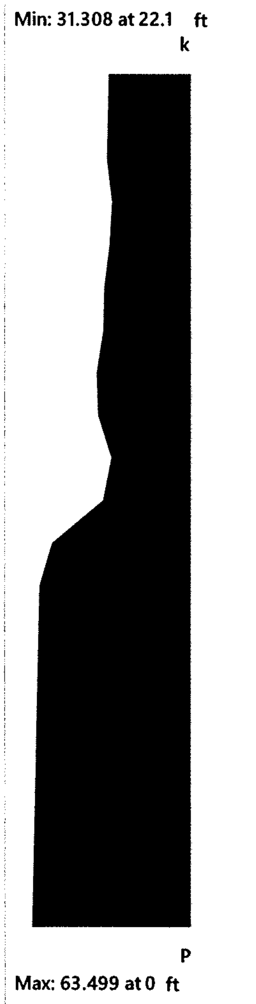
Geometry

Total Height (ft):	26
Total Length (ft):	3
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

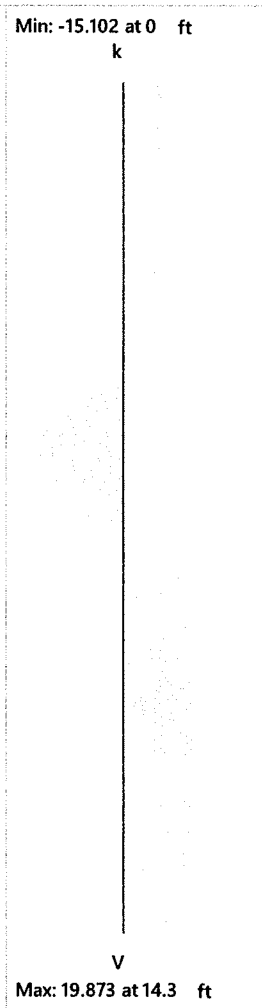
Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.5404	PASS
Shear Details	1	17.2664 k	70.4788 k	0.245	PASS
Deflection Details	1		0.1005 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

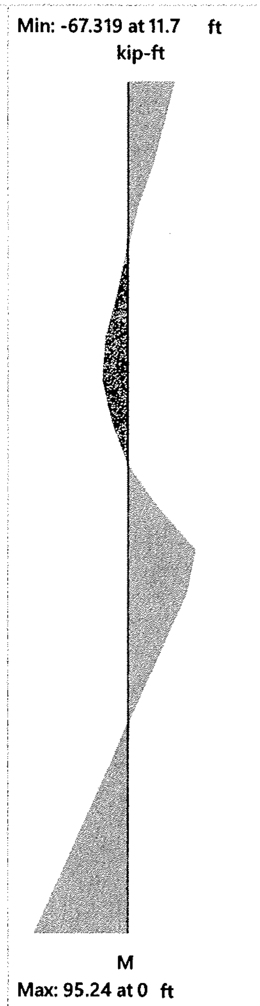
Region R2 (In-Plane) **0.2856** **PASS**



Axial Diagram



Shear Diagram



Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	10
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

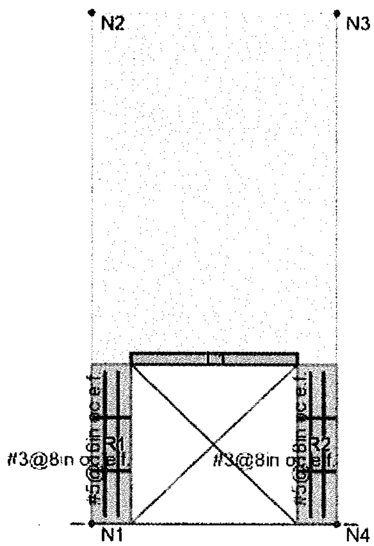
Geometry

Total Height (ft):	26
Total Length (ft):	3
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.2355	PASS
Shear Details	1	-15.1023 k	52.8837 k	0.2856	PASS
Deflection Details	1		0.1009 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Detail Report: WP1 **Enveloped Results**



Input Data:

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Transfer In?:	No
Transfer Out?:	No
Group Wall?:	No

Material Properties:

Material Set:	Conc5000NW	Conc Density (k/ft³):	0.145	Vert Bar Fy (ksi):	60
Concrete f'c (ksi):	5	Lambda:	1	Horz Bar Fy (ksi):	60
Concrete E (ksi):	4074	Conc Str Blk:	Rectangular	Steel E (ksi):	29000
Concrete G (ksi):	1771				

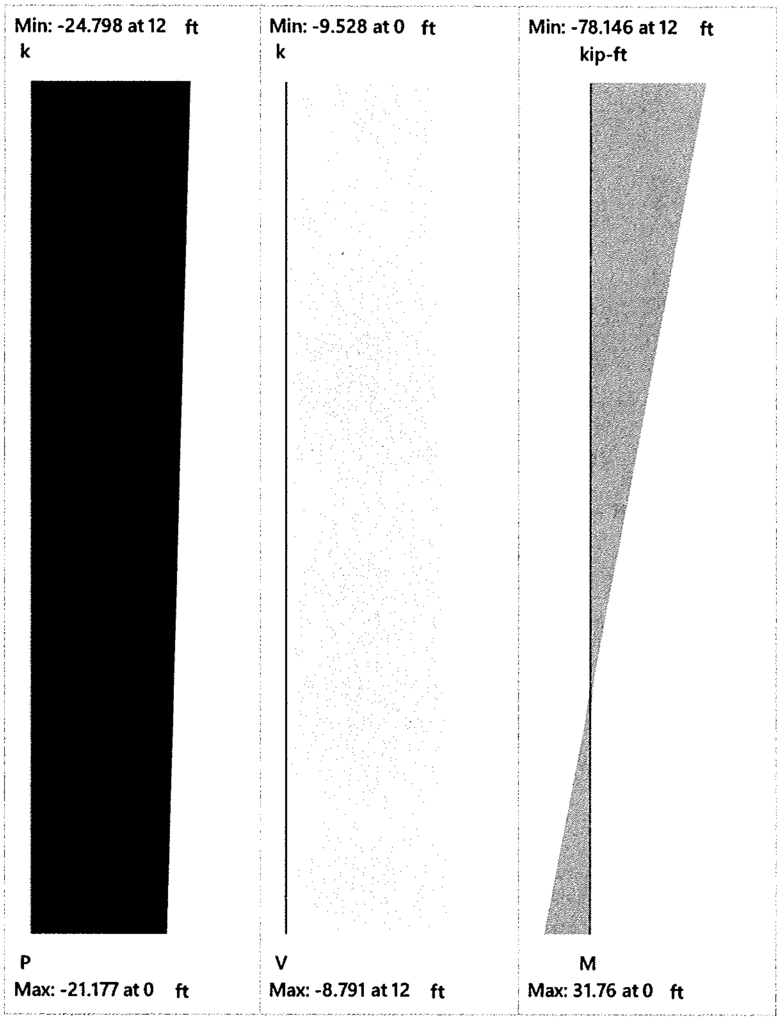
Geometry:

Total Height (ft):	38.44	Int Cover (-z, in):	1	Use Cracked ?:	Yes
Total Length (ft):	18.5	Ext Cover (+z, in):	1	In Icr Factor:	0.7
Thickness (in):	9.25	Cover Open/Edge (in):	2	Out Icr Factor:	0.35
K:	1				

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
UC Max In-Plane				0.3123	PASS
R1	1			0.2495	PASS
R2	1			0.3123	PASS
UC Shear In-Plane				0.3301	PASS
R1	1	0 k	50.2235 k	0.175	PASS
R2	1	0 k	62.1499 k	0.3301	PASS
Delta Max In-Plane					
R1	1	0.0807 in			
R2	1	0.0798 in			

Region Design		Result
Region R1 (In-Plane)	0.2495	PASS



Axial Diagram   Shear Diagram   Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	16
Horz Bar Spac (in):	8
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft³):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

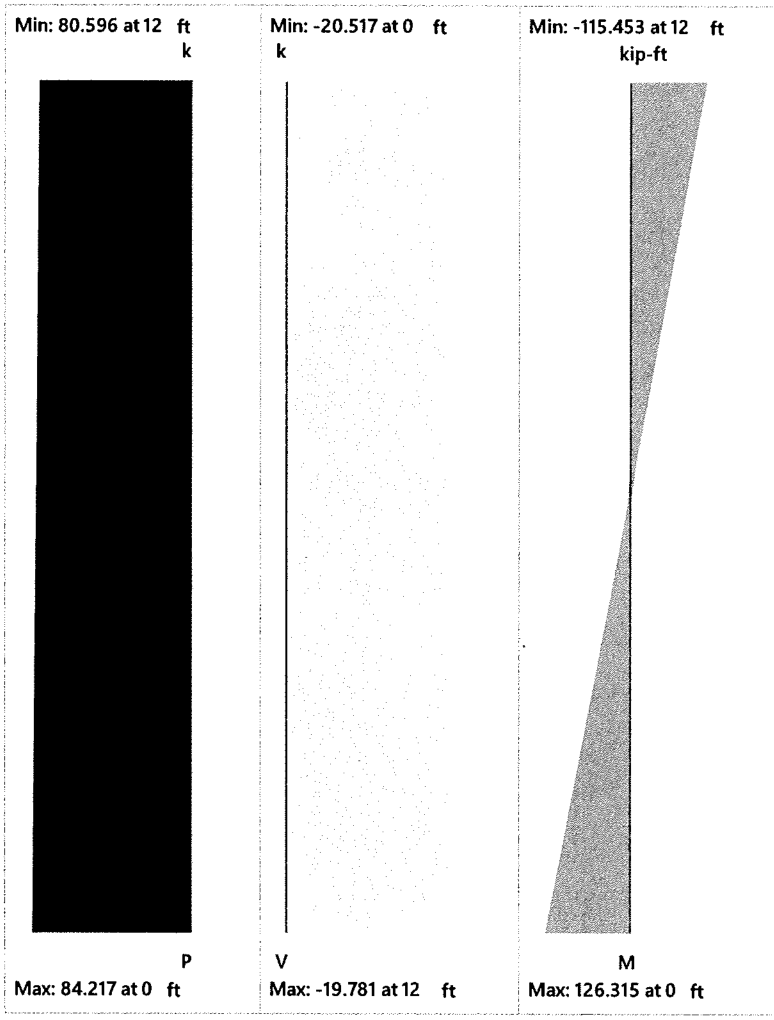
Geometry

Total Height (ft):	12
Total Length (ft):	3
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.2495	PASS
Shear Details	1	-8.7914 k	50.2235 k	0.175	PASS
Deflection Details	1		0.0807 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Region R2 (In-Plane) 0.3301 PASS



Axial Diagram · Shear Diagram · Moment Diagram

Region Criteria

Code:	ACI 318-14
Design Rule:	R2
Seismic Rule:	None
Loc of r/f:	Each Face
Outer Bars:	Horizontal
Vert Bar Size:	#5
Horz Bar Size:	#3
Vert Bar Spac (in):	18
Horz Bar Spac (in):	8
Group Wall?:	No

Materials

Material Set:	Conc5000NW
Concrete f'c (ksi):	5
Concrete E (ksi):	4074
Concrete G (ksi):	1771
Conc Density (k/ft^3):	0.145
Lambda:	1
Conc Str Blk:	Rectangular
Vert Bar Fy (ksi):	60
Horz Bar Fy (ksi):	60
Steel E (ksi):	29000

Geometry

Total Height (ft):	12
Total Length (ft):	3
Thickness (in):	9.25
Int Cover (-z, in):	1
Ext Cover (+z, in):	1
Cover Open/Edge (in):	2
K:	1
Use Cracked?:	Yes
Icr Factor:	0.7

Code Check:

Limit State	Gov. LC	Required	Available	Unity Check	Result
Axial/Bending Details	1			0.3123	PASS
Shear Details	1	-20.5173 k	62.1499 k	0.3301	PASS
Deflection Details	1		0.0798 in		
Slender Bending Span Results					
Wall Segment Section Properties					
Reinforcement Details					
In-Plane Wall Interaction Diagram					
Cross Section Detailing					

Project
Subject
With/To
Address
Date

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Phone
Fax #
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By

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☐ Calculations
☐ Fax
☐ Memorandum
☐ Meeting Minutes
☐ Telephone Memo

AHBL

Civil Engineers
Structural Engineers
Landscape Architects
Community Planners
Land Surveyors

Foundation Design

Exterior Wall

$P = 150(9.25/12)(43') + (12psf + 25psf)(63.5'/2) = 6147\, plf$

Req'd ftg Width = 6147 plf / 2500 psf = 2.46'

Use 3'-0" x 12" Ftg min w/ (3) #5 longitudinal

Interior Column

$P = 37psf(63.5'/2)(120'/2) + (37psf(60'/2) + 41psf(60'/2))56'/2 = 136.0^K$

See Enclosed output for design, use 8'-0" x 8'-0" x 16" w/ (8) #6 Ea. Way.

Column Design

$P = 136.0^K$
$Ht = 42.81'$

Use HSS 12x12 x 1/4 - See Enclosed output

Project ______________________ Project No. ______________________ ☐ Page _____ of _____
Subject ______________________ Phone ______________________ ☐ Calculations
With/To ______________________ Fax # ______________________ ☐ Fax
Address ______________________ # Faxed Pages ______________________ ☐ Memorandum
Date ______________________ By ______________________ ☐ Meeting Minutes
☐ Telephone Memo

AHBL
Civil Engineers
Structural Engineers
Landscape Architects
Community Planners
Land Surveyors

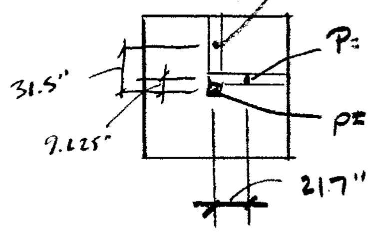
Node Col Ftgs

Grid B & 2.5

$P = 150\left(\frac{9.25}{12}\right)(40.1')(2.75') + (12+25)\frac{6.75}{2}(2.75)$
$= 13.1^k$ (12.9^k DL, $.20^k$ SL)

$P = 150\left(\frac{9.25}{12}\right)(34.77')(4.39') + (12+25)\frac{60}{2}(4.39') = 22.5^k$
(19.2^k DL, 3.3^k SL)

$P = (12\text{psf} + 25\text{psf})\left(\frac{60+20}{2}\right)\frac{56}{2} = 41.44^k$
(13.44^k DL, 28^k SL)

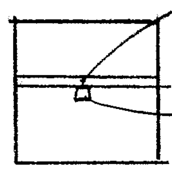


See Computer Output, Use 8x8x16" ftg
w/ (8) #6 Ea. Way

Grid B & 1

$P = 150(9.25/12)(38.44')(8') + (12+25)\left(\frac{63.5}{2}\right)(8') = 44.96^k$
(38.61^k DL, 6.35^k SL)

$P = (12+25)\frac{65}{2}(63.5/2) = 38.2^k$
(12.4^k DL, 25.8^k SL)



See Computer Output, Use 8'x8'x16" ftg
w/ (8) #6 Ea. Way.

General Footing

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: --None--

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	No
Use ftg wt for stability, moments & shears	:	No
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	350.0 pcf
Soil/Concrete Friction Coeff.	=	0.350

Increases based on footing Depth

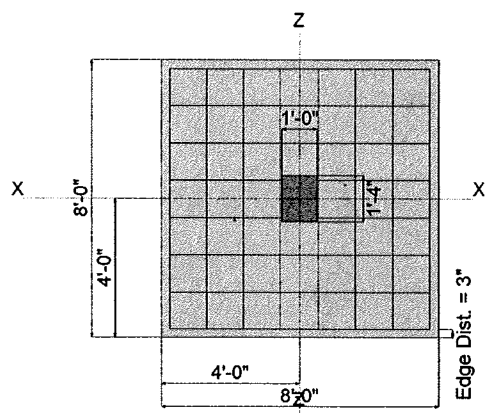
Footing base depth below soil surface	=	2.330 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	8.0 ft
Length parallel to Z-Z Axis	=	8.0 ft
Footing Thickness	=	16.0 in



Pedestal dimensions...

px : parallel to X-X Axis	=	12.0 in
pz : parallel to Z-Z Axis	=	16.0 in
Height	=	16.0 in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in

Reinforcing

Bars parallel to X-X Axis

Number of Bars	=	8
Reinforcing Bar Size	=	# 6

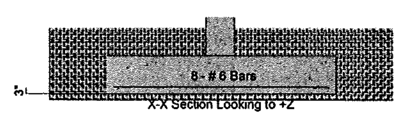
Bars parallel to Z-Z Axis

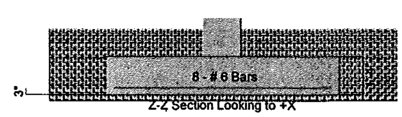
Number of Bars	=	8
Reinforcing Bar Size	=	# 6

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation n/a

# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	46.380			89.630				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC

DESCRIPTION: --None--

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8932	Soil Bearing	2.233 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.7648	Z Flexure (+X)	19.034 k-ft/ft	24.886 k-ft/ft	+1.20D+1.60S
PASS	0.7648	Z Flexure (-X)	19.034 k-ft/ft	24.886 k-ft/ft	+1.20D+1.60S
PASS	0.6937	X Flexure (+Z)	17.264 k-ft/ft	24.886 k-ft/ft	+1.20D+1.60S
PASS	0.6937	X Flexure (-Z)	17.264 k-ft/ft	24.886 k-ft/ft	+1.20D+1.60S
PASS	0.5819	1-way Shear (+X)	47.810 psi	82.158 psi	+1.20D+1.60S
PASS	0.5819	1-way Shear (-X)	47.810 psi	82.158 psi	+1.20D+1.60S
PASS	0.5431	1-way Shear (+Z)	44.622 psi	82.158 psi	+1.20D+1.60S
PASS	0.5431	1-way Shear (-Z)	44.622 psi	82.158 psi	+1.20D+1.60S
PASS	0.7949	2-way Punching	130.609 psi	164.317 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.8320	0.8320	n/a	n/a	0.333
X-X, +D+S	2.50	n/a	0.0	2.233	2.233	n/a	n/a	0.893
X-X, +D+0.750S	2.50	n/a	0.0	1.882	1.882	n/a	n/a	0.753
X-X, +0.60D	2.50	n/a	0.0	0.4992	0.4992	n/a	n/a	0.200
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.8320	0.8320	0.333
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.233	2.233	0.893
Z-Z, +D+0.750S	2.50	0.0	n/a	n/a	n/a	1.882	1.882	0.753
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.4992	0.4992	0.200

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	5.618	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.40D	5.618	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D	4.816	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D	4.816	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.50S	8.706	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.50S	8.706	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+1.60S	17.264	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+1.60S	17.264	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.20S	6.372	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.20S	6.372	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +0.90D	3.612	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +0.90D	3.612	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.40D	6.194	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.40D	6.194	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D	5.309	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D	5.309	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.50S	9.598	-X	Bottom	0.3456	AsMin	0.440	24.886	OK

General Footing

Project File: Panattoni Tumwater.ec6

DESCRIPTION: --None--

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	9.598	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+1.60S	19.034	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+1.60S	19.034	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.20S	7.025	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.20S	7.025	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +0.90D	3.982	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +0.90D	3.982	+X	Bottom	0.3456	AsMin	0.440	24.886	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	15.56 psi	15.56 psi	14.52 psi	14.52 psi	15.56 psi	82.16 psi	0.19	OK
+1.20D	13.34 psi	13.34 psi	12.45 psi	12.45 psi	13.34 psi	82.16 psi	0.16	OK
+1.20D+0.50S	24.11 psi	24.11 psi	22.50 psi	22.50 psi	24.11 psi	82.16 psi	0.29	OK
+1.20D+1.60S	47.81 psi	47.81 psi	44.62 psi	44.62 psi	47.81 psi	82.16 psi	0.58	OK
+1.20D+0.20S	17.65 psi	17.65 psi	16.47 psi	16.47 psi	17.65 psi	82.16 psi	0.21	OK
+0.90D	10.00 psi	10.00 psi	9.34 psi	9.34 psi	10.00 psi	82.16 psi	0.12	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	42.51 psi	164.32psi	0.2587	OK
+1.20D	36.43 psi	164.32psi	0.2217	OK
+1.20D+0.50S	65.86 psi	164.32psi	0.4008	OK
+1.20D+1.60S	130.61 psi	164.32psi	0.7949	OK
+1.20D+0.20S	48.21 psi	164.32psi	0.2934	OK
+0.90D	27.33 psi	164.32psi	0.1663	OK

Steel Column

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Typ Column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name : **HSS12x12x1/4**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 50 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 42.81 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 42.81 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 42.81 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,688.0 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 42.810 ft, D = 46.380, S = 89.630 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9874** : 1	
Load Combination	+D+S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	137.698	k
Pn / Omega : Allowabl	139.454	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	86.793	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	86.793	k-ft
PASS Maximum Shear Stress Rati	**0.0** : 1	
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

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

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

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.345	PASS	0.00 ft	1.00	1.00	107.25	107.25	0.000	PASS	0.00 ft
+D+S	0.987	PASS	0.00 ft	1.00	1.00	107.25	107.25	0.000	PASS	0.00 ft
+D+0.750S	0.827	PASS	0.00 ft	1.00	1.00	107.25	107.25	0.000	PASS	0.00 ft
+0.60D	0.207	PASS	0.00 ft	1.00	1.00	107.25	107.25	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments (k-ft) @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
D Only	48.068								
+D+S	137.698								
+D+0.750S	115.290								
+0.60D	28.841								
S Only	89.630								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments (k-ft) @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
Axial @ Base	Maximum	137.698								
"	Minimum	28.841								

Steel Column

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Typ Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Reaction, X-X Axis Base	Maximum	48.068								
"	Minimum	48.068								
Reaction, Y-Y Axis Base	Maximum	48.068								
"	Minimum	48.068								
Reaction, X-X Axis Top	Maximum	48.068								
"	Minimum	48.068								
Reaction, Y-Y Axis Top	Maximum	48.068								
"	Minimum	48.068								
Moment, X-X Axis Base	Maximum	48.068								
"	Minimum	48.068								
Moment, Y-Y Axis Base	Maximum	48.068								
"	Minimum	48.068								
Moment, X-X Axis Top	Maximum	48.068								
"	Minimum	48.068								
Moment, Y-Y Axis Top	Maximum	48.068								
"	Minimum	48.068								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS12x12x1/4

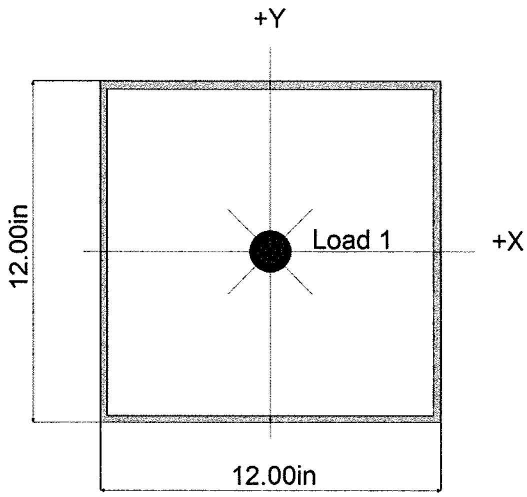
Depth	=	12.000 in	I xx	=	248.00 in^4	J	=	384.000 in^4
Design Thick	=	0.233 in	S xx	=	41.40 in^3			
Width	=	12.000 in	R xx	=	4.790 in			
Wall Thick	=	0.250 in	Zx	=	47.600 in^3			
Area	=	10.800 in^2	I yy	=	248.000 in^4	C	=	64.500 in^3
Weight	=	39.430 plf	S yy	=	41.400 in^3			
			R yy	=	4.790 in			
Ycg	=	0.000 in						

Steel Column

Project File: Panattoni Tumwater.ec6

DESCRIPTION: Typ Column

Sketches



Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Node footing at B and 2.5

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	No
Use ftg wt for stability, moments & shears	:	No
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	350.0 pcf
Soil/Concrete Friction Coeff.	=	0.350

Increases based on footing Depth

Footing base depth below soil surface	=	2.330 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	8.0 ft
Length parallel to Z-Z Axis	=	8.0 ft
Footing Thickness	=	16.0 in

Load location offset from footing center...

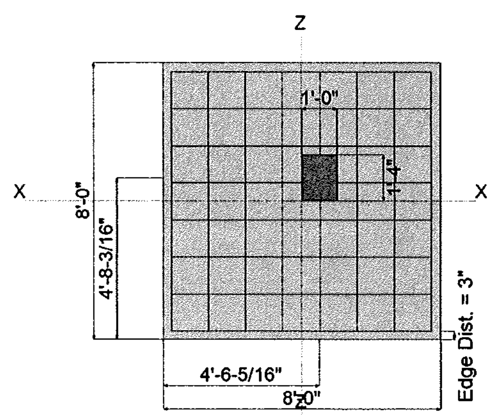
ex : Prll to X-X Axis	=	6.34 in
ez : Prll to Z-Z Axis	=	8.17 in

Pedestal dimensions...

px : parallel to X-X Axis	=	12.0 in
pz : parallel to Z-Z Axis	=	16.0 in
Height	=	16.0 in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



Reinforcing

Bars parallel to X-X Axis

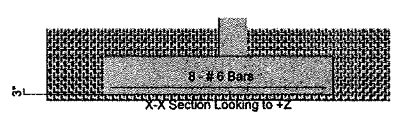
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6

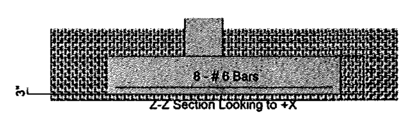
Bars parallel to Z-Z Axis

Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation

Bars required within zone
Bars required on each side of zone





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	45.540			31.50				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC

DESCRIPTION: Node footing at B and 2.5

DESIGN SUMMARY

Design N.G.

Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	Soil Bearing	ksf	ksf	
FAIL	Overturning - X-X	k-ft	k-ft	
FAIL	Overturning - Z-Z	k-ft	k-ft	
FAIL	Sliding - X-X	k	k	
FAIL	Sliding - Z-Z	k	k	
FAIL	Uplift	k	k	
PASS	Z Flexure (+X)	k-ft/ft	k-ft/ft	
PASS	Z Flexure (-X)	k-ft/ft	k-ft/ft	
PASS	X Flexure (+Z)	k-ft/ft	k-ft/ft	
PASS	X Flexure (-Z)	k-ft/ft	k-ft/ft	
PASS	1-way Shear (+X)	psi	psi	
PASS	1-way Shear (-X)	psi	psi	
PASS	1-way Shear (+Z)	psi	psi	
PASS	1-way Shear (-Z)	psi	psi	
PASS	2-way Punching	psi	psi	

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom Left	Top Left	Top Right	Bottom Right	Actual / Allow Ratio
, D Only								0.000
, 52.2 deg CCW	2.50	5.491	7.076	0.8992	0.1821	0.7386	1.456	0.582
, +D+S								0.000
, 52.2 deg CCW	2.50	5.810	7.487	1.447	0.2324	1.175	2.390	0.956
, +D+0.750S								0.000
, 52.2 deg CCW	2.50	5.755	7.416	1.310	0.2198	1.066	2.156	0.862
, +0.60D								0.000
, 52.2 deg CCW	2.50	5.491	7.076	0.5395	0.1093	0.4432	0.8734	0.349

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	6.223	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.40D	9.312	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D	5.334	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D	7.981	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.50S	6.739	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.50S	9.976	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+1.60S	9.832	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+1.60S	14.366	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.20S	5.896	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +1.20D+0.20S	8.779	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +0.90D	4.0	+Z	Bottom	0.3456	AsMin	0.440	24.886	OK
X-X, +0.90D	5.986	-Z	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.40D	9.381	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.40D	7.615	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D	8.041	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D	6.527	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.50S	10.051	-X	Bottom	0.3456	AsMin	0.440	24.886	OK

General Footing

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC

DESCRIPTION: Node footing at B and 2.5

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	8.243	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+1.60S	14.473	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+1.60S	12.019	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.20S	8.845	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +1.20D+0.20S	7.214	+X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +0.90D	6.031	-X	Bottom	0.3456	AsMin	0.440	24.886	OK
Z-Z, +0.90D	4.896	+X	Bottom	0.3456	AsMin	0.440	24.886	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	22.20 psi	21.27 psi	19.74 psi	18.25 psi	22.20 psi	82.16 psi	0.27	OK
+1.20D	19.03 psi	18.23 psi	16.92 psi	15.65 psi	19.03 psi	82.16 psi	0.23	OK
+1.20D+0.50S	23.79 psi	23.02 psi	21.07 psi	19.77 psi	23.79 psi	82.16 psi	0.29	OK
+1.20D+1.60S	34.28 psi	33.57 psi	30.22 psi	28.85 psi	34.28 psi	82.16 psi	0.42	OK
+1.20D+0.20S	20.94 psi	20.15 psi	18.58 psi	17.30 psi	20.94 psi	82.16 psi	0.25	OK
+0.90D	14.27 psi	13.67 psi	12.69 psi	11.74 psi	14.27 psi	82.16 psi	0.17	OK

Two-Way "Punching" Shear All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	47.53 psi	164.32psi	0.2893	OK
+1.20D	40.74 psi	164.32psi	0.2479	OK
+1.20D+0.50S	50.93 psi	164.32psi	0.3099	OK
+1.20D+1.60S	73.33 psi	164.32psi	0.4463	OK
+1.20D+0.20S	44.81 psi	164.32psi	0.2727	OK
+0.90D	30.56 psi	164.32psi	0.186	OK

Steel Column

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Node Column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name : **HSS8x8x5/16**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 38.44 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 38.44 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 38.44 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,223.93 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 38.440 ft, D = 12.40, S = 38.20 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.8548**	: 1
Load Combination	+D+S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	51.824	k
Pn / Omega : Allowable	60.624	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	62.625	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	62.625	k-ft
PASS Maximum Shear Stress Ratio	**0.0**	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

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

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

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.225	PASS	0.00 ft	1.00	1.00	147.37	147.37	0.000	PASS	0.00 ft
+D+S	0.855	PASS	0.00 ft	1.00	1.00	147.37	147.37	0.000	PASS	0.00 ft
+D+0.750S	0.697	PASS	0.00 ft	1.00	1.00	147.37	147.37	0.000	PASS	0.00 ft
+0.60D	0.135	PASS	0.00 ft	1.00	1.00	147.37	147.37	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
D Only	13.624								
+D+S	51.824								
+D+0.750S	42.274								
+0.60D	8.174								
S Only	38.200								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
Axial @ Base	Maximum	51.824								
"	Minimum	8.174								

Steel Column

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Node Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Reaction, X-X Axis Base	Maximum	13.624								
"	Minimum	13.624								
Reaction, Y-Y Axis Base	Maximum	13.624								
"	Minimum	13.624								
Reaction, X-X Axis Top	Maximum	13.624								
"	Minimum	13.624								
Reaction, Y-Y Axis Top	Maximum	13.624								
"	Minimum	13.624								
Moment, X-X Axis Base	Maximum	13.624								
"	Minimum	13.624								
Moment, Y-Y Axis Base	Maximum	13.624								
"	Minimum	13.624								
Moment, X-X Axis Top	Maximum	13.624								
"	Minimum	13.624								
Moment, Y-Y Axis Top	Maximum	13.624								
"	Minimum	13.624								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS8x8x5/16

Depth	=	8.000 in	I xx	=	85.60 in^4	J	=	136.000 in^4
Design Thick	=	0.291 in	S xx	=	21.40 in^3			
Width	=	8.000 in	R xx	=	3.130 in			
Wall Thick	=	0.313 in	Zx	=	25.100 in^3			
Area	=	8.760 in^2	I yy	=	85.600 in^4	C	=	34.500 in^3
Weight	=	31.840 plf	S yy	=	21.400 in^3			
			R yy	=	3.130 in			
Ycg	=	0.000 in						

Project Title:
Engineer:
Project ID:
Project Descr:

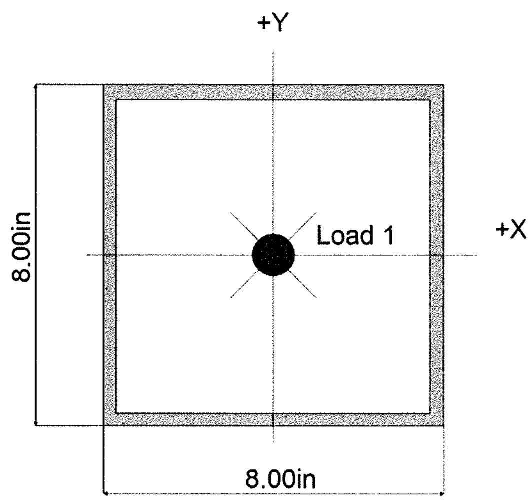
Steel Column

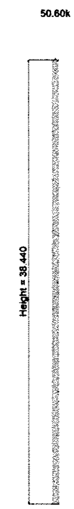
Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

DESCRIPTION: Node Column

Sketches



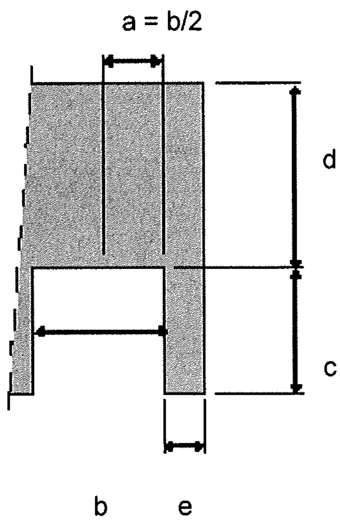


Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 3
Wall Description = 11 ft opening

Wall Ht =	38.44	ft
b =	11	ft
c =	21	ft
e =	3.00	ft
d =	17.44	ft
a =	5.5	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	4922	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	55	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	935	plf
equiv SL =	1947.9167	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	45.3	psf
P seismic equiv =	80.3	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC

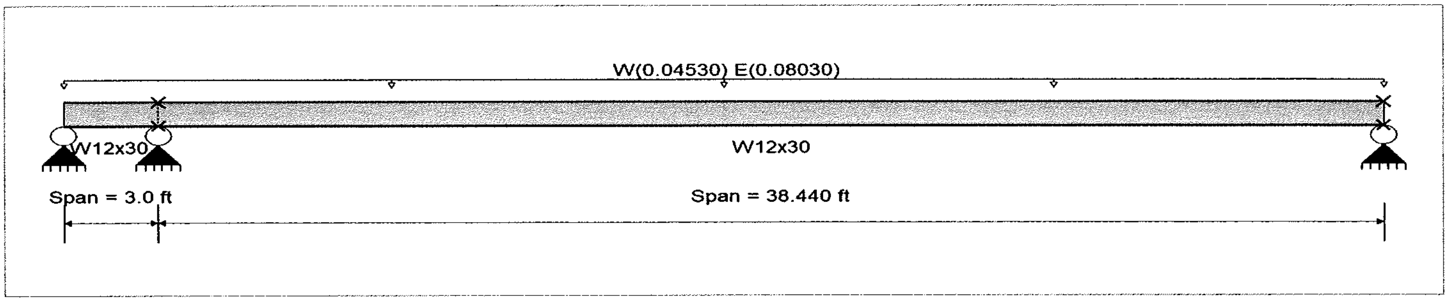
DESCRIPTION: Panel 3 Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04530, E = 0.08030 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.085** : 1	Maximum Shear Stress Ratio =	**0.049** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	13.765 k-ft	Vu : Applied	4.709 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.261 in	Ratio =	1,767	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in	Ratio =	18,188	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.183 in	Ratio =	2525	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.001 in	Ratio =	25984	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.024	0.014		-3.88	3.88	179.58	161.63	1.00	1.00	1.33	95.94	95.94
Dsgn. L = 38.44 ft	2	0.024	0.006	2.47	-3.88	3.88	179.58	161.63	1.00	1.00	0.54	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.048	0.028		-7.77	7.77	179.58	161.63	1.00	1.00	2.66	95.94	95.94
Dsgn. L = 38.44 ft	2	0.048	0.011	4.93	-7.77	7.77	179.58	161.63	1.00	1.00	1.07	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.085	0.049		-13.76	13.76	179.58	161.63	1.00	1.00	4.71	95.94	95.94
Dsgn. L = 38.44 ft	2	0.085	0.020	8.75	-13.76	13.76	179.58	161.63	1.00	1.00	1.90	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

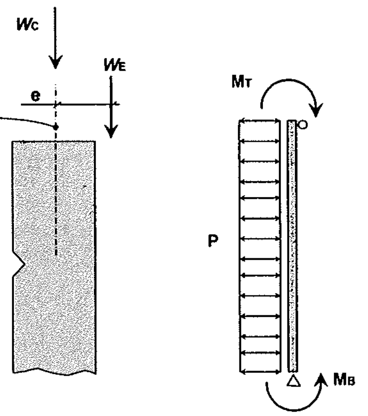
AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	3
Wall Description =	11 ft opening

See ACI 11.8.2.2 for distribution of concentrated forces

℄ of structural thickness



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	42.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	3.00
Number of Bars Ea Face (or at Center) of Pier	6.99
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 5" o.c.
Max Deflection	L / 680
% of Flexural Capacity	75%

Hand Input	**OK**
Potential Hand Input	
Output	

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds
f_1 =	0.5
f_2 =	0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	4922	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = We * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 Mtop	
Eccentricity (in)	6.75				
Dead - D (plf)	935	D =	526	D =	263
Snow - S (plf)	1947.916667	S =	1096	S =	548
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(Mtop)	(Mbot)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (Mtop + Mbot)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-13765	E =	-6883
Wind - W (lb-ft/ft)	0	-7765	W =	-3883

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	80.3	E =	14837
Wind - W (psf)	45.3	W =	8373

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	8542
Snow - S (plf)	1948
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	263
Snow - S (lb-ft/ft)	548
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	7954
Wind - W (lb-ft/ft)	4491

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	42.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	5.15	OK
As per foot (in^2/ft)	0.71	(This is the area of tension steel only)
Total As in Pier (in^2)	0.71	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.33	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρ_v min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0129	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0089	= As per ft / (12 * d)
$\rho max = 0.6\ \rho b = 0.6 * 0.85 * \beta_1 * fc / fy * 87000 / (87000 + fy)$	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓ w (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓ c (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓ c / 150 (in)	3.0752	

OK

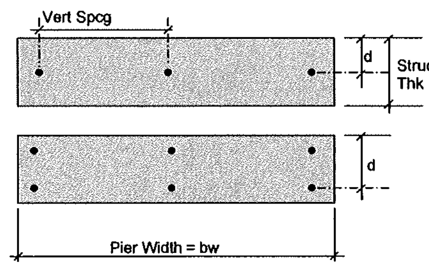
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 3
Wall Description = 11 ft opening

IBC-2015	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	11959	11224	13367	13367	11224	13174	7688	6128	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	4418	7074	14306	41250	60963	104417	56729	97716	
Pu / Ag (psi)	117	110	131	131	110	129	75	60	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.84	0.83	0.86	0.86	0.83	0.85	0.80	0.78	

a (in) =(Ase*fy) / (0.85*f'c*lw)	0.99	0.98	1.01	1.01	0.98	1.01	0.94	0.92	
$C_U = C_{ULTIMATE} = a / \beta_1$	1.24	1.23	1.26	1.26	1.23	1.26	1.17	1.15	
$I_{cr\,U}$ (in4) = $I_{cr\,ULTIMATE}$ = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	187.42	186.30	189.55	189.55	186.30	189.26	180.78	178.28	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	8302	12672	29622	85415	109204	213255	82432	130643	
Mn (lb-in) = Ase * fy * (d - a/2)	312670	310009	317762	317762	310009	317064	297134	291424	
Cu / d	0.19	0.18	0.19	0.19	0.18	0.19	0.18	0.17	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	281403	279008	285985	285985	279008	285357	267420	262281	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	3%	5%	10%	30%	39%	75%	31%	50%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	10490	8542	8542	9516	10490	10490	
Applied Moment at Mid Ht = Msa (lb-in/ft)	9730	35489	69972	38776	25897	76546	
Ase (in^2) = (Ps + As*fy) / fy	0.89	0.86	0.86	0.87	0.89	0.89	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.300	0.300	0.300	0.300	0.300	0.300	
$C_E = C_{ELASTIC}$ = k * d	2.00	2.00	2.00	2.00	2.00	2.00	
$I_{cr\,E}$ (in4) = $I_{cr\,ELASTIC}$ = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	172.61	167.49	167.49	170.05	172.61	172.61	
M_1 = Msa (lb-in)	9730	35489	69972	38776	25897	76546	
I_{e1} (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * I_{e1}))	10739	38428	75768	42388	28581	84482	
I_{e2} (in4)	614	614	614	614	614	502	
M_3 (lb-in)	10739	38428	75768	42388	28581	86481	
I_{e3} (in4)	614	614	614	614	614	480	
M_4 (lb-in)	10739	38428	75768	42388	28581	87007	
I_{e4} (in4)	614	614	614	614	614	474	
M_5 (lb-in)	10739	38428	75768	42388	28581	87146	
I_{e5} (in4)	614	614	614	614	614	473	
M_6 (lb-in)	10739	38428	75768	42388	28581	87183	
I_{e6} (in4)	614	614	614	614	614	472	
M_7 (lb-in)	10739	38428	75768	42388	28581	87192	
I_{e7} (in4)	614	614	614	614	614	472	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * I_{e7})	0.10	0.34	0.68	0.38	0.26	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

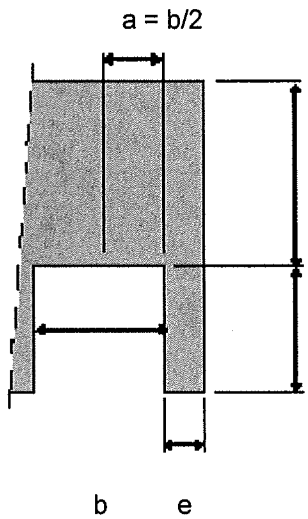
Job Name = Tumwate
Job Number = 2E+06
Wall Type = 3
Wall Description = 11 ft oper

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 1
Wall Description = 7.5 ft opening

Wall Ht =	38.44	ft
b =	7.5	ft
c =	21	ft
e =	2.17	ft
d =	17.44	ft
a =	3.75	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	4640	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	55	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	900.2765	plf
equiv SL =	1875.576	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	43.6	psf
P seismic equiv =	78.1	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam	Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

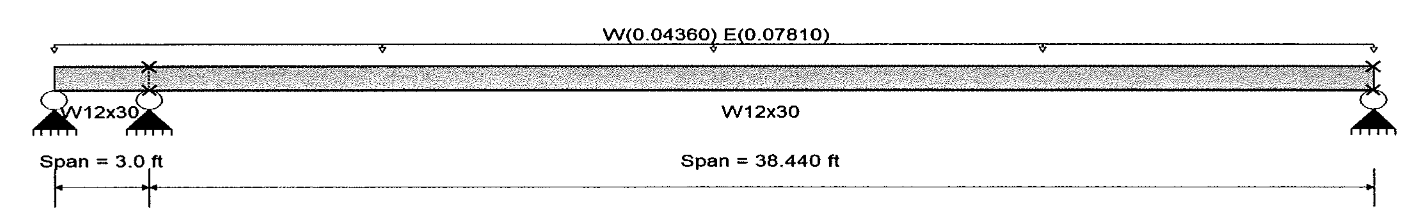
DESCRIPTION: Panel 1 Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04360, E = 0.07810 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.083** : 1	Maximum Shear Stress Ratio =	**0.048** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	13.387 k-ft	Vu : Applied	4.580 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.254 in	Ratio =	1,817	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in	Ratio =	18,701	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.178 in	Ratio =	2596	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.001 in	Ratio =	26716	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.023	0.013		-3.74	3.74	179.58	161.63	1.00	1.00	1.28	95.94	95.94
Dsgn. L = 38.44 ft	2	0.023	0.005	2.37	-3.74	3.74	179.58	161.63	1.00	1.00	0.52	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.046	0.027		-7.47	7.47	179.58	161.63	1.00	1.00	2.56	95.94	95.94
Dsgn. L = 38.44 ft	2	0.046	0.011	4.75	-7.47	7.47	179.58	161.63	1.00	1.00	1.03	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.083	0.048		-13.39	13.39	179.58	161.63	1.00	1.00	4.58	95.94	95.94
Dsgn. L = 38.44 ft	2	0.083	0.019	8.51	-13.39	13.39	179.58	161.63	1.00	1.00	1.85	95.94	95.94

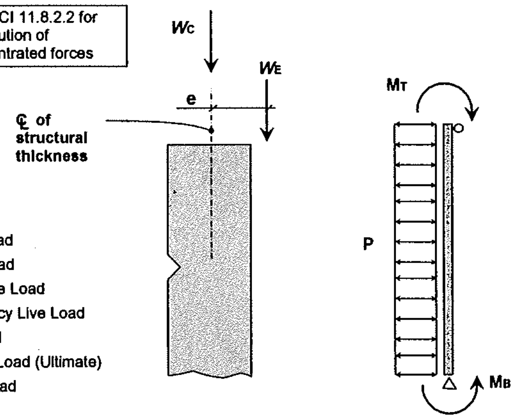
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	1
Wall Description =	7.5 ft opening

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	42.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.17
Number of Bars Ea Face (or at Center) of Pier	7.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 4" o.c.
Max Deflection	L / 701
% of Flexural Capacity	48%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 =	0.5
f_2 =	0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	4640	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (W_E)

		Moment at Top (lb-ft/ft) = W_E * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	6.75				
Dead - D (plf)	900.2764977	D =	506	D =	253
Snow - S (plf)	1875.576037	S =	1055	S =	528
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-13387	E =	-6694
Wind - W (lb-ft/ft)	0	-7474	W =	-3737

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	78.1	E =	14433
Wind - W (psf)	43.6	W =	8062

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	8225
Snow - S (plf)	1876
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	253
Snow - S (lb-ft/ft)	528
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	7739
Wind - W (lb-ft/ft)	4325

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	42.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	3.72	OK
As per foot (in^2/ft)	0.99	(This is the area of tension steel only)
Total As in Pier (in^2)	0.99	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	3.23	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0178	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0123	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

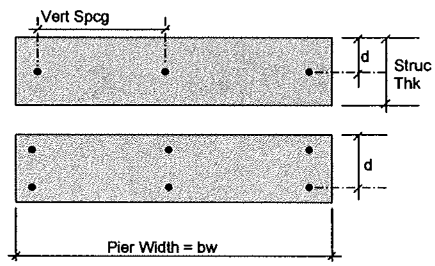
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 1
Wall Description = 7.5 ft opening

IBC-2015	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	11515	10807	12871	12871	10807	12684	7402	5900	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	4254	6811	13774	39726	58715	101505	54638	95053	
Pu / Ag (psi)	113	106	126	126	106	124	73	58	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	1.11	1.10	1.13	1.13	1.10	1.12	1.07	1.05	

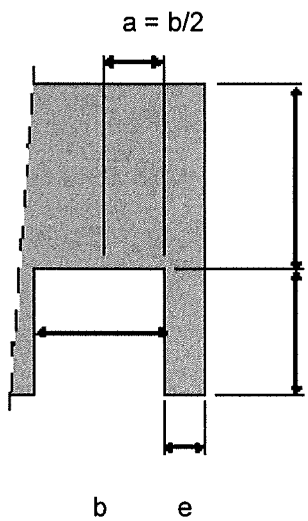
a (in) =(Ase*fy) / (0.85*f'c*lw)	1.31	1.30	1.32	1.32	1.30	1.32	1.26	1.24	
Cu = C ULTIMATE = a / β_1	1.63	1.62	1.66	1.66	1.62	1.65	1.57	1.55	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	221.67	220.83	223.27	223.27	220.83	223.05	216.70	214.84	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	6871	10623	23858	68810	91575	174097	72895	119021	
Mn (lb-in) = Ase * fy * (d - a/2)	402428	400008	407056	407056	400008	406422	388301	383109	
Cu / d	0.24	0.24	0.25	0.25	0.24	0.25	0.23	0.23	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	362185	360007	366351	366351	360007	365779	349471	344798	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	2%	3%	7%	19%	25%	48%	21%	35%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	10100	8225	8225	9163	10100	10100	
Applied Moment at Mid Ht = Msa (lb-in/ft)	9369	34181	68050	37346	24940	74380	
Ase (in^2) = (Ps + As*fy) / fy	1.16	1.13	1.13	1.14	1.16	1.16	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.342	0.342	0.342	0.342	0.342	0.342	
C_E = C ELASTIC = k * d	2.29	2.29	2.29	2.29	2.29	2.29	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-C_E)^2 + 1/3*ℓw*C_E^3	209.21	204.85	204.85	207.03	209.21	209.21	
M_1 = Msa (lb-in)	9369	34181	68050	37346	24940	74380	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	10300	36898	73460	40684	27419	81776	
Ie_2 (in4)	614	614	614	614	614	542	
M_3 (lb-in)	10300	36898	73460	40684	27419	82865	
Ie_3 (in4)	614	614	614	614	614	529	
M_4 (lb-in)	10300	36898	73460	40684	27419	83097	
Ie_4 (in4)	614	614	614	614	614	527	
M_5 (lb-in)	10300	36898	73460	40684	27419	83147	
Ie_5 (in4)	614	614	614	614	614	526	
M_6 (lb-in)	10300	36898	73460	40684	27419	83157	
Ie_6 (in4)	614	614	614	614	614	526	
M_7 (lb-in)	10300	36898	73460	40684	27419	83160	
Ie_7 (in4)	614	614	614	614	614	526	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.09	0.33	0.66	0.36	0.25	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 1
Wall Description = 7.5 ft ope

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 5
Wall Description = 11.5 ft opening

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**		
b =	11.5	ft	Wt of Concrete=	150	pcf
c =	21	ft	Wall Thickness=	9.25	in.
e =	2.17	ft	Concentric Load=	7114	plf
d =	17.44	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	5.75	ft			



Roof Weight

Joist Span=	6.75	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	147.81567	plf
equiv SL =	307.94931	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	58.4	psf
P seismic equiv =	97.3	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC

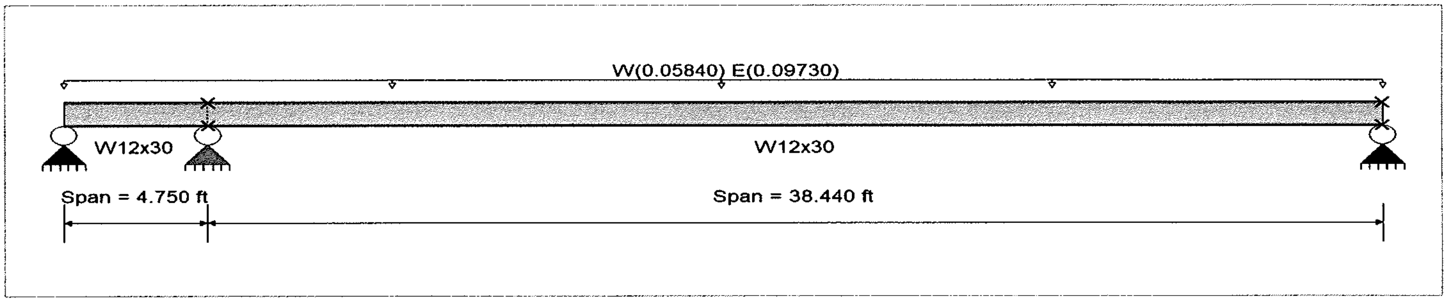
DESCRIPTION: Panel 5 Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.05840, E = 0.09730 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.099** : 1	Maximum Shear Stress Ratio =	**0.038** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	16.025 k-ft	Vu : Applied	3.605 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.330 in	Ratio =	1,395	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.006 in	Ratio =	10,036	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.231 in	Ratio =	1994	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.004 in	Ratio =	14337	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.030	0.011		-4.81	4.81	179.58	161.63	1.00	1.00	1.08	95.94	95.94
Dsgn. L = 38.44 ft	2	0.030	0.007	3.26	-4.81	4.81	179.58	161.63	1.00	1.00	0.69	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.060	0.023		-9.62	9.62	179.58	161.63	1.00	1.00	2.16	95.94	95.94
Dsgn. L = 38.44 ft	2	0.060	0.014	6.51	-9.62	9.62	179.58	161.63	1.00	1.00	1.37	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.099	0.038		-16.03	16.03	179.58	161.63	1.00	1.00	3.60	95.94	95.94
Dsgn. L = 38.44 ft	2	0.099	0.024	10.85	-16.03	16.03	179.58	161.63	1.00	1.00	2.29	95.94	95.94

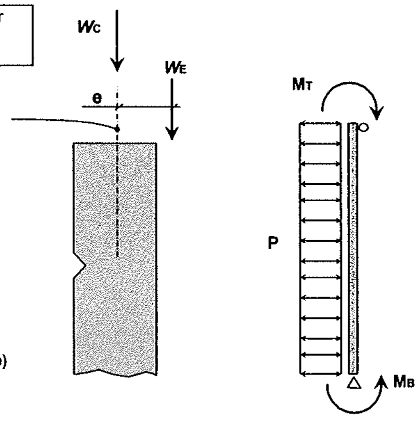
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	5
Wall Description =	11.5 ft opening

See ACI 11.8.2.2 for distribution of concentrated forces

℄ of structural thickness



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	42.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.17
Number of Bars Ea Face (or at Center) of Pier	5.99
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 4" o.c.
Max Deflection	L / 339
% of Flexural Capacity	73%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	7114 (tributary wall weight at midheight)
Snow - S (plf)	0
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Uniform Eccentric Applied Loads (W_E)

		Moment at Top (lb-ft/ft) = W_E * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 M_{TOP}
Eccentricity (in)	6.75		
Dead - D (plf)	147.8156682	D = 83	D = 42
Snow - S (plf)	307.9493088	S = 173	S = 87
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(M_{TOP})	(M_{BOT})	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (M_{TOP} + M_{BOT})
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-16025	E = -8013
Wind - W (lb-ft/ft)	0	-9619	W = -4810

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	97.3	E = 17971
Wind - W (psf)	58.4	W = 10786

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	9947
Snow - S (plf)	308
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	42
Snow - S (lb-ft/ft)	87
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	9958
Wind - W (lb-ft/ft)	5977

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	42.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 — 0.625 in
Vert Rebar o.c. Spacing (in)	4.35	OK
As per foot (in^2/ft)	0.85	(This is the area of tension steel only)
Total As in Pier (in^2)	0.85	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.76	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0152	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0105	= As per ft / (12 * d)
$\rho max = 0.6\ \rho b = 0.6 * 0.85 * \beta_1 * fc / fy * 87000 / (87000 + fy)$	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

OK

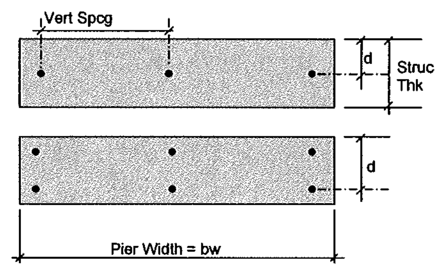
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

65

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 5
Wall Description = 11.5 ft opening

IBC-2015	Load Combo 16-1 U = 1.4D	Load Combo 16-2 U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	Load Combo 16-3(a) U = 1.2D + 1.6(Lr or S) + f_1 L	Load Combo 16-3(b) U = 1.2D + 1.6(Lr or S) + 0.5W	Load Combo 16-4 U = 1.2D + W + f_1 L + 0.5(Lr or S)	Load Combo 16-5* U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	Load Combo 16-6 U = 0.9D + W + 1.6H	Load Combo 16-7* U = (0.9-0.2Sds)D + 1.0E + 1.6H	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	13925	12090	12429	12429	12090	13968	8952	7136	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	698	1118	2262	38121	72837	120918	72168	119858	
Pu / Ag (psi)	137	119	122	122	119	137	88	70	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.99	0.97	0.98	0.98	0.97	0.99	0.94	0.92	

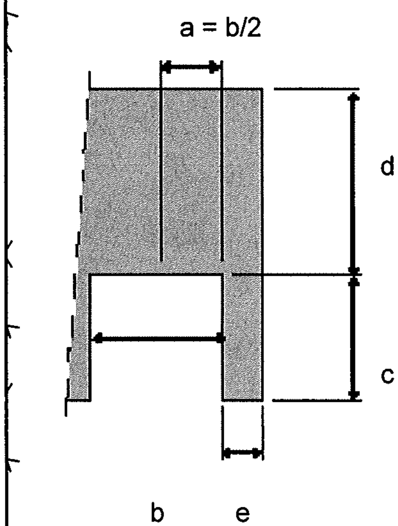
a (in) =(Ase*fy) / (0.85*f'c*lw)	1.17	1.15	1.15	1.15	1.15	1.17	1.11	1.08	
C_U = C ULTIMATE = a / β_1	1.46	1.43	1.44	1.44	1.43	1.46	1.38	1.36	
Icr U (in4) = Icr ULTIMATE = n*Ase*(d-C_U)^2 + 1/3*ℓw*C_U^3	207.78	205.34	205.80	205.80	205.34	207.84	201.07	198.54	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	1373	1968	4059	68418	128167	238390	107145	162748	
Mn (lb-in) = Ase * fy * (d - a/2)	363915	357466	358658	358658	357466	364064	346376	339922	
Cu / d	0.22	0.21	0.22	0.22	0.21	0.22	0.21	0.20	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	327524	321719	322792	322792	321719	327658	311739	305930	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	0%	1%	1%	21%	40%	73%	34%	53%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	10255	9947	9947	10101	10255	10255	
Applied Moment at Mid Ht = Msa (lb-in/ft)	1538	43530	84149	44050	23054	85188	
Ase (in^2) = (Ps + As*fy) / fy	1.02	1.01	1.01	1.01	1.02	1.02	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.321	0.321	0.321	0.321	0.321	0.321	
C_E = C ELASTIC = k * d	2.15	2.15	2.15	2.15	2.15	2.15	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-C_E)^2 + 1/3*ℓw*C_E^3	190.48	189.72	189.72	190.10	190.48	190.48	
M_1 = Msa (lb-in)	1538	43530	84149	44050	23054	85188	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	510	614	614	499	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	1694	47786	94253	48430	25385	96045	
Ie_2 (in4)	614	614	418	614	614	406	
M_3 (lb-in)	1694	47786	96825	48430	25385	98942	
Ie_3 (in4)	614	614	400	614	614	387	
M_4 (lb-in)	1694	47786	97474	48430	25385	99705	
Ie_4 (in4)	614	614	396	614	614	383	
M_5 (lb-in)	1694	47786	97637	48430	25385	99905	
Ie_5 (in4)	614	614	395	614	614	382	
M_6 (lb-in)	1694	47786	97678	48430	25385	99957	
Ie_6 (in4)	614	614	395	614	614	381	
M_7 (lb-in)	1694	47786	97688	48430	25385	99970	
Ie_7 (in4)	614	614	395	614	614	381	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.02	0.43	1.36	0.43	0.23	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 5
Wall Description = 11.5 ft op

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 6
Wall Description = Solid Panel

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**		
b =	0.01	ft	Wt of Concrete=	150	pcf
c =	0.01	ft	Wall Thickness=	9.25	in.
e =	1.00	ft	Concentric Load=	13	plf
d =	38.43	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	0.005	ft			



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	382.905	plf
equiv SL =	797.71875	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	16.1	psf
P seismic equiv =	42.4	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

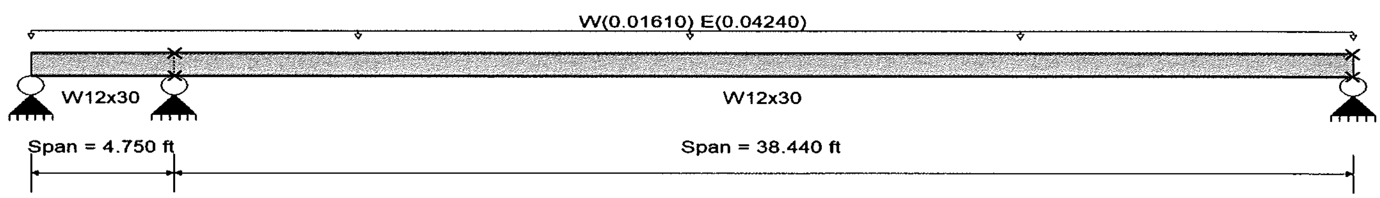
DESCRIPTION: Panel 6 Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.01610, E = 0.04240 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.043** : 1	Maximum Shear Stress Ratio =	**0.016** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	6.983 k-ft	Vu : Applied	1.571 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.144 in	Ratio = 3,202	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in	Ratio = 23,030	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.101 in	Ratio = 4575	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.002 in	Ratio = 32901	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.008	0.003		-1.33	1.33	179.58	161.63	1.00	1.00	0.30	95.94	95.94
Dsgn. L = 38.44 ft	2	0.008	0.002	0.90	-1.33	1.33	179.58	161.63	1.00	1.00	0.19	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.016	0.006		-2.65	2.65	179.58	161.63	1.00	1.00	0.60	95.94	95.94
Dsgn. L = 38.44 ft	2	0.016	0.004	1.80	-2.65	2.65	179.58	161.63	1.00	1.00	0.38	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.043	0.016		-6.98	6.98	179.58	161.63	1.00	1.00	1.57	95.94	95.94
Dsgn. L = 38.44 ft	2	0.043	0.010	4.73	-6.98	6.98	179.58	161.63	1.00	1.00	1.00	95.94	95.94

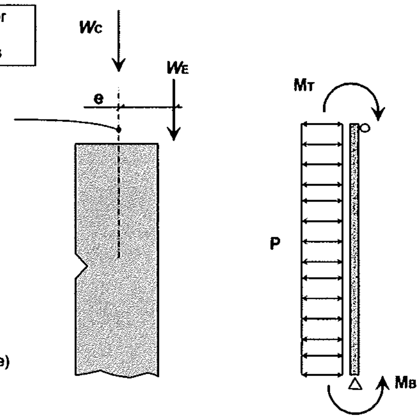
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	6
Wall Description =	Solid Panel

See ACI 11.8.2.2 for distribution of concentrated forces

℄ of structural thickness



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	42.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	1.00
Number of Bars Ea Face (or at Center) of Pier	1.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 12" o.c.
Max Deflection	L / 1325
% of Flexural Capacity	70%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	13	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (W_E)

		Moment at Top (lb-ft/ft) = W_E * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 M_{TOP}
Eccentricity (in)	6.75		
Dead - D (plf)	382.905	D = 215	D = 108
Snow - S (plf)	797.71875	S = 449	S = 224
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(M_{TOP})	(M_{BOT})	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (M_{TOP} + M_{BOT})
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-6983	E = -3492
Wind - W (lb-ft/ft)	0	-2652	W = -1326

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	42.4	E = 7838
Wind - W (psf)	16.1	W = 2970

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	3081
Snow - S (plf)	798
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	108
Snow - S (lb-ft/ft)	224
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	4347
Wind - W (lb-ft/ft)	1644

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	42.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 0.625 in
Vert Rebar o.c. Spacing (in)	12	OK
As per foot (in^2/ft)	0.31	(This is the area of tension steel only)
Total As in Pier (in^2)	0.31	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρ_v min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0055	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0038	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * f'c / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

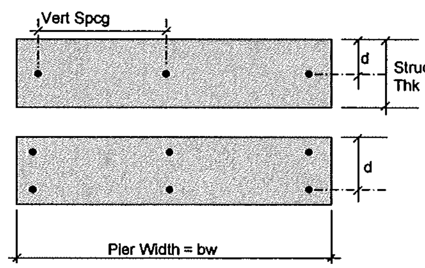
OK

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 6
Wall Description = Solid Panel

IBC-2015	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	4314	4096	4974	4974	4096	4818	2773	2210	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1809	2897	5858	15723	22625	55834	20892	53089	
Pu / Ag (psi)	42	40	49	49	40	47	27	22	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.35	0.35	0.36	0.36	0.35	0.36	0.34	0.33	

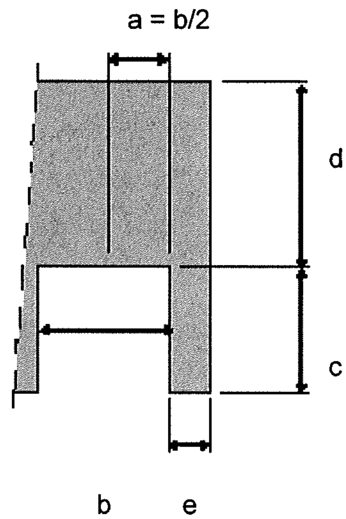
a (in) =(Ase*fy) / (0.85*f'c*lw)	0.41	0.41	0.42	0.42	0.41	0.42	0.40	0.39	
Cu = C ULTIMATE = a / β_1	0.52	0.51	0.53	0.53	0.51	0.53	0.49	0.49	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	97.08	96.54	98.70	98.70	96.54	98.32	93.26	91.84	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2684	4205	9292	24936	32843	87149	26717	64465	
Mn (lb-in) = Ase * fy * (d - a/2)	137050	136183	139680	139680	136183	139061	130899	128648	
Cu / d	0.08	0.08	0.08	0.08	0.08	0.08	0.07	0.07	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	123345	122565	125712	125712	122565	125155	117809	115783	
ϕMn > Mcr ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Mu / ϕMn	2%	3%	7%	20%	27%	70%	23%	56%	**POSITIVI**
ϕMn > Mu ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	3879	3081	3081	3480	3879	3879	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3985	13129	37806	14476	9903	40498	
Ase (in^2) = (Ps + As*fy) / fy	0.37	0.36	0.36	0.36	0.37	0.37	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.209	0.209	0.209	0.209	0.209	0.209	
CE = C ELASTIC = k * d	1.40	1.40	1.40	1.40	1.40	1.40	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	85.72	83.04	83.04	84.38	85.72	85.72	
M_1 = Msa (lb-in)	3985	13129	37806	14476	9903	40498	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	4128	13502	38879	14941	10260	41956	
Ie_2 (in4)	614	614	614	614	614	614	
M_3 (lb-in)	4128	13502	38879	14941	10260	41956	
Ie_3 (in4)	614	614	614	614	614	614	
M_4 (lb-in)	4128	13502	38879	14941	10260	41956	
Ie_4 (in4)	614	614	614	614	614	614	
M_5 (lb-in)	4128	13502	38879	14941	10260	41956	
Ie_5 (in4)	614	614	614	614	614	614	
M_6 (lb-in)	4128	13502	38879	14941	10260	41956	
Ie_6 (in4)	614	614	614	614	614	614	
M_7 (lb-in)	4128	13502	38879	14941	10260	41956	
Ie_7 (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.04	0.12	0.35	0.13	0.09	E+S is N/A	
	OK	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 6
Wall Description = Solid Par

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier between rollup door and man door

Wall Ht =	36.81	ft	**Wall Weight at Mid Height**		
b =	15.33	ft	Wt of Concrete=	150	pcf
c =	7	ft	Wall Thickness=	9.25	in.
e =	4.00	ft	Concentric Load=	4964	plf
d =	29.81	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	7.665	ft			



Roof Weight

Joist Span=	20	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	349.95	plf
equiv SL =	729.0625	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	46.7	psf
P seismic equiv =	117.4	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

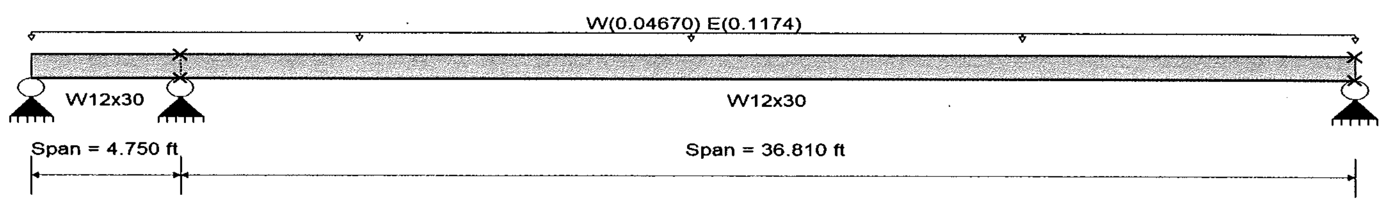
DESCRIPTION: Panel 7 Moments - Pier A

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04670, E = 0.1174 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.109** : 1	Maximum Shear Stress Ratio =	**0.042** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	17.649 k-ft	Vu : Applied	3.995 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.337 in	Ratio =	1,310	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.006 in	Ratio =	9,136	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.236 in	Ratio =	1872	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.004 in	Ratio =	13053	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 36.81 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.022	0.008		-3.51	3.51	179.58	161.63	1.00	1.00	0.79	95.94	95.94
Dsgn. L = 36.81 ft	2	0.022	0.005	2.39	-3.51	3.51	179.58	161.63	1.00	1.00	0.53	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.043	0.017		-7.02	7.02	179.58	161.63	1.00	1.00	1.59	95.94	95.94
Dsgn. L = 36.81 ft	2	0.043	0.011	4.79	-7.02	7.02	179.58	161.63	1.00	1.00	1.05	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.109	0.042		-17.65	17.65	179.58	161.63	1.00	1.00	3.99	95.94	95.94
Dsgn. L = 36.81 ft	2	0.109	0.028	12.04	-17.65	17.65	179.58	161.63	1.00	1.00	2.64	95.94	95.94

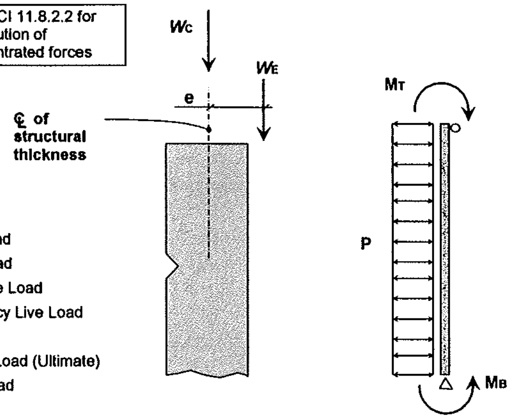
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	**Tumwater**
Job Number =	**2210856.20**
Wall Type =	**7**
Wall Description =	**Pier between rollup door and man door**

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	36.81
Total Wall Ht w/ Parapet (ft)	40.81
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	4.00
Number of Bars Ea Face (or at Center) of Pier	8.99
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 5" o.c.
Max Deflection	L / 268
% of Flexural Capacity	86%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 =	0.5
f_2 =	0.7

Uniform Concentric Applied Loads (*Wc*)

Dead - D (plf)	4964	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*WE*)

		Moment at Top (lb-ft/ft) = WE * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP
Eccentricity (in)	6.75		
Dead - D (plf)	349.95	D = 197	D = 98
Snow - S (plf)	729.0625	S = 410	S = 205
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-17649	E = -8825
Wind - W (lb-ft/ft)	0	-7021	W = -3511

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	117.4	E = 19884
Wind - W (psf)	46.7	W = 7903

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	7905
Snow - S (plf)	729
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	98
Snow - S (lb-ft/ft)	205
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	11059
Wind - W (lb-ft/ft)	4392

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	36.81	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	40.81	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 / 0.625 in
Vert Rebar o.c. Spacing (in)	5.34	OK
As per foot (in^2/ft)	0.69	(This is the area of tension steel only)
Total As in Pier (in^2)	0.69	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.25	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρ_v min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0124	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0086	= As per ft / (12 * d)
$\rho max = 0.6\ \rho b = 0.6 * 0.85 * \beta_1 * fc / fy * 87000 / (87000 + fy)$	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓ w (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓ c (in)	441.72	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓ c / 150 (in)	2.9448	

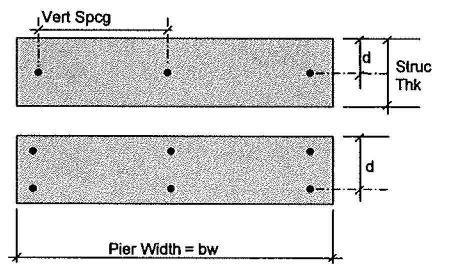
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier between rollup door and man door

IBC-2015	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	11067	9850	10652	10652	9850	11439	7114	5671	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1654	2648	5354	31709	55356	136064	53772	133556	
Pu / Ag (psi)	108	97	104	104	97	112	70	56	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.81	0.79	0.80	0.80	0.79	0.81	0.76	0.75	

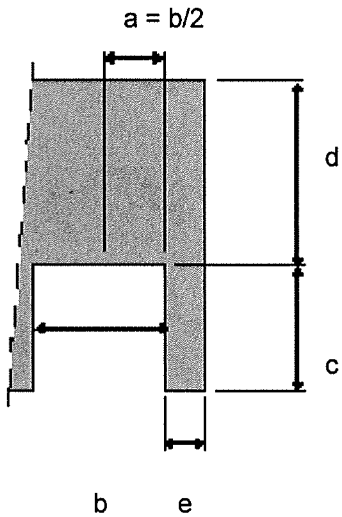
a (in) =(Ase*fy) / (0.85*fc*lw)	0.95	0.93	0.94	0.94	0.93	0.95	0.90	0.88	
Cu = C ULTIMATE = a / β_1	1.19	1.17	1.18	1.18	1.17	1.19	1.12	1.10	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	182.33	180.40	181.67	181.67	180.40	182.91	175.97	173.58	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2794	4183	8839	52345	87468	234795	73844	171150	
Mn (lb-in) = Ase * fy * (d - a/2)	300702	296260	299190	299190	296260	302061	286226	280909	
Cu / d	0.18	0.17	0.18	0.18	0.17	0.18	0.17	0.16	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	270632	266634	269271	269271	266634	271855	257604	252818	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	1%	2%	3%	19%	33%	86%	29%	68%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	8634	7905	7905	8269	8634	8634	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3642	32806	94077	34037	19454	96538	
Ase (in^2) = (Ps + As*fy) / fy	0.83	0.82	0.82	0.83	0.83	0.83	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.295	0.295	0.295	0.295	0.295	0.295	
CE = C ELASTIC = k * d	1.97	1.97	1.97	1.97	1.97	1.97	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	163.98	162.04	162.04	163.01	163.98	163.98	
M_1 = Msa (lb-in)	3642	32806	94077	34037	19454	96538	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	406	614	614	389	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	3920	35084	104308	36516	20939	108699	
Ie_2 (in4)	614	614	341	614	614	322	
M_3 (lb-in)	3920	35084	106517	36516	20939	111647	
Ie_3 (in4)	614	614	330	614	614	310	
M_4 (lb-in)	3920	35084	106985	36516	20939	112338	
Ie_4 (in4)	614	614	328	614	614	307	
M_5 (lb-in)	3920	35084	107083	36516	20939	112499	
Ie_5 (in4)	614	614	328	614	614	306	
M_6 (lb-in)	3920	35084	107104	36516	20939	112536	
Ie_6 (in4)	614	614	328	614	614	306	
M_7 (lb-in)	3920	35084	107109	36516	20939	112544	
Ie_7 (in4)	614	614	328	614	614	306	
ℓc / 150 (in)	2.9448	2.9448	2.9448	2.9448	2.9448	2.9448	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.03	0.29	1.65	0.30	0.17	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 7
Wall Description = Pier betw

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier adjacent to roll up door

Wall Ht =	36.81	ft	**Wall Weight at Mid Height**		
b =	12	ft	Wt of Concrete=	150	pcf
c =	14	ft	Wall Thickness=	9.25	in.
e =	6.55	ft	Concentric Load=	2373	plf
d =	22.81	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	6	ft			



Roof Weight

Joist Span=	20	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	229.92366	plf
equiv SL =	479.00763	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	30.7	psf
P seismic equiv =	70.5	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

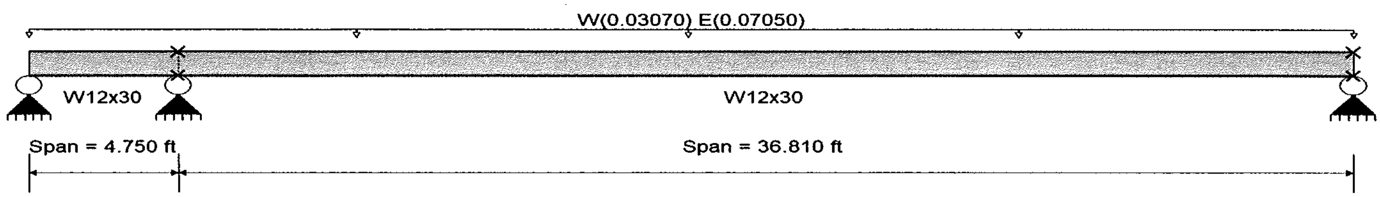
DESCRIPTION: Panel 7 Moments - Pier B

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.03070, E = 0.07050 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.066** : 1	Maximum Shear Stress Ratio =	**0.025** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	10.599 k-ft	Vu : Applied	2.399 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.202 in	Ratio = 2,182	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.004 in	Ratio = 15,215	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.142 in	Ratio = 3118	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.003 in	Ratio = 21736	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 36.81 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.014	0.005		-2.31	2.31	179.58	161.63	1.00	1.00	0.52	95.94	95.94
Dsgn. L = 36.81 ft	2	0.014	0.004	1.57	-2.31	2.31	179.58	161.63	1.00	1.00	0.35	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.029	0.011		-4.62	4.62	179.58	161.63	1.00	1.00	1.04	95.94	95.94
Dsgn. L = 36.81 ft	2	0.029	0.007	3.15	-4.62	4.62	179.58	161.63	1.00	1.00	0.69	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.066	0.025		-10.60	10.60	179.58	161.63	1.00	1.00	2.40	95.94	95.94
Dsgn. L = 36.81 ft	2	0.066	0.017	7.23	-10.60	10.60	179.58	161.63	1.00	1.00	1.59	95.94	95.94

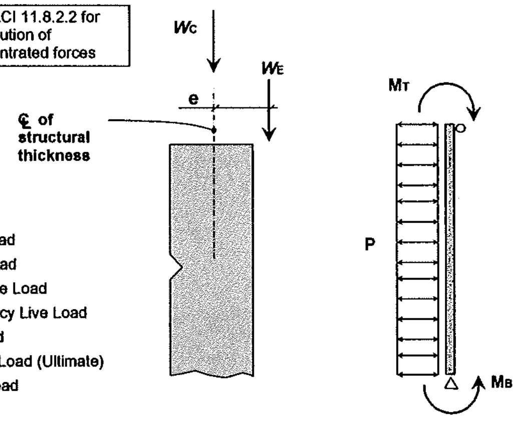
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	7
Wall Description =	Pier adjacent to roll up door

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	36.81
Total Wall Ht w/ Parapet (ft)	40.81
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	6.55
Number of Bars Ea Face (or at Center) of Pier	8.98
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 9" o.c.
Max Deflection	L / 910
% of Flexural Capacity	78%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	2373	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	6.75				
Dead - D (plf)	229.9236641	D =	129	D =	65
Snow - S (plf)	479.0076336	S =	269	S =	135
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-10599	E =	-5300
Wind - W (lb-ft/ft)	0	-4615	W =	-2308

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	70.5	E =	11945
Wind - W (psf)	30.7	W =	5192

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	5194
Snow - S (plf)	479
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	65
Snow - S (lb-ft/ft)	135
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	6646
Wind - W (lb-ft/ft)	2885

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	36.81	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	40.81	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 — 0.625 in
Vert Rebar o.c. Spacing (in)	8.75	OK
As per foot (in^2/ft)	0.42	(This is the area of tension steel only)
Total As in Pier (in^2)	0.42	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.37	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0076	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0052	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * f'c / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	441.72	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	2.9448	

OK

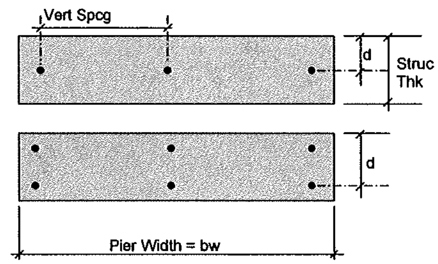
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier adjacent to roll up door

IBC-2015	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	7271	6472	6999	6999	6472	7516	4674	3726	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1086	1740	3518	20827	36358	81952	35317	80304	
Pu / Ag (psi)	71	63	69	69	63	74	46	37	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.50	0.49	0.49	0.49	0.49	0.50	0.47	0.46	

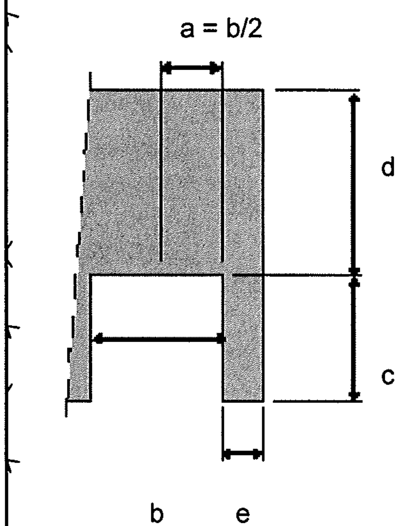
a (in) =(Ase*fy) / (0.85*f'c*lw)	0.59	0.58	0.58	0.58	0.58	0.59	0.55	0.54	
Cu = C ULTIMATE = a / β_1	0.73	0.72	0.73	0.73	0.72	0.74	0.69	0.68	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	128.60	126.88	128.01	128.01	126.88	129.12	122.97	120.87	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	1753	2647	5563	32933	55335	134650	47442	101298	
Mn (lb-in) = Ase * fy * (d - a/2)	190983	187881	189927	189927	187881	191932	180886	177186	
Cu / d	0.11	0.11	0.11	0.11	0.11	0.11	0.10	0.10	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	171885	169093	170934	170934	169093	172739	162798	159467	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	1%	2%	3%	19%	33%	78%	29%	64%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	5673	5194	5194	5433	5673	5673	
Applied Moment at Mid Ht = Msa (lb-in/ft)	2393	21547	56599	22355	12778	58216	
Ase (in^2) = (Ps + As*fy) / fy	0.52	0.51	0.51	0.51	0.52	0.52	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.240	0.240	0.240	0.240	0.240	0.240	
CE = C ELASTIC = k * d	1.60	1.60	1.60	1.60	1.60	1.60	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	112.33	110.85	110.85	111.59	112.33	112.33	
M_1 = Msa (lb-in)	2393	21547	56599	22355	12778	58216	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	2510	22507	59120	23399	13402	61060	
Ie_2 (in4)	614	614	614	614	614	614	
M_3 (lb-in)	2510	22507	59120	23399	13402	61060	
Ie_3 (in4)	614	614	614	614	614	614	
M_4 (lb-in)	2510	22507	59120	23399	13402	61060	
Ie_4 (in4)	614	614	614	614	614	614	
M_5 (lb-in)	2510	22507	59120	23399	13402	61060	
Ie_5 (in4)	614	614	614	614	614	614	
M_6 (lb-in)	2510	22507	59120	23399	13402	61060	
Ie_6 (in4)	614	614	614	614	614	614	
M_7 (lb-in)	2510	22507	59120	23399	13402	61060	
Ie_7 (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	2.9448	2.9448	2.9448	2.9448	2.9448	2.9448	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.02	0.18	0.49	0.19	0.11	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 7
Wall Description = Pier adjac

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier adjacent to man-door

Wall Ht =	36.81	ft	**Wall Weight at Mid Height**		
b =	3.33	ft	Wt of Concrete=	150	pcf
c =	7	ft	Wall Thickness=	9.25	in.
e =	2.50	ft	Concentric Load=	1725	plf
d =	29.81	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	1.665	ft			



Roof Weight

Joist Span=	20	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	199.92	plf
equiv SL =	416.5	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	26.7	psf
P seismic equiv =	68.4	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

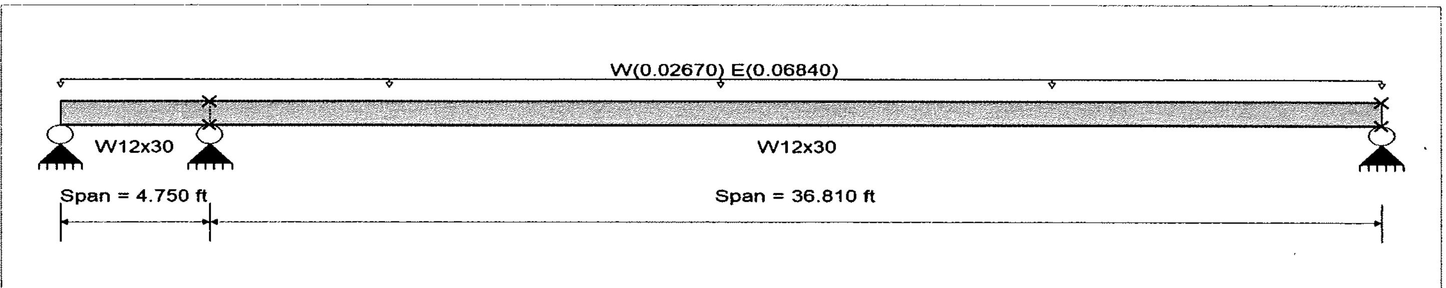
DESCRIPTION: Panel 7 Moments - Pier C

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.02670, E = 0.06840 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.064** : 1	Maximum Shear Stress Ratio =	**0.024** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	10.283 k-ft	Vu : Applied	2.327 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.196 in	Ratio = 2,249	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.004 in	Ratio = 15,682	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.137 in	Ratio = 3214	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.003 in	Ratio = 22403	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 36.81 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.012	0.005		-2.01	2.01	179.58	161.63	1.00	1.00	0.45	95.94	95.94
Dsgn. L = 36.81 ft	2	0.012	0.003	1.37	-2.01	2.01	179.58	161.63	1.00	1.00	0.30	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.025	0.009		-4.01	4.01	179.58	161.63	1.00	1.00	0.91	95.94	95.94
Dsgn. L = 36.81 ft	2	0.025	0.006	2.74	-4.01	4.01	179.58	161.63	1.00	1.00	0.60	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.064	0.024		-10.28	10.28	179.58	161.63	1.00	1.00	2.33	95.94	95.94
Dsgn. L = 36.81 ft	2	0.064	0.016	7.01	-10.28	10.28	179.58	161.63	1.00	1.00	1.54	95.94	95.94

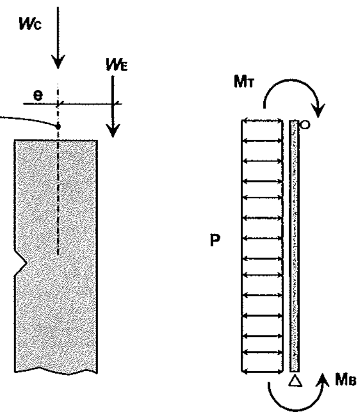
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	7
Wall Description =	Pier adjacent to man-door

See ACI 11.8.2.2 for distribution of concentrated forces

℄ of structural thickness



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	36.81
Total Wall Ht w/ Parapet (ft)	40.81
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.50
Number of Bars Ea Face (or at Center) of Pier	4.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 8" o.c.
Max Deflection	L / 946
% of Flexural Capacity	60%

Hand Input	OK
Potential Hand Input	
Output	

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 =	0.5
f_2 =	0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	1725	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	6.75				
Dead - D (plf)	199.92	D =	112	D =	56
Snow - S (plf)	416.5	S =	234	S =	117
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-10283	E =	-5142
Wind - W (lb-ft/ft)	0	-4014	W =	-2007

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter <u>positive numbers to increase</u> the moment induced at the mid-height of the wall being designed and <u>negative numbers to reduce</u> the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	68.4	E =	11577
Wind - W (psf)	26.7	W =	4515

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	4516
Snow - S (plf)	417
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	56
Snow - S (lb-ft/ft)	117
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	6435
Wind - W (lb-ft/ft)	2508

Note that these totals represent the *<u>unfactored</u>* forces at the mid-height of the wall *<u>including</u>* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *<u>not</u>* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	36.81	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	40.81	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	7.5	OK
As per foot (in^2/ft)	0.49	(This is the area of tension steel only)
Total As in Pier (in^2)	0.49	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.60	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0088	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0061	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	441.72	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	2.9448	

OK

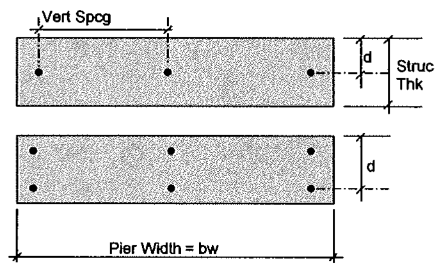
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 7
Wall Description = Pier adjacent to man-door

IBC-2015	U = 1.4D Load Combo 16-1	U = 1.2D + 1.6(L+H) + 0.5(Lr or S) Load Combo 16-2	U = 1.2D + 1.6(Lr or S) + f_1 L Load Combo 16-3(a)	U = 1.2D + 1.6(Lr or S) + 0.5W Load Combo 16-3(b)	U = 1.2D + W + f_1 L + 0.5(Lr or S) Load Combo 16-4	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2 Load Combo 16-5*	U = 0.9D + W + 1.6H Load Combo 16-6	U = (0.9-0.2Sds)D + 1.0E + 1.6H Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	6322	5627	6085	6085	5627	6535	4064	3240	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	945	1513	3059	18105	31606	79141	30701	77708	
Pu / Ag (psi)	62	55	60	60	55	64	40	32	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.56	0.55	0.56	0.56	0.55	0.56	0.53	0.53	

a (in) =(Ase*fy) / (0.85*f'c*lw)	0.66	0.65	0.65	0.65	0.65	0.66	0.63	0.62	
Cu = C ULTIMATE = a / β_1	0.82	0.81	0.82	0.82	0.81	0.82	0.79	0.77	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	140.37	138.97	139.90	139.90	138.97	140.80	135.77	134.06	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	1355	2078	4323	25590	43430	115042	38437	92784	
Mn (lb-in) = Ase * fy * (d - a/2)	212850	210184	211942	211942	210184	213666	204175	200997	
Cu / d	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	191565	189166	190748	190748	189166	192299	183758	180897	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	1%	1%	2%	13%	23%	60%	21%	51%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	4932	4516	4516	4724	4932	4932	
Applied Moment at Mid Ht = Msa (lb-in/ft)	2080	18731	54732	19434	11108	56137	
Ase (in^2) = (Ps + As*fy) / fy	0.57	0.57	0.57	0.57	0.57	0.57	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.256	0.256	0.256	0.256	0.256	0.256	
CE = C ELASTIC = k * d	1.71	1.71	1.71	1.71	1.71	1.71	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	122.15	120.91	120.91	121.53	122.15	122.15	
M_1 = Msa (lb-in)	2080	18731	54732	19434	11108	56137	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	2168	19452	56839	20218	11577	58507	
Ie_2 (in4)	614	614	614	614	614	614	
M_3 (lb-in)	2168	19452	56839	20218	11577	58507	
Ie_3 (in4)	614	614	614	614	614	614	
M_4 (lb-in)	2168	19452	56839	20218	11577	58507	
Ie_4 (in4)	614	614	614	614	614	614	
M_5 (lb-in)	2168	19452	56839	20218	11577	58507	
Ie_5 (in4)	614	614	614	614	614	614	
M_6 (lb-in)	2168	19452	56839	20218	11577	58507	
Ie_6 (in4)	614	614	614	614	614	614	
M_7 (lb-in)	2168	19452	56839	20218	11577	58507	
Ie_7 (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	2.9448	2.9448	2.9448	2.9448	2.9448	2.9448	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.02	0.16	0.47	0.17	0.10	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

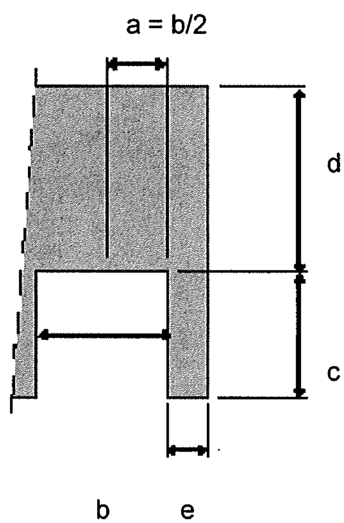
Job Name = Tumwate
Job Number = 2E+06
Wall Type = 7
Wall Description = Pier adja

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 9
Wall Description = Dock Door

Wall Ht =	35.27	ft
b =	9	ft
c =	10	ft
e =	2.50	ft
d =	25.27	ft
a =	4.5	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	3670	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	60	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	2.25	inch
equiv DL =	1008	plf
equiv SL =	2100	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	44.8	psf
P seismic equiv =	106.5	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

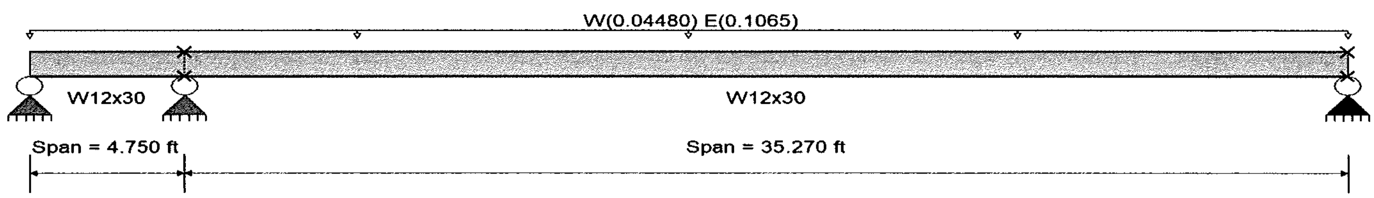
DESCRIPTION: Panel 9 - Dock Door Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04480, E = 0.1065 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.091** : 1	Maximum Shear Stress Ratio =	**0.035** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	14.630 k-ft	Vu : Applied	3.333 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.259 in	Ratio = 1,634	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.005 in	Ratio = 11,053	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.181 in	Ratio = 2335	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.004 in	Ratio = 15791	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

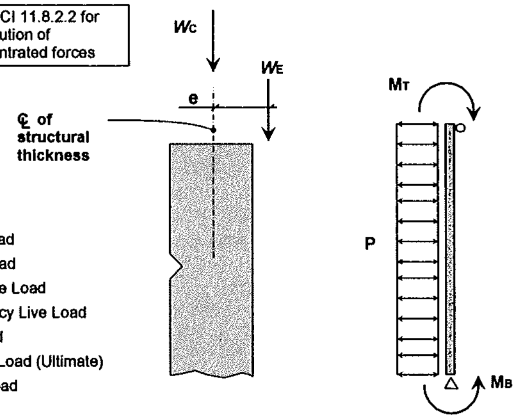
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 35.27 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.019	0.007		-3.08	3.08	179.58	161.63	1.00	1.00	0.70	95.94	95.94
Dsgn. L = 35.27 ft	2	0.019	0.005	2.11	-3.08	3.08	179.58	161.63	1.00	1.00	0.48	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.038	0.015		-6.15	6.15	179.58	161.63	1.00	1.00	1.40	95.94	95.94
Dsgn. L = 35.27 ft	2	0.038	0.010	4.23	-6.15	6.15	179.58	161.63	1.00	1.00	0.96	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.091	0.035		-14.63	14.63	179.58	161.63	1.00	1.00	3.33	95.94	95.94
Dsgn. L = 35.27 ft	2	0.091	0.024	10.05	-14.63	14.63	179.58	161.63	1.00	1.00	2.29	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	9
Wall Description =	Dock Door

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	35.27
Total Wall Ht w/ Parapet (ft)	35.27
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.50
Number of Bars Ea Face (or at Center) of Pier	5.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 6" o.c.
Max Deflection	L / 546
% of Flexural Capacity	76%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	3670	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	2.25				
Dead - D (plf)	1008	D =	189	D =	95
Snow - S (plf)	2100	S =	394	S =	197
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-14630	E =	-7315
Wind - W (lb-ft/ft)	0	-6154	W =	-3077

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	106.5	E =	16561
Wind - W (psf)	44.8	W =	6966

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	6717
Snow - S (plf)	2100
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	95
Snow - S (lb-ft/ft)	197
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	9246
Wind - W (lb-ft/ft)	3889

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	35.27	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	35.27	
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	**OK**
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	6	**OK**
As per foot (in^2/ft)	0.61	(This is the area of tension steel only)
Total As in Pier (in^2)	0.61	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρ_v min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0111	**OK** Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	**OK**
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	**OK**
ρ	0.0076	= As per ft / (12 * d)
$\rho max = 0.6\ \rho b = 0.6 * 0.85 * \beta_1 * fc / fy * 87000 / (87000 + fy)$	0.0201	**OK**
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	423.24	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	2.8216	

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

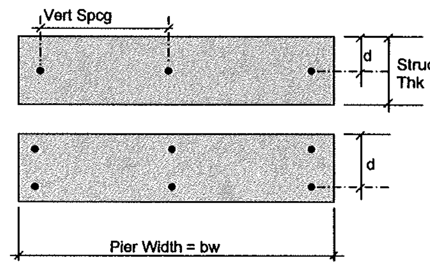
The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 9
Wall Description = Dock Door

IBC-2015	Load Combo 16-1 U = 1.4D	Load Combo 16-2 U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	Load Combo 16-3(a) U = 1.2D + 1.6(Lr or S) + f1 L	Load Combo 16-3(b) U = 1.2D + 1.6(Lr or S) + 0.5W	Load Combo 16-4 U = 1.2D + W + f1 L + 0.5(Lr or S)	Load Combo 16-5* U = (1.2+0.2Sds)D + 1.0E + f1 L + f2	Load Combo 16-6 U = 0.9D + W + 1.6H	Load Combo 16-7* U = (0.9-0.2Sds)D + 1.0E + 1.6H	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	9404	9111	11421	11421	9111	10757	6046	4819	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1588	2542	5141	28476	49213	114175	47692	111767	
Pu / Ag (psi)	92	89	112	112	89	105	59	47	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.71	0.71	0.73	0.73	0.71	0.73	0.68	0.66	

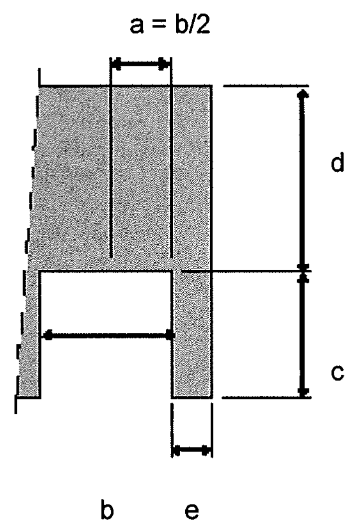
a (in) =(Ase*fy) / (0.85*fc*lw)	0.84	0.84	0.86	0.86	0.84	0.86	0.80	0.78	
$C_U = C_{ULTIMATE} = a / \beta_1$	1.05	1.04	1.08	1.08	1.04	1.07	1.00	0.98	
$I_{cr\,U}$ (in4) = $I_{cr\,ULTIMATE} = n*Ase*(d-C_U)^2 + 1/3*\ell w*C_U^3$	167.77	167.26	171.22	171.22	167.26	170.09	161.87	159.66	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2428	3830	8739	48408	74142	187293	61981	137359	
Mn (lb-in) = Ase * fy * (d - a/2)	268220	267129	275699	275699	267129	273242	255692	251094	
Cu / d	0.16	0.16	0.16	0.16	0.16	0.16	0.15	0.15	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	241398	240416	248129	248129	240416	245918	230123	225985	
ϕMn > Mcr ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Mu / ϕMn	1%	2%	4%	20%	31%	76%	27%	61%	**POSITIVI**
ϕMn > Mu ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 9
Wall Description = Dock Doc

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	8817	6717	6717	7767	8817	8817	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3497	29137	78801	30318	17498	81164	
Ase (in^2) = (Ps + As*fy) / fy	0.76	0.73	0.73	0.74	0.76	0.76	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.281	0.281	0.281	0.281	0.281	0.281	
$C_E = C_{ELASTIC}$ = k * d	1.88	1.88	1.88	1.88	1.88	1.88	
$I_{cr\,E}$ (in4) = $I_{cr\,ELASTIC} = n*Ase*(d-C_E)^2 + 1/3*\ell w*C_E^3$	153.05	147.23	147.23	150.14	153.05	153.05	
M_1 = Msa (lb-in)	3497	29137	78801	30318	17498	81164	
I_{e1} (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * $I_{cr\,E}$ } < Ig	614	614	577	614	614	541	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * I_{e1}))	3745	30691	83293	32203	18744	87786	
I_{e2} (in4)	614	614	511	614	614	460	
M_3 (lb-in)	3745	30691	83909	32203	18744	89072	
I_{e3} (in4)	614	614	503	614	614	447	
M_4 (lb-in)	3745	30691	83996	32203	18744	89327	
I_{e4} (in4)	614	614	502	614	614	444	
M_5 (lb-in)	3745	30691	84008	32203	18744	89378	
I_{e5} (in4)	614	614	502	614	614	444	
M_6 (lb-in)	3745	30691	84009	32203	18744	89388	
I_{e6} (in4)	614	614	502	614	614	444	
M_7 (lb-in)	3745	30691	84010	32203	18744	89390	
I_{e7} (in4)	614	614	502	614	614	444	
ℓc / 150 (in)	2.8216	2.8216	2.8216	2.8216	2.8216	2.8216	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * I_{e7})	0.03	0.23	0.78	0.24	0.14	E+S is N/A	
	OK	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 12
Wall Description = Pier A next to Man Door

Wall Ht =	35.27	ft	**Wall Weight at Mid Height**		
b =	9	ft	Wt of Concrete=	150	pcf
c =	3	ft	Wall Thickness=	9.25	in.
e =	2.50	ft	Concentric Load=	3670	plf
d =	32.27	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	4.5	ft			



Roof Weight

Joist Span=	60	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	2.25	inch
equiv DL =	1008	plf
equiv SL =	2100	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	44.8	psf
P seismic equiv =	117.1	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

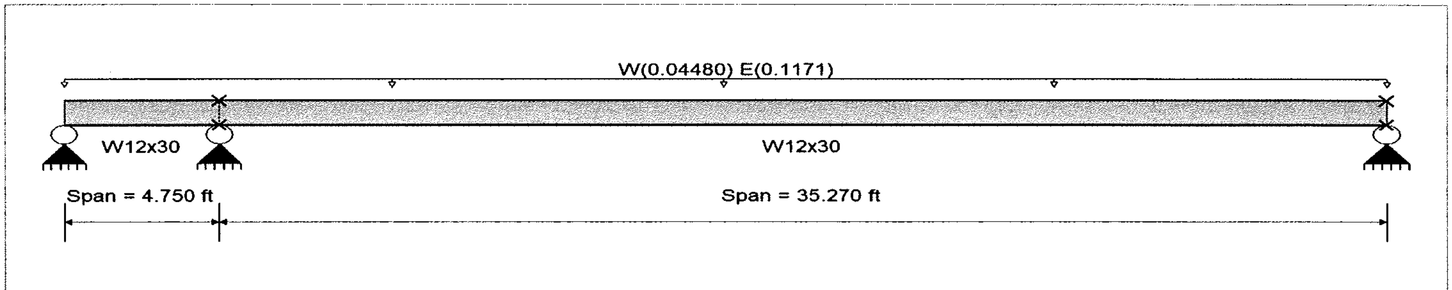
DESCRIPTION: Panel 12 - Pier A man door

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04480, E = 0.1171 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.100 : 1**	Maximum Shear Stress Ratio =	**0.038 : 1**
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	16.087 k-ft	Vu : Applied	3.665 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.285 in	Ratio = 1,486	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.006 in	Ratio = 10,053	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.199 in	Ratio = 2124	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.004 in	Ratio = 14362	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

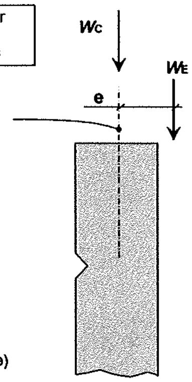
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 35.27 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.019	0.007		-3.08	3.08	179.58	161.63	1.00	1.00	0.70	95.94	95.94
Dsgn. L = 35.27 ft	2	0.019	0.005	2.11	-3.08	3.08	179.58	161.63	1.00	1.00	0.48	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.038	0.015		-6.15	6.15	179.58	161.63	1.00	1.00	1.40	95.94	95.94
Dsgn. L = 35.27 ft	2	0.038	0.010	4.23	-6.15	6.15	179.58	161.63	1.00	1.00	0.96	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.100	0.038		-16.09	16.09	179.58	161.63	1.00	1.00	3.66	95.94	95.94
Dsgn. L = 35.27 ft	2	0.100	0.026	11.05	-16.09	16.09	179.58	161.63	1.00	1.00	2.52	95.94	95.94

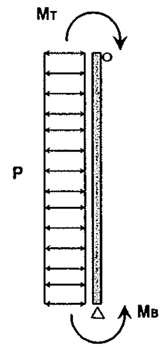
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	12
Wall Description =	Pier A next to Man Door

See ACI 11.8.2.2 for distribution of concentrated forces





D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	35.27
Total Wall Ht w/ Parapet (ft)	35.27
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.50
Number of Bars Ea Face (or at Center) of Pier	5.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 6" o.c.
Max Deflection	L / 391
% of Flexural Capacity	84%

Hand Input	OK
Potential Hand Input	
Output	

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 =	0.5
f_2 =	0.7

Uniform Concentric Applied Loads (*Wc*)

Dead - D (plf)	3670	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*WE*)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	2.25				
Dead - D (plf)	1008	D =	189	D =	95
Snow - S (plf)	2100	S =	394	S =	197
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-16087	E =	-8044
Wind - W (lb-ft/ft)	0	-6154	W =	-3077

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL²	
Seismic (Ultimate) - E (psf)	117.1	E =	18215
Wind - W (psf)	44.8	W =	6966

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	6717
Snow - S (plf)	2100
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	95
Snow - S (lb-ft/ft)	197
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	10171
Wind - W (lb-ft/ft)	3889

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

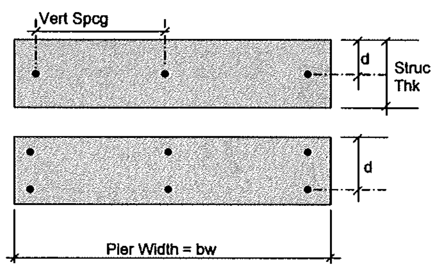
Wall Height Between Supports (ft)	35.27	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	35.27	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	65
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	6	OK
As per foot (in^2/ft)	0.61	(This is the area of tension steel only)
Total As in Pier (in^2)	0.61	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0111	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0076	= As per ft / (12 * d)
$\rho max = 0.6\ \rho b = 0.6 * 0.85 * \beta_1 * fc / fy * 87000 / (87000 + fy)$	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	423.24	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	2.8216	

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"

Verify "d" with hand calcs also



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 12
Wall Description = Pier A next to Man Door

IBC-2015	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	9404	9111	11421	11421	9111	10757	6046	4819	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1588	2542	5141	28476	49213	125274	47692	122866	
Pu / Ag (psi)	92	89	112	112	89	105	59	47	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.71	0.71	0.73	0.73	0.71	0.73	0.68	0.66	

a (in) =(Ase*fy) / (0.85*f'c*lw)	0.84	0.84	0.86	0.86	0.84	0.86	0.80	0.78	
Cu = C ULTIMATE = a / β_1	1.05	1.04	1.08	1.08	1.04	1.07	1.00	0.98	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	167.77	167.26	171.22	171.22	167.26	170.09	161.87	159.66	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2428	3830	8739	48408	74142	205499	61981	150999	
Mn (lb-in) = Ase * fy * (d - a/2)	268220	267129	275699	275699	267129	273242	255692	251094	
Cu / d	0.16	0.16	0.16	0.16	0.16	0.16	0.15	0.15	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	241398	240416	248129	248129	240416	245918	230123	225985	
ϕMn > Mcr ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Mu / ϕMn	1%	2%	4%	20%	31%	84%	27%	67%	**POSITIVI**
ϕMn > Mu ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	8817	6717	6717	7767	8817	8817	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3497	29137	86570	30318	17498	88933	
Ase (in^2) = (Ps + As*fy) / fy	0.76	0.73	0.73	0.74	0.76	0.76	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.281	0.281	0.281	0.281	0.281	0.281	
C_E = C ELASTIC = k * d	1.88	1.88	1.88	1.88	1.88	1.88	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-C_E)^2 + 1/3*ℓw*C_E^3	153.05	147.23	147.23	150.14	153.05	153.05	
M_1 = Msa (lb-in)	3497	29137	86570	30318	17498	88933	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	471	614	614	448	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	3745	30691	92689	32203	18744	97848	
Ie_2 (in4)	614	614	411	614	614	375	
M_3 (lb-in)	3745	30691	93655	32203	18744	99811	
Ie_3 (in4)	614	614	403	614	614	362	
M_4 (lb-in)	3745	30691	93809	32203	18744	100246	
Ie_4 (in4)	614	614	402	614	614	359	
M_5 (lb-in)	3745	30691	93834	32203	18744	100342	
Ie_5 (in4)	614	614	402	614	614	358	
M_6 (lb-in)	3745	30691	93838	32203	18744	100363	
Ie_6 (in4)	614	614	402	614	614	358	
M_7 (lb-in)	3745	30691	93839	32203	18744	100368	
Ie_7 (in4)	614	614	402	614	614	358	
ℓc / 150 (in)	2.8216	2.8216	2.8216	2.8216	2.8216	2.8216	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.03	0.23	1.08	0.24	0.14	E+S is N/A	
	OK	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

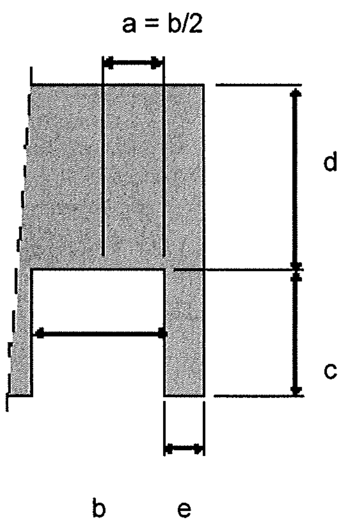
Job Name = Tumwate
Job Number = 2E+06
Wall Type = 12
Wall Description = Pier A ne

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 12
Wall Description = Pier B between OH and Man Door

Wall Ht =	35.27	ft
b =	18	ft
c =	3	ft
e =	5.00	ft
d =	32.27	ft
a =	9	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	3670	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	60	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	2.25	inch
equiv DL =	1008	plf
equiv SL =	2100	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	44.8	psf
P seismic equiv =	117.1	psf

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

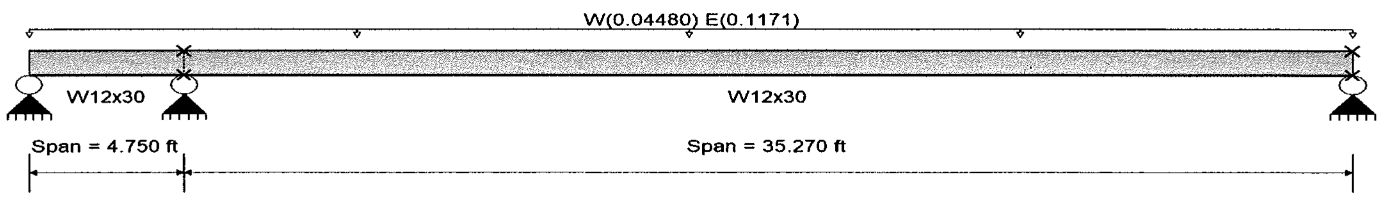
DESCRIPTION: Panel 12 - Pier B Btwn OH and man door

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04480, E = 0.1171 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.100** : 1	Maximum Shear Stress Ratio =	**0.038** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	16.087 k-ft	Vu : Applied	3.665 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	4.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.285 in	Ratio =	1,486	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.006 in	Ratio =	10,053	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.199 in	Ratio =	2124	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.004 in	Ratio =	14362	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

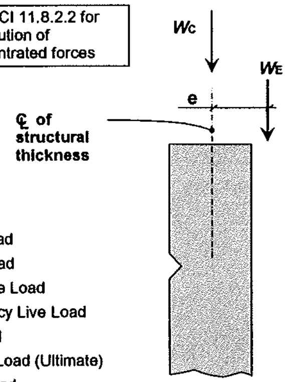
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 4.75 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 35.27 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 4.75 ft	1	0.019	0.007		-3.08	3.08	179.58	161.63	1.00	1.00	0.70	95.94	95.94
Dsgn. L = 35.27 ft	2	0.019	0.005	2.11	-3.08	3.08	179.58	161.63	1.00	1.00	0.48	95.94	95.94
W Only													
Dsgn. L = 4.75 ft	1	0.038	0.015		-6.15	6.15	179.58	161.63	1.00	1.00	1.40	95.94	95.94
Dsgn. L = 35.27 ft	2	0.038	0.010	4.23	-6.15	6.15	179.58	161.63	1.00	1.00	0.96	95.94	95.94
E Only													
Dsgn. L = 4.75 ft	1	0.100	0.038		-16.09	16.09	179.58	161.63	1.00	1.00	3.66	95.94	95.94
Dsgn. L = 35.27 ft	2	0.100	0.026	11.05	-16.09	16.09	179.58	161.63	1.00	1.00	2.52	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

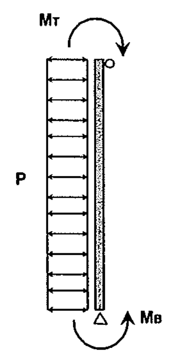
If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	12
Wall Description =	Pier B between OH and Man Door

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load



DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	35.27
Total Wall Ht w/ Parapet (ft)	35.27
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	5.00
Number of Bars Ea Face (or at Center) of Pier	10.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 6" o.c.
Max Deflection	L / 391
% of Flexural Capacity	84%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*Wc*)

Dead - D (plf)	3670	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*WE*)

		Moment at Top (lb-ft/ft) = WE * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP
Eccentricity (in)	2.25		
Dead - D (plf)	1008	D = 189	D = 95
Snow - S (plf)	2100	S = 394	S = 197
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-16087	E = -8044
Wind - W (lb-ft/ft)	0	-6154	W = -3077

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	117.1	E = 18215
Wind - W (psf)	44.8	W = 6966

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	6717
Snow - S (plf)	2100
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	95
Snow - S (lb-ft/ft)	197
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	10171
Wind - W (lb-ft/ft)	3889

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	35.27	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	35.27	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	65
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	6	OK
As per foot (in^2/ft)	0.61	(This is the area of tension steel only)
Total As in Pier (in^2)	0.61	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0111	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0076	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	423.24	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	2.8216	

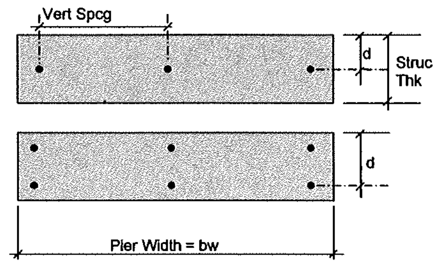
OK

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 12
Wall Description = Pier B between OH and Man Door

IBC-2015	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	9404	9111	11421	11421	9111	10757	6046	4819	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1588	2542	5141	28476	49213	125274	47692	122866	
Pu / Ag (psi)	92	89	112	112	89	105	59	47	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.71	0.71	0.73	0.73	0.71	0.73	0.68	0.66	

a (in) =(Ase*fy) / (0.85*f'c*lw)	0.84	0.84	0.86	0.86	0.84	0.86	0.80	0.78	
C_U = C ULTIMATE = a / β_1	1.05	1.04	1.08	1.08	1.04	1.07	1.00	0.98	
Icr U (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	167.77	167.26	171.22	171.22	167.26	170.09	161.87	159.66	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2428	3830	8739	48408	74142	205499	61981	150999	
Mn (lb-in) = Ase * fy * (d - a/2)	268220	267129	275699	275699	267129	273242	255692	251094	
Cu / d	0.16	0.16	0.16	0.16	0.16	0.16	0.15	0.15	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	241398	240416	248129	248129	240416	245918	230123	225985	
ϕMn > Mcr ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Mu / ϕMn	1%	2%	4%	20%	31%	84%	27%	67%	**POSITIVI**
ϕMn > Mu ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	8817	6717	6717	7767	8817	8817	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3497	29137	86570	30318	17498	88933	
Ase (in^2) = (Ps + As*fy) / fy	0.76	0.73	0.73	0.74	0.76	0.76	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.281	0.281	0.281	0.281	0.281	0.281	
C_E = C ELASTIC = k * d	1.88	1.88	1.88	1.88	1.88	1.88	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-C_E)^2 + 1/3*ℓw*C_E^3	153.05	147.23	147.23	150.14	153.05	153.05	
M_1 = Msa (lb-in)	3497	29137	86570	30318	17498	88933	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	471	614	614	448	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	3745	30691	92689	32203	18744	97848	
Ie_2 (in4)	614	614	411	614	614	375	
M_3 (lb-in)	3745	30691	93655	32203	18744	99811	
Ie_3 (in4)	614	614	403	614	614	362	
M_4 (lb-in)	3745	30691	93809	32203	18744	100246	
Ie_4 (in4)	614	614	402	614	614	359	
M_5 (lb-in)	3745	30691	93834	32203	18744	100342	
Ie_5 (in4)	614	614	402	614	614	358	
M_6 (lb-in)	3745	30691	93838	32203	18744	100363	
Ie_6 (in4)	614	614	402	614	614	358	
M_7 (lb-in)	3745	30691	93839	32203	18744	100368	
Ie_7 (in4)	614	614	402	614	614	358	
ℓc / 150 (in)	2.8216	2.8216	2.8216	2.8216	2.8216	2.8216	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.03	0.23	1.08	0.24	0.14	E+S is N/A	
	OK	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

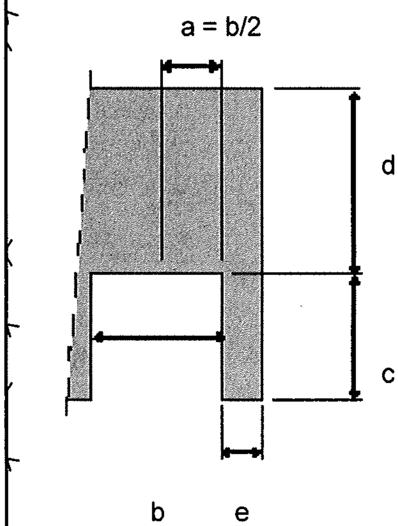
Job Name = Tumwate
Job Number = 2E+06
Wall Type = 12
Wall Description = Pier B be

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 43
Wall Description = 12.33 ' opening

Wall Ht =	38.44	ft
b =	12.33	ft
c =	12	ft
e =	3.00	ft
d =	26.44	ft
a =	6.165	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	5517	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	1163.955	plf
equiv SL =	2424.9063	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	48.9	psf
P seismic equiv =	112.9	psf

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

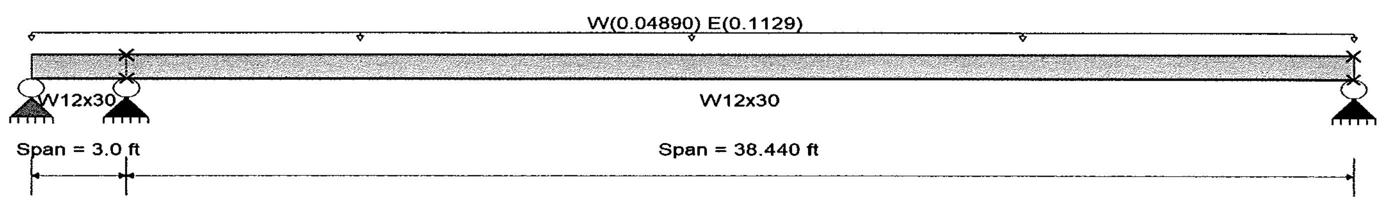
DESCRIPTION: Panel 43 Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04890, E = 0.1129 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.120** : 1	Maximum Shear Stress Ratio =	**0.069** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	19.353 k-ft	Vu : Applied	6.620 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.367 in	Ratio = 1,257	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.003 in	Ratio = 12,936	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.257 in	Ratio = 1796	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.002 in	Ratio = 18481	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

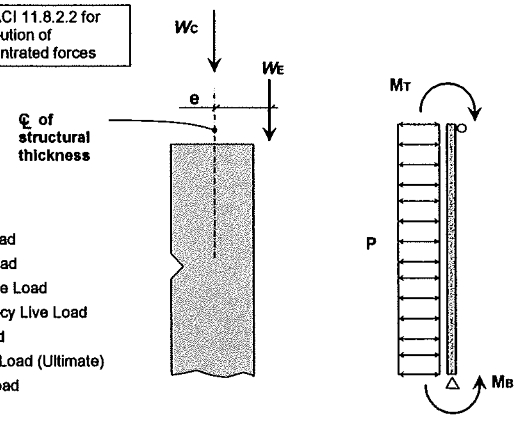
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.026	0.015		-4.19	4.19	179.58	161.63	1.00	1.00	1.43	95.94	95.94
Dsgn. L = 38.44 ft	2	0.026	0.006	2.66	-4.19	4.19	179.58	161.63	1.00	1.00	0.58	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.052	0.030		-8.38	8.38	179.58	161.63	1.00	1.00	2.87	95.94	95.94
Dsgn. L = 38.44 ft	2	0.052	0.012	5.33	-8.38	8.38	179.58	161.63	1.00	1.00	1.16	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.120	0.069		-19.35	19.35	179.58	161.63	1.00	1.00	6.62	95.94	95.94
Dsgn. L = 38.44 ft	2	0.120	0.028	12.30	-19.35	19.35	179.58	161.63	1.00	1.00	2.67	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	43
Wall Description =	12.33 ' opening

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	42.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	3.00
Number of Bars Ea Face (or at Center) of Pier	8.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 5" o.c.
Max Deflection	L / 215
% of Flexural Capacity	95%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	5517	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	6.75				
Dead - D (plf)	1163.955	D =	655	D =	327
Snow - S (plf)	2424.90625	S =	1364	S =	682
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-19353	E =	-9677
Wind - W (lb-ft/ft)	0	-8382	W =	-4191

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	112.9	E =	20855
Wind - W (psf)	48.9	W =	9028

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	9366
Snow - S (plf)	2425
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	327
Snow - S (lb-ft/ft)	682
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	11179
Wind - W (lb-ft/ft)	4837

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	4	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	42.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 / 0.625 in
Vert Rebar o.c. Spacing (in)	4.5	OK
As per foot (in^2/ft)	0.82	(This is the area of tension steel only)
Total As in Pier (in^2)	0.82	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.67	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0147	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0102	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

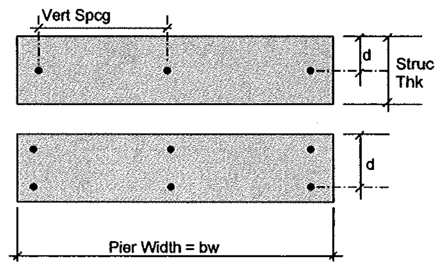
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 43
Wall Description = 12.33 ' opening

IBC-2015	Load Combo 16-1 U = 1.4D	Load Combo 16-2 U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	Load Combo 16-3(a) U = 1.2D + 1.6(Lr or S) + f_1 L	Load Combo 16-3(b) U = 1.2D + 1.6(Lr or S) + 0.5W	Load Combo 16-4 U = 1.2D + W + f_1 L + 0.5(Lr or S)	Load Combo 16-5* U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	Load Combo 16-6 U = 0.9D + W + 1.6H	Load Combo 16-7* U = (0.9-0.2Sds)D + 1.0E + 1.6H	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	13112	12452	15119	15119	12452	14647	8429	6719	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	5500	8806	17809	46833	66854	145307	61584	136965	
Pu / Ag (psi)	129	122	148	148	122	144	83	66	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.96	0.95	0.98	0.98	0.95	0.97	0.91	0.89	

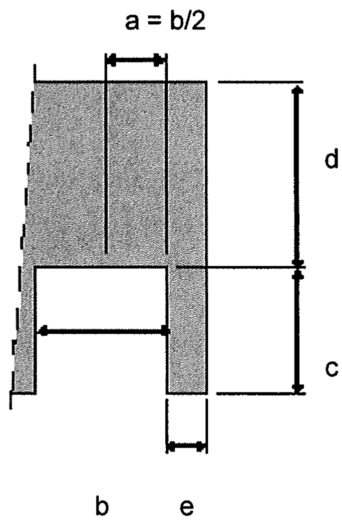
a (in) =(Ase*fy) / (0.85*f'c*lw)	1.13	1.12	1.15	1.15	1.12	1.15	1.07	1.05	
Cu = C ULTIMATE = a / β_1	1.41	1.40	1.44	1.44	1.40	1.43	1.33	1.31	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	203.13	202.22	205.83	205.83	202.22	205.20	196.59	194.13	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	10442	16054	38595	101497	121881	304862	89823	183545	
Mn (lb-in) = Ase * fy * (d - a/2)	351676	349339	358752	358752	349339	357090	335037	328917	
Cu / d	0.21	0.21	0.22	0.22	0.21	0.21	0.20	0.20	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	316508	314405	322877	322877	314405	321381	301533	296025	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	3%	5%	12%	31%	39%	95%	30%	62%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	11791	9366	9366	10579	11791	11791	
Applied Moment at Mid Ht = Msa (lb-in/ft)	12112	38757	97831	42849	29527	106015	
Ase (in^2) = (Ps + As*fy) / fy	1.01	0.97	0.97	0.99	1.01	1.01	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.317	0.317	0.317	0.317	0.317	0.317	
CE = C ELASTIC = k * d	2.12	2.12	2.12	2.12	2.12	2.12	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	190.45	184.37	184.37	187.41	190.45	190.45	
M_1 = Msa (lb-in)	12112	38757	97831	42849	29527	106015	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	391	614	614	350	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	13542	42305	112676	47333	33012	130082	
Ie_2 (in4)	614	614	320	614	614	277	
M_3 (lb-in)	13542	42305	116628	47333	33012	138405	
Ie_3 (in4)	614	614	306	614	614	262	
M_4 (lb-in)	13542	42305	117608	47333	33012	140816	
Ie_4 (in4)	614	614	303	614	614	259	
M_5 (lb-in)	13542	42305	117845	47333	33012	141468	
Ie_5 (in4)	614	614	303	614	614	258	
M_6 (lb-in)	13542	42305	117903	47333	33012	141641	
Ie_6 (in4)	614	614	302	614	614	258	
M_7 (lb-in)	13542	42305	117916	47333	33012	141686	
Ie_7 (in4)	614	614	302	614	614	257	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.12	0.38	2.14	0.42	0.30	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 43
Wall Description = 12.33 ' op

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 49
Wall Description = Man Door

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**		
b =	3.33	ft	Wt of Concrete=	150	pcf
c =	7	ft	Wall Thickness=	9.25	in.
e =	2.00	ft	Concentric Load=	1850	plf
d =	31.44	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	1.665	ft			



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	698.1825	plf
equiv SL =	1454.5469	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	29.3	psf
P seismic equiv =	75.1	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

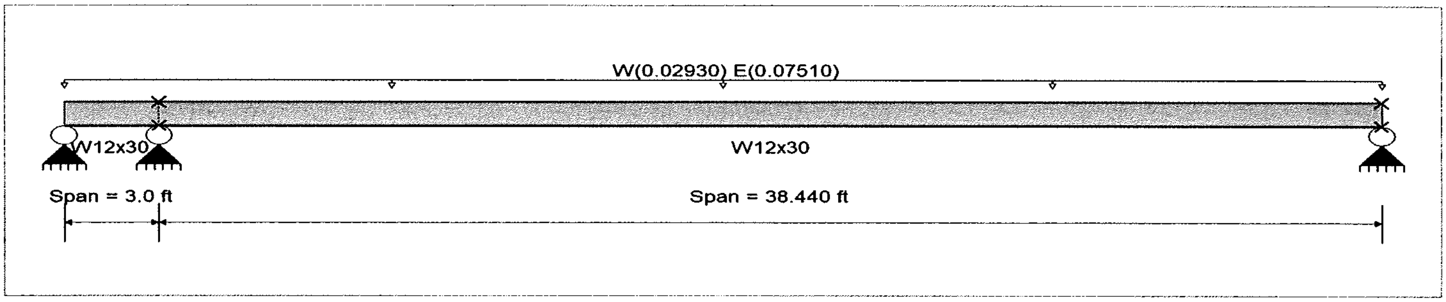
DESCRIPTION: Panel 49 Man Door Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.02930, E = 0.07510 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.080** : 1	Maximum Shear Stress Ratio =	**0.046** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	12.873 k-ft	Vu : Applied	4.404 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.244 in	Ratio =	1,889	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in	Ratio =	19,448	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.171 in	Ratio =	2700	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.001 in	Ratio =	27783	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

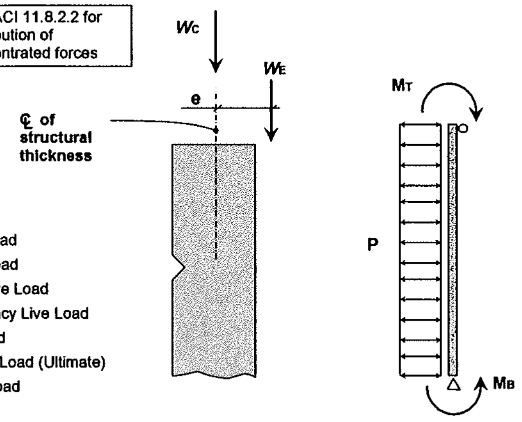
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.016	0.009		-2.51	2.51	179.58	161.63	1.00	1.00	0.86	95.94	95.94
Dsgn. L = 38.44 ft	2	0.016	0.004	1.60	-2.51	2.51	179.58	161.63	1.00	1.00	0.35	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.031	0.018		-5.02	5.02	179.58	161.63	1.00	1.00	1.72	95.94	95.94
Dsgn. L = 38.44 ft	2	0.031	0.007	3.19	-5.02	5.02	179.58	161.63	1.00	1.00	0.69	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.080	0.046		-12.87	12.87	179.58	161.63	1.00	1.00	4.40	95.94	95.94
Dsgn. L = 38.44 ft	2	0.080	0.019	8.18	-12.87	12.87	179.58	161.63	1.00	1.00	1.78	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	49
Wall Description =	Man Door

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	38.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.00
Number of Bars Ea Face (or at Center) of Pier	4.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 6" o.c.
Max Deflection	L / 761
% of Flexural Capacity	62%

Hand Input	**OK**
Potential Hand Input	
Output	

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*W*c)

Dead - D (plf)	1850	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*W*E)

		Moment at Top (lb-ft/ft) = WE * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP	
Eccentricity (in)	6.75				
Dead - D (plf)	698.1825	D =	393	D =	196
Snow - S (plf)	1454.546875	S =	818	S =	409
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-12873	E =	-6437
Wind - W (lb-ft/ft)	0	-5022	W =	-2511

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	75.1	E =	13869
Wind - W (psf)	29.3	W =	5416

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	4771
Snow - S (plf)	1455
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	196
Snow - S (lb-ft/ft)	409
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	7432
Wind - W (lb-ft/ft)	2905

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	38.44	
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	**OK**
Vert Rebar Size	5	0.31 in² — 0.625 in
Vert Rebar o.c. Spacing (in)	6	**OK**
As per foot (in²/ft)	0.61	(This is the area of tension steel only)
Total As in Pier (in²)	0.61	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0111	**OK** Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in²/ft)	0.28	**OK**
Min Tensile Flexural Reinf 2 = As min 2 (in²/ft)	0.27	**OK**
ρ	0.0076	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	**OK**
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in²/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in⁴/ft)	614	= 1/12 * 12 * Struc Thk³
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.

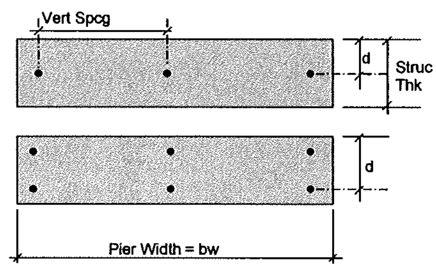
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in²)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 49
Wall Description = Man Door

IBC-2015	Load Combo 16-1 U = 1.4D	Load Combo 16-2 U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	Load Combo 16-3(a) U = 1.2D + 1.6(Lr or S) + f_1 L	Load Combo 16-3(b) U = 1.2D + 1.6(Lr or S) + 0.5W	Load Combo 16-4 U = 1.2D + W + f_1 L + 0.5(Lr or S)	Load Combo 16-5* U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	Load Combo 16-6 U = 0.9D + W + 1.6H	Load Combo 16-7* U = (0.9-0.2Sds)D + 1.0E + 1.6H	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	6679	6452	8052	8052	6452	7614	4294	3422	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	3299	5282	10682	28109	40137	95882	36975	90879	
Pu / Ag (psi)	65	63	79	79	63	75	42	34	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.68	0.68	0.70	0.70	0.68	0.69	0.66	0.65	

	16-1	16-2	16-3(a)	16-3(b)	16-4	16-5*	16-6	16-7*	
a (in) =(Ase*fy) / (0.85*f'c*lw)	0.81	0.80	0.82	0.82	0.80	0.82	0.78	0.76	
Cu = C ULTIMATE = a / β_1	1.01	1.00	1.03	1.03	1.00	1.02	0.97	0.96	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	163.00	162.59	165.42	165.42	162.59	164.65	158.70	157.11	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	4716	7450	16611	43709	56607	145072	46125	108153	
Mn (lb-in) = Ase * fy * (d - a/2)	258061	257212	263187	263187	257212	261553	249121	245845	
Cu / d	0.15	0.15	0.15	0.15	0.15	0.15	0.14	0.14	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	232255	231491	236868	236868	231491	235398	224209	221261	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	2%	3%	7%	18%	24%	62%	21%	49%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 49
Wall Description = Man Doo

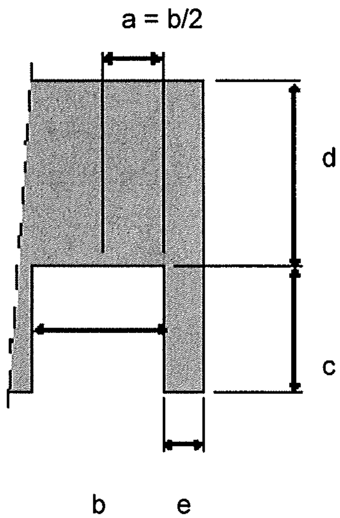
	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	6225	4771	4771	5498	6225	6225	
Applied Moment at Mid Ht = Msa (lb-in/ft)	7265	23269	64788	25724	17722	69697	
Ase (in^2) = (Ps + As*fy) / fy	0.72	0.69	0.69	0.71	0.72	0.72	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.281	0.281	0.281	0.281	0.281	0.281	
CE = C ELASTIC = k * d	1.88	1.88	1.88	1.88	1.88	1.88	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	145.86	141.83	141.83	143.85	145.86	145.86	
M_1 = Msa (lb-in)	7265	23269	64788	25724	17722	69697	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	7694	24307	67679	27055	18768	73812	
Ie_2 (in4)	614	614	614	614	614	614	
M_3 (lb-in)	7694	24307	67679	27055	18768	73812	
Ie_3 (in4)	614	614	614	614	614	614	
M_4 (lb-in)	7694	24307	67679	27055	18768	73812	
Ie_4 (in4)	614	614	614	614	614	614	
M_5 (lb-in)	7694	24307	67679	27055	18768	73812	
Ie_5 (in4)	614	614	614	614	614	614	
M_6 (lb-in)	7694	24307	67679	27055	18768	73812	
Ie_6 (in4)	614	614	614	614	614	614	
M_7 (lb-in)	7694	24307	67679	27055	18768	73812	
Ie_7 (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.07	0.22	0.61	0.24	0.17	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 54
Wall Description = Solid Panel

Wall Ht =	38.44	ft
b =	0.01	ft
c =	0.01	ft
e =	1.00	ft
d =	38.43	ft
a =	0.005	ft

Wall Weight at Mid Height

Wt of Concrete=	150	pcf
Wall Thickness=	9.25	in.
Concentric Load=	11	plf
Seismic Fp=.4Sd*=	0.3652	Wp



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	6.75	inch
equiv DL =	382.905	plf
equiv SL =	797.71875	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	16.1	psf
P seismic equiv =	42.4	psf

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

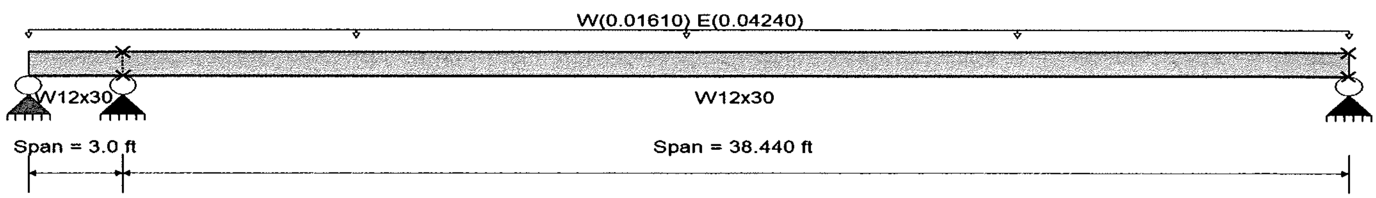
DESCRIPTION: Panel 54 Solid Panel Moments

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.01610, E = 0.04240 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.045** : 1	Maximum Shear Stress Ratio =	**0.026** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	7.268 k-ft	Vu : Applied	2.486 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.138 in	Ratio =	3,347	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.001 in	Ratio =	34,447	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.096 in	Ratio =	4782	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.001 in	Ratio =	49210	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.009	0.005		-1.38	1.38	179.58	161.63	1.00	1.00	0.47	95.94	95.94
Dsgn. L = 38.44 ft	2	0.009	0.002	0.88	-1.38	1.38	179.58	161.63	1.00	1.00	0.19	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.017	0.010		-2.76	2.76	179.58	161.63	1.00	1.00	0.94	95.94	95.94
Dsgn. L = 38.44 ft	2	0.017	0.004	1.75	-2.76	2.76	179.58	161.63	1.00	1.00	0.38	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.045	0.026		-7.27	7.27	179.58	161.63	1.00	1.00	2.49	95.94	95.94
Dsgn. L = 38.44 ft	2	0.045	0.010	4.62	-7.27	7.27	179.58	161.63	1.00	1.00	1.00	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

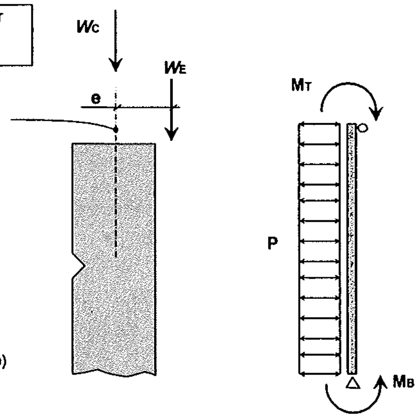
AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	54
Wall Description =	Solid Panel

See ACI 11.8.2.2 for distribution of concentrated forces

℄ of structural thickness



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	38.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	1.00
Number of Bars Ea Face (or at Center) of Pier	1.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 12" o.c.
Max Deflection	L / 1374
% of Flexural Capacity	64%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	11	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (W_E)

		Moment at Top (lb-ft/ft) = W_E * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 M_{TOP}
Eccentricity (in)	6.75		
Dead - D (plf)	382.905	D = 215	D = 108
Snow - S (plf)	797.71875	S = 449	S = 224
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(M_{TOP})	(M_{BOT})	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (M_{TOP} + M_{BOT})
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-7268	E = -3634
Wind - W (lb-ft/ft)	0	-2760	W = -1380

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	42.4	E = 7838
Wind - W (psf)	16.1	W = 2970

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	2616
Snow - S (plf)	798
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	108
Snow - S (lb-ft/ft)	224
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	4204
Wind - W (lb-ft/ft)	1590

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

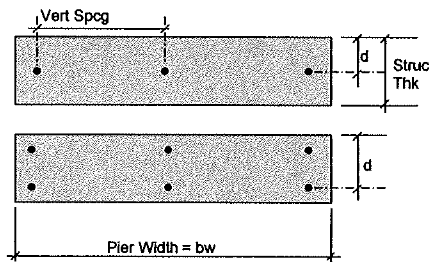
Parameter	Value	Note
Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	38.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	65
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	12	OK
As per foot (in^2/ft)	0.31	(This is the area of tension steel only)
Total As in Pier (in^2)	0.31	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρ_v min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0055	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0038	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



OK

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 54
Wall Description = Solid Panel

IBC-2015	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	3663	3538	4416	4416	3538	4176	2355	1877	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1809	2897	5858	15399	21977	54124	20244	51379	
Pu / Ag (psi)	36	35	43	43	35	41	23	18	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.35	0.34	0.35	0.35	0.34	0.35	0.33	0.33	

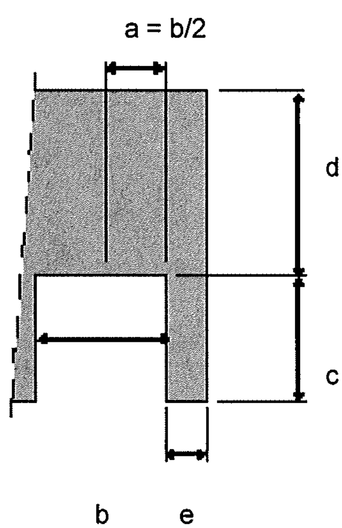
a (in) =(Ase*fy) / (0.85*fc*lw)	0.41	0.41	0.42	0.42	0.41	0.41	0.39	0.38	
Cu = C ULTIMATE = a / β_1	0.51	0.51	0.52	0.52	0.51	0.52	0.49	0.48	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	95.47	95.16	97.33	97.33	95.16	96.74	92.21	91.00	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2517	3983	8779	23075	30215	79185	24907	60534	
Mn (lb-in) = Ase * fy * (d - a/2)	134454	133957	137458	137458	133957	136500	129226	127313	
Cu / d	0.08	0.08	0.08	0.08	0.08	0.08	0.07	0.07	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	121009	120562	123712	123712	120562	122850	116303	114582	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	2%	3%	7%	19%	25%	64%	21%	53%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	3414	2616	2616	3015	3414	3414	
Applied Moment at Mid Ht = Msa (lb-in/ft)	3985	12741	36609	14087	9709	39301	
Ase (in^2) = (Ps + As*fy) / fy	0.36	0.35	0.35	0.36	0.36	0.36	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.209	0.209	0.209	0.209	0.209	0.209	
CE = C ELASTIC = k * d	1.40	1.40	1.40	1.40	1.40	1.40	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	84.16	81.48	81.48	82.82	84.16	84.16	
M_1 = Msa (lb-in)	3985	12741	36609	14087	9709	39301	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	4110	13046	37487	14478	10015	40541	
Ie_2 (in4)	614	614	614	614	614	614	
M_3 (lb-in)	4110	13046	37487	14478	10015	40541	
Ie_3 (in4)	614	614	614	614	614	614	
M_4 (lb-in)	4110	13046	37487	14478	10015	40541	
Ie_4 (in4)	614	614	614	614	614	614	
M_5 (lb-in)	4110	13046	37487	14478	10015	40541	
Ie_5 (in4)	614	614	614	614	614	614	
M_6 (lb-in)	4110	13046	37487	14478	10015	40541	
Ie_6 (in4)	614	614	614	614	614	614	
M_7 (lb-in)	4110	13046	37487	14478	10015	40541	
Ie_7 (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.04	0.12	0.34	0.13	0.09	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 54
Wall Description = Solid Par

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 128
Wall Description = CENTER Pier

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**	
b =	12.67	ft	Wt of Concrete=	150 pcf
c =	7	ft	Wall Thickness=	9.25 in.
e =	5.00	ft	Concentric Load=	2816 plf
d =	31.44	ft	Seismic Fp=.4Sd*=	0.3652 Wp
a =	6.335	ft		



Roof Weight

Joist Span=	63.5 feet
Dead Load=	12 psf
Snow Load=	25 psf
Live Roof =	0 psf
Live Floor=	0 psf
eccentricity	2.25 inch
equiv DL =	863.727 plf
equiv SL =	1799.4313 plf
equiv Lr =	0 plf
equiv LL =	0 plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	36.3	psf
P seismic equiv =	92.2	psf

Steel Beam

Project File: Panattoni Tumwater.ec6

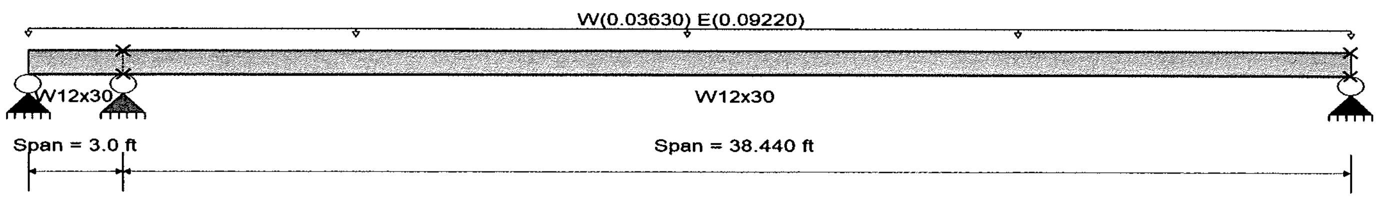
DESCRIPTION: Panel 28 center pier

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.03630, E = 0.09220 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.098** : 1	Maximum Shear Stress Ratio =	**0.056** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	15.804 k-ft	Vu : Applied	5.406 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.300 in	Ratio = 1,539	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in	Ratio = 15,841	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.210 in	Ratio = 2199	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.002 in	Ratio = 22630	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

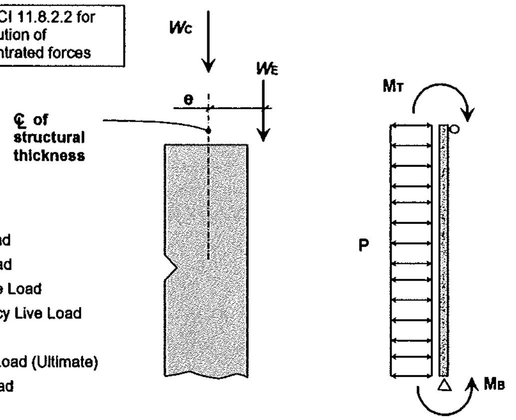
Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.019	0.011		-3.11	3.11	179.58	161.63	1.00	1.00	1.06	95.94	95.94
Dsgn. L = 38.44 ft	2	0.019	0.004	1.98	-3.11	3.11	179.58	161.63	1.00	1.00	0.43	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.038	0.022		-6.22	6.22	179.58	161.63	1.00	1.00	2.13	95.94	95.94
Dsgn. L = 38.44 ft	2	0.038	0.009	3.95	-6.22	6.22	179.58	161.63	1.00	1.00	0.86	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.098	0.056		-15.80	15.80	179.58	161.63	1.00	1.00	5.41	95.94	95.94
Dsgn. L = 38.44 ft	2	0.098	0.023	10.04	-15.80	15.80	179.58	161.63	1.00	1.00	2.18	95.94	95.94

Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	128
Wall Description =	Center pier

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	38.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	5.00
Number of Bars Ea Face (or at Center) of Pier	8.96
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 7" o.c.
Max Deflection	L / 518
% of Flexural Capacity	91%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	2816	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (W_E)

		Moment at Top (lb-ft/ft) = W_E * e		Moment at Mid-Ht (lb-ft/ft) = 1/2 M_{TOP}	
Eccentricity (in)	2.25				
Dead - D (plf)	863.727	D =	162	D =	81
Snow - S (plf)	1799.43125	S =	337	S =	169
Roof Live - Lr (plf)	0	Lr =	0	Lr =	0
Occupancy Live - L (plf)	0	L =	0	L =	0
Soil - H (plf)	0	H =	0	H =	0

Uniform Moments Applied

	(M_{TOP})	(M_{BOT})	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (M_{TOP} + M_{BOT})	
Dead - D (lb-ft/ft)	0	0	D =	0
Snow - S (lb-ft/ft)	0	0	S =	0
Roof Live - Lr (lb-ft/ft)	0	0	Lr =	0
Occupancy Live - L (lb-ft/ft)	0	0	L =	0
Soil - H (lb-ft/ft)	0	0	H =	0
Seismic (Ultimate) - E (lb-ft/ft)	0	-15804	E =	-7902
Wind - W (lb-ft/ft)	0	-6222	W =	-3111

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2	
Seismic (Ultimate) - E (psf)	92.2	E =	17037
Wind - W (psf)	36.3	W =	6700

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	5902
Snow - S (plf)	1799
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	81
Snow - S (lb-ft/ft)	169
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	9135
Wind - W (lb-ft/ft)	3589

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Parameter	Value	Note
Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	38.44	
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	**OK**
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	6.7	**OK**
As per foot (in^2/ft)	0.55	(This is the area of tension steel only)
Total As in Pier (in^2)	0.55	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.79	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0099	**OK** Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	**OK**
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	**OK**
ρ	0.0068	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	**OK**
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

OK

The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.

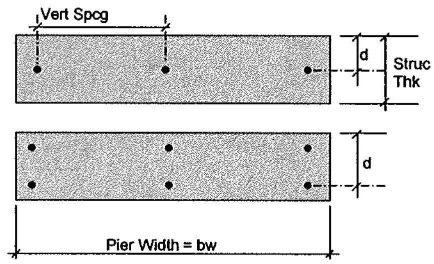
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 128
Wall Description = Center pier

IBC-2015	U = 1.4D Load Combo 16-1	U = 1.2D + 1.6(L+H) + 0.5(Lr or S) Load Combo 16-2	U = 1.2D + 1.6(Lr or S) + f_1 L Load Combo 16-3(a)	U = 1.2D + 1.6(Lr or S) + 0.5W Load Combo 16-3(b)	U = 1.2D + W + f_1 L + 0.5(Lr or S) Load Combo 16-4	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2 Load Combo 16-5*	U = 0.9D + W + 1.6H Load Combo 16-6	U = (0.9-0.2Sds)D + 1.0E + 1.6H Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	8262	7982	9961	9961	7982	9419	5312	4234	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1360	2178	4405	25937	45241	112376	43938	110313	
Pu / Ag (psi)	81	78	98	98	78	92	52	42	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.64	0.63	0.65	0.65	0.63	0.65	0.61	0.59	

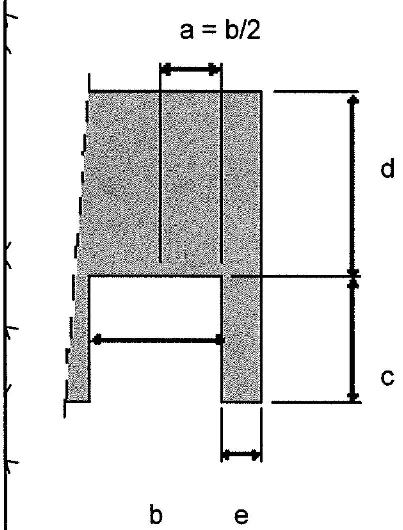
a (in) =(Ase*fy) / (0.85*f'c*lw)	0.75	0.75	0.77	0.77	0.75	0.76	0.71	0.70	
Cu = C ULTIMATE = a / β_1	0.94	0.93	0.96	0.96	0.93	0.95	0.89	0.87	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	154.86	154.34	158.00	158.00	154.34	157.00	149.28	147.19	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2235	3509	8192	48233	72875	200632	59447	139796	
Mn (lb-in) = Ase * fy * (d - a/2)	241275	240216	247675	247675	240216	245636	230105	226009	
Cu / d	0.14	0.14	0.14	0.14	0.14	0.14	0.13	0.13	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	217148	216195	222907	222907	216195	221072	207095	203408	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	1%	2%	4%	22%	34%	91%	29%	69%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	7701	5902	5902	6801	7701	7701	
Applied Moment at Mid Ht = Msa (lb-in/ft)	2996	26810	77703	27822	15915	79727	
Ase (in^2) = (Ps + As*fy) / fy	0.68	0.65	0.65	0.66	0.68	0.68	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.268	0.268	0.268	0.268	0.268	0.268	
Ce = C ELASTIC = k * d	1.80	1.80	1.80	1.80	1.80	1.80	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-Ce)^2 + 1/3*ℓw*Ce^3	139.87	134.71	134.71	137.29	139.87	139.87	
M_1 = Msa (lb-in)	2996	26810	77703	27822	15915	79727	
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	595	614	614	561	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	3218	28305	82189	29626	17094	86237	
Ie_2 (in4)	614	614	523	614	614	473	
M_3 (lb-in)	3218	28305	82840	29626	17094	87574	
Ie_3 (in4)	614	614	514	614	614	458	
M_4 (lb-in)	3218	28305	82937	29626	17094	87857	
Ie_4 (in4)	614	614	513	614	614	455	
M_5 (lb-in)	3218	28305	82952	29626	17094	87918	
Ie_5 (in4)	614	614	513	614	614	454	
M_6 (lb-in)	3218	28305	82954	29626	17094	87930	
Ie_6 (in4)	614	614	513	614	614	454	
M_7 (lb-in)	3218	28305	82954	29626	17094	87933	
Ie_7 (in4)	614	614	513	614	614	454	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.03	0.25	0.89	0.27	0.15	E+S is N/A	
	OK	OK	OK	OK	OK	OK	OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 128
Wall Description = Center pi

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 128
Wall Description = End Pier

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**		
b =	6.33	ft	Wt of Concrete=	150	pcf
c =	7	ft	Wall Thickness=	9.25	in.
e =	3.67	ft	Concentric Load=	1917	plf
d =	31.44	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	3.165	ft			



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	2.25	inch
equiv DL =	709.57357	plf
equiv SL =	1478.2783	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	29.8	psf
P seismic equiv =	76.3	psf

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

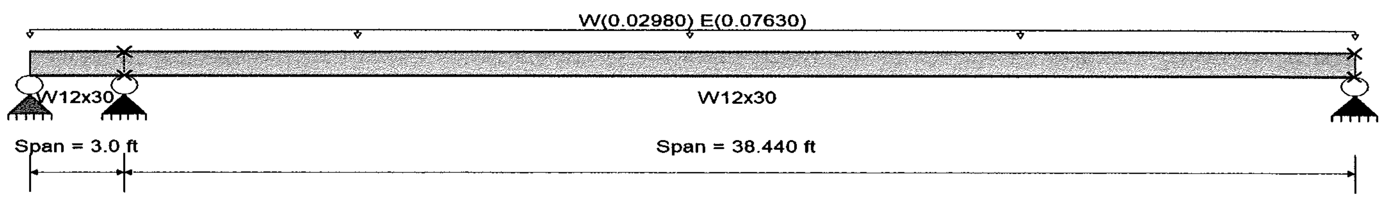
DESCRIPTION: Panel 28 end pier

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.02980, E = 0.07630 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.081** : 1	Maximum Shear Stress Ratio =	**0.047** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	13.079 k-ft	Vu : Applied	4.474 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.248 in Ratio =	1,860	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.002 in Ratio =	19,142	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.174 in Ratio =	2658	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.001 in Ratio =	27346	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.016	0.009		-2.55	2.55	179.58	161.63	1.00	1.00	0.87	95.94	95.94
Dsgn. L = 38.44 ft	2	0.016	0.004	1.62	-2.55	2.55	179.58	161.63	1.00	1.00	0.35	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.032	0.018		-5.11	5.11	179.58	161.63	1.00	1.00	1.75	95.94	95.94
Dsgn. L = 38.44 ft	2	0.032	0.007	3.25	-5.11	5.11	179.58	161.63	1.00	1.00	0.71	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.081	0.047		-13.08	13.08	179.58	161.63	1.00	1.00	4.47	95.94	95.94
Dsgn. L = 38.44 ft	2	0.081	0.019	8.31	-13.08	13.08	179.58	161.63	1.00	1.00	1.81	95.94	95.94

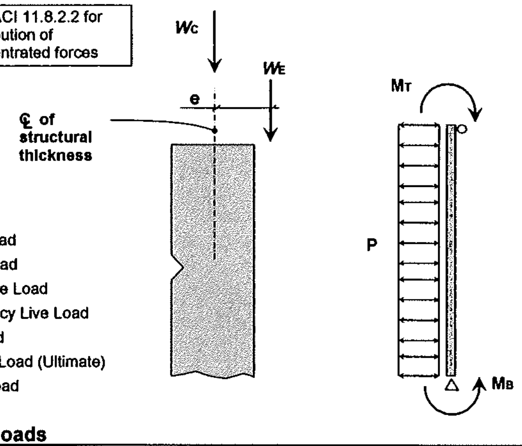
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	128
Wall Description =	End pier

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	38.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	3.67
Number of Bars Ea Face (or at Center) of Pier	4.98
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 9" o.c.
Max Deflection	L / 768
% of Flexural Capacity	97%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (W_C)

Dead - D (plf)	1917 (tributary wall weight at midheight)
Snow - S (plf)	0
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Uniform Eccentric Applied Loads (W_E)

Eccentricity (in) = 2.25

	Load	Moment at Top (lb-ft/ft) = WE * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 MTOP
Dead - D (plf)	709.5735695	D = 133	D = 67
Snow - S (plf)	1478.27827	S = 277	S = 139
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(MTOP)	(MBOT)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (MTOP + MBOT)
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-13079	E = -6540
Wind - W (lb-ft/ft)	0	-5108	W = -2554

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	76.3	E = 14087
Wind - W (psf)	29.8	W = 5504

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	4848
Snow - S (plf)	1478
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	67
Snow - S (lb-ft/ft)	139
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	7547
Wind - W (lb-ft/ft)	2950

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	38.44	The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 ; 0.625 in
Vert Rebar o.c. Spacing (in)	8.85	OK
As per foot (in^2/ft)	0.42	(This is the area of tension steel only)
Total As in Pier (in^2)	0.42	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	1.36	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0075	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0052	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

OK

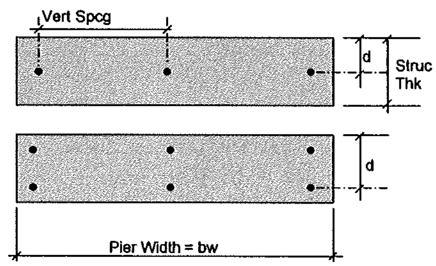
Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

65

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 128
Wall Description = End pier

IBC-2015	U = 1.4D	U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	U = 1.2D + 1.6(Lr or S) + f_1 L	U = 1.2D + 1.6(Lr or S) + 0.5W	U = 1.2D + W + f_1 L + 0.5(Lr or S)	U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	U = 0.9D + W + 1.6H	U = (0.9-0.2Sds)D + 1.0E + 1.6H	
	Load Combo 16-1	Load Combo 16-2	Load Combo 16-3(a)	Load Combo 16-3(b)	Load Combo 16-4	Load Combo 16-5*	Load Combo 16-6	Load Combo 16-7*	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	6788	6557	8183	8183	6557	7738	4364	3478	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1118	1789	3619	21318	37188	92836	36117	91141	
Pu / Ag (psi)	67	64	80	80	64	76	43	34	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.49	0.49	0.50	0.50	0.49	0.50	0.46	0.45	

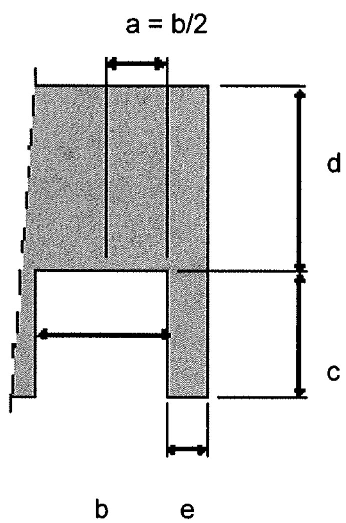
a (in) =(Ase*fy) / (0.85*f'c*lw)	0.57	0.57	0.59	0.59	0.57	0.59	0.54	0.53	
Cu = C ULTIMATE = a / β_1	0.72	0.71	0.74	0.74	0.71	0.73	0.68	0.67	
Icr U (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	126.59	126.09	129.59	129.59	126.09	128.64	121.29	119.32	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	1842	2892	6739	39701	60106	166098	49058	115916	
Mn (lb-in) = Ase * fy * (d - a/2)	187365	186469	192779	192779	186469	191054	177923	174463	
Cu / d	0.11	0.11	0.11	0.11	0.11	0.11	0.10	0.10	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	168628	167822	173501	173501	167822	171949	160131	157017	
ϕMn > Mcr ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Mu / ϕMn	1%	2%	4%	23%	36%	97%	31%	74%	**POSITIVI**
ϕMn > Mu ?	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 128
Wall Description = End pier

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E	
D	1	1	1	1	1	1	
S	1	0	0	0.5	1	1	
Lr	0	0	0	0	0	0	
L	1	1	1	1	1	1	
H	1	1	1	1	1	1	
E	0	0	0.70	0	0	0.70	
W	0	0.6	0	0.6	0.3	0	
Axial Load at Mid Ht = Ps (lb/ft)	6327	4848	4848	5588	6327	6327	
Applied Moment at Mid Ht = Msa (lb-in/ft)	2461	22037	64196	22869	13081	65859	
Ase (in^2) = (Ps + As*fy) / fy	0.52	0.50	0.50	0.51	0.52	0.52	
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.238	0.238	0.238	0.238	0.238	0.238	
CE = C ELASTIC = k * d	1.59	1.59	1.59	1.59	1.59	1.59	
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	113.54	108.94	108.94	111.24	113.54	113.54	
M_1 = Msa (lb-in)	2461	22037	64196	22869	13081	65859	
I_{e1} (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	614	614	614	614	
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * I_{e1}))	2609	23038	67109	24073	13866	69814	
I_{e2} (in4)	614	614	614	614	614	614	
M_3 (lb-in)	2609	23038	67109	24073	13866	69814	
I_{e3} (in4)	614	614	614	614	614	614	
M_4 (lb-in)	2609	23038	67109	24073	13866	69814	
I_{e4} (in4)	614	614	614	614	614	614	
M_5 (lb-in)	2609	23038	67109	24073	13866	69814	
I_{e5} (in4)	614	614	614	614	614	614	
M_6 (lb-in)	2609	23038	67109	24073	13866	69814	
I_{e6} (in4)	614	614	614	614	614	614	
M_7 (lb-in)	2609	23038	67109	24073	13866	69814	
I_{e7} (in4)	614	614	614	614	614	614	
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752	
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * I_{e7})	0.02	0.21	0.60	0.22	0.12	E+S is N/A	
	OK	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

Job Name = Tumwater
Job Number = 2210856.2
Wall Type = 128
Wall Description = End Pier w/ Window

Wall Ht =	38.44	ft	**Wall Weight at Mid Height**		
b =	7.5	ft	Wt of Concrete=	150	pcf
c =	3	ft	Wall Thickness=	9.25	in.
e =	2.50	ft	Concentric Load=	3333	plf
d =	35.44	ft	Seismic Fp=.4Sd*=	0.3652	Wp
a =	3.75	ft			



Roof Weight

Joist Span=	63.5	feet
Dead Load=	12	psf
Snow Load=	25	psf
Live Roof =	0	psf
Live Floor=	0	psf
eccentricity	2.25	inch
equiv DL =	952.5	plf
equiv SL =	1984.375	plf
equiv Lr =	0	plf
equiv LL =	0	plf

Equivalent Wind and Seismic Load

P wind =	16.0	psf
P seismic =	42.2	psf
P wind equiv =	40.0	psf
P seismic equiv =	104.8	psf

Steel Beam

Project File: Panattoni Tumwater.ec6

LIC# : KW-06014847, Build:20.22.2.9 AHBL, INC (c) ENERCALC INC 1983-2022

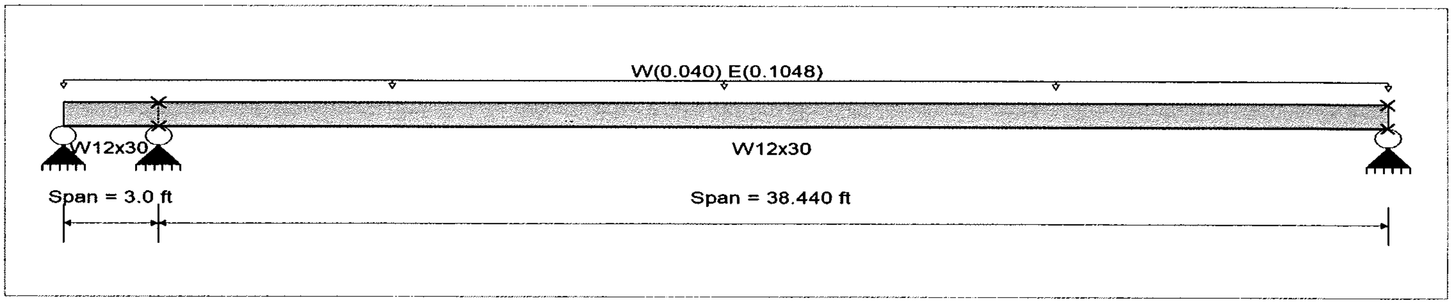
DESCRIPTION: Panel 28 end pier w/ window

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-10

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.040, E = 0.1048 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.111** : 1	Maximum Shear Stress Ratio =	**0.064** : 1
Section used for this span	**W12x30**	Section used for this span	**W12x30**
Mu : Applied	17.964 k-ft	Vu : Applied	6.145 k
Mn * Phi : Allowable	161.625 k-ft	Vn * Phi : Allowable	95.940 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	3.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.340 in	Ratio =	1,354	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.003 in	Ratio =	13,936	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.238 in	Ratio =	1935	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.002 in	Ratio =	19909	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
Dsgn. L = 3.00 ft	1		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
Dsgn. L = 38.44 ft	2		0.000				179.58	161.63	1.00	1.00	-0.00	95.94	95.94
+0.50W													
Dsgn. L = 3.00 ft	1	0.021	0.012		-3.43	3.43	179.58	161.63	1.00	1.00	1.17	95.94	95.94
Dsgn. L = 38.44 ft	2	0.021	0.005	2.18	-3.43	3.43	179.58	161.63	1.00	1.00	0.47	95.94	95.94
W Only													
Dsgn. L = 3.00 ft	1	0.042	0.024		-6.86	6.86	179.58	161.63	1.00	1.00	2.35	95.94	95.94
Dsgn. L = 38.44 ft	2	0.042	0.010	4.36	-6.86	6.86	179.58	161.63	1.00	1.00	0.95	95.94	95.94
E Only													
Dsgn. L = 3.00 ft	1	0.111	0.064		-17.96	17.96	179.58	161.63	1.00	1.00	6.15	95.94	95.94
Dsgn. L = 38.44 ft	2	0.111	0.026	11.42	-17.96	17.96	179.58	161.63	1.00	1.00	2.48	95.94	95.94

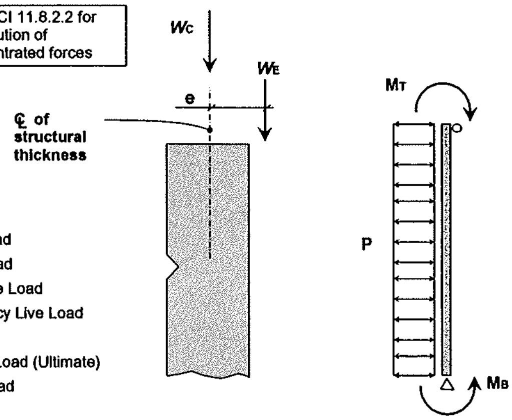
Alternate Concrete Slender Wall Design (ACI 318-14 Sect 11.8)

AHBL

If you need to make modifications to any other part of the spreadsheet besides the yellow cells the password is "save"

Job Name =	Tumwater
Job Number =	2210856.20
Wall Type =	128
Wall Description =	End Pier w/ Window

See ACI 11.8.2.2 for distribution of concentrated forces



D = Dead Load
S = Snow Load
Lr = Roof Live Load
L = Occupancy Live Load
H = Soil Load
E = Seismic Load (Ultimate)
W = Wind Load

DESIGN SUMMARY	
Wall Ht Btwn Supports (ft)	38.44
Total Wall Ht w/ Parapet (ft)	38.44
Total Wall Thickness (in)	9.25
Reveal Depth (in)	0.75
Structural Thickness (in)	8.5
Pier Width (ft)	2.50
Number of Bars Ea Face (or at Center) of Pier	5.00
Concrete Strength (psi)	5000
Reinforcement	(2) Layer #5 Rebar @ 6" o.c.
Max Deflection	L / 321
% of Flexural Capacity	95%

Hand Input
Potential Hand Input
Output

OK

Applied Loads

S	What is the controlling type of roof load? Snow or Roof Live Load? (Enter "S" or "Lr")
NO	Are you applying occupancy live loads for places of public assembly, or live loads in excess of 100 psf, or parking garage live loads? (YES: f_1 = 1.0, NO: f_1 = 0.5)
YES	Do you have a roof config that prevents snow from shedding off the structure? (YES: f_2 = 0.7, NO: f_2 = 0.2)
YES	Is the design snow load less than or equal to 30 psf?
0.913	Seismic: Sds

f_1 = 0.5
f_2 = 0.7

Uniform Concentric Applied Loads (*Wc*)

Dead - D (plf)	3333	(tributary wall weight at midheight)
Snow - S (plf)	0	
Roof Live - Lr (plf)	0	
Occupancy Live - L (plf)	0	
Soil - H (plf)	0	

Uniform Eccentric Applied Loads (*We*)

		Moment at Top (lb-ft/ft) = We * e	Moment at Mid-Ht (lb-ft/ft) = 1/2 Mtop
Eccentricity (in)	2.25		
Dead - D (plf)	952.5	D = 179	D = 89
Snow - S (plf)	1984.375	S = 372	S = 186
Roof Live - Lr (plf)	0	Lr = 0	Lr = 0
Occupancy Live - L (plf)	0	L = 0	L = 0
Soil - H (plf)	0	H = 0	H = 0

Uniform Moments Applied

	(Mtop)	(Mbot)	Moment @ Mid-Ht (lb-ft/ft) = 1/2 (Mtop + Mbot)
Dead - D (lb-ft/ft)	0	0	D = 0
Snow - S (lb-ft/ft)	0	0	S = 0
Roof Live - Lr (lb-ft/ft)	0	0	Lr = 0
Occupancy Live - L (lb-ft/ft)	0	0	L = 0
Soil - H (lb-ft/ft)	0	0	H = 0
Seismic (Ultimate) - E (lb-ft/ft)	0	-17964	E = -8982
Wind - W (lb-ft/ft)	0	-6857	W = -3429

The uniform moments applied to the top and bottom of the wall can be used to model loads from a wall above or below, or to model lateral parapet forces. Enter *positive numbers to increase* the moment induced at the mid-height of the wall being designed and *negative numbers to reduce* the moment.

Note that soil forces are not allowed to counteract wind or seismic forces. In addition, soil forces that counteract other forces are not allowed to be factored and should be accounted for in hand calcs.

Equivalent Uniform Lateral Applied Loads (P)

		Moment @ Mid-Ht (lb-ft/ft) = 1/8 PL^2
Seismic (Ultimate) - E (psf)	104.8	E = 19356
Wind - W (psf)	40.0	W = 7388

Total Uniform Axial Load at Mid-Height of Wall

Dead - D (plf)	6508
Snow - S (plf)	1984
Roof Live - Lr (plf)	0
Occupancy Live - L (plf)	0
Soil - H (plf)	0

Total Uniform Moment at Mid-Height of Wall

Dead - D (lb-ft/ft)	89
Snow - S (lb-ft/ft)	186
Roof Live - Lr (lb-ft/ft)	0
Occupancy Live - L (lb-ft/ft)	0
Soil - H (lb-ft/ft)	0
Seismic (Ultimate) - E (lb-ft/ft)	10374
Wind - W (lb-ft/ft)	3960

Note that these totals represent the *unfactored* forces at the mid-height of the wall *including* the self wt of the wall (this spreadsheet automatically calcs wall self wt). P-Δ effects have *not* been accounted for. These forces can be overridden by entering your own mid-height axial loads and moments determined from hand calculations. You will still have to enter information describing the loads so that the proper f_1, f_2 and f_3 load factors are properly applied. Remember to enter the loads unfactored and include the self-weight of the section of wall being analyzed.

Wall Parameters

Parameter	Value	Note
Wall Height Between Supports (ft)	38.44	(Not including parapet)
Parapet Height (ft)	0	(This is used to calc the self-weight of the wall only)
Total Wall Height (ft)	38.44	
Concrete Strength f'c (psi)	5000	
Concrete Unit Weight (pcf)	150	
Rebar Yield Stress fy (psi)	60000	
Width of Pier Being Designed (ft)	1	(Width of pier, or enter 1 ft for analyzing unit width)
Total Wall Thickness (in)	9.25	
Depth of Reveal (in)	0.75	
Structural Thickness (in)	8.50	= Total Thk - Reveal Depth
(1) or (2) Layers of Reinf?	2	OK
Vert Rebar Size	5	0.31 in^2 — 0.625 in
Vert Rebar o.c. Spacing (in)	6	OK
As per foot (in^2/ft)	0.61	(This is the area of tension steel only)
Total As in Pier (in^2)	0.61	(This is the area of tension steel only)
Number of Bars within Pier (Ea Face)	2.00	
Are You Providing Confinement Reinf?	NO	
Confinement Rebar Size	3	0 in
Conc Cover at Ext Side of Wall Exp to Weather/Earth (in)	1.5	
Conc Cover at Int Side of Wall Not Exp to Weather/Earth (in)	1	
Min Depth to Tension Rebar = d (in)	6.7	(w/ 2 layers of rebar, d = Struc Width - Max Cover - Confine ϕ - 1/2 Vert ϕ)
Min Vertical Steel Ratio - ρ_v min	0.0025	(ρv min may be reduced if the shear force is low. See ACI 21.7.2)
Actual Vertical Steel Ratio - ρ_v	0.0111	OK — Based on total wall thk not struc thk = (Rebar A * # Layers / Spacing) / (Total Thk)
Min Tensile Flexural Reinf 1 = As min 1 (in^2/ft)	0.28	OK
Min Tensile Flexural Reinf 2 = As min 2 (in^2/ft)	0.27	OK
ρ	0.0076	= As per ft / (12 * d)
ρmax = 0.6 ρb = 0.6 * 0.85 * β_1 * fc / fy * 87000 / (87000 + fy)	0.0201	OK
Ec (psi)	4030509	= 57000 * sqrt (f'c)
Es (psi)	29000000	
n	7.2	= Es / Ec
ℓw (in)	12	= 12"
Ag (in^2/ft)	102	= Struc Thk * 12
0.06 f'c (psi)	300	
ℓc (in)	461.28	= Wall Ht * 12
β_1	0.8	
Ig (in^4/ft)	614	= 1/12 * 12 * Struc Thk^3
fr (psi)	530	= 7.5 * sqrt (f'c)
yt (in)	4.25	= Struc Thk / 2
Mcr (lb-in)	76633	= fr * Ig / yt
ℓc / 150 (in)	3.0752	

OK

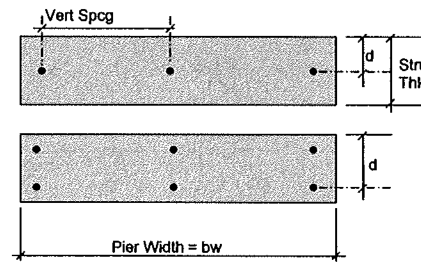
The width of the pier doesn't affect the structural design since loads are input **per linear foot**. Pier width is for your reference so you can track your calculations. This **does** calculate the actual number of bars required within the pier width you input.

Per ACI 11.7.4.1 lateral ties need not be provided where vert reinf is not req'd as compression reinf. Thus walls designed using this method do not need to have confinement steel. But in many cases is still advisable, particularly with 2 layers of rebar.

Verify "d" with hand calcs also

Rebar	Dia (in)	A (in^2)
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56

ACI Min Cover Reqments:	
Exposed to Weather:	#5 & Smaller - 1 1/2"
	#6 & Larger - 2"
Not Exposed to Weather:	#11 & Smaller = 3/4"



Job Name = **Tumwater**
Job Number = 2210856.2
Wall Type = 128
Wall Description = End Pier w/ Window

IBC-2015	Load Combo 16-1 U = 1.4D	Load Combo 16-2 U = 1.2D + 1.6(L+H) + 0.5(Lr or S)	Load Combo 16-3(a) U = 1.2D + 1.6(Lr or S) + f_1 L	Load Combo 16-3(b) U = 1.2D + 1.6(Lr or S) + 0.5W	Load Combo 16-4 U = 1.2D + W + f_1 L + 0.5(Lr or S)	Load Combo 16-5* U = (1.2+0.2Sds)D + 1.0E + f_1 L + f_2	Load Combo 16-6 U = 0.9D + W + 1.6H	Load Combo 16-7* U = (0.9-0.2Sds)D + 1.0E + 1.6H	*ASCE 7 12.4.2.3
D	1.4	1.2	1.2	1.2	1.2	1.3826	0.9	0.7174	
S	0	0.5	1.6	1.6	0.5	0.7	0	0	
Lr	0	0	0	0	0	0	0	0	
L	0	1.6	0.5	0	0.5	0.5	0	0	
H	0	1.6	0	0	0	0	1.6	1.6	
E	0	0	0	0	0	1.0	0	1.0	
W	0	0	0	0.5	1	0	1	0	
Factored Axial Load at Mid Ht = Pu (lb/ft)	9112	8802	10985	10985	8802	10387	5857	4669	
Factored Applied Moment at Mid Ht = Mua (lb-in/ft)	1500	2402	4858	28616	49918	127536	48480	125261	
Pu / Ag (psi)	89	86	108	108	86	102	57	46	
Vert Stress at Mid-Ht Wall ok? Pu / Ag < 0.06 f'c?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Ase (in^2) = (Pu(h/2d) + As*fy) / fy	0.71	0.71	0.73	0.73	0.71	0.72	0.68	0.66	

a (in) =(Ase*fy) / (0.85*f'c*lw)	0.84	0.83	0.86	0.86	0.83	0.85	0.79	0.78	
Cu = C ULTIMATE = a / β_1	1.04	1.04	1.07	1.07	1.04	1.06	0.99	0.98	
Icr u (in4) = Icr ULTIMATE = n*Ase*(d-Cu)^2 + 1/3*ℓw*Cu^3	167.27	166.73	170.48	170.48	166.73	169.46	161.53	159.39	
Mu (lb-in) = Mua / (1 - (5 * Pu * Lc^2)/(0.75 * 48 * Ec * Icr)) =	2498	3919	9208	54243	81445	231649	66039	159525	
Mn (lb-in) = Ase * fy * (d - a/2)	267132	265981	274085	274085	265981	271870	254988	250531	
Cu / d	0.16	0.16	0.16	0.16	0.16	0.16	0.15	0.15	
ϕ = 0.23 + 0.25 /(Cu / d)	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
ϕMn (lb-in)	240419	239382	246677	246677	239382	244683	229489	225478	
ϕMn > Mcr ?	OK	OK	OK	OK	OK	OK	OK	OK	OK
Mu / ϕMn	1%	2%	4%	22%	34%	95%	29%	71%	POSITIVI
ϕMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK

	A = D + L + (Lr or S)	A = D + L + .6W	A = D + L + 0.7E	A = D + L + .6W + S/2	A = D + L + S + .3W	A = D + L + S + .7E
D	1	1	1	1	1	1
S	1	0	0	0.5	1	1
Lr	0	0	0	0	0	0
L	1	1	1	1	1	1
H	1	1	1	1	1	1
E	0	0	0.70	0	0	0.70
W	0	0.6	0	0.6	0.3	0
Axial Load at Mid Ht = Ps (lb/ft)	8493	6508	6508	7500	8493	8493
Applied Moment at Mid Ht = Msa (lb-in/ft)	3304	29581	88216	30697	17559	90448
Ase (in^2) = (Ps + As*fy) / fy	0.76	0.72	0.72	0.74	0.76	0.76
k = Sqrt ((n*p)^2 + 2*n*p) - n*p	0.281	0.281	0.281	0.281	0.281	0.281
CE = C ELASTIC = k * d	1.88	1.88	1.88	1.88	1.88	1.88
Icr E (in4) = Icr ELASTIC = n*Ase*(d-CE)^2 + 1/3*ℓw*CE^3	152.15	146.65	146.65	149.40	152.15	152.15
M_1 = Msa (lb-in)	3304	29581	88216	30697	17559	90448
Ie_1 (in4) = { (Mcr / M)^3 * Ig + (1-(Mcr / M)^3) * Icr E } < Ig	614	614	453	614	614	433
M_2 (lb-in) = Msa / (1 - (5 * Ps * Lc^2) / (48 * Ec * Ie_1))	3576	31412	95782	32908	19004	101380
Ie_2 (in4)	614	614	386	614	614	352
M_3 (lb-in)	3576	31412	97230	32908	19004	104299
Ie_3 (in4)	614	614	376	614	614	335
M_4 (lb-in)	3576	31412	97509	32908	19004	105081
Ie_4 (in4)	614	614	374	614	614	331
M_5 (lb-in)	3576	31412	97563	32908	19004	105290
Ie_5 (in4)	614	614	373	614	614	330
M_6 (lb-in)	3576	31412	97574	32908	19004	105345
Ie_6 (in4)	614	614	373	614	614	330
M_7 (lb-in)	3576	31412	97576	32908	19004	105360
Ie_7 (in4)	614	614	373	614	614	330
ℓc / 150 (in)	3.0752	3.0752	3.0752	3.0752	3.0752	3.0752
Δs (in) = (5 * M_7 * Lc^2) / (48 * Ec * Ie_7)	0.03	0.28	1.44	0.29	0.17	E+S is N/A
	OK	OK	OK	OK	OK	OK

OK

Job Name = Tumwate
Job Number = 2E+06
Wall Type = 128
Wall Description = End Pier