ENGINEERING DIAGNOSTICS, LLC.
CALCULATION COVER SHEET

Project Name: Commerce Bank Building Columbia	Performed By: Campion	Date: 7-25-24
Project Number: F29-5077-K02	Checked By:	Date:

Calculation Package No.:

Scope: Design parapet masonry wall with reinforced CMU wall, use 8" CMU with partially grouted and reinforced with #4 at 32 inches on center reinforced of cell, maximum parapet wall height 6'-6".

Conclusions: Wall design OK.

STATE OF MISSOURI
TERRY A. CAMPION
NUMBER
E-20900
REGISTERED PROFESSIONAL ENGINEER
7/26/24

Hand Calculations/Computer Analysis

(File Name: ___________________)

CHECKLIST

Incl.	N/A		Incl.	N/A	Loading
X		Diagrams			Dead
		Dimensions			Live
		Assumptions			Self Weight
		Codes	X		Wind/Snow
X		Product Data			Thermal
		Equation Sources			Shrinkage/Creep
		Cost Sources			Settlement
		Quantity Take-off			Test

Variables/Boundary Conditions:

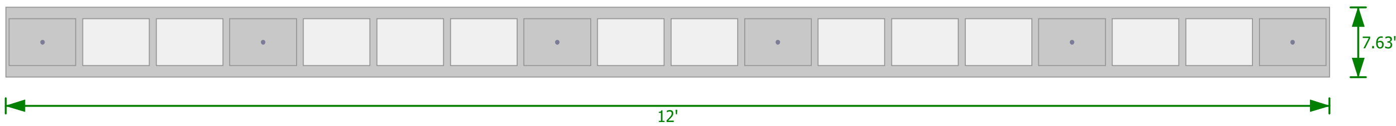
Result:

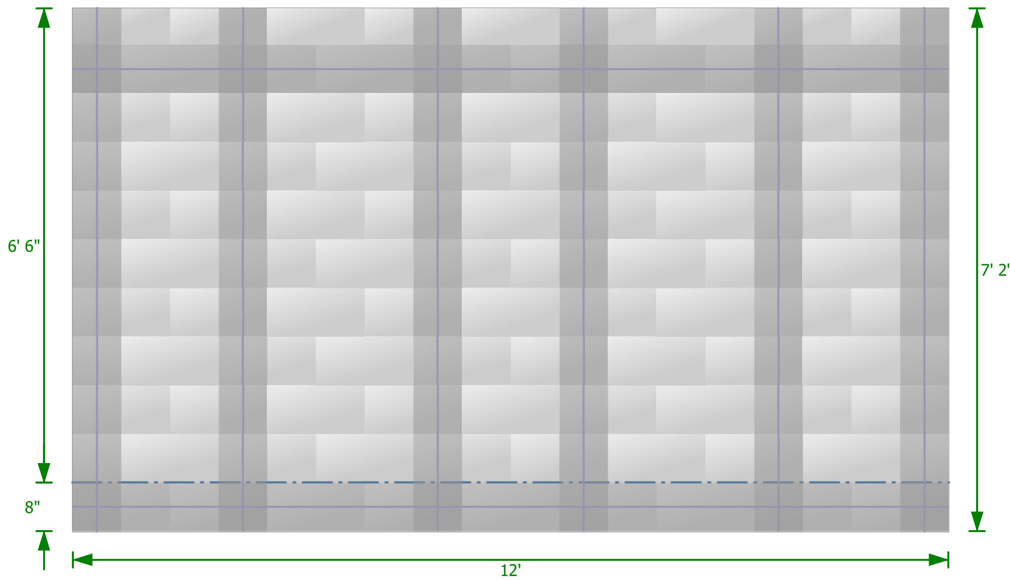
COMMERCE BANK COLUMBIA WIND LOAD AND CMU CELL REINFORCEMENT DEVELOPMENT										
COMMERCE BANK COLUMBIA WIND ON PARAPET WALL										
7/25/2024										
ASCE 7-16		IBC 2018		BASIC WIND SPEED 120 MPH						
P	=	0.00256	Kz	Kzt	Kd	Ke	V^2	Qz		
		0.00256	0.7	1	0.85	1	120	21.93408	PSF	
WINDWARD GCf		1.5	NOTE: ADDING BOTH WINDWARD AND LEEWARD CREATES LARGER PRESSURE THAN EITHER INDIVIDUAL GCf							
LEEWARD GCf		1								
		Qz	GCf	DESING PRESSURE						
		21.93408	1.5	33	PSF					
HILTI HY 200 ADHESIVE BAR DEVELOPMENT LENGTH										
F'c	4,000 PSI									
FROM ELEMASONRY SOFTWARE ANALYSIS DEMAND FLEXURAL MAXIMUM DEMAND ΦMn/Mu				0.4	NOTE: RATIO BASED ON MAXIMUM EITHER CMU STRENGTH OR REINFORCEMENT STRENGTH					
FROM HILTI HY 200 ADHESIVE DATA MAXIMUM BOND LOAD UNCRACKED CONCRETE #4 EMBED 6"				6,900	LB					
FROM HILTI HY 200 ADHESIVE DATA MAXIMUM DESIGN STRENGTH OF #4 BAR				17,000	LB					
MAXIMUM BAR TENSILE LOAD										
DEMAND RATIO	BAR DESIGN STRENGTH ΦN	BOND LOAD (DEMAND RATIO X BAR DESIGN STRENGTH)		DEMAND/ PROVIDED RATIO						
0.4	17,000	6,800		0.99	OK					

Terry Campion
Engineering Diagnostics LLC
123 Designer Ave.
Masonville, USA 12345
(123) 456-7890

Wall 1

Masonry Wall Design Calculations





Masonry Unit: CMU - 8 in.
Bond: Running
Grouting: Partial

Vertical Distributed Rebar:
No. 4, spaced as shown (1 per cell)
Horizontal (Bond Beam) Rebar:
No. 4, spaced as shown (1 per course)

Design Criteria

Building Code: TMS 402-2016 (Strength Design)
Choose Load Combinations Manually: No
Strength Check Load Combinations: ASCE 7-16 Strength Combinations
Service Check Load Combinations: ASCE 7-16 ASD Combinations
Apply Sds to Seismic Combinations for Ev: No
f'm: 1350 psi
fy: 60000 psi
Mortar Type: Type N - PCL (Portland Cement/Lime)
Shear Wall Seismic Designation: Ordinary
Use R=1.5 for Shear Walls: No

Material (CMU/Clay): Taken from Unit
Specify Wall Weight Manually: No
Block Weight: Normal Weight
Built on Concrete with Rough Surface: No

Secondary Moment Approach: Moment Magnifier
Include Wall Self-Weight: Yes
Neglect Lateral Load on Parapet: No
Include Wall Wt In Virtual Eccentricity: Yes
Always use I-cracked: No
Amplify Axial Stress For Slenderness: No
End Bars Only For Shear Wall Flexural/Axial Analysis: No
Multiply Seismic Shear By 1.5 (ASD only): No

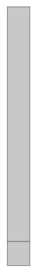
Checks Summary

☐	Ratio	Check	Critical Combination
	Out-Of-Plane Checks		
✔	0.018	Axial Stress	1.4D
✔	0.000	Axial Stress (w slend)	1.4D
✔	0.011	Axial Force	1.4D
✔	0.613	Shear	0.9D + 1Wind$_{C\&C+Wall}$
✔	0.414	Flexural+Axial Interaction	0.9D + 1Wind$_{C\&C+Wall}$
✔	0.222	Maximum Reinforcement	1.2D + 0.5Wind$_{C\&C+Wall}$
✔	0.001	Deflection (TMS)	1D + 0.6Wind$_{C\&C+Wall}$
✔	0.001	Deflection (IBC)	1D + 0.6Wind$_{C\&C+Wall}$

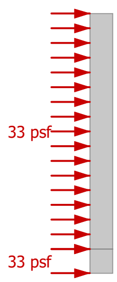
☐	Ratio	Check	Critical Combination
	In-Plane Checks		
✔	0.000	Flexural+Axial Interaction	1.4D
✔	0.000	Shear	1.4D
✔	0.000	Shear Friction	1.4D
✔	0.075	Maximum Reinforcement	1.4D
✔	0.000	Deflection (Drift)	1.4D
	Prescriptive Checks		
✔	1.000	Vert Area	
✔	0.267	Vert Spacing	
✔	1.000	Horz Area	
✔	0.067	Horz Spacing	

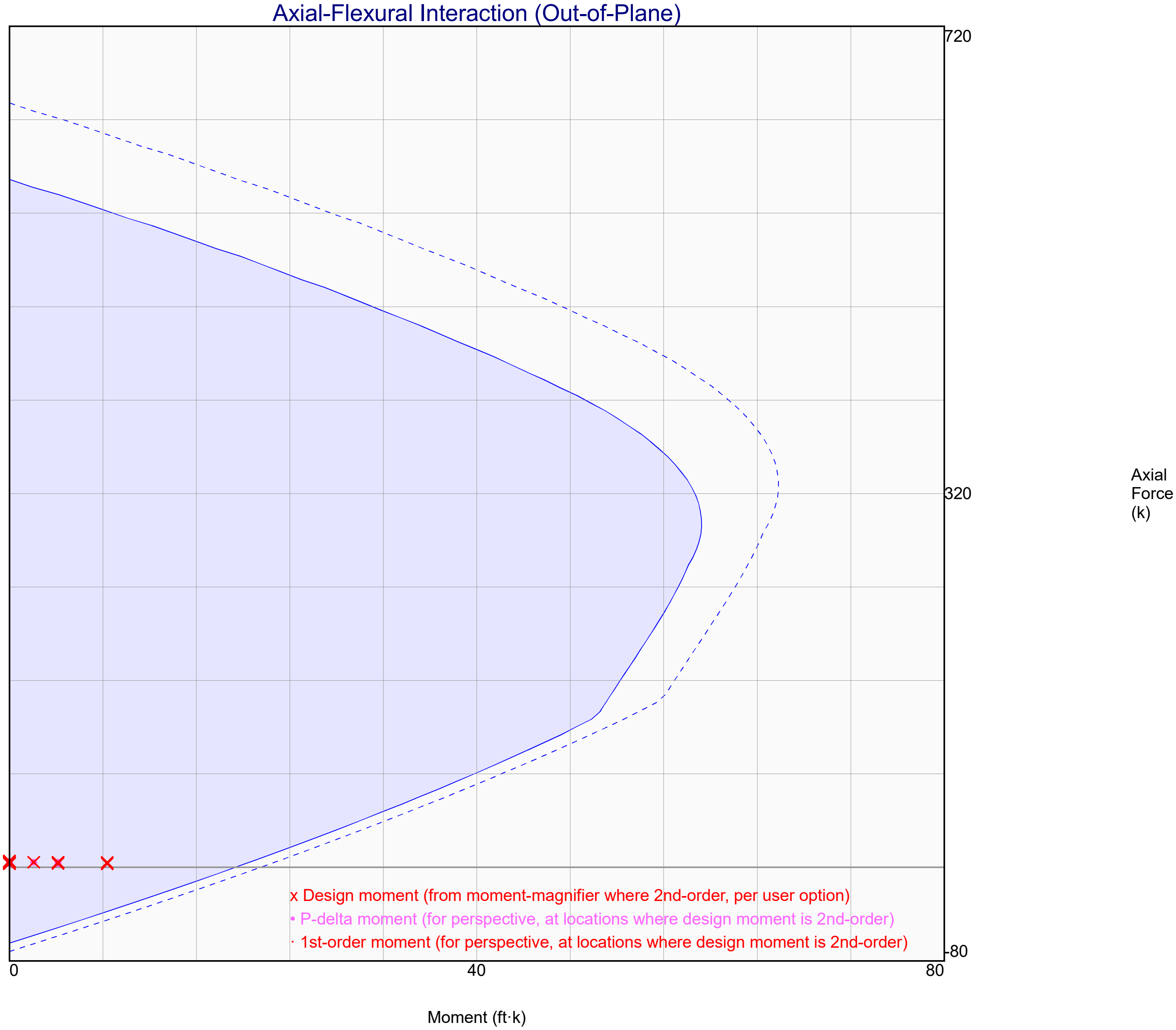
Loads on the Wall

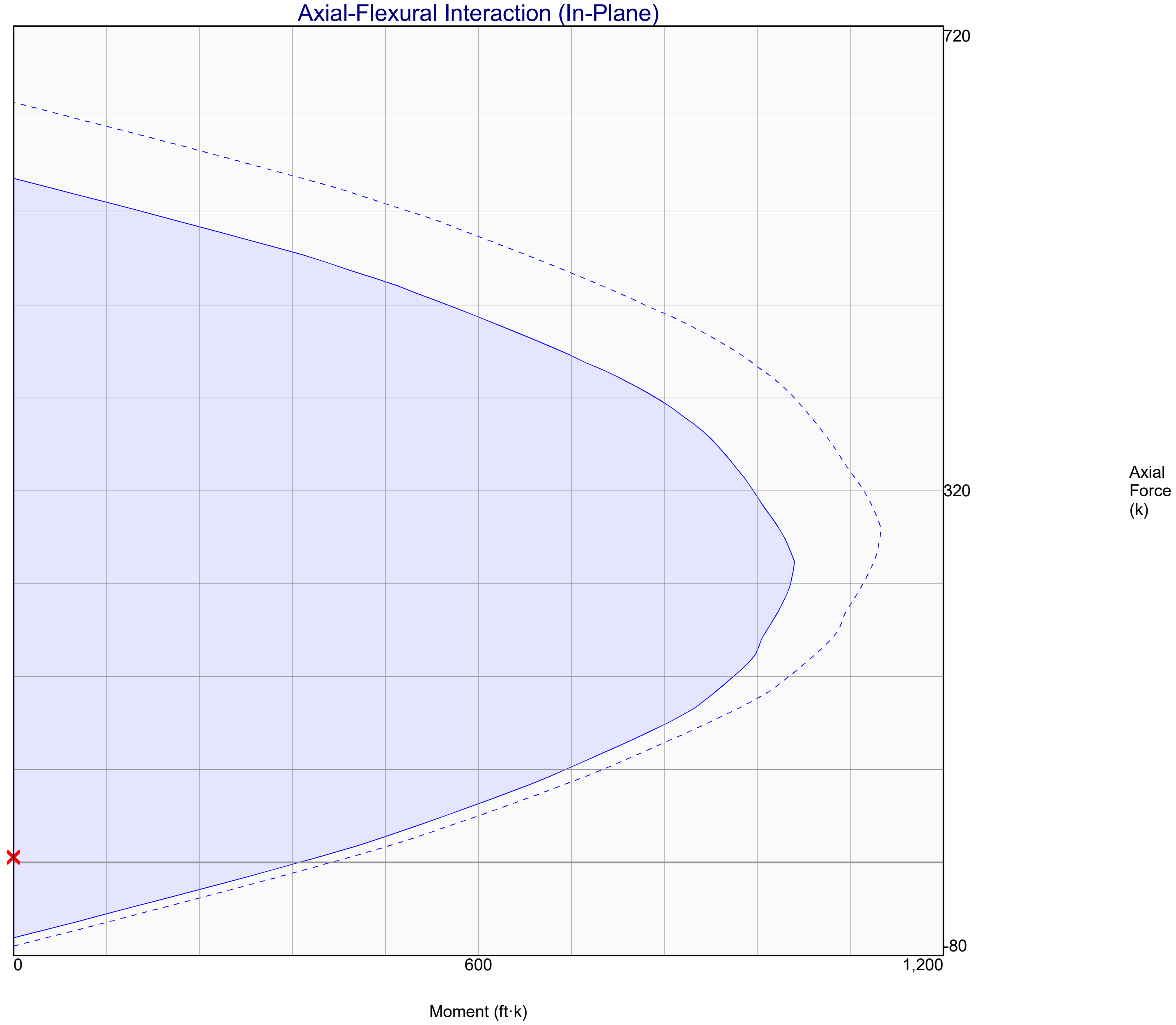
copping unit [Source: Dead]



wind [Source: WindCnCPositiveOnWall]




Axial-Flexural Interaction (Out-of-Plane)
720
320
-80
Axial
Force
(k)
0
40
80
Moment (ft·k)
x Design moment (from moment-magnifier where 2nd-order, per user option)
• P-delta moment (for perspective, at locations where design moment is 2nd-order)
· 1st-order moment (for perspective, at locations where design moment is 2nd-order)


Axial-Flexural Interaction (In-Plane)
720
320
-80
Axial
Force
(k)
0
600
1,200
Moment (ft·k)

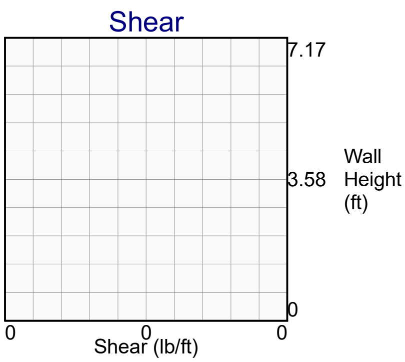
Out-of-Plane Design Calculations for Combination 1.4D

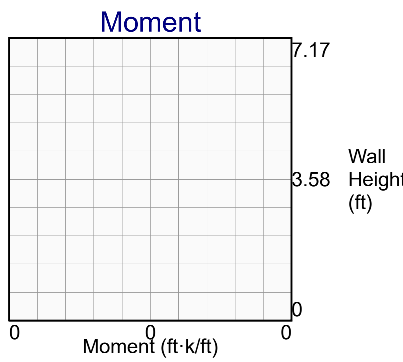
Load Combination: 1.4D

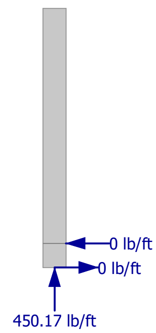
Wall weight: For this partially grouted wall, the average volume of masonry per cell is calculated as 265.06 in³ based on 32 in grout spacing. Given a masonry density of 130 lb/ft³, the average weight per face of wall is 44.87 psf and the weight per length for this wall is 321.55 lb/ft (all unfactored values).

Factored wall weight = 62.81 psf.

	Per-Length Axial Force (lb/ft)	**Per-Length Out-of-Plane Shear (lb/ft)**	**Per-Length Out-of-Plane Moment (ft·k/ft)**	**Total Wall Axial Force (k)**	**Total Wall Out-of-Plane Shear (k)**	**Total Wall Out-of-Plane Moment (ft·k)**	**Total Wall Out-of-Plane P-δ Moment (ft·k)**	**Total Wall Out-of-Plane Mag. Moment (ft·k)**
7.17 ft from base	0 lb/ft	0 lb/ft	0 ft·k/ft	0 k	0 k	0 ft·k	N/A	N/A
0.67 ft from base	408.29 lb/ft	0 lb/ft	0 ft·k/ft	4.9 k	0 k	0 ft·k	N/A	N/A
0.33 ft from base	429.23 lb/ft	0 lb/ft	0 ft·k/ft	5.15 k	0 k	0 ft·k	0 ft·k	0 ft·k
0 ft from base	450.17 lb/ft	0 lb/ft	0 ft·k/ft	5.4 k	0 k	0 ft·k	N/A	N/A







Secondary Moments

The strength design provisions of TMS 402 require consideration of a secondary moment. Both P-Delta and moment magnifier options are given. The P-delta approach may be used under some conditions and the moment magnifier may be used under all conditions. Both are calculated here for perspective, but per user option the value used for design will be that from the moment magnifier approach.

Cracking Moment

The distance from the neutral axis to the extreme tension fiber 'c_t' is half the wall thickness: 3.81 in.

$$S_n = I_n/c_t = \left(4{,}242.98\ \text{in}^4\right)/\left(3.81\ \text{in}\right) = 1{,}112.91\ \text{in}^3$$

$$f_r = 87.5\ \text{psi}$$

$$M_{cr} = S_n f_r = \left(1{,}112.91\ \text{in}^3\right)\left(87.5\ \text{psi}\right) = 8.11\ \text{ft}\cdot\text{k}$$

[NCMA TEK note 14-4B (2008), p. 3]

Secondary Moment Calculation (At Midspan)

Note that the I_{cr} value used here is interpolated from the values in the Section Analysis calculations based on the axial force at this location.

P-Delta

Because $M_{u1} < M_{cr}$, M_u is calculated by:

$$M_u = \frac{M_{u1}}{1 - \frac{5P_u h^2}{48E_m I_n}} = \frac{\left(\left(0\ \text{ft}\cdot\text{k}\right)\right)}{\left(1 - \frac{\left(5\left(5.15\ \text{k}\right)\left(0.67\ \text{ft}\right)^2\right)}{\left(48\left(1{,}215{,}000\ \text{psi}\right)\left(4{,}242.98\ \text{in}^4\right)\right)}\right)} = 0\ \text{ft}\cdot\text{k}$$

[Masonry Designers Guide 2016, Eqn 12.4-23 (substituting M_{u1} for first order moment expression)]

Moment Magnifier

Because $M_{u1} < M_{cr}$, $I_{eff} = 0.75I_n = 0.75(4{,}242.98\ \text{in}^4) = 3{,}182.23\ \text{in}^4$.

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} = \left(\frac{\left((3.1416)^2 (1{,}215{,}000\text{ psi})(3{,}182.23\text{ in}^4) \right)}{(0.67\text{ ft})^2} \right) = 596{,}249.39\text{ k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-29]

$$\psi = \frac{1}{1-\frac{P_u}{P_e}} = \frac{1}{\left(1-\left(\frac{(5.15\text{ k})}{(596{,}249.39\text{ k})}\right)\right)} = 1$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-28]

$$M_u = \psi M_{u0} = (1)(0\text{ ft}\cdot\text{k}) = 0\text{ ft}\cdot\text{k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-27]

Out-of-Plane Strength Checks: Combination 1.4D

Design forces for this load combination:

- P_u @ top = 4.9 k
- P_u @ mid-span = 5.15 k
- P_u @ base = 5.4 k
- V_u = 0 k
- M_u @ top = 0 ft·k
- M_u @ mid-span = 0 ft·k (from moment magnifier; first-order moment was 0 ft·k)

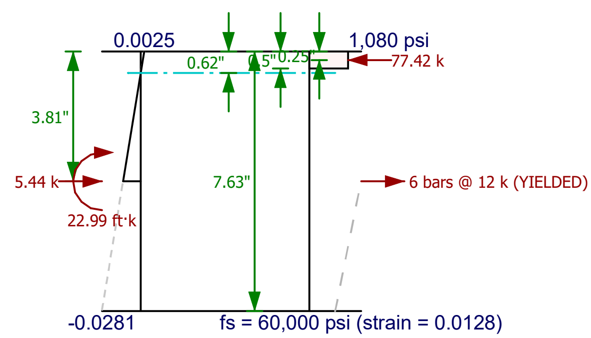
Moment Check @ Axial Load Application Point

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (4.9 k) and thus P_n = 5.44 k. The associated M_n (22.99 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 4.9 k, the interaction diagram gives a moment capacity of 20.71 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0

Compression area = 0.5 ft². Depth of compression zone = 0.5 in.



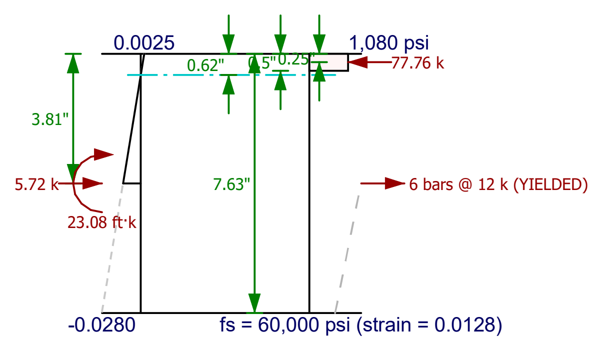
Moment Check @ Mid-span

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (5.15 k) and thus P_n = 5.72 k. The associated M_n (23.08 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 5.15 k, the interaction diagram gives a moment capacity of 20.76 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0

Compression area = 0.5 ft². Depth of compression zone = 0.5 in.



Axial Stress Check

$$\sigma_p = P_u/A_g = (5.4\text{ k})/(7.63\text{ ft}^2) = 4.92\text{ psi}$$

[TMS 402-16 Section 9.3.5.4.2]

$$\sigma_{pmax} = 0.2f'_m = 0.2(1{,}350\text{ psi}) = 270\text{ psi}$$

[TMS 402-16 Section 9.3.5.4.2]

✔ $\sigma_{Pmax} \geq \sigma_P$ ...utilization ratio 0.018

Axial Stress Check with Slender Wall

$$\frac{h}{t}=\frac{(0.67\ \text{ft})}{(7.63\ \text{in})}=1.05$$

[TMS 402-16 Section 9.3.5.4.2]

Slenderness ratio does not exceed 30; check does not apply

✔ $\sigma_{Pmax} \geq \sigma_P$ ...utilization ratio 0

Axial Force Check

$$\frac{h}{r}=\frac{(0.67\ \text{ft})}{(2.65\ \text{in})}=3.02$$

[TMS 402-16 Section 9.3.4.1.1]

$$P_n=0.80\left[0.80f'_m\left(A_n-A_{st}\right)+f_yA_{st}\right]\left[1-\left(\frac{h}{140r}\right)^2\right]=0.80\left(0.80\left(1{,}350\ \text{psi}\right)\left(\left(4.21\ \text{ft}^2\right)-\left(0\ \text{in}^2\right)\right)+\left(60{,}000\ \text{psi}\right)\left(0\ \text{in}^2\right)\right)\left(1-\left(\frac{(0.67\ \text{ft})}{(140(2.65\ \text{in}))}\right)^2\right)=523.34\ \text{k}$$

[TMS 402-16 Section 9.3.4.1.1, Equation 9-15]

$$\phi P_n=\phi P_n=(0.9)(523.34\ \text{k})=471.01\ \text{k}$$

[TMS 402-16 Section 9.1.3, 9.1.4.4]

$$P_u=5.15\ \text{k}$$

✔ $\phi P_n \geq P_u$ ...utilization ratio 0.011

Shear Check

There is no applied shear force in this load combination; check passes.

✔ $\phi V_n \geq V_u$ ...utilization ratio 0

Maximum Reinforcement Check

Maximum reinforcement is checked based on the provisions of TMS 402-16 9.3.3.2.

$$\frac{M_u}{V_ud_v}=\frac{(0\ \text{ft}\cdot\text{k})}{((0\ \text{k})(7.63\ \text{in}))}=0$$

[TMS 402-16 Section 9.3.4.1.2]

Because this ratio does not exceed 1.0 and R <= 1.5, there is no limit on flexural tensile reinforcement.

✔ $\varepsilon_s \geq \varepsilon_{min}$ ...utilization ratio 0

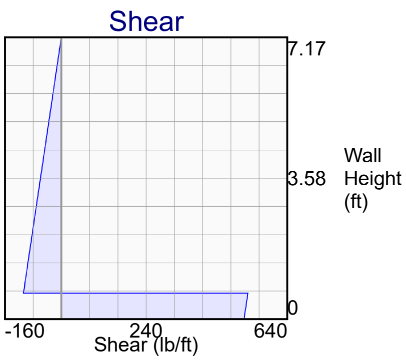
Out-of-Plane Design Calculations for Combination 1.2D + 0.5Wind$_{C\&C+Wall}$

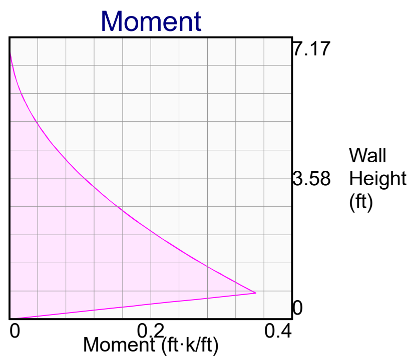
Load Combination: 1.2D + 0.5Wind$_{C\&C+Wall}$

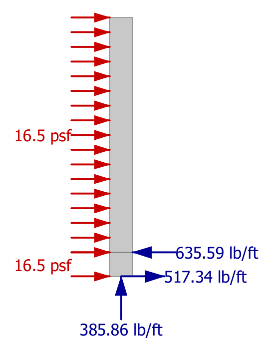
Wall weight: For this partially grouted wall, the average volume of masonry per cell is calculated as 265.06 in³ based on 32 in grout spacing. Given a masonry density of 130 lb/ft³, the average weight per face of wall is 44.87 psf and the weight per length for this wall is 321.55 lb/ft (all unfactored values).

Factored wall weight = 53.84 psf.

	Per-Length Axial Force (lb/ft)	Per-Length Out-of-Plane Shear (lb/ft)	Per-Length Out-of-Plane Moment (ft·k/ft)	Total Wall Axial Force (k)	Total Wall Out-of-Plane Shear (k)	Total Wall Out-of-Plane Moment (ft·k)	Total Wall Out-of-Plane P-δ Moment (ft·k)	Total Wall Out-of-Plane Mag. Moment (ft·k)
7.17 ft from base	0 lb/ft	0 lb/ft	0 ft·k/ft	0 k	0 k	0 ft·k	N/A	N/A
0.67 ft from base	349.97 lb/ft	-107.25 lb/ft	0.35 ft·k/ft	4.2 k	-1.29 k	4.18 ft·k	N/A	N/A
		528.34 lb/ft	0.35 ft·k/ft	4.2 k	6.34 k	4.18 ft·k	N/A	N/A
0.33 ft from base	367.91 lb/ft	522.84 lb/ft	0.17 ft·k/ft	4.41 k	6.27 k	2.08 ft·k	2.08 ft·k	2.08 ft·k
0 ft from base	385.86 lb/ft	517.34 lb/ft	0 ft·k/ft	4.63 k	6.21 k	0 ft·k	N/A	N/A







Secondary Moments

The strength design provisions of TMS 402 require consideration of a secondary moment. Both P-Delta and moment magnifier options are given. The P-delta approach may be used under some conditions and the moment magnifier may be used under all conditions. Both are calculated here for perspective, but per user option the value used for design will be that from the moment magnifier approach.

Cracking Moment

The distance from the neutral axis to the extreme tension fiber 'c_t' is half the wall thickness: 3.81 in.

$$S_n = I_n/c_t = \left(4{,}242.98\ \text{in}^4\right)/\left(3.81\ \text{in}\right) = 1{,}112.91\ \text{in}^3$$

$$f_r = 87.5\ \text{psi}$$

$$M_{cr} = S_n f_r = \left(1{,}112.91\ \text{in}^3\right)\left(87.5\ \text{psi}\right) = 8.11\ \text{ft·k}$$

[NCMA TEK note 14-4B (2008), p. 3]

Secondary Moment Calculation (At Midspan)

Note that the I_{cr} value used here is interpolated from the values in the Section Analysis calculations based on the axial force at this location.

P-Delta

Because $M_{u1} < M_{cr}$, M_u is calculated by:

$$M_u = \frac{M_{u1}}{1-\frac{5P_u h^2}{48E_m I_n}} = \frac{\left(\left(2.08\ \text{ft·k}\right)\right)}{\left(1-\frac{\left(5\left(4.41\ \text{k}\right)\left(0.67\ \text{ft}\right)^2\right)}{\left(48\left(1{,}215{,}000\ \text{psi}\right)\left(4{,}242.98\ \text{in}^4\right)\right)}\right)} = 2.08\ \text{ft·k}$$

[Masonry Designers Guide 2016, Eqn 12.4-23 (substituting M_{u1} for first order moment expression)]

Moment Magnifier

Because $M_{u1} < M_{cr}$, $I_{eff} = 0.75I_n = 0.75(4{,}242.98\ \text{in}^4) = 3{,}182.23\ \text{in}^4$.

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} = \left(\frac{\left((3.1416)^2 (1{,}215{,}000\ \text{psi})(3{,}182.23\ \text{in}^4) \right)}{(0.67\ \text{ft})^2} \right) = 596{,}249.39\ \text{k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-29]

$$\psi = \frac{1}{1-\frac{P_u}{P_e}} = \frac{1}{\left(1-\left(\frac{(4.41\ \text{k})}{(596{,}249.39\ \text{k})}\right)\right)} = 1$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-28]

$$M_u = \psi M_{u0} = (1)(2.08\ \text{ft·k}) = 2.08\ \text{ft·k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-27]

Out-of-Plane Strength Checks: Combination 1.2D + 0.5Wind$_{C\&C+Wall}$

Design forces for this load combination:

- P_u @ top = 4.2 k
- P_u @ mid-span = 4.41 k
- P_u @ base = 4.63 k
- V_u = 6.34 k
- M_u @ top = 4.18 ft·k
- M_u @ mid-span = 2.08 ft·k (from moment magnifier; first-order moment was 2.08 ft·k)

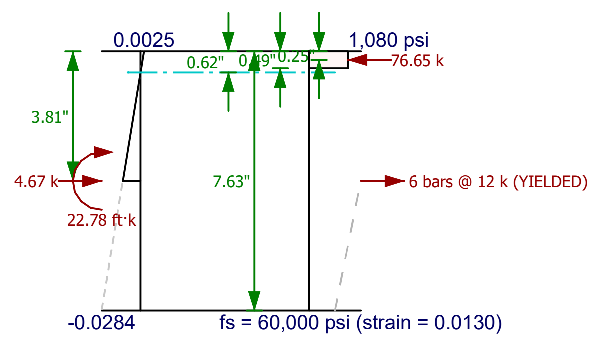
Moment Check @ Axial Load Application Point

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (4.2 k) and thus P_n = 4.67 k. The associated M_n (22.78 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 4.2 k, the interaction diagram gives a moment capacity of 20.5 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0.204

Compression area = 0.49 ft². Depth of compression zone = 0.49 in.



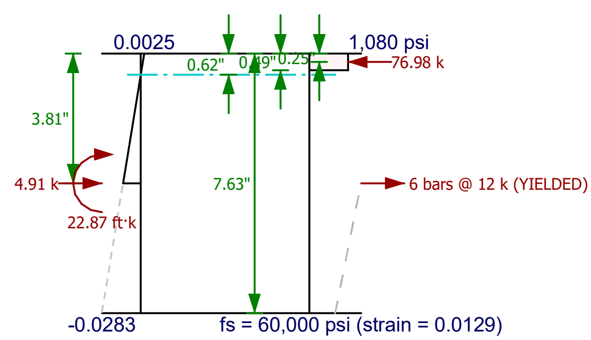
Moment Check @ Mid-span

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (4.41 k) and thus P_n = 4.91 k. The associated M_n (22.87 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 4.41 k, the interaction diagram gives a moment capacity of 20.56 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0.101

Compression area = 0.49 ft². Depth of compression zone = 0.49 in.



Axial Stress Check

$$\sigma_p = P_u/A_g = (4.63\ \text{k})/(7.63\ \text{ft}^2) = 4.22\ \text{psi}$$

[TMS 402-16 Section 9.3.5.4.2]

$$\sigma_{pmax} = 0.2f'_m = 0.2(1{,}350\ \text{psi}) = 270\ \text{psi}$$

[TMS 402-16 Section 9.3.5.4.2]

✔ $\sigma_{P\max} \geq \sigma_P$ ...utilization ratio 0.016

Axial Stress Check with Slender Wall

$$\frac{h}{t}=\frac{(0.67\text{ ft})}{(7.63\text{ in})}=1.05$$

[TMS 402-16 Section 9.3.5.4.2]

Slenderness ratio does not exceed 30; check does not apply

✔ $\sigma_{P\max} \geq \sigma_P$ ...utilization ratio 0

Axial Force Check

$$\frac{h}{r}=\frac{(0.67\text{ ft})}{(2.65\text{ in})}=3.02$$

[TMS 402-16 Section 9.3.4.1.1]

$$P_n=0.80\left[0.80f'_m\left(A_n-A_{st}\right)+f_yA_{st}\right]\left[1-\left(\frac{h}{140r}\right)^2\right]=0.80\left(0.80\left(1{,}350\text{ psi}\right)\left(\left(4.21\text{ ft}^2\right)-\left(0\text{ in}^2\right)\right)+\left(60{,}000\text{ psi}\right)\left(0\text{ in}^2\right)\right)\left(1-\left(\frac{(0.67\text{ ft})}{(140(2.65\text{ in}))}\right)^2\right)=523.34\text{ k}$$

[TMS 402-16 Section 9.3.4.1.1, Equation 9-15]

$$\phi P_n=\phi P_n=(0.9)(523.34\text{ k})=471.01\text{ k}$$

[TMS 402-16 Section 9.1.3, 9.1.4.4]

$$P_u=4.41\text{ k}$$

✔ $\phi P_n \geq P_u$ ...utilization ratio 0.009

Shear Check

The net shear area is taken as the area bounded by the extreme compression fiber and the reinforcement nearest the opposite face, subject to the effective compression width.

$$A_{nv}=2.1\text{ ft}^2$$

Shear capacity due to masonry, conservatively taking M/Vd = 1.0:

$$V_{nm}=\left[4.0-1.75(1.0)\right]A_{nv}\sqrt{f'_m}+0.25P_u=\left(4.0-1.75(1.0)\right)\left(2.1\text{ ft}^2\right)\sqrt{\left(1{,}350\text{ psi}\right)}+0.25(4.2\text{ k})=26.1\text{ k}$$

[TMS 402-16 Section 9.3.4.1.2.1, Equation 9-20 with conservative M/Vd = 1.0]

No reinforcement for out-of-plane shear:

$$V_{ns}=0\text{ k}$$

Nominal shear capacity:

$$V_n=\left(V_{nm}+V_{ns}\right)\gamma_g=\left((26.1\text{ k})+(0\text{ k})\right)(1)=26.1\text{ k}$$

[TMS 402-16 Section 9.3.4.1.2, Equation 9-17]

Limited by:

$$V_{n\_limit}=\left(4A_{nv}\sqrt{f'_m}\right)\gamma_g=\left(4\left(2.1\text{ ft}^2\right)\sqrt{\left(1{,}350\text{ psi}\right)}\right)(1)=44.53\text{ k}$$

[TMS 402-16 Section 9.3.4.1.2, Equation 9-19]

...Limit on V_n does not control

$$\phi V_n=\phi V_n=(0.8)(26.1\text{ k})=20.88\text{ k}$$

[TMS 402-16 Section 9.1.3, 9.1.4.5]

✔ $\phi V_n \geq V_u$ ...utilization ratio 0.304

Maximum Reinforcement Check

Maximum reinforcement is checked based on the provisions of TMS 402-16 9.3.3.2.

$$\frac{M_u}{V_ud_v}=\frac{(4.18\text{ ft}\cdot\text{k})}{((1.29\text{ k})(7.63\text{ in}))}=5.11$$

[TMS 402-16 Section 9.3.4.1.2]

The code gives the limit on area of reinforcement as that established by an analysis where the maximum tension bar strain is set to a code-mandated extreme strain value. The provision will be enforced by determining the strain corresponding to the actual specified reinforcement and comparing to the code-mandated minimum strain.

The member must be reinforced such that the tensile strain at equilibrium exceeds 1.5 times the yield strain.

$$\varepsilon_{min} = 1.5\varepsilon_y = 1.5\left(0.00207\right) = 0.0031$$

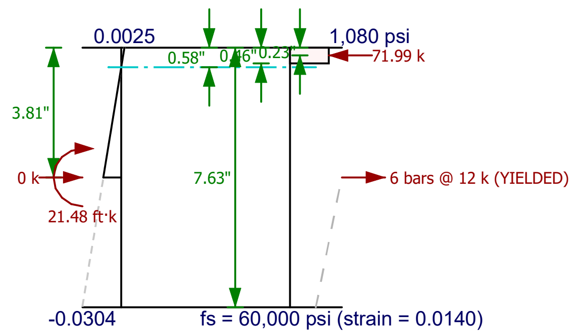
[TMS 402-16 Section 9.3.3.2.1(a)]

The axial load to be used in the analysis is given by the special load combination D + 0.75L + 0.525Q_E, which in this case gives 0 k.

The tensile strain that will occur at equilibrium with the current reinforcement is illustrated below:

$$\varepsilon_s = 0.01397$$

✔ $\varepsilon_s \geq \varepsilon_{min}$ ...utilization ratio 0.222



From the analysis, strain in extreme tension bar is 0.0140.

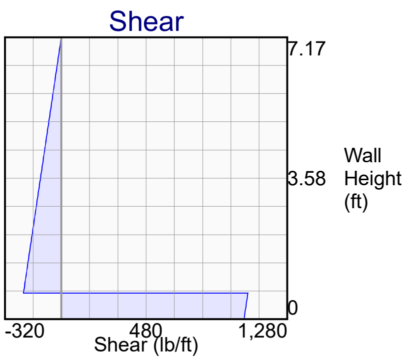
Out-of-Plane Design Calculations for Combination 0.9D + 1Wind$_{C\&C+Wall}$

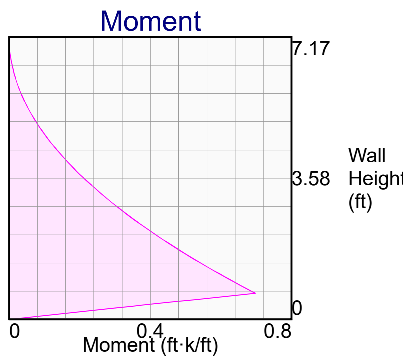
Load Combination: 0.9D + 1Wind$_{C\&C+Wall}$

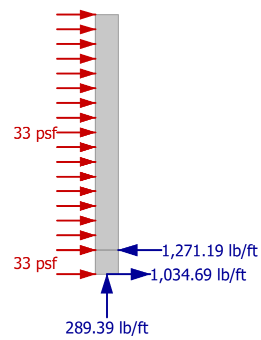
Wall weight: For this partially grouted wall, the average volume of masonry per cell is calculated as 265.06 in³ based on 32 in grout spacing. Given a masonry density of 130 lb/ft³, the average weight per face of wall is 44.87 psf and the weight per length for this wall is 321.55 lb/ft (all unfactored values).

Factored wall weight = 40.38 psf.

	Per-Length Axial Force (lb/ft)	Per-Length Out-of-Plane Shear (lb/ft)	Per-Length Out-of-Plane Moment (ft·k/ft)	Total Wall Axial Force (k)	Total Wall Out-of-Plane Shear (k)	Total Wall Out-of-Plane Moment (ft·k)	Total Wall Out-of-Plane P-δ Moment (ft·k)	Total Wall Out-of-Plane Mag. Moment (ft·k)
7.17 ft from base	0 lb/ft	0 lb/ft	0 ft·k/ft	0 k	0 k	0 ft·k	N/A	N/A
0.67 ft from base	262.47 lb/ft	-214.5 lb/ft	0.7 ft·k/ft	3.15 k	-2.57 k	8.37 ft·k	N/A	N/A
		1,056.69 lb/ft	0.7 ft·k/ft	3.15 k	12.68 k	8.37 ft·k	N/A	N/A
0.33 ft from base	275.93 lb/ft	1,045.69 lb/ft	0.35 ft·k/ft	3.31 k	12.55 k	4.16 ft·k	4.16 ft·k	4.16 ft·k
0 ft from base	289.39 lb/ft	1,034.69 lb/ft	0 ft·k/ft	3.47 k	12.42 k	0 ft·k	N/A	N/A







Secondary Moments

The strength design provisions of TMS 402 require consideration of a secondary moment. Both P-Delta and moment magnifier options are given. The P-delta approach may be used under some conditions and the moment magnifier may be used under all conditions. Both are calculated here for perspective, but per user option the value used for design will be that from the moment magnifier approach.

Cracking Moment

The distance from the neutral axis to the extreme tension fiber 'c_t' is half the wall thickness: 3.81 in.

$$S_n = I_n/c_t = \left(4{,}242.98\ \text{in}^4\right)/\left(3.81\ \text{in}\right) = 1{,}112.91\ \text{in}^3$$

$$f_r = 87.5\ \text{psi}$$

$$M_{cr} = S_n f_r = \left(1{,}112.91\ \text{in}^3\right)\left(87.5\ \text{psi}\right) = 8.11\ \text{ft·k}$$

[NCMA TEK note 14-4B (2008), p. 3]

Secondary Moment Calculation (At Midspan)

Note that the I_{cr} value used here is interpolated from the values in the Section Analysis calculations based on the axial force at this location.

P-Delta

Because $M_{u1} < M_{cr}$, M_u is calculated by:

$$M_u = \frac{M_{u1}}{1-\frac{5P_uh^2}{48E_mI_n}} = \frac{\left(\left(4.16\ \text{ft·k}\right)\right)}{\left(1-\frac{\left(5\left(3.31\ \text{k}\right)\left(0.67\ \text{ft}\right)^2\right)}{\left(48\left(1{,}215{,}000\ \text{psi}\right)\left(4{,}242.98\ \text{in}^4\right)\right)}\right)} = 4.16\ \text{ft·k}$$

[Masonry Designers Guide 2016, Eqn 12.4-23 (substituting M_{u1} for first order moment expression)]

Moment Magnifier

Because $M_{u1} < M_{cr}$, $I_{eff} = 0.75I_n$ = 0.75(4,242.98 in^4) = 3,182.23 in^4.

$$P_e=\frac{\pi^2 E_m I_{eff}}{h^2}=\left(\frac{\left((3.1416)^2(1{,}215{,}000\text{ psi})(3{,}182.23\text{ in}^4)\right)}{(0.67\text{ ft})^2}\right)=596{,}249.39\text{ k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-29]

$$\psi=\frac{1}{1-\frac{P_u}{P_e}}=\frac{1}{\left(1-\left(\frac{(3.31\text{ k})}{(596{,}249.39\text{ k})}\right)\right)}=1$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-28]

$$M_u=\psi M_{u0}=(1)(4.16\text{ ft·k})=4.16\text{ ft·k}$$

[TMS 402-16 Section 9.3.5.4.3, Equation 9-27]

Out-of-Plane Strength Checks: Combination 0.9D + 1Wind$_{C\&C+Wall}$

Design forces for this load combination:

- P_u @ top = 3.15 k
- P_u @ mid-span = 3.31 k
- P_u @ base = 3.47 k
- V_u = 12.68 k
- M_u @ top = 8.37 ft·k
- M_u @ mid-span = 4.16 ft·k (from moment magnifier; first-order moment was 4.16 ft·k)

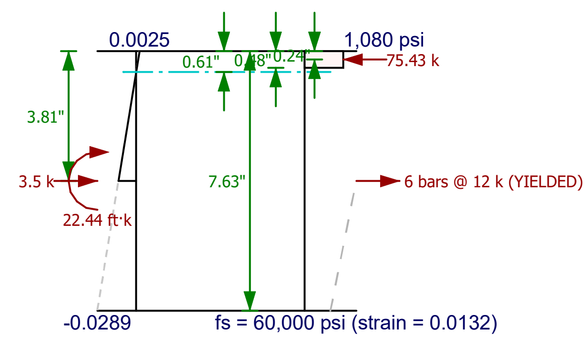
Moment Check @ Axial Load Application Point

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (3.15 k) and thus P_n = 3.5 k. The associated M_n (22.44 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 3.15 k, the interaction diagram gives a moment capacity of 20.2 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0.414

Compression area = 0.48 ft². Depth of compression zone = 0.48 in.



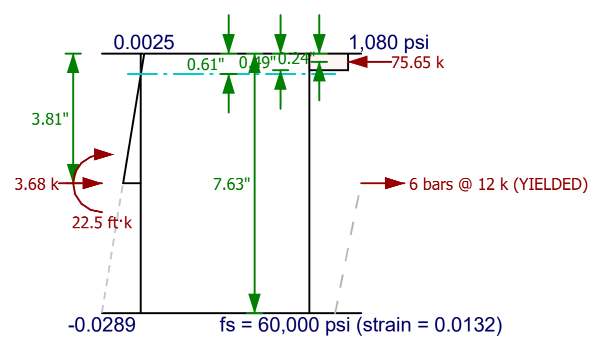
Moment Check @ Mid-span

The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (3.31 k) and thus P_n = 3.68 k. The associated M_n (22.5 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 3.31 k, the interaction diagram gives a moment capacity of 20.24 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0.206

Compression area = 0.49 ft². Depth of compression zone = 0.49 in.



Axial Stress Check

$$\sigma_p=P_u/A_g=(3.47\text{ k})/(7.63\text{ ft}^2)=3.16\text{ psi}$$

[TMS 402-16 Section 9.3.5.4.2]

$$\sigma_{pmax}=0.2f'_m=0.2(1{,}350\text{ psi})=270\text{ psi}$$

[TMS 402-16 Section 9.3.5.4.2]

✔ $\sigma_{Pmax} \geq \sigma_P$ ...utilization ratio 0.012

Axial Stress Check with Slender Wall

$$\frac{h}{t} = \frac{(0.67\ \text{ft})}{(7.63\ \text{in})} = 1.05$$

[TMS 402-16 Section 9.3.5.4.2]

Slenderness ratio does not exceed 30; check does not apply

✔ $\sigma_{Pmax} \geq \sigma_P$ ...utilization ratio 0

Axial Force Check

$$\frac{h}{r} = \frac{(0.67\ \text{ft})}{(2.65\ \text{in})} = 3.02$$

[TMS 402-16 Section 9.3.4.1.1]

$$P_n = 0.80\left[0.80f'_m\left(A_n - A_{st}\right) + f_y A_{st}\right]\left[1 - \left(\frac{h}{140r}\right)^2\right] = 0.80\left(0.80\left(1{,}350\ \text{psi}\right)\left(\left(4.21\ \text{ft}^2\right) - \left(0\ \text{in}^2\right)\right) + \left(60{,}000\ \text{psi}\right)\left(0\ \text{in}^2\right)\right)\left(1 - \left(\frac{(0.67\ \text{ft})}{\left(140\left(2.65\ \text{in}\right)\right)}\right)^2\right) = 523.34\ \text{k}$$

[TMS 402-16 Section 9.3.4.1.1, Equation 9-15]

$$\phi P_n = \phi P_n = (0.9)(523.34\ \text{k}) = 471.01\ \text{k}$$

[TMS 402-16 Section 9.1.3, 9.1.4.4]

$$P_u = 3.31\ \text{k}$$

✔ $\phi P_n \geq P_u$ ...utilization ratio 0.007

Shear Check

The net shear area is taken as the area bounded by the extreme compression fiber and the reinforcement nearest the opposite face, subject to the effective compression width.

$$A_{nv} = 2.1\ \text{ft}^2$$

Shear capacity due to masonry, conservatively taking M/Vd = 1.0:

$$V_{nm} = \left[4.0 - 1.75(1.0)\right]A_{nv}\sqrt{f'_m} + 0.25P_u = \left(4.0 - 1.75(1.0)\right)\left(2.1\ \text{ft}^2\right)\sqrt{\left(1{,}350\ \text{psi}\right)} + 0.25(3.15\ \text{k}) = 25.84\ \text{k}$$

[TMS 402-16 Section 9.3.4.1.2.1, Equation 9-20 with conservative M/Vd = 1.0]

No reinforcement for out-of-plane shear:

$$V_{ns} = 0\ \text{k}$$

Nominal shear capacity:

$$V_n = \left(V_{nm} + V_{ns}\right)\gamma_g = \left((25.84\ \text{k}) + (0\ \text{k})\right)(1) = 25.84\ \text{k}$$

[TMS 402-16 Section 9.3.4.1.2, Equation 9-17]

Limited by:

$$V_{n\_limit} = \left(4A_{nv}\sqrt{f'_m}\right)\gamma_g = \left(4\left(2.1\ \text{ft}^2\right)\sqrt{\left(1{,}350\ \text{psi}\right)}\right)(1) = 44.53\ \text{k}$$

[TMS 402-16 Section 9.3.4.1.2, Equation 9-19]

...Limit on V_n does not control

$$\phi V_n = \phi V_n = (0.8)(25.84\ \text{k}) = 20.67\ \text{k}$$

[TMS 402-16 Section 9.1.3, 9.1.4.5]

✔ $\phi V_n \geq V_u$ ...utilization ratio 0.613

Maximum Reinforcement Check

Maximum reinforcement is checked based on the provisions of TMS 402-16 9.3.3.2.

$$\frac{M_u}{V_u d_v} = \frac{(8.37\ \text{ft}\cdot\text{k})}{\left((2.57\ \text{k})(7.63\ \text{in})\right)} = 5.11$$

[TMS 402-16 Section 9.3.4.1.2]

The code gives the limit on area of reinforcement as that established by an analysis where the maximum tension bar strain is set to a code-mandated extreme strain value. The provision will be enforced by determining the strain corresponding to the actual specified reinforcement and comparing to the code-mandated minimum strain.

The member must be reinforced such that the tensile strain at equilibrium exceeds 1.5 times the yield strain.

$$\varepsilon_{min} = 1.5\varepsilon_y = 1.5\left(0.00207\right) = 0.0031$$

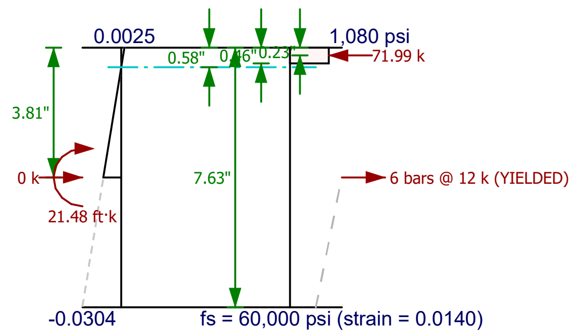
[TMS 402-16 Section 9.3.3.2.1(a)]

The axial load to be used in the analysis is given by the special load combination D + 0.75L + 0.525Q_E, which in this case gives 0 k.

The tensile strain that will occur at equilibrium with the current reinforcement is illustrated below:

$$\varepsilon_s = 0.01397$$

✔ $\varepsilon_s \geq \varepsilon_{min}$ ...utilization ratio 0.222



From the analysis, strain in extreme tension bar is 0.0140.

Out-of-Plane Service Checks for Combination 1D + 0.6Wind$_{C\&C+Wall}$

Out-of-Plane Service Checks: Combination 1D + 0.6Wind$_{C\&C+Wall}$

Design forces for this load combination:

- P @ mid-span = 3.68 k
- M @ mid-span = 2.5 ft·k

Deflection Check per TMS 402

The distance from the neutral axis to the extreme tension fiber 'c_t' is half the wall thickness: 3.81 in.

$$S_n = I_n/c_t = \left(4{,}242.98\ \text{in}^4\right)/\left(3.81\ \text{in}\right) = 1{,}112.91\ \text{in}^3$$

$$f_r = 87.5\ \text{psi}$$

$$M_{cr} = S_n f_r = \left(1{,}112.91\ \text{in}^3\right)\left(87.5\ \text{psi}\right) = 8.11\ \text{ft·k}$$

[NCMA TEK note 14-4B (2008), p. 3]

$$E_m = 900f'_m = 900\left(1{,}350\ \text{psi}\right) = 1{,}215{,}000\ \text{psi}$$

[TMS 402-16 Section 4.2.2]

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} = \frac{\left(\left(60{,}000\ \text{in}^2\right)\left(1.2\ \text{psi}\right) + \left(3.68\ \text{k}\right)\right)}{\left(0.64\left(1{,}350\ \text{psi}\right)\left(144\ \text{in}\right)\right)} = 0.61\ \text{in}$$

[TMS 402-16 Section 9.3.5.4.5, Equation 9-31]

$$n = \frac{E_s}{E_m} = \frac{\left(29{,}000{,}000\ \text{psi}\right)}{\left(1{,}215{,}000\ \text{psi}\right)} = 23.87$$

$$I_{cr} = n\left(A_s + \frac{P_u t_{sp}}{f_y 2d}\right)(d-c)^2 + b\frac{c^3}{3} = \left(23.87\right)\left(\left(1.2\ \text{in}^2\right) + \left(\frac{\left(3.68\ \text{k}\right)}{\left(60{,}000\ \text{psi}\right)}\right)\left(\frac{\left(7.63\ \text{in}\right)}{\left(2\left(3.81\ \text{in}\right)\right)}\right)\right)\left(\left(3.81\ \text{in}\right) - \left(0.61\ \text{in}\right)\right)^2 + \left(\left(144\ \text{in}\right)\frac{\left(0.61\ \text{in}\right)^3}{3}\right) = 319.9\ \text{in}^4$$

[TMS 402-16 Section 9.3.5.4.5, Equation 9-30]

P-delta analysis converged to a deflection of 0 in after 2 iterations. Note that the service moment shown in the equation for the final iteration below (2.5 ft·k) is typically larger than the first-order service moment (2.5 ft·k) due to the addition of second order moment resulting from the deflection.

$$\delta_s = \frac{5M_{ser}h^2}{48E_m I_n} = \frac{\left(5\left(2.5\ \text{ft·k}\right)\left(0.67\ \text{ft}\right)^2\right)}{\left(48\left(1{,}215{,}000\ \text{psi}\right)\left(4{,}242.98\ \text{in}^4\right)\right)} = 0\ \text{in}$$

[TMS 402-16 Section 9.3.5.5.1, Equation 9-25 modified for service conditions]

$$\delta_{max} = 0.007h = 0.007\left(0.67\ \text{ft}\right) = 0.06\ \text{in}$$

[TMS 402-16 Section 9.3.5.5, Equation 9-32]

✔ $\delta_{max} \geq \delta_s$ ...utilization ratio 0.001

Deflection Check per IBC

The service moment due only to Components and Cladding wind pressures is 5.02 ft·k.

Per IBC Table 1604.3 footnote (f), the wind pressure can be taken as 42% of this value for this check. The resulting value is:

$$M_{ser} = 2.11\ \text{ft·k}$$

$$\delta_s = \frac{5M_{ser}h^2}{48E_m I_n} = \frac{\left(5\left(2.11\ \text{ft·k}\right)\left(0.67\ \text{ft}\right)^2\right)}{\left(48\left(1{,}215{,}000\ \text{psi}\right)\left(4{,}242.98\ \text{in}^4\right)\right)} = 0\ \text{in}$$

[TMS 402-16 Section 9.3.5.5.1, Equation 9-25 modified for service conditions]

$$\delta_{max} = \frac{h}{240} = \frac{0.67\ \text{ft}}{240} = 0\ \text{ft}$$

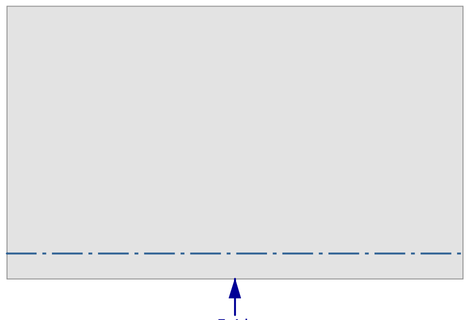
[IBC Section 1604.3.1, Footnote f to Table 1604.3]

✔ $\delta_{max} \geq \delta_s$ ...utilization ratio 0.001

In-Plane Design Calculations for Combination 1.4D

Load Combination: 1.4D

Factored wall weight per height of wall = 753.77 lb/ft.Factored total wall weight = 5.4 k.



In-Plane Strength Checks: Combination 1.4D

Design forces for this load combination:

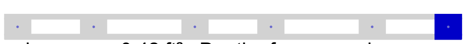
- P_u = 5.4 k
- V_u = 0 k
- M_u = 0 ft·k

Moment Check

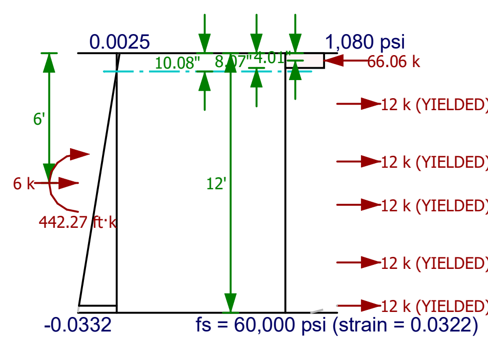
The moment capacity is determined by considering the point on the interaction diagram where ϕP_n is equal to the axial load at this section (5.4 k) and thus P_n = 6 k. The associated M_n (442.27 ft·k) is multiplied by ϕ to obtain design moment capacity.

At an axial load of 5.4 k, the interaction diagram gives a moment capacity of 397.8 ft·k.

✔ $\phi M_n \geq M_u$ ...utilization ratio 0



Compression area = 0.42 ft². Depth of compression zone = 8.07 in.



Shear Check

There is no applied shear force in this load combination; check passes.

✔ $\phi V_n \geq V_u$ ...utilization ratio 0

Shear Friction Check

There is no applied shear force in this load combination; check passes.

✔ $\phi V_n \geq V_u$ ...utilization ratio 0

Maximum Reinforcement Check

Maximum reinforcement is checked based on the provisions of TMS 402-16 9.3.3.2.

$$\frac{M_u}{V_u d_v} = \frac{\left(0\ \text{ft}\cdot\text{k}\right)}{\left(\left(0\ \text{k}\right)\left(144\ \text{in}\right)\right)} = 0$$

[TMS 402-16 Section 9.3.4.1.2]

The code gives the limit on area of reinforcement as that established by an analysis where the maximum tension bar strain is set to a code-mandated extreme strain value. The provision will be enforced by determining the strain corresponding to the actual specified reinforcement and comparing to the code-mandated minimum strain.

For a Ordinary Reinforced Shear Wall: The member must be reinforced such that the tensile strain at equilibrium exceeds 1.5 times the yield strain.

$\varepsilon_{min} = 1.5\varepsilon_y = 1.5(0.00207) = 0.0031$

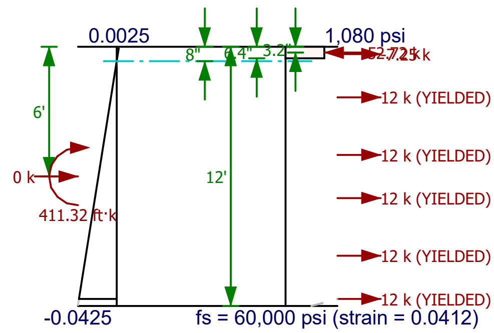
[TMS 402-16 Section 9.3.3.2.1(a)]

The axial load to be used in the analysis is given by the special load combination D + 0.75L + 0.525Q_E, which in this case gives 0 k.

The tensile strain that will occur at equilibrium with the current reinforcement is illustrated below:

$\varepsilon_s = 0.04124$

✔ $\varepsilon_s \geq \varepsilon_{min}$ ...utilization ratio 0.075



From the analysis, strain in extreme tension bar is 0.0412.

Deflection (Drift) Check

Applying a factor to I_g to represent the effects of cracking:

$I_g = t_{sp} l_w^3 / 12 = (7.63\text{ in})(12\text{ ft})^3 / 12 = 1{,}897{,}344\text{ in}^4$

$I_{eff} = C_{cr} I_g = (0.4)(1{,}897{,}344\text{ in}^4) = 758{,}937.6\text{ in}^4$

$E_m = 900 f'_m = 900(1{,}350\text{ psi}) = 1{,}215{,}000\text{ psi}$

Calculate elastic deflection:

$A_g = 2 t_{fs} l_w = 2(1.25\text{ in})(12\text{ ft}) = 2.5\text{ ft}^2$

$$\delta_u = \frac{V_u h}{(5/6.0) A_g 0.4 E_m} + \frac{V_u h^3}{3 E_m I_{eff}} = \frac{((0\text{ k})(0.67\text{ ft}))}{((5/6.0)(2.5\text{ ft}^2)0.4(1{,}215{,}000\text{ psi}))} + \frac{((0\text{ k})(0.67\text{ ft})^3)}{(3(1{,}215{,}000\text{ psi})(758{,}937.6\text{ in}^4))} = 0\text{ in}$$

[Masonry Designers Guide 2013, Eqn 7.4-1 (modified form)]

Calculate inelastic deflection, taking C_d from ASCE 7 Table 12.2-1:

$\delta_{inelastic} = C_d \delta_u = (1.75)(0\text{ in}) = 0\text{ in}$

Maximum allowable deflection:

$\delta_{max} = 0.01h = 0.01(0.67\text{ ft}) = 0.08\text{ in}$

[ASCE 7-10 Table 12.12-1]

✔ $\delta_{max} \geq \delta_{inelastic}$ ...utilization ratio 0

Prescriptive Checks

Maximum Vertical Steel Spacing

For this Ordinary Reinforced Shear Wall, the maximum spacing of vertical reinforcement is:

s_{max}= 120 in

[TMS 402-16 7.3.2.3.1]

The spacing of provided distributed vertical reinforcing bars is:

s= 32 in

✔ $s_{max} \geq s$ ...utilization ratio 0.267

Minimum Distributed Vertical Steel Area

For this Ordinary Reinforced Shear Wall, the minimum area of vertical reinforcement is:

A_{s_min}= 0.2 in²

[TMS 402-16 7.3.2.3.1]

The area of provided distributed vertical reinforcing bars is:

A_s= 0.2 in²

✔ $A_{s\_min} \geq A_s$ ...utilization ratio 1

Maximum Horizontal Steel Spacing

For this Ordinary Reinforced Shear Wall, the maximum spacing of horizontal reinforcement is:

s_{max}= 120 in

[TMS 402-16 7.3.2.3.1]

The spacing of provided distributed horizontal reinforcement is:

s= 8 in

✔ $s_{max} \geq s$ ...utilization ratio 0.067

Minimum Distributed Horizontal Steel Area

For this Ordinary Reinforced Shear Wall, the minimum area of horizontal reinforcement is bond beam reinforcement of minimum area:

A_{s_min}= 0.2 in²

[TMS 402-16 7.3.2.3.1]

The area of provided bond beam bar size is:

A_s= 0.2 in²

✔ $A_{s\_min} \geq A_s$ ...utilization ratio 1

Table 20 - Hilti HIT-HY 200 adhesive design strength with concrete / bond failure for rebar in cracked concrete[1,2,3,4,5,6,7,8,9]

Rebar size	Effective embedment in. (mm)	Tension — ϕN_n f'_c = 2,500 psi (17.2 MPa) lb (kN)	Tension — ϕN_n f'_c = 3,000 psi (20.7 MPa) lb (kN)	Tension — ϕN_n f'_c = 4,000 psi (27.6 MPa) lb (kN)	Tension — ϕN_n f'_c = 6,000 psi (41.4 MPa) lb (kN)	Shear — ϕV_n f'_c = 2,500 psi (17.2 MPa) lb (kN)	Shear — ϕV_n f'_c = 3,000 psi (20.7 MPa) lb (kN)	Shear — ϕV_n f'_c = 4,000 psi (27.6 MPa) lb (kN)	Shear — ϕV_n f'_c = 6,000 psi (41.4 MPa) lb (kN)
#3	3-3/8 (86)	2,790 (12.4)	2,845 (12.7)	2,925 (13.0)	3,045 (13.5)	6,010 (26.7)	6,120 (27.2)	6,300 (28.0)	6,560 (29.2)
	4-1/2 (114)	3,720 (16.5)	3,790 (16.9)	3,900 (17.3)	4,060 (18.1)	8,015 (35.7)	8,165 (36.3)	8,400 (37.4)	8,750 (38.9)
	7-1/2 (191)	6,205 (27.6)	6,315 (28.1)	6,500 (28.9)	6,770 (30.1)	13,360 (59.4)	13,605 (60.5)	14,005 (62.3)	14,580 (64.9)
#4	4-1/2 (114)	4,960 (22.1)	5,055 (22.5)	5,200 (23.1)	5,415 (24.1)	10,690 (47.6)	10,885 (48.4)	11,200 (49.8)	11,665 (51.9)
	6 (152)	6,615 (29.4)	6,740 (30.0)	6,935 (30.8)	7,220 (32.1)	14,250 (63.4)	14,510 (64.5)	14,935 (66.4)	15,555 (69.2)
	10 (254)	11,025 (49.0)	11,230 (50.0)	11,560 (51.4)	12,035 (53.5)	23,750 (105.6)	24,185 (107.6)	24,895 (110.7)	25,925 (115.3)
#5	5-5/8 (143)	7,370 (32.8)	7,970 (35.5)	8,200 (36.5)	8,540 (38.0)	15,875 (70.6)	17,165 (76.4)	17,665 (78.6)	18,395 (81.8)
	7-1/2 (191)	10,435 (46.4)	10,625 (47.3)	10,935 (48.6)	11,390 (50.7)	22,470 (100.0)	22,885 (101.8)	23,555 (104.8)	24,530 (109.1)
	12-1/2 (318)	17,390 (77.4)	17,710 (78.8)	18,225 (81.1)	18,980 (84.4)	37,455 (166.6)	38,145 (169.7)	39,255 (174.6)	40,880 (181.8)
#6	6-3/4 (171)	9,690 (43.1)	10,615 (47.2)	11,810 (52.5)	12,300 (54.7)	20,870 (92.8)	22,860 (101.7)	25,440 (113.2)	26,490 (117.8)
	9 (229)	14,920 (66.4)	15,300 (68.1)	15,745 (70.0)	16,400 (73.0)	32,130 (142.9)	32,955 (146.6)	33,915 (150.9)	35,320 (157.1)
	15 (381)	25,040 (111.4)	25,500 (113.4)	26,245 (116.7)	27,330 (121.6)	53,935 (239.9)	54,925 (244.3)	56,530 (251.5)	58,870 (261.9)
#7	7-7/8 (200)	11,750 (52.3)	11,965 (53.2)	12,315 (54.8)	12,825 (57.0)	25,305 (112.6)	25,770 (114.6)	26,525 (118.0)	27,620 (122.9)
	10-1/2 (267)	15,665 (69.7)	15,955 (71.0)	16,420 (73.0)	17,100 (76.1)	33,740 (150.1)	34,360 (152.8)	35,365 (157.3)	36,830 (163.8)
	17-1/2 (445)	26,110 (116.1)	26,590 (118.3)	27,365 (121.7)	28,500 (126.8)	56,235 (250.1)	57,270 (254.7)	58,940 (262.2)	61,380 (273.0)
#8	9 (229)	14,920 (66.4)	15,720 (69.9)	16,180 (72.0)	16,850 (75.0)	32,130 (142.9)	33,860 (150.6)	34,850 (155.0)	36,295 (161.4)
	12 (305)	20,585 (91.6)	20,960 (93.2)	21,575 (96.0)	22,465 (99.9)	44,335 (197.2)	45,150 (200.8)	46,470 (206.7)	48,390 (215.2)
	20 (508)	34,305 (152.6)	34,935 (155.4)	35,955 (159.9)	37,445 (166.6)	73,890 (328.7)	75,250 (334.7)	77,445 (344.5)	80,650 (358.7)
#9	10-1/8 (257)	17,800 (79.2)	19,500 (86.7)	20,720 (92.2)	21,580 (96.0)	38,340 (170.5)	42,000 (186.8)	44,635 (198.5)	46,480 (206.8)
	13-1/2 (343)	26,360 (117.3)	26,845 (119.4)	27,630 (122.9)	28,775 (128.0)	56,780 (252.6)	57,825 (257.2)	59,510 (264.7)	61,975 (275.7)
	22-1/2 (572)	43,935 (195.4)	44,745 (199.0)	46,050 (204.8)	47,955 (213.3)	94,630 (420.9)	96,370 (428.7)	99,185 (441.2)	103,290 (459.5)
#10	11-1/4 (286)	20,850 (92.7)	22,840 (101.6)	25,585 (113.8)	26,640 (118.5)	44,905 (199.7)	49,190 (218.8)	55,105 (245.1)	57,385 (255.3)
	15 (381)	32,095 (142.8)	33,145 (147.4)	34,110 (151.7)	35,525 (158.0)	69,135 (307.5)	71,385 (317.5)	73,470 (326.8)	76,510 (340.3)
	25 (635)	54,240 (241.3)	55,240 (245.7)	56,850 (252.9)	59,205 (263.4)	116,830 (519.7)	118,980 (529.2)	122,450 (544.7)	127,515 (567.2)

1 See section 3.1.8 for explanation on development of load values.
2 See section 3.1.8 to convert design strength (factored resistance) value to ASD value.
3 Linear interpolation between embedment depths and concrete compressive strengths is not permitted.
4 Apply spacing, edge distance, and concrete thickness factors in tables 22 - 37 as necessary to the above values. Compare to the steel values in table 21. The lesser of the values is to be used for the design.
5 Data is for temperature range A: Max. short term temperature = 130° F (55° C), max. long term temperature = 110° F (43° C).
For temperature range B: Max. short term temperature = 176° F (80° C), max. long term temperature = 110° F (43° C) multiply above values by 0.92.
For temperature range C: Max. short term temperature = 248° F (120° C), max. long term temperature = 162° F (72° C) multiply above values by 0.78.
Short term elevated concrete temperatures are those that occur over brief intervals, e.g., as a result of diurnal cycling. Long term concrete temperatures are roughly constant over significant periods of time.
6 Tabular values are for dry concrete conditions. For water saturated concrete multiply design strength by 0.85.
7 Tabular values are for short term loads only. For sustained loads including overhead use, see section 3.1.8.
8 Tabular values are for normal-weight concrete only. For lightweight concrete, multiply design strength (factored resistance) by λ_a as follows:
For sand-lightweight, λ_a = 0.51. For all-lightweight, λ_a = 0.45.
9 Tabular values are for static loads only. For seismic loads, multiply cracked concrete tabular values in tension and shear by the following reduction factors:
#3 to #6 - α_{seis} = 0.60, #7 - α_{seis} = 0.64, #8 - α_{seis} = 0.68, #9 - α_{seis} = 0.71, #10 - α_{seis} = 0.75
See section 3.1.8 for additional information on seismic applications.

Table 21 - Steel design strength for US rebar[1,2]

Rebar size	ASTM A615 Grade 40 [4]			ASTM A615 Grade 60 [4]			ASTM A706 Grade 60 [4]		
	Tensile[3] ϕN_{sa} lb (kN)	Shear[4] ϕV_{sa} lb (kN)	Seismic[5] Shear $\phi V_{sa,eq}$ lb (kN)	Tensile[3] ϕN_{sa} lb (kN)	Shear[4] ϕV_{sa} lb (kN)	Seismic[5] Shear $\phi V_{sa,eq}$ lb (kN)	Tensile[3] ϕN_{sa} lb (kN)	Shear[4] ϕV_{sa} lb (kN)	Seismic[5] Shear $\phi V_{sa,eq}$ lb (kN)
#3	4,290 (19.1)	2,375 (10.6)	1,665 (7.4)	6,435 (28.6)	3,565 (15.9)	2,495 (11.1)	6,600 (29.4)	3,430 (15.3)	2,400 (10.7)
#4	7,800 (34.7)	4,320 (19.2)	3,025 (13.4)	11,700 (52.0)	6,480 (28.8)	4,535 (20.2)	12,000 (53.4)	6,240 (27.8)	4,370 (19.5)
#5	12,090 (53.8)	6,695 (29.8)	4,685 (20.9)	18,135 (80.7)	10,045 (44.7)	7,030 (31.3)	18,600 (82.7)	9,670 (43.0)	6,770 (30.1)
#6	17,160 (76.3)	9,505 (42.3)	6,655 (29.6)	25,740 (114.5)	14,255 (63.4)	9,980 (44.4)	26,400 (117.4)	13,730 (61.1)	9,610 (42.8)
#7	23,400 (104.1)	12,960 (57.6)	9,070 (40.3)	35,100 (156.1)	19,440 (86.5)	13,610 (60.6)	36,000 (160.1)	18,720 (83.3)	13,105 (58.3)
#8	30,810 (137.0)	17,065 (75.9)	11,945 (53.1)	46,215 (205.6)	25,595 (113.9)	17,915 (79.7)	47,400 (210.8)	24,650 (109.6)	17,255 (76.7)
#9	39,000 (173.5)	21,600 (96.1)	15,120 (67.3)	58,500 (260.2)	32,400 (144.1)	22,680 (100.9)	60,000 (266.9)	31,200 (138.8)	21,840 (97.2)
#10	49,530 (220.3)	27,430 (122.0)	19,200 (85.4)	74,295 (330.5)	41,150 (183.0)	28,805 (128.1)	76,200 (339.0)	39,625 (176.3)	27,740 (123.4)

1 See Section 3.1.8 to convert design strength value to ASD value.
2 ASTM A706 Grade 60 rebar are considered ductile steel elements. ASTM A615 Grade 40 and 60 rebar are considered brittle steel elements.
3 Tensile = $\phi A_{se,N} f_{uta}$ as noted in ACI 318-14 Chapter 17.
4 Shear = $\phi\ 0.60 A_{se,N} f_{uta}$ as noted in ACI 318-14 Chapter 17.
5 Seismic Shear = $\alpha_{V,seis}\ \phi\ V_{sa}$: Reduction for seismic shear only.
See section 3.1.8 for additional information on seismic applications.