Concrete Masonry Design Program (CMD21©) Usability

Architectural Engineering

California Polytechnic State University, San Luis Obispo

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Nomenclature

A_{br} = bearing area, in.^{2}

A_g = gross cross-sectional area of a member, in.^{2}

A_n = net cross-sectional area of a member, in.^{2}

A_s = area of nonprestressed longitudinal tension reinforcement, in.^{2}

A_{st} = total area of laterally tied longitudinal reinforcing steel, in.^{2}

a = depth of an equivalent compression stress block at nominal strength, in.

b = width of section, in.

C_m = force due to compression stress of masonry, lb

c = distance from the maximum compressive strain to the neutral axis, in.

c_b = distance from the maximum compressive strain to the neutral axis at the balanced condition, in.

C_d = deflection amplification factor (unitless)

D = dead load or related internal moments and forces, lb

d_i = distance of extreme compression fiber to centroid of tension reinforcement, in.

E = load effect of earthquake or related internal moments and forces, lb, in.-lb

E_m = modulus of elasticity of masonry in compression, psi

E_s = modulus of elasticity of steel, psi

e = eccentricity of axial load, in.

f’m = specified compressive strength of clay masonry or concrete masonry, psi

f_s = calculated tensile or compressive stress in reinforcement, psi

h = effective height of wall, in.

I_{eff} = effective moment of inertia, in.^{4}

I_g = moment of inertia of gross cross-sectional area of a member, in.^{4}

I_n = moment of inertia of net-cross-sectional area of a member, in.^{4}

L = live load or related internal moments and forces, lb, in.-lb

l = clear span between supports, in.

M	=	maximum moment at the section under consideration, in.-lb
P	=	axial load, lb
P_b	=	balanced axial load, lb
P_u	=	ultimate axial load, lb
P_n	=	nominal axial load, lb
P_{Lr}	=	axial load due to the reduced live loading, lb
P_D	=	axial load due to the dead loading, lb
R	=	response modification coefficient (unitless)
r	=	radius of gyration, in
T_{si}	=	force due to tension in steel, lb
ε_{mu}	=	maximum usable compressive strain on masonry, in/in
ε_{si}	=	tension strain in steel reinforcing members, in/in
ρ	=	reinforcement ratio (unitless)
ϕ	=	strength reduction factor (unitless)

1.0 Introduction:

Spring quarter of 2023 began the collaboration with senior project advisor, Dr. Craig Baltimore, in order to assess the usability for a first-time user of the Concrete Masonry Design Program, CMD21, program, developed by Alan Robinson, through the Concrete Masonry Association of California and Nevada[1]. The CMD21 program is being used in this project to design concrete masonry unit shear walls, although the program has the capacity to assess beams, columns, and more using either concrete or masonry materials. Designing masonry shear walls is extremely relevant to the structural engineering industry, as the production of materials such as Portland and masonry cement in the US has increased from 66,447,000 metric tons in 2010 to 95,000,000 metric tons in 2022.

1.1 Background of Structural Programs:

This project demonstrates the learning curve for a design software, which is a common occurrence in the structural engineering consultation industry. Design-software learning is emphasized greatly in Cal Poly's architectural engineering program curriculum as students are exposed to many software programs throughout their education. Such software includes:

- RISA 2D: Structural analysis and design in two dimensions
- ETabs: Structural analysis and design in three dimensions
- MATLAB: Programming language and numeric computing environment
- Python: High-level programming language and environment
- AutoCAD: Industry standard building information modeling in two dimensions
- Revit: Industry standard building information modeling in three dimensions
- ARCHICAD: Building information modeling in three dimensions
- SPColumn: Finite element analysis
- Excel: Data processing environment and cell-based coding

1.2 Relevant CMD21 Manual Contents:

Chapter 1 of the manual instructs the user how to install the CMD21 program. Chapters 2 and 3 outline the usage of each button on the user interface. Chapter 4 focuses on educating the user on the CMD21 Input Form and what each symbol represents. Chapter 5 outlines relevant equations for strength design with code citations. Chapters 6 through 10 cover different types of strength design such as strength design of walls for out-of-plane loads, for in-plane loads, of beams, of piers, and of columns. This project solely represents the usability of the design program for strength design of walls for in-plane loads. The remaining chapters of the manual perform design tasks using working stress design, which is beyond the scope of this project.

[1] Concrete Masonry Association of California and Nevada, 6060 Sunrise Vista Dr., Suite 1990, Citrus Heights, CA 95610, Telephone: 916-722-1700, Website: cmacn.org

2.0 Purpose:

The purpose of this project is to verify the first-time usability of the CMD21 program for designing masonry shear walls with in-plane loads using strength design. Another aspect of this project is being exposed to using and understanding this software, as learning new software pertains to being a practicing engineer. The practice of hand-calculations for a shear wall also demonstrates an understanding of non-linear stress behavior and free body diagrams.

3.0 Program Verification:

The use of the CMD21 program to analyze a shear wall with in-plane loading will be verified by liberally referencing an example of a wall-line for a single-story office building with the following conditions. The wall is 8" CMU, and it is configured as shown in figures 3.1 and 3.2 below. The site is classified as Site Class D. The wall is fully grouted, using medium weight concrete masonry units laid in running bond with type S mortar. The masonry has a specified compressive strength, f'_m, of 2000 psi. The steel reinforcement is Grade 60 (f_y = 60 ksi) with design coefficients of R = 5.0 and C_d = 3.5.

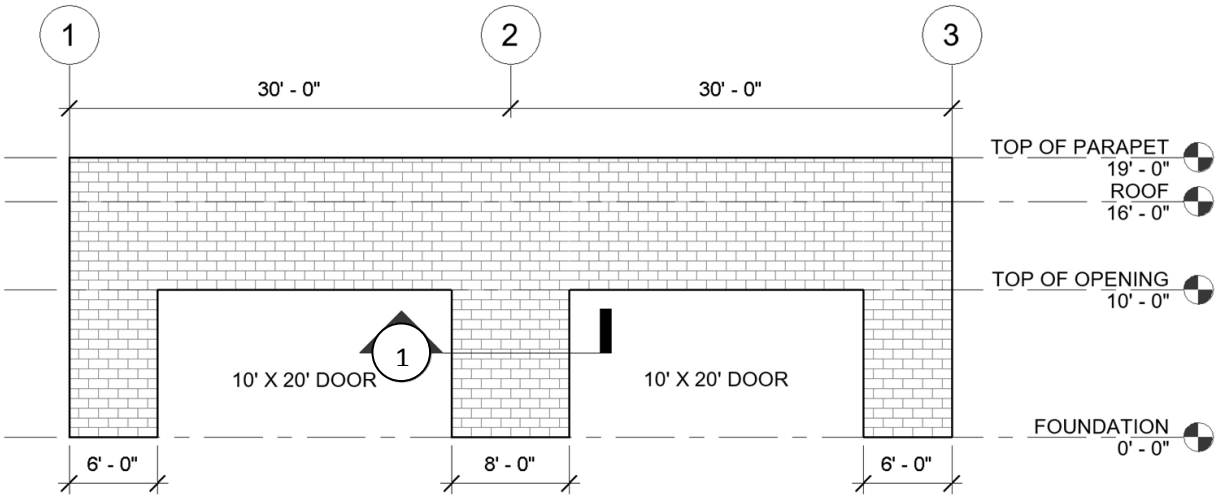


Figure 3.1: An elevation of the wall with a section cut through the wall being analyzed.

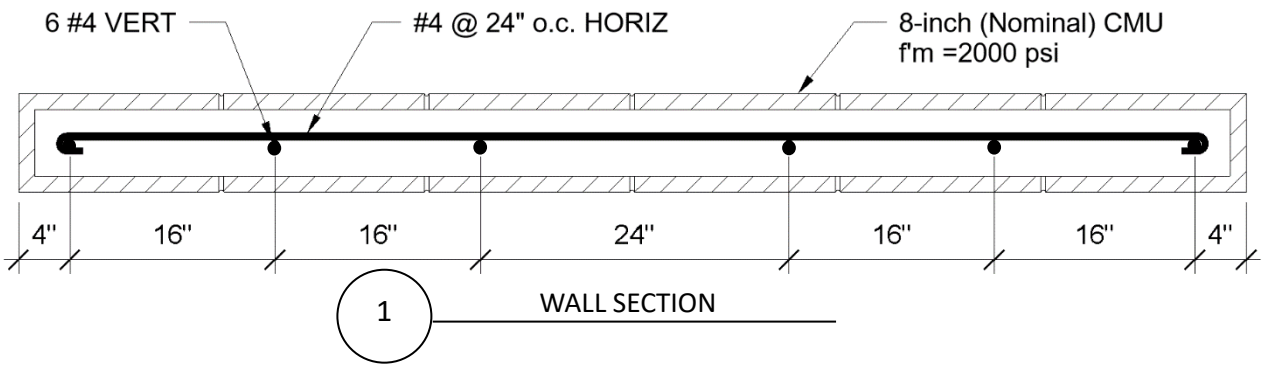


Figure 3.2: A section cut through the wall segment being analyzed that shows the reinforcement spacing.

Loading Diagrams:

The following figures define the realistic dead loads and earthquake loads for this project needed to analyze the center wall section and define the demand loads using load combinations from ASCE 7-16.

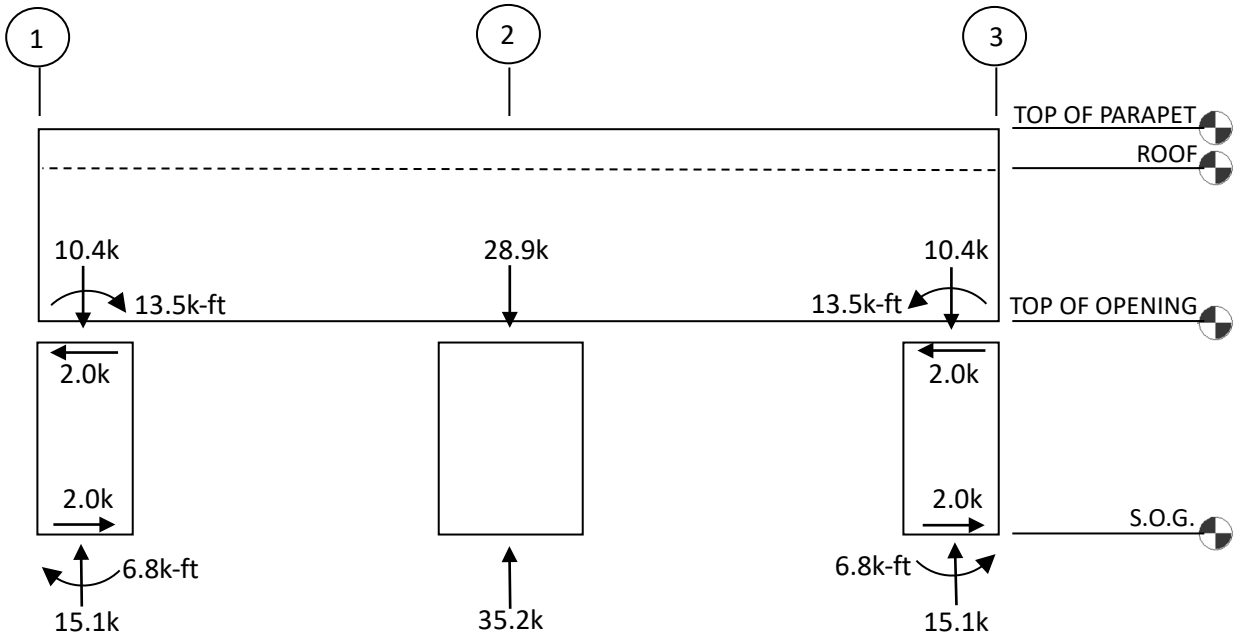


Figure 3.3: Dead loads along the wall-line

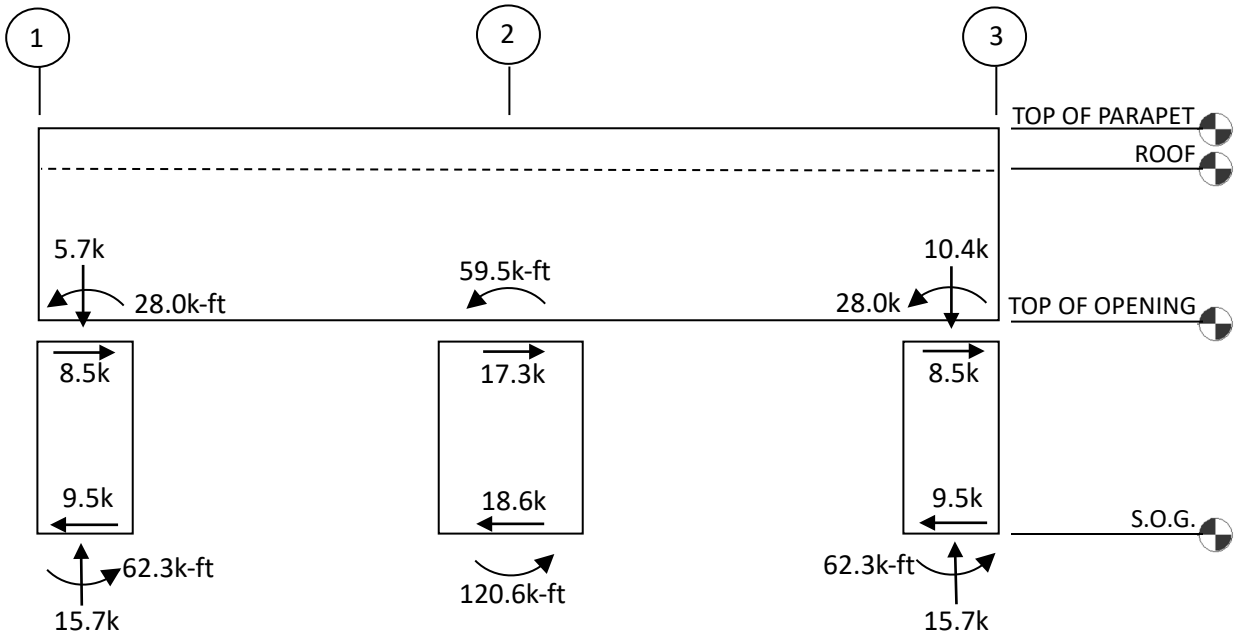


Figure 3.4: Earthquake loads along the wall-line

3.1 Hand Calculations:

Load Combinations:

ϕ=0.9 (TMS 402-16 Section 9.1.4)

1. 1.2D + 1.6L

   P_u = 49.1 kips; M_u = 0 kip-ft; V_u = 0 kips (at the top of the wall segment)

   P_u = 56.6 kips; M_u = 0 kip-ft; V_u = 0 kips (at the bottom of the wall segment)

2. 1.2D + E = 1.4D + E_h

   P_u = 40.5 kips; M_u = 59.5 kip-ft; V_u = 17.3 kips (at the top of the wall segment)

   P_u = 49.3 kips; M_u = 120.6 kip-ft; V_u = 18.7 kips (at the bottom of the wall segment)

3. 0.9D + E = 0.7D + E_h

   P_u = 20.2 kips; M_u = 59.5 kip-ft; V_u = 17.3 kips (at the top of the wall segment)

   P_u = 24.6 kips; M_u = 120.6 kip-ft; V_u = 18.7 kips (at the bottom of the wall segment)

To assess the axial force and moment interaction, the three points of the interaction diagram, the pure axial condition, the pure flexural condition, and the balanced condition, must be calculated.

3.1a Pure Axial Condition:

Radius of gyration for a rectangular cross-section: $r = \sqrt{\frac{I}{A}} = \frac{t}{\sqrt{12}} = \frac{7.63\ in}{\sqrt{12}} = 2.20\ in$

Slenderness ratio: $\frac{h}{r} = \frac{16ft(12in/ft)}{2.20in} = 87.3 < 99$

$P_n = 0.80\left[0.80f'_m(A_n - A_{st}) + f_y A_{st}\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$ (TMS 402-16 Eq. 9-15)

$P_n = 0.80[0.80(2ksi)(7.63in \times 96in - 0) + 60ksi(0)]\left[1 - \left(\frac{87.3}{140}\right)^2\right] = 573\ kips$

$\Phi P_n = 0.9(573) = 516kips > P_u$

In the pure flexural condition, the distance from the furthest compression fiber to the neutral axis, c, is unknown. In order to solve for c, a process of trial and error must be completed such that the tension and compression forces from the rebar and masonry, respectively, are equally balanced. In this condition, the masonry has reached its maximum strain, which is, according to TMS 402-16 section 9.3.2, a strain of 0.0025. A value of c is chosen, and the following calculations are completed until the tension and compression forces are equal.

3.1b Pure Flexural Condition:

$a = 0.8c$ (per TMS 402-16 Section 9.3.2)

$C_m = 0.8abf'_m$ (per TMS 402-16 Section 9.3.2)

$\varepsilon_{si} = \varepsilon_{mu}\left(\frac{d_i - c}{c}\right)$ (by similar triangles)

$T_{si} = \varepsilon_{si}E_sA_{si} = \varepsilon_{mu}\left(\frac{d_i - c}{c}\right)E_sA_{si} \leq f_yA_{si}$

$\Sigma T_{si} + P_u = C_m$ (P_u=0 in the pure flexural condition)

$\varepsilon_{mu} = 0.0025$ (per TMS 402-16 Section 9.3.2)

Through trial and error, the neutral axis has been selected (c=6.14in)

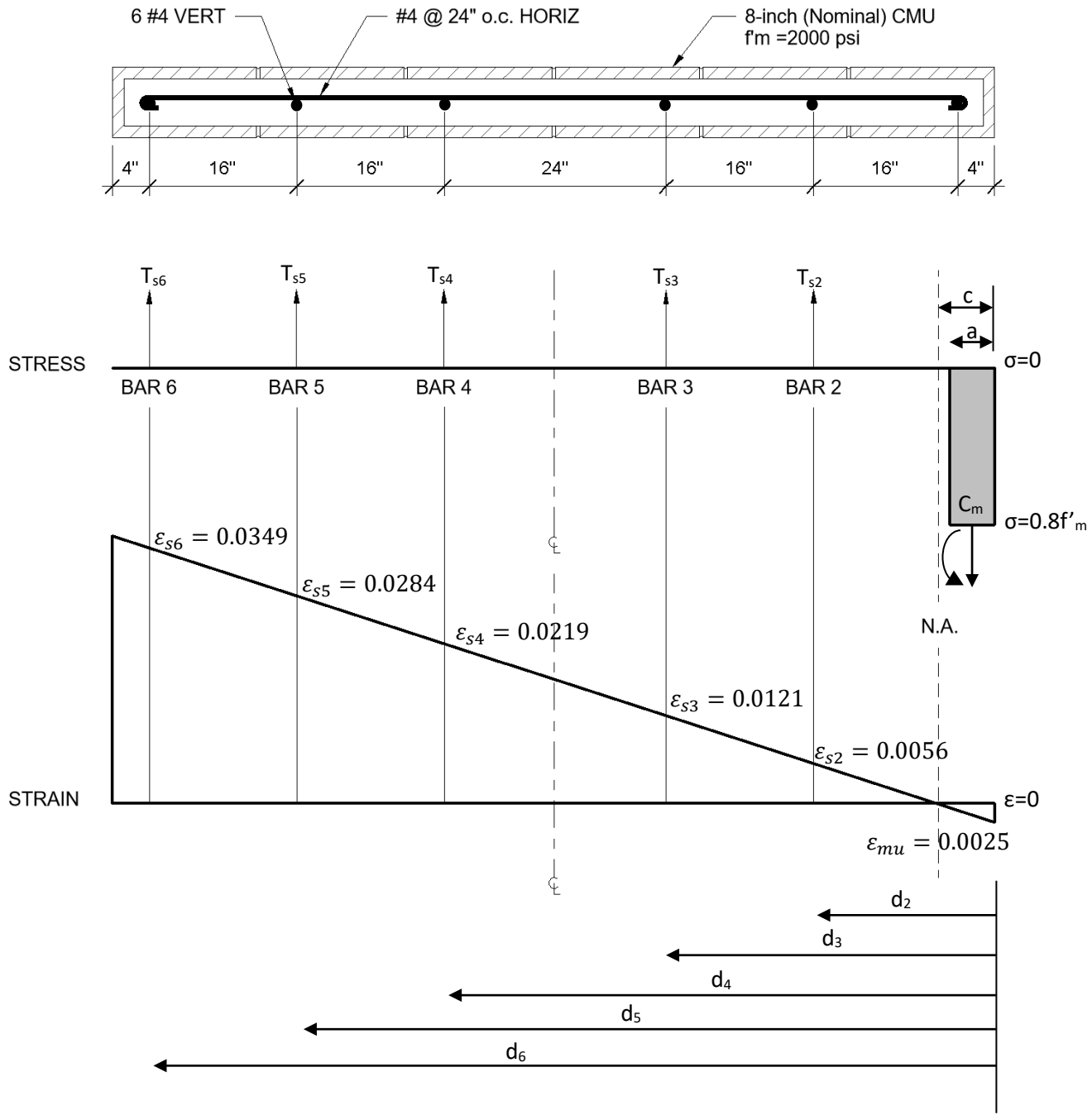


Figure 3.5: Stress and Strain diagrams labeled with the appropriate force resultants and strain values at the pure flexural condition

Table 3.1: Used to calculate total axial loads in the y direction and the flexural loads about the center line for the pure axial condition

	A_{si} (in^2)	d_i (in)	ε_{si} (in/in)	f_{si} (ksi)	P_{nCL} (kips)	Moment Arm	M_{nCL} (kip-in)
Masonry		2.46			60.00	45.540”	2732.60
Steel							
Bar 1	0.20	4.00	*	*	*	*	*
Bar 2	0.20	20.00	-0.0056	-60.00	-12.00	28.000”	-336.00
Bar 3	0.20	36.00	-0.0121	-60.00	-12.00	12.000”	-144.00
Bar 4	0.20	60.00	-0.0219	-60.00	-12.00	12.000”	144.00
Bar 5	0.20	76.00	-0.0284	-60.00	-12.00	28.000”	336.00
Bar 6	0.20	92.00	-0.0349	-60.00	-12.00	44.000”	528.00
Total	**1.20**				**0.00**		**3260.60**

*Compressive steel is not calculated as a compressive force per TMS 402-16 Section 9.3.2

$$\phi M_{nCL} = 0.9\left(\frac{3260.6\ kip-in}{12\ in/ft}\right) = 240\ kip-ft$$

Unlike the iterative process needed to determine the neutral axis location in the pure flexural condition, the balanced condition has the maximum strain values at yielding. For the tension steel, the ultimate strain is equal to 0.0021, and, for the compression masonry, the ultimate strain is equal to 0.0025. A line is then drawn connecting both strain values, and strain at each bar of tension rebar can be calculated using similar triangles. The value at which the line crosses the zero strain line is the new neutral axis value, c_b, which is given from the furthest compression fiber.

3.1c Balanced Condition:

$$c_b = \frac{d}{\left(\frac{f_y}{E_s \varepsilon_{mu}} + 1\right)} = \frac{d}{\left(\frac{60ksi}{29{,}000ksi \times 0.0025} + 1\right)} = 0.547d = 0.547(92in) = 50.3"$$

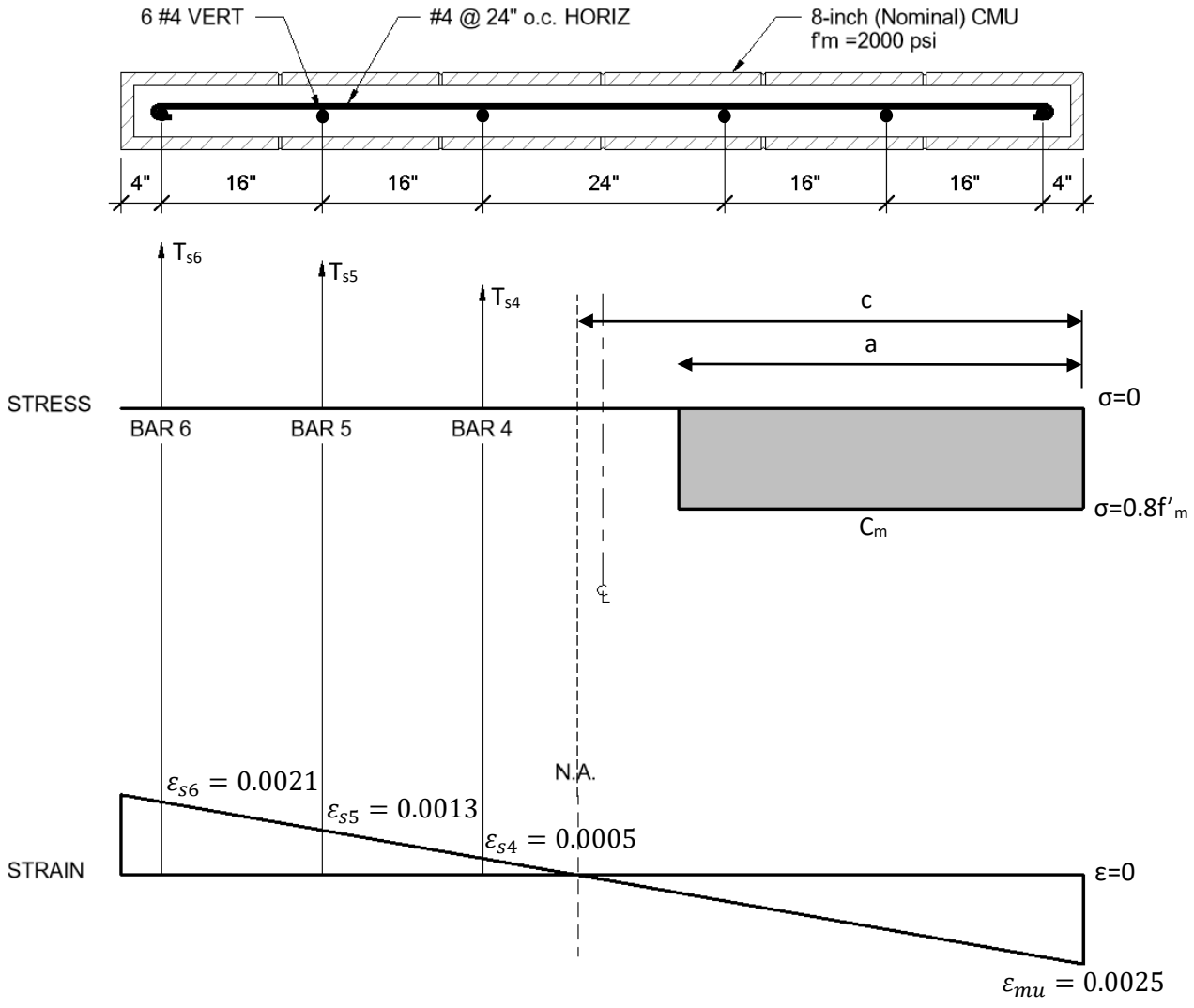


Figure 3.6: Stress and Strain diagrams labeled with the appropriate force resultants and strain values at the balanced condition.

Table 3.2: Used to calculate total axial loads in the y direction and the flexural loads about the center line for the balanced condition

	A_{si} (in^2)	d_i (in)	ε_{si} (in/in)	f_{si} (ksi)	P_{nCL} (kips)	Moment Arm	M_{nCL} (kip-in)
Masonry		20.13			491.50	5.560”	2732.60
Steel							
Bar 1	0.20	4.00	*	*	*	*	*
Bar 2	0.20	20.00	*	*	*	*	*
Bar 3	0.20	36.00	*	*	*	*	*
Bar 4	0.20	60.00	-0.0005	-13.90	-2.80	12.000”	33.50
Bar 5	0.20	76.00	-0.0013	-37.00	-7.40	28.000”	207.10
Bar 6	0.20	92.00	-0.0021	-60.00	-12.00	44.000”	528.00
Total	**1.20**				**469.30**		**14,466.50**

*Compressive steel is not calculated as a compressive force per TMS 402-16 Section 9.3.2

$$\phi P_b = 0.9(469.3kips) = 422\ kips$$

$$\phi M_b = 0.9\left(\frac{14{,}466.5\ kip-in}{12\ in/ft}\right) = 1085\ kip-ft$$

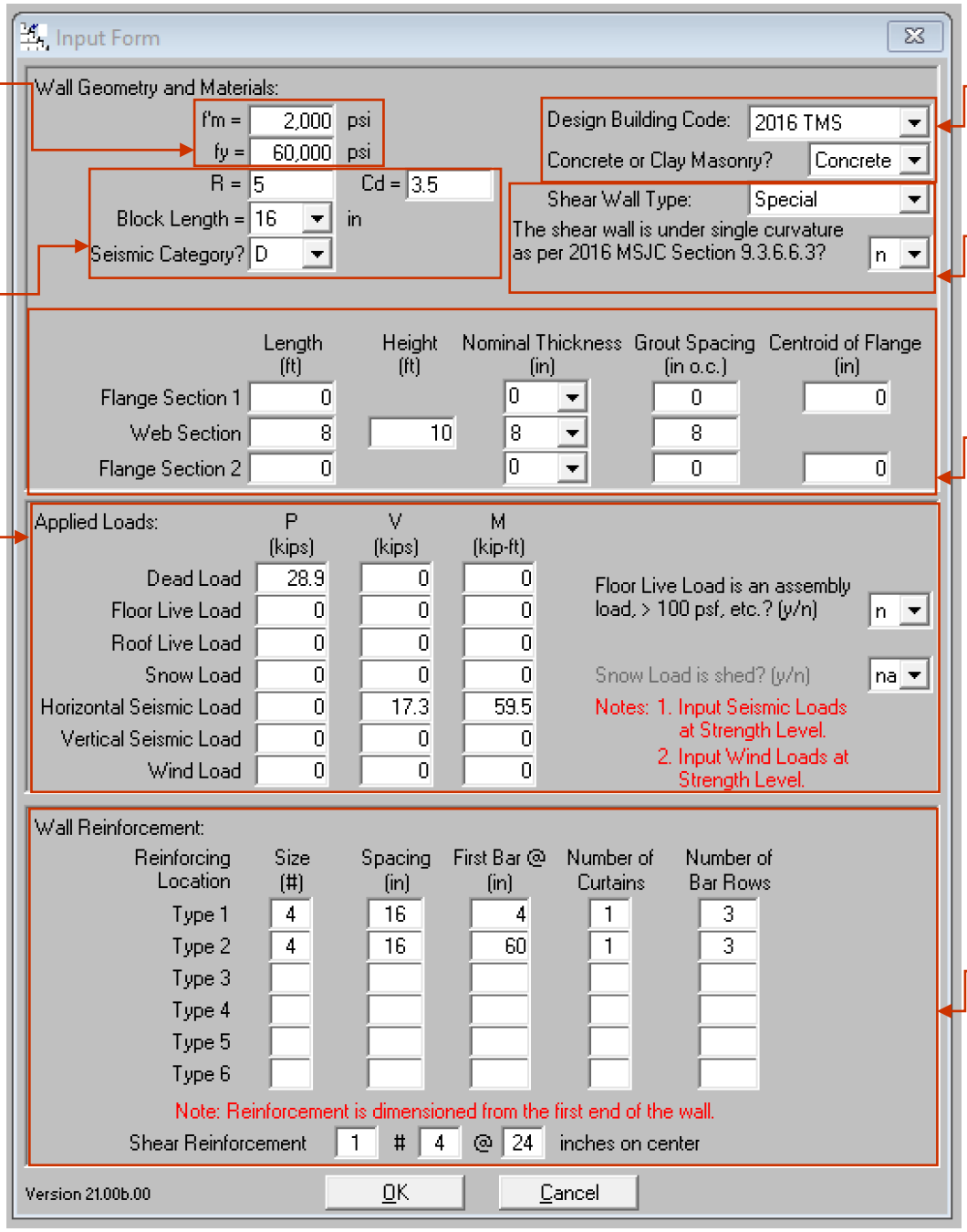
3.2 CMD21 Input (10’ Wall):

Using the CMD21 design program, input values were filled in for the 8’ wide, 10’ tall shear wall.

The given material properties.

The given site properties and geometry.

Given or calculated loads are input here, and, given that this problem is calculating a strength design shear wall, live load is not considered. The program will not calculate the loads at the bottom of the wall, so if the loads at the base of the wall are the ultimate loads, they will need to be calculated and input here.



This example problem used the 2016 TMS, although other codes are available.

This shear wall is designed, fully grouted, and reinforced as a special shear wall.

This 8’ shear wall is 10’ tall with no flanges, and it is fully grouted, so grout spacing is indicated as 8” o.c., meaning both cells of the 16” blocks are grouted.

This rebar spacing follows the given diagram, figure 3.2, of the shear wall. Two types of rebar must be input to account for the change in spacing, the 24” gap between bars of rebar, in the center of the wall.

3.3 CMD21 Output (10’ Wall):

The CMD21 program outputs the following pages, which include the input properties, a plan view sketch of the wall, output values, some general calculations, load combinations, and a separate output of the interaction diagram was included as well.

Project:

Number:
Date: May 18, 2023
By:
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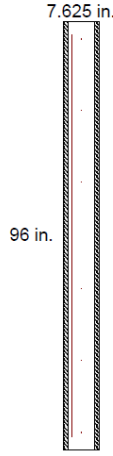
CMD21.00b.00 Filename: SHEAR WALL VERIFICATION 10 FC

Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 TMS Section 9.3.6
- Ultimate Strength Design -

Input:



3 Sets of 1 # 4 @ 16 in. o.c. between 60.00 in. and 92.00 in.

3 Sets of 1 # 4 @ 16 in. o.c. between 4.00 in. and 36.00 in.

Shear Steel: 1 # 4 @ 24 in. o.c.
Plan View

Reinforcing dimensioned from this end of wall:

Wall Geometry:
10.00' high X 8.00' long CMU wall

	Length feet	Nominal Thickness inches	Grout Spacing inches	Effective Thickness/Width inches
Flange 1	None			
Web	8.00	8	8	7.625
Flange 2	None			

Wall Material Properties:
f'_m = 2,000 psi E_m = 1,800 ksi
f_y = 60,000 psi E_s = 29,000 ksi
n = 16.11

Applied Loads:	P kips	V kips	M kip-ft
Dead Load	28.9	0.00	0.00
Floor Live Load	0.0	0.00	0.00
Roof Live Load	0.0	0.00	0.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	0.0	17.30	59.50
Vertical Seismic Load	0.0	0.00	0.00
Wind Load	0.0	0.00	0.00

Seismic Design Category: D

Output:

P vs M Diagram Points:

		Positive Moments P_n, M_n	Positive Moments $\phi P_n, \phi M_n$	Positive Moments ϕ	Negative Moments P_n, M_n	Negative Moments $\phi P_n, \phi M_n$	Negative Moments ϕ
Maximum Axial Load (kips)	P_{max} =	994	715	0.90	994	715	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	0	675	0.90	0	-675	0.90
Balanced Axial Load (kips)	P_b =	398	358	0.90	398	358	0.90
Balanced Moment (kip-ft)	M_b =	1,205	1,084	0.90	-1,205	-1,084	0.90
Moment at P=0 (kip-ft)	M_o =	273	246	0.90	-273	-246	0.90

General Calculations:

Total A_s = 1.20 in², ρ = 0.0016 ≤ 0.0741 = ρ_{max} - OK
Modulus of Rupture: f_r = 267 psi
Gross Section Moduli: S_g+ = 11,712 in³
S_g- = 11,712 in³
Cracking Moment strengths: M_{cr}+ = 260.6 kip-ft
M_{cr}- = -260.6 kip-ft

Minimum Nominal Shear Strength of the Wall, V_n = 136.8 kips ≥ V = 2.5 times maximum V_u = 43.3 kips
and, V = 2.5 times maximum V_u < (V corresponding to 1.25M_n)/ϕ = 196.9 kips - OK

		Project:	Program Verification
		Number:	1
		Date:	May 18, 2023
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CMD21.00b.00 Filename: SHEAR WALL VERIFICATION 10 FOC

Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 TMS Section 9.3.6
- Ultimate Strength Design -

Strength Checks - Moments:

	P_n kips	M_n kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	45.0	450	0.90	40.5	405	40.5	0	0.06
1.2D+1.6L+0.5Lr	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.6L+0.5S	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.6Lr+0.5L	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.6Lr+0.5W	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.6S+0.5L	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.6S+0.5W	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.0W+0.5L+0.5Lr	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.0W+0.5L+0.5S	38.5	426	0.90	34.7	384	34.7	0	0.05
1.2D+1.0W	38.5	426	0.90	34.7	384	34.7	0	0.05
0.9D+1.0W	28.9	390	0.90	26.0	351	26.0	0	0.04
1.2D+1.0E+0.5L+0.2S	38.5	426	0.90	34.7	384	34.7	60	0.16
1.2D-1.0E+0.5L+0.2S	38.5	-426	0.90	34.7	-384	34.7	-60	0.16
1.2D+1.0E	38.5	426	0.90	34.7	384	34.7	60	0.16
1.2D-1.0E	38.5	-426	0.90	34.7	-384	34.7	-60	0.16
0.9D+1.0E	28.9	390	0.90	26.0	351	26.0	60	0.17
0.9D-1.0E	28.9	-390	0.90	26.0	-351	26.0	-60	0.17

Strength Checks - Shears:

	$1¼M_n$ kip-ft	$V\{1¼M_n\}$ kips	V_n kips	ϕ	ϕV_n kips	V_u kips	Stress Ratio
1.4D	542	157.6	165.1	0.80	132.0	0.0	0.00
1.2D+1.6L+0.5Lr	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.6L+0.5S	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.6Lr+0.5L	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.6Lr+0.5W	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.6S+0.5L	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.6S+0.5W	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W+0.5L+0.5Lr	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W+0.5L+0.5S	542	157.6	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W	542	157.6	163.6	0.80	130.9	0.0	0.00
0.9D+1.0W	542	157.6	161.4	0.80	129.2	0.0	0.00
1.2D+1.0E+0.5L+0.2S	542	157.6	139.0	0.80	111.2	17.3	0.16
1.2D-1.0E+0.5L+0.2S	542	157.6	139.0	0.80	111.2	-17.3	0.16
1.2D+1.0E	542	157.6	139.0	0.80	111.2	17.3	0.16
1.2D-1.0E	542	157.6	139.0	0.80	111.2	-17.3	0.16
0.9D+1.0E	542	157.6	136.8	0.80	109.5	17.3	0.16
0.9D-1.0E	542	157.6	136.8	0.80	109.5	-17.3	0.16

	Project:
	Number:
	Date: May 18, 2023
T:	By:
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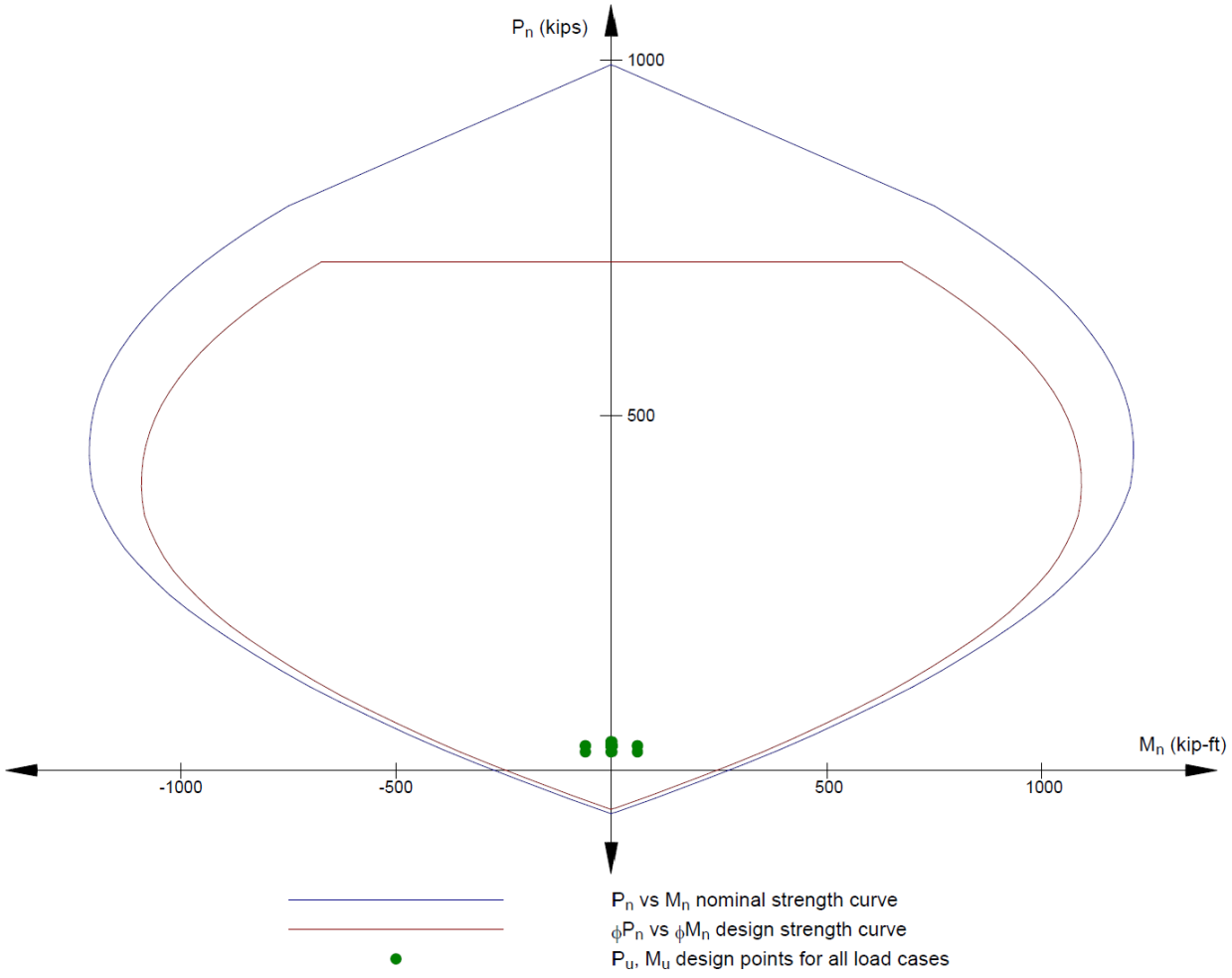
CMD21.00b.00 Filename: SHEAR WALL VERIFICATION 10 FOO

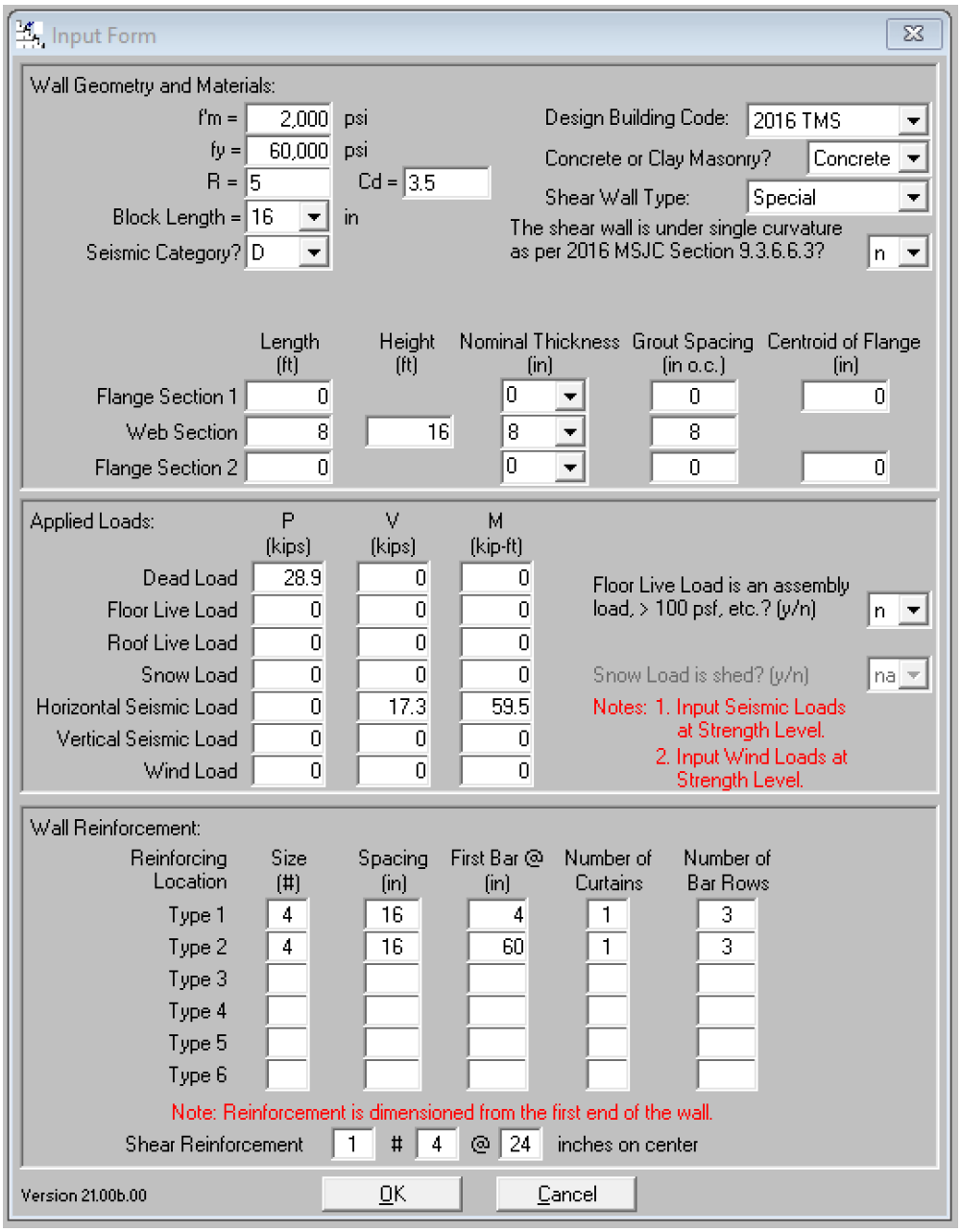
Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 TMS Section 9.3.6
- Ultimate Strength Design -

Axial Load vs Moment Diagram



3.4 CMD21 Input (16’ Wall):

The CMD21 input was populated with the same values once more, but the height of the wall was changed to 16’ to account for out-of-plane slenderness.



3.5 CMD21 Output (16’ Wall):

The most notable change from the 10’ wall output values to the 16’ wall output values is a decrease in axial capacity in the “Output” section, under “P vs M Diagram Points.”

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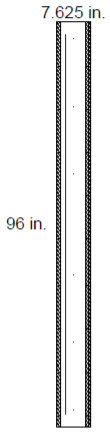
Input:



3 Sets of 1 # 4 @ 16 in. o.c. between 60.00 in. and 92.00 in.

3 Sets of 1 # 4 @ 16 in. o.c. between 4.00 in. and 36.00 in.

Shear Steel: 1 # 4 @ 24 in. o.c.
Plan View

Reinforcing dimensioned from this end of wall:

Wall Geometry:
16.00' high X 8.00' long CMU wall

	Length feet	Nominal Thickness inches	Grout Spacing inches	Effective Thickness/Width inches
Flange 1	None			
Web	8.00	8	8	7.625
Flange 2	None			

Wall Material Properties:

f'_m =	2,000 psi	E_m =	1,800 ksi
f_y =	60,000 psi	E_s =	29,000 ksi
		n =	16.11

Applied Loads:	P kips	V kips	M kip-ft
Dead Load	28.9	0.00	0.00
Floor Live Load	0.0	0.00	0.00
Roof Live Load	0.0	0.00	0.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	0.0	17.30	59.50
Vertical Seismic Load	0.0	0.00	0.00
Wind Load	0.0	0.00	0.00

Seismic Design Category: D

Output:

P vs M Diagram Points:

		Positive Moments P_n, M_n	Positive Moments $\phi P_n, \phi M_n$	Positive Moments ϕ	Negative Moments P_n, M_n	Negative Moments $\phi P_n, \phi M_n$	Negative Moments ϕ
Maximum Axial Load (kips)	P_{max} =	717	516	0.90	717	516	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	0	675	0.90	0	-675	0.90
Balanced Axial Load (kips)	P_b =	287	258	0.90	287	258	0.90
Balanced Moment (kip-ft)	M_b =	1,205	1,084	0.90	-1,205	-1,084	0.90
Moment at P=0 (kip-ft)	M_o =	273	246	0.90	-273	-246	0.90

General Calculations:

Total A_s = 1.20 in², ρ = 0.0016 $\leq$ 0.0741 = ρ_{max} - OK

Modulus of Rupture:	f_r =	267	psi
Gross Section Moduli:	S_g+ =	11,712	in³
	S_g- =	11,712	in³
Cracking Moment strengths:	M_{cr}+ =	260.6	kip-ft
	M_{cr}- =	-260.6	kip-ft

Minimum Nominal Shear Strength of the Wall, V_n = 136.8 kips $\geq$ V = 2.5 times maximum V_u = 43.3 kips
and, V = 2.5 times maximum V_u < (V corresponding to $1.25M_n/\phi$ = 223.1 kips - OK

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- Ultimate Strength Design -

Strength Checks - Moments:

	P_n kips	M_n kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	45.0	513	0.90	40.5	462	40.5	0	0.08
1.2D+1.6L+0.5Lr	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.6L+0.5S	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.6Lr+0.5L	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.6Lr+0.5W	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.6S+0.5L	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.6S+0.5W	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.0W+0.5L+0.5Lr	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.0W+0.5L+0.5S	38.5	482	0.90	34.7	434	34.7	0	0.07
1.2D+1.0W	38.5	482	0.90	34.7	434	34.7	0	0.07
0.9D+1.0W	28.9	432	0.90	26.0	389	26.0	0	0.05
1.2D+1.0E+0.5L+0.2S	38.5	482	0.90	34.7	434	34.7	60	0.14
1.2D-1.0E+0.5L+0.2S	38.5	-482	0.90	34.7	-434	34.7	-60	0.14
1.2D+1.0E	38.5	482	0.90	34.7	434	34.7	60	0.14
1.2D-1.0E	38.5	-482	0.90	34.7	-434	34.7	-60	0.14
0.9D+1.0E	28.9	432	0.90	26.0	389	26.0	60	0.15
0.9D-1.0E	28.9	-432	0.90	26.0	-389	26.0	-60	0.15

Strength Checks - Shears:

	$1¼M_n$ kip-ft	$V\{1¼M_n\}$ kips	V_n kips	ϕ	ϕV_n kips	V_u kips	Stress Ratio
1.4D	614	178.5	165.1	0.80	132.0	0.0	0.00
1.2D+1.6L+0.5Lr	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.6L+0.5S	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.6Lr+0.5L	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.6Lr+0.5W	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.6S+0.5L	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.6S+0.5W	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W+0.5L+0.5Lr	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W+0.5L+0.5S	614	178.5	163.6	0.80	130.9	0.0	0.00
1.2D+1.0W	614	178.5	163.6	0.80	130.9	0.0	0.00
0.9D+1.0W	614	178.5	161.4	0.80	129.2	0.0	0.00
1.2D+1.0E+0.5L+0.2S	614	178.5	139.0	0.80	111.2	17.3	0.16
1.2D-1.0E+0.5L+0.2S	614	178.5	139.0	0.80	111.2	-17.3	0.16
1.2D+1.0E	614	178.5	139.0	0.80	111.2	17.3	0.16
1.2D-1.0E	614	178.5	139.0	0.80	111.2	-17.3	0.16
0.9D+1.0E	614	178.5	136.8	0.80	109.5	17.3	0.16
0.9D-1.0E	614	178.5	136.8	0.80	109.5	-17.3	0.16

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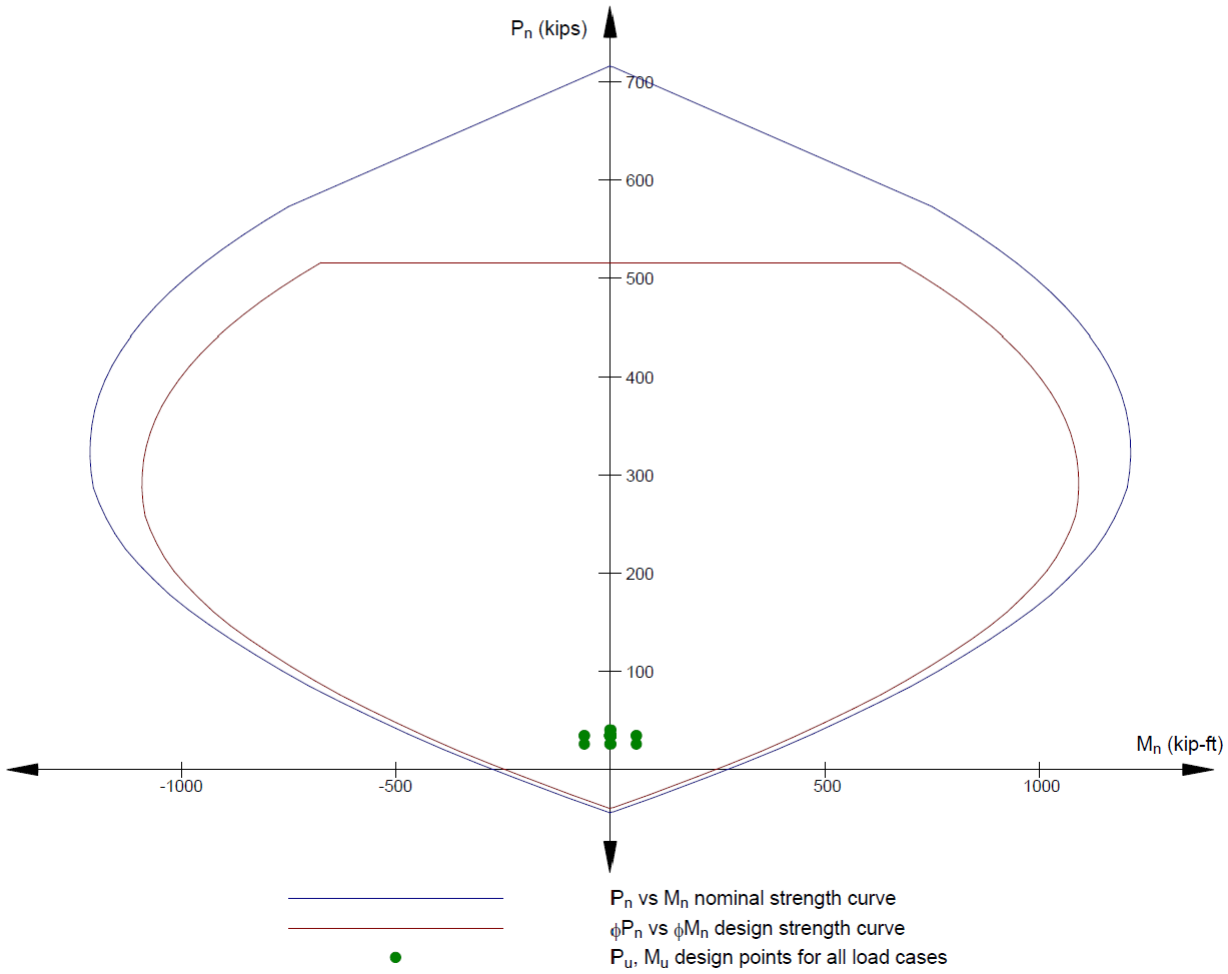
Axial Load vs Moment Diagram



3.6 Summary Comparing Hand Calculations to CMD21 Results:

Hand Calculations:

- Pure Axial: $\phi P_n = 516 kips$
- Balanced Condition: $\phi P_b = 422\ kips$, $\phi M_b = 1085\ kip - ft$
- Pure Flexural: $\phi M_n = 240\ kip - ft$

CMD21 Result (10’ Wall Height):

- Pure Axial: $\phi P_n = 675 kips$
- Balanced Condition: $\phi P_b = 358\ kips$, $\phi M_b = 1084\ kip - ft$
- Pure Flexural: $\phi M_n = 246\ kip - ft$

CMD21 Result (16’ Wall Height):

- Pure Axial: $\phi P_n = 516 kips$
- Balanced Condition: $\phi P_b = 258\ kips$, $\phi M_b = 1084\ kip - ft$
- Pure Flexural: $\phi M_n = 246\ kip - ft$

4.0 Ease of Use Based on Program Verification:

The CMD21 program was assessed based on the program verification in section 3.0 of this report. Three inaccuracies and a few observations beyond the scope of the CMD21 manual were recorded, and, in response, there are suggested improvements to be made to the program.

4.1 Inaccuracies:

- The CMD21 manual has an example problem, on page 79, that describes a shear wall loaded in-plane. The example problem defines the distributed dead load as 4.5 klf, which produces a point load of 49.88 kips, while the CMD21 input uses 6 klf multiplied by the 8’ length of the wall to produce a 61.888 kip point load at the center of the wall to represent the dead load.
- An error message with the code E3102 was displayed with the message, “Grout Spacing must be in even multiples of cell spacing,” while the same error in the CMD21 manual is listed as error code E3602.
- The strength design in-plane loading shear wall input form in CMD21 includes an input that reads, “The shear wall is under single curvature as per MSJC Section 9.3.6.6.3.” The input should read, “The shear wall is under single curvature per the TMS 402-16 Section 9.3.6.6.3.”

4.2 Observations:

- In an example problem such as the one in section 3.0 of this report, slenderness is represented out-of-plane at a 16-foot height, while the wall loads are being applied at the opening height of 10 feet. A CMD21 analysis, equivalent to hand calculations, would need to account for this out-of-plane slenderness in order to delineate accurate results.
- Every time input values are adjusted, the design building code input defaults to the 2022 CBC, which can lead to accidental inaccuracies.
- The sheet number "__ of __" never populates with actual values.
- CMD21 does not calculate design values, so ultimate values at the base of the wall must be calculated separately and input by the user in the "Applied Loads" section.
- The interaction diagram does not differentiate between nominal and design values when the diagram is printed in black and white.

4.3 Suggested Improvements:

- Graph points should be labeled or listed somewhere on the interaction diagram because it is printed as a separate report and not included in the complete report.
- Indicating fully grouted in the grout spacing input should be clearer.
- Interaction diagram lines should be indicated by different line weights or dashes.
- In-plane axial load is affected by out-of-plane slenderness, so there needs to be a warning or a software modification to account for this.

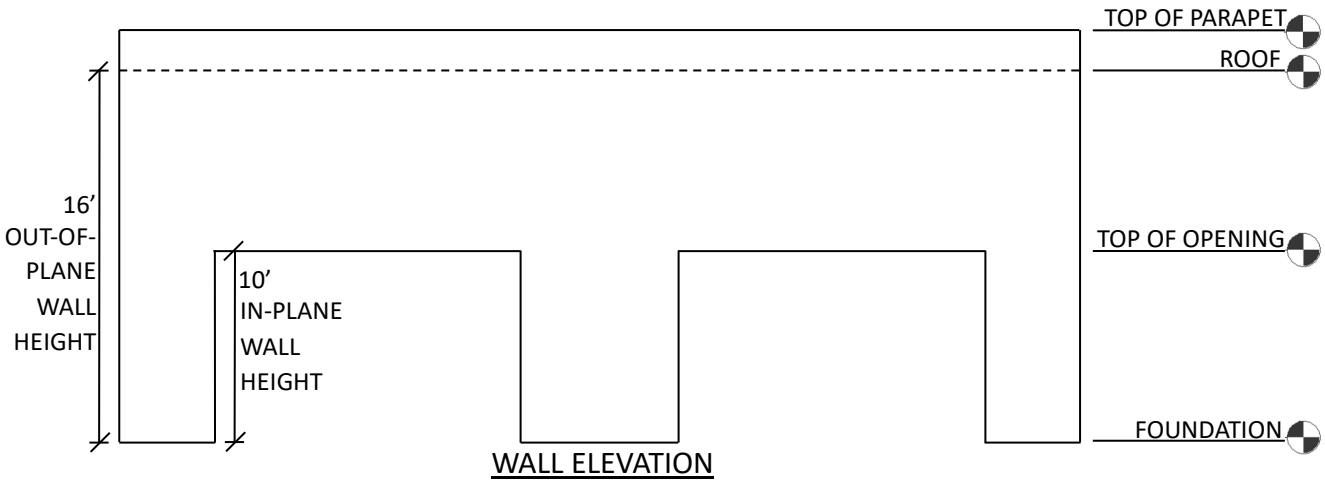


Figure 4.1: Wall elevation depicting both in-plane and out-of-plane heights

5.0 Conclusion:

Learning how to use CMD21 throughout this spring quarter was an exciting experience. I successfully practiced self-learning through the use of the CMD21 user manual and masonry hand calculations. Overall, I would recommend using the CMD21 software to others for in-plane strength design of shear walls, given my experience with the program, as long as the user is aware of the most pressing issue, the height discrepancy between the out-of-plane slenderness height and actual in-plane height of the wall. Upon combining results using both possible wall heights, I was able to determine fairly accurate capacity values, as listed in section 3.6. The capacity values acquired were somewhat conservative, but, overall, not too different from the hand-calculation values. Therefore, I would be able to save time and calculate masonry shear wall capacities efficiently by using the program in industry. The program environment is, for the most part, simple and straight forward. There are three inaccuracies that I discovered within the program and user manual, as listed in section 4.1, though none of them alter the effectiveness of the program itself. The observations that I made, as listed in section 4.2, are most likely to lead to inaccurate results and user errors, thus they are logically followed up by suggested improvements in section 4.3. I enjoyed using the CMD21 program, and I will likely use it again in the future.

6.0 Appendix

6.1 CMD21 User Manual Table of Contents:

Table of Contents

6.2 CMD21 User Manual Chapter 7:

Chapter 7 Strength Design of Masonry Walls for In-Plane Loads

SUMMARY

This program is written for concrete or clay masonry walls with in-plane shear and moments. The program uses the strength design procedures found in the 2008 MSJC Section 3.3.6 (referenced by the 2009 IBC/2010 CBC), 2011 MSJC Section 3.3.6 (referenced by the 2012 IBC/2013 CBC), 2013 MSJC Section 9.3.6 (referenced by the 2015 IBC/2016 CBC) or 2016 TMS Section 9.3.6 (referenced by the 2018 IBC/2019 CBC/2021 IBC/2022 CBC).

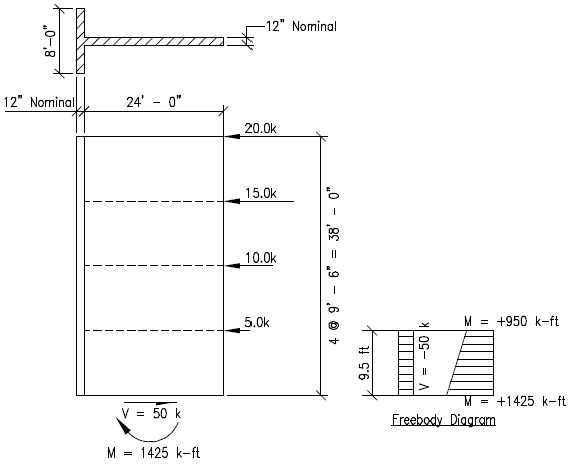
The program is capable of analyzing walls of T, L, I and C shapes as well as rectangular. Any flange section can be included at either end or both ends of the wall. The program allows partial grouting of the flange sections for all standard block thicknesses, 6", 8", 10", 12", and 16". If the user inputs a T, L, I or C shape with flange block thickness greater than 16" the flange must be fully grouted.

The program uses ultimate strength design to determine if the wall, with input reinforcement, is adequate to support the required load combinations. The program checks ultimate shear, axial and moment loadings. The program calculates one nominal axial load versus moment curve based on a maximum masonry strain of 0.0025 for concrete masonry and 0.0035 for clay masonry and a maximum compression stress in the masonry of $0.80f'_m$. The program determines a demand capacity ratio of ultimate load versus nominal load and whether boundary members are required. The program requires the designer to specify the amount of reinforcing steel.

ASSUMPTIONS MADE IN THE PROGRAM:

1. For partially grouted walls, the program requires the cells at the ends of the walls to be grouted solid and the grout spacing is limited to a maximum of one half (½) the length of the wall. The grout spacing is limited per the maximum vertical steel spacing depending on the type of shear wall being designed; *ordinary*, *intermediate* or *special reinforced masonry shear wall*. (2008 MSJC Sections 1.17.3.2.4 through 1.17.3.2.6, 2011 MSJC Sections 1.18.3.2.4 through 1.18.3.2.6, 2013 MSJC/2016 TMS Sections 7.3.2.4 through 7.3.2.6)
2. The program calculates the stresses for partially grouted sections of walls based on an equivalent thickness of masonry, counting only the grouted cells and the flanges between the grouted cells for resisting the required loads. The masonry unit webs which are not surrounding a grouted cell are assumed to have no mortar and are not counted in the equivalent thickness or the effective area. (See Table #1, page 40, Table #2, page 40, Table #3, page 41, and ,Figure 3 page 42)
3. Shear stresses are only resisted by the web section of the wall.

4 The value of f'_m used is limited as required by 2008 MSJC Section 3.1.8.1.1 (referenced by the 2009 IBC/2010 CBC), 2011 MSJC Section 3.1.8.1.1 (referenced by the 2012 IBC/2013 CBC) or 2013 MSJC/2016 TMS Section 9.1.9.1.1.

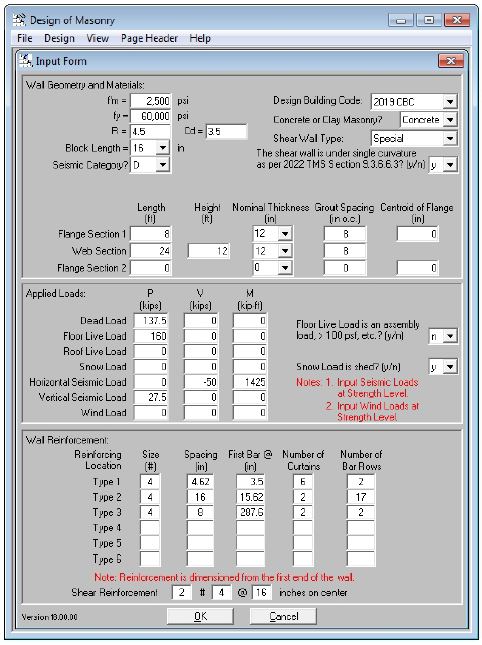


Example Shear Wall - Elevation and Loading

Figure 8

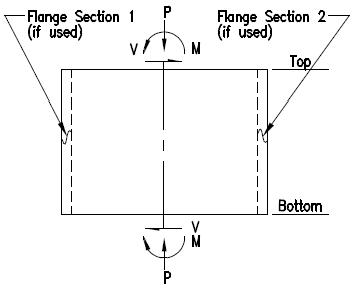
Input Screen

Whenever the user starts the **Input** option, the following screen will appear.



The required input is described in Chapter 4. The input for most of the Wall Geometry and Materials and the inputs for the Applied Loads are consistent with the standard input described in that chapter.

The input loads must follow the sign convention used for the program. See Figure 9, page 65 for the assumed positive sign convention for the input loads. See Figure 8 page 64 for the example problem loading.



Positive Sign Convention

Figure 9

These loads are combined into the various load combinations required by the design code as shown in Chapter 5.

The R value used to calculate the seismic loads on the wall and type of shear wall are used in determining the maximum reinforcement percentage, ρ_{max}, equal to A_s/bd_v, for the wall. The maximum reinforcement percentage when $M_u/V_u d_v \geq 1.0$ is calculated per the following sections of the masonry codes 2008/2011 MSJC Section 3.3.3.5, 2013 MSJC Section 9.3.3.5 or 2016 TMS Section 9.3.3.2 as ρ_{max} =

$$\frac{\left[0.80f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)-\frac{P_u}{A_e}\right]}{\left[1.25f_y\left(1-\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\left(\frac{2C-1}{2C}\right)-\min\left\{\left(\frac{1}{2}\right)\varepsilon_{mu},\varepsilon_y\left(\frac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)+f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\right]}$$

for rectangular shaped walls or flanged walls with the compression entirely within the flange. For walls where $0.85d_v\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+\varepsilon_y}\right) > t_{flange}$, then $\rho_{\max}$ =

$$\frac{\left[0.80f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)+f'_m 0.80t_{flange}\left(\frac{b-b_w}{b_w d_v}\right)-\frac{P_u}{b_w d_v}\right]}{\left[1.25f_y\left(1-\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\left(\frac{2C-1}{2C}\right)-\min\left\{\left(\frac{1}{2}\right)\varepsilon_{mu},\varepsilon_y\left(\frac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)+f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\right]}.$$

In these equations, the '*C*' factor for the steel strain is equal to 4.0 for *Special Reinforced Masonry Shear Walls*, 2.0 for *Intermediate Reinforced Masonry Shear Walls* or 1.5 for all other shear walls. The maximum reinforcement percentage when $M_u/V_u d_v < 1.0$ and '*R*' is greater than 1.5 is calculated as $\rho_{\max}$ =

$$\frac{\left[0.80f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)-\frac{P_u}{A_e}\right]}{\left[f_y\left(1-\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)\left(\frac{2\times1.5-1}{2\times1.5}\right)-\min\left\{\left(\frac{1}{2}\right)\varepsilon_{mu},\varepsilon_y\left(\frac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)+f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)\right]}$$

for rectangular shaped walls or flanged walls with the compression entirely within the flange. For walls where $0.85d_v\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+\varepsilon_y}\right) > t_{flange}$, then $\rho_{\max}$ =

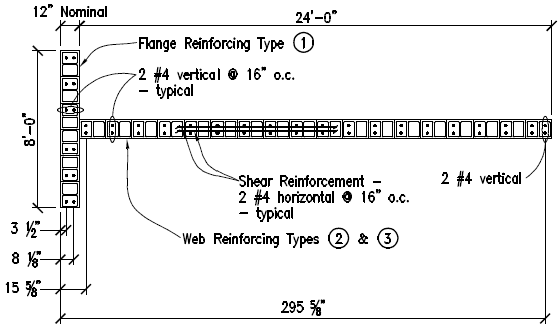
$$\frac{\left[0.80f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)+f'_m 0.80t_{flange}\left(\frac{b-b_w}{b_w d_v}\right)-\frac{P_u}{b_w d_v}\right]}{\left[f_y\left(1-\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)\left(\frac{2\times1.5-1}{2\times1.5}\right)-\min\left\{\left(\frac{1}{2}\right)\varepsilon_{mu},\varepsilon_y\left(\frac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)+f'_m 0.80\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+1.5\varepsilon_y}\right)\right]}.$$

There are no maximum reinforcement requirements when $M_u/V_u d_v < 1.0$ and '*R*' is less than 1.5. Unless specifically instructed by the user, the program will not check any design with a reinforcement ratio greater than the permitted value.

There is an input for the C_d value and there is a selection for indicating whether the wall exhibits single curvature behavior as described in 2008/2011 MSJC Section 3.3.6.5.3, 2013 MSJC Section 9.3.6.5.3 or 2016 TMS Section 9.3.6.6.3. These inputs are used to determine whether boundary element reinforcing is required per 2008/2011 MSJC Sections 3.3.6.5.1 through 3.3.6.5.5 or 2013 MSJC Sections 9.3.6.5.1 through 9.3.6.5.5. Based on the input reinforcing, the program determines whether the input wall meets the requirements for maximum reinforcing as defined in 2008/2011 MSJC Section 3.3.3.5, 2013 MSJC Section 9.3.3.5 or 2016 TMS Section 9.3.3.2. If the maximum reinforcing requirements are exceeded, the user is given the choice of designing with boundary elements or revising the input reinforcing to meet the maximum reinforcing requirements. In addition, if the wall is bending in single curvature, the program may also request input on the wall height for determining deflections and maximum

deflection at that height if the program determines that boundary reinforcing may be required. This input will not be requested if boundary members are not required.

When using a plan shape other than a simple rectangular wall, the input for the location of the reinforcing becomes critical. When placing reinforcing in the flange section, the user must determine the number of bars along the length of the flange (perpendicular to the web) at a given distance from the face of the flange. This number becomes the number of "curtains" of steel for that reinforcing type in the input. The spacing of bars in the flange is the distance between the actual curtains of steel in the flange. See Figure 10, page 67.



Example Shear Wall - Plan and Reinforcing

Figure 10

Error Flags and Notes

The following 30 explanations list the more important error messages which may appear when the input is checked by clicking the "OK" button or selecting "Run" from the pull down menu. If an error occurs, the error message will appear in a pop up window.

Note Number	Text of note
E3101	Grout spacing cannot be closer than the cell spacing
E3102	Grout spacing must be in even multiples of the cell spacing
E3103	This program only supports solid grouted wall flanges/end pilasters, which are greater than 16 inches nominal thickness.
E3105	Grout spacing in ordinary reinforced masonry shear walls cannot exceed the maximum vertical steel spacing which cannot exceed 10 feet per 2008 MSJC Section 1.17.3.2.4, 2011 MSJC Section 1.18.3.2.4 or 2013 MSJC/2016 TMS Section 7.3.2.4.
E3106	Grout spacing in intermediate reinforced masonry shear walls cannot exceed the maximum vertical steel spacing which cannot exceed 48 inches per 2008 MSJC Section 1.17.3.2.5, 2011 MSJC Section 1.18.3.2.5 or 2013 MSJC/2016 TMS Section 7.3.2.5.
E3107	Grout spacing in special reinforced masonry shear walls cannot exceed the maximum vertical steel spacing which cannot exceed 48 inches per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.
E3108	Grout spacing in special reinforced masonry shear walls cannot exceed the maximum vertical steel spacing which cannot exceed one third the length of the wall per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.
E3109	Grout spacing in special reinforced masonry shear walls cannot exceed the maximum vertical steel spacing which cannot exceed one third the height of the wall per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.
E4101	Only bar sizes #3 through #11 are permitted in this program. Maximum reinforcing steel size is #9 per 2008/2011 MSJC Section 3.3.3.1 or 2013 MSJC/2016 TMS Section 9.3.3.1.
E4102	Reinforcing steel spacing cannot be closer than the cell spacing.

E4103 Reinforcing steel spacing must be in even multiples of the grout spacing.

E4104 The input reinforcing overlaps. At least one input steel start location plus the reinforcing steel spacing times the number of bar rows is greater than another input steel start location.

E4106 Vertical steel spacing in ordinary reinforced masonry shear walls cannot exceed 10 feet per 2008 MSJC Section 1.17.3.2.4, 2011 MSJC Section 1.18.3.2.4 or 2013 MSJC/2016 TMS Section 7.3.2.4.

E4107 Vertical steel spacing in intermediate reinforced masonry shear walls cannot exceed 48 inches per 2008 MSJC Section 1.17.3.2.5, 2011 MSJC Section 1.18.3.2.5 or 2013 MSJC/2016 TMS Section 7.3.2.5.

E4108 Vertical steel spacing in special reinforced masonry shear walls cannot exceed 48 inches per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.

E4109 Vertical steel spacing in special reinforced masonry shear walls cannot exceed one third the length of the wall per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.

E4110 Vertical steel spacing in special reinforced masonry shear walls cannot exceed one third the height of the wall per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.

E4112 The amount of vertical steel must be at least one third of the horizontal steel per 2008/2011 MSJC Section 3.3.6.2 or 2013 MSJC/2016 TMS Section 9.3.6.2.

E4113 Vertical reinforcing steel spacing cannot exceed 10 feet in Seismic Design Category C per 2008 MSJC Section 1.17.4.3, 2011 MSJC Section 1.18.4.3 or 2013 MSJC/2016 TMS Section 7.4.3.

E4114 Horizontal reinforcing steel spacing cannot exceed 10 feet in Seismic Design Category C per 2008 MSJC Section 1.17.4.3, 2011 MSJC Section 1.18.4.3 or 2013 MSJC/2016 TMS Section 7.4.3.

E4115 Vertical reinforcing steel spacing cannot exceed 48 inches in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4116 Vertical steel spacing cannot exceed one third the length of the wall in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4117 Vertical steel spacing cannot exceed one third the length of the wall in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4118 Horizontal reinforcing steel spacing cannot exceed 48 inches in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4119 Horizontal steel spacing cannot exceed one third the length of the wall in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4120 Horizontal steel spacing cannot exceed one third the height of the wall in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.4.4.2 referencing Section 1.17.3.2.6, 2011 MSJC Section 1.18.4.4.2 referencing Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.4.4.2 referencing Section 7.3.2.6.

E4121 The sum of the areas of vertical and horizontal steel must be at least 0.002 times the gross cross-sectional area of the wall and the minimum area of reinforcing in either direction must be at least 0.0007 times the gross cross-sectional area of the wall in Seismic Design Categories D, E and F per 2008 MSJC Section 1.17.3.2.6, 2011 MSJC Section 1.18.3.2.6 or 2013 MSJC/2016 TMS Section 7.3.2.6.

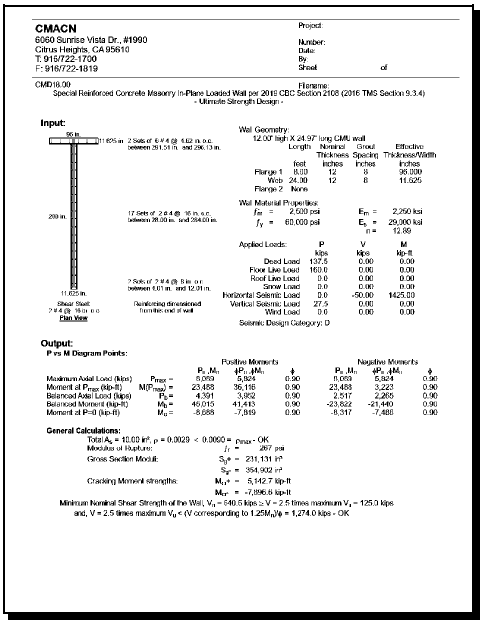
E5101 Masonry strength is not per 2008/2011 MSJC Section 3.1.8.1.1 or 2013 MSJC/2016 TMS Section 9.1.9.1.1. The masonry strength is less than the minimum allowable strength.

E5102 Masonry strength is not per 2008/2011 MSJC 3.1.8.1.1 or 2013 MSJC/2016 TMS Section 9.1.9.1.1. The masonry strength is greater than the maximum allowable strength.

E5104 Steel yield is not per 2008/2011 MSJC Section 3.1.8.3 or 2013 MSJC/2016 TMS Section 9.1.9.3.

Printed Output

When the user chooses the print option in the main menu the following information will be printed. The program also has an option for printing a summary of the design. This summary prints out the five (5) worst load combinations for each of the tables described below. Pages 72 through 73 show the complete printed output. A description of the program calculations and output follows on page 74. Note: the output shown below is hypothetical.



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CMD16.00 Filename:
Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 CBC Section 2108 (2016 TMS Section 9.3.4)
- Ultimate Strength Design -

Input:

Wall Geometry:
12.00' high X 24.97' long CMU wall

	Length feet	Nominal Thickness inches	Grout Spacing inches	Effective Thickness/Width inches
Flange 1	0.00	12	8	39.000
Web	24.00	12	8	11.625
Flange 2	None			

Wall Material Properties:
f'_m = 2,500 psi E_m = 2,250 ksi
f_y = 60,000 psi E_s = 29,000 ksi
n = 12.89

Applied Loads:

	P kips	V kips	M kip-ft
Dead Load	137.5	0.00	0.00
Floor Live Load	160.0	0.00	0.00
Roof Live Load	0.0	0.00	0.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	0.0	-50.00	1425.00
Vertical Seismic Load	27.5	0.00	0.00
Wind Load	0.0	0.00	0.00

Seismic Design Category: D

Output:

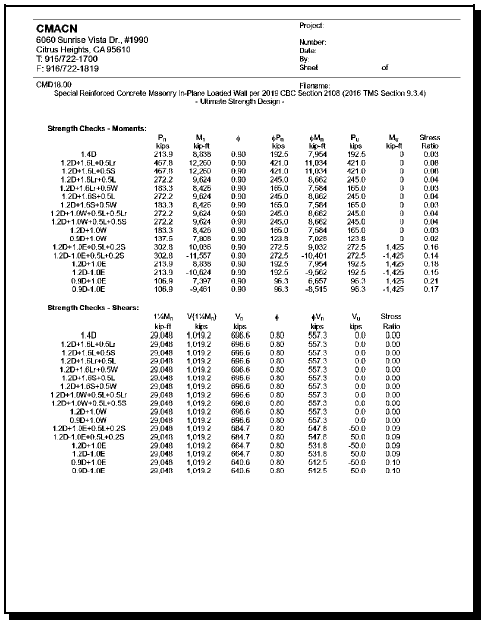
P vs M Diagram Points:

		Positive Moments P_n, M_n	ϕP_n, ϕM_n	ϕ	Negative Moments P_n, M_n	ϕP_n, ϕM_n	ϕ
Maximum Axial Load (kips)	P_{max} =	5,059	5,824	0.90	5,059	5,824	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	23,468	39,116	0.90	23,468	3,223	0.90
Balanced Axial Load (kips)	P_b =	4,191	3,952	0.90	2,517	2,265	0.90
Balanced Moment (kip-ft)	M_b =	46,015	41,413	0.90	-23,822	-21,440	0.90
Moment at P=0 (kip-ft)	M_o =	-8,588	-7,729	0.90	-8,317	-7,486	0.90

General Calculations:

Total A_s = 10.00 in², ρ = 0.0029 < 0.0060 = ρ_{max} - OK
Modulus of Rupture: f_r = 267 psi
Gross Section Moduli: S_g = 231,131 in³
S_g = 384,902 in³
Cracking Moment strengths: M_{cr} = 5,142.7 kip-ft
M_{cr} = -7,896.8 kip-ft

Minimum Nominal Shear Strength of the Wall, V_n = 640.5 kips ≥ V = 2.5 times maximum V_u = 125.0 kips
and, V = 2.5 times maximum V_u < (V corresponding to 1.25M_n/e = 1,274.0 kips - OK



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Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 CBC Section 2108 (2016 TMS Section 9.3.4)
- Ultimate Strength Design -

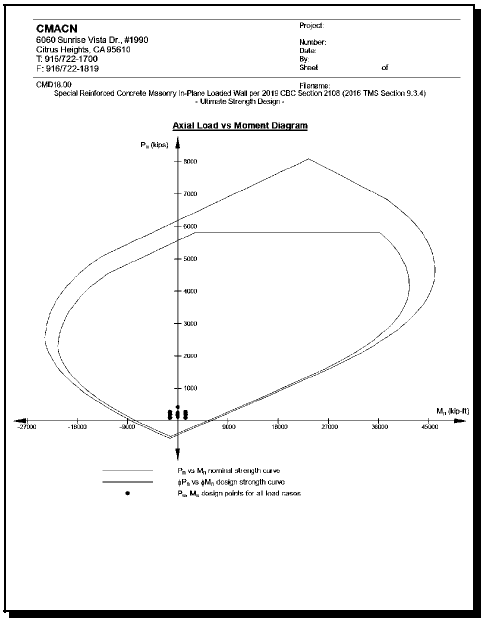
Strength Checks - Moments:

	P_u kips	M_u kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	213.9	8,838	0.90	192.5	7,954	0	0	0.00
1.2D+1.6L+0.5Lr	467.8	12,260	0.90	421.0	11,034	421.0	0	0.00
1.2D+1.6L+0.5S	467.8	12,260	0.90	421.0	11,034	421.0	0	0.00
1.2D+1.6Lr+0.5L	272.2	9,624	0.90	245.0	8,662	245.0	0	0.04
1.2D+1.6L+0.5W	183.3	8,425	0.90	165.0	7,584	165.0	0	0.00
1.2D+1.6S+0.5L	272.2	9,624	0.90	245.0	8,662	245.0	0	0.04
1.2D+1.6S+0.5W	183.3	8,425	0.90	165.0	7,584	165.0	0	0.00
1.2D+1.0W+0.5L+0.5Lr	272.2	9,624	0.90	245.0	8,662	245.0	0	0.04
1.2D+1.0W+0.5L+0.5S	272.2	9,624	0.90	245.0	8,662	245.0	0	0.04
1.2D+1.0W	183.3	8,425	0.90	165.0	7,584	165.0	0	0.00
0.9D+1.0W	137.5	7,908	0.90	123.8	7,028	123.8	0	0.00
1.2D+1.0E+0.5L+0.2S	302.8	10,935	0.90	272.5	9,932	272.5	1,425	0.16
1.2D+1.0E+0.5L+0.2S	302.8	-11,557	0.90	272.5	-10,401	272.5	-1,425	0.14
1.2D+1.0E	213.9	8,838	0.90	192.5	7,954	192.5	1,425	0.18
1.2D-1.0E	213.9	-10,624	0.90	192.5	-9,562	192.5	-1,425	0.15
0.9D+1.0E	106.9	7,397	0.90	96.3	6,657	96.3	1,425	0.21
0.9D-1.0E	106.9	-9,461	0.90	96.3	-8,515	96.3	-1,425	0.17

Strength Checks - Shears:

	1/4M_n kip-ft	V(1/4M_n) kips	V_s kips	ϕ	ϕV_n kips	V_u kips	Stress Ratio
1.4D	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6L+0.5Lr	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6L+0.5S	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6Lr+0.5L	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6L+0.5W	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6S+0.5L	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.6S+0.5W	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.0W+0.5L+0.5Lr	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.0W+0.5L+0.5S	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.0W	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
0.9D+1.0W	29,648	1,019.2	696.6	0.80	557.3	0.0	0.00
1.2D+1.0E+0.5L+0.2S	29,648	1,019.2	684.7	0.80	547.8	50.0	0.09
1.2D-1.0E+0.5L+0.2S	29,648	1,019.2	684.7	0.80	547.8	50.0	0.09
1.2D+1.0E	29,648	1,019.2	664.7	0.80	531.8	-50.0	0.09
1.2D-1.0E	29,648	1,019.2	664.7	0.80	531.8	50.0	0.09
0.9D+1.0E	29,648	1,019.2	640.6	0.80	512.5	-50.0	0.10
0.9D-1.0E	29,648	1,019.2	640.6	0.80	512.5	50.0	0.10

Printed Output (continued)



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Filename:

Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2019 CBC Section 2108 (2016 TMS Section 9.3.4)
- Ultimate Strength Design -

Axial Load vs Moment Diagram

P_n (kips)

5000
4000
3000
2000
1000

-27000 -18000 -9000 9000 18000 27000 36000 45000

M_n (kip-ft)

P_n vs M_n nominal strength curve
ϕP_n vs ϕM_n design strength curve
P_{nx}, M_{nx} design points for all load cases

Calculations and Output

The first section of the output is a listing of all of the data input by the user. A sketch of the plan geometry of the wall is printed, including the input reinforcing.

The actual calculated output starts with a table containing points for the axial load versus moment diagrams for the positive and negative moments. The program calculates the nominal and reduced nominal strength, φ times the nominal strength, curves for both positive and negative moments. The calculations determine forty points on the curves. The printed output contains a simplified version of these curves for verification of the design. The output prints the three main points on each of these curves. The three points are the maximum axial load capacity, P_o, when the entire wall is under compression, the moment capacity when axial load is zero, M_o, and the balanced axial load and positive moment capacity, P_b and M_b, when both the masonry and the steel are stressed to their respective yield points. The moment corresponding to the maximum axial load, $M(P_o)$ is zero if the wall shape and the reinforcing are symmetrical about the centerline of the wall. If the wall reinforcing or shape is not symmetrical then the moment at the maximum axial load will be off of the axial load axis. See Figure 11 page 74.

These three points will allow the designer to plot an approximate graph of the ultimate and nominal curves for the wall. This graph can be used to determine the adequacy of the wall for load combinations not input and for a double check of the output.

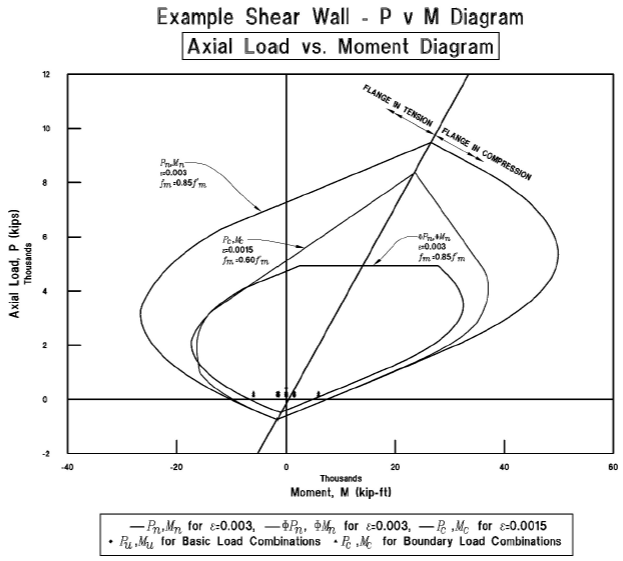


Figure 11

The section following the graph points contains some general design calculations. The first line shows the total area of steel in the shear wall and the calculated reinforcing steel ratio, ρ. The calculated steel reinforcing ratio is compared with the maximum reinforcement ratio. The maximum reinforcement percentage when $M_u/V_u d_v \geq 1.0$ is calculated per 2008/2011 MSJC Section 3.3.3.5, 2013 MSJC Section 9.3.3.5 or 2016 TMS Section 9.3.3.2 as $\rho_{\max} = \dfrac{\left[0.80 f'_m 0.80\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right) - \dfrac{P_u}{A_e}\right]}{\left[f_y\left(1-\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\left(\dfrac{2C-1}{2C}\right) - \min\left\{\left(\dfrac{1}{2}\right)\varepsilon_{mu}, \varepsilon_y\left(\dfrac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right) + f'_m 0.80\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\right]}$ for rectangular shaped walls or flanged walls with the compression entirely within the flange. For walls where $0.85 d_v\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+\varepsilon_y}\right) > t_{flange}$, then $\rho_{\max} = \dfrac{\left[0.80 f'_m 0.80\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right) + f'_m 0.80 t_{flange}\left(\dfrac{b-b_w}{b_w d_v}\right) - \dfrac{P_u}{b_w d_v}\right]}{\left[f_y\left(1-\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\left(\dfrac{2C-1}{2C}\right) - \min\left\{\left(\dfrac{1}{2}\right)\varepsilon_{mu}, \varepsilon_y\left(\dfrac{2\varepsilon_{mu}-\varepsilon_y}{2\varepsilon_{mu}}\right)\right\}E_s\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right) + f'_m 0.80\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+C\varepsilon_y}\right)\right]}$.

In these equations, when $M_u/V_u d_v \geq 1.0$, the '*C*' factor for the steel strain is equal to 1.5 for *Ordinary Reinforced Masonry Shear Walls*, 3.0 for *Intermediate Reinforced Masonry Shear Walls* and 4.0 for *Special Reinforced Masonry Shear Walls*. When $M_u/V_u d_v < 1.0$ and '*R*' is greater than 1.5, then the 'C' factor is equal to 1.5. When $M_u/V_u d_v < 1.0$ and '*R*' is less than 1.5, then there are no maximum reinforcing steel percentage requirements. The next line is the modulus of rupture of the masonry, f_r = 200 psi for fully grouted walls and 125 psi for partially grouted walls per 2008/2011 MSJC Section 3.1.8.2 or 267 psi for fully grouted walls and 167 psi for partially grouted walls per 2013 MSJC/2016 TMS Section 9.1.9.2. The third and fourth lines are the gross sectional modulus of the wall for both positive and negative moments, $S_g = I_g/c$, where I_g is the gross moment of inertia about the neutral axis of the horizontal wall area and c is the distance from the tension end of the wall to the neutral axis. The next two lines are the cracking moment strength of the wall for both positive and negative moments, $M_{cr} = S_g f_r$.

Following the previous section, the output prints some notes regarding the design of the shear wall. The nominal shear strength of *Special Reinforced Masonry Shear walls* must be greater than 1.25 times the shear corresponding to the nominal flexural strength. The nominal shear strength, however, does not need to exceed 2.5 times the maximum ultimate shear. See 2008 MSJC Section 1.17.3.2.6.1.1, 2008 MSJC Section 1.18.3.2.6.1.1 or 2013 MSJC/2016 TMS Section 7.3.2.6.1.1.

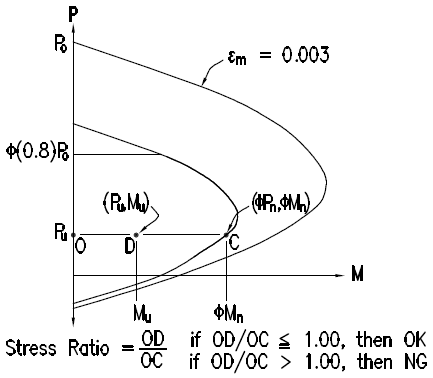
The remainder of the output of the program is contained in two or three tables where each row of the table is for one of the load combinations listed in chapter 5 for the selected design code. The output may contain two or three tables, depending on whether the design is in accordance with the boundary element requirements of 2008/2011 MSJC Sections 3.3.6.5.1 through 3.3.6.5.5, 2008/2011 MSJC Section 3.3.6.5, 2013 MSJC Sections 9.3.6.5.1 through 9.3.6.5.5, 2013 MSJC Section 9.3.6.5, 2016 TMS Sections 9.3.6.6.1 through 9.3.6.6.5 or 2013 MSJC Section 9.3.6.6. The program shows which of these load combinations causes the greatest demand on the wall.

The first of the main load combination tables is the axial load and moment strength check table. The first two columns are the nominal axial load and positive moment strength for the wall given that the maximum compressive stress in the masonry is equal to f'_m and the maximum strain in the masonry, ε_{mu}, is equal to 0.0025 for concrete masonry and 0.0035 for clay masonry. The Whitney stress block is used for calculating the nominal axial strength versus moment strength of the wall when the maximum masonry strain is equal to ε_{mu}, see ? page ?. The nominal strength for each load case is determined by first dividing the factored axial load, P_u, for that load combination by the φ factor corresponding to that factored axial load, this is the nominal axial strength of the wall for the given load case, P_n. The moment strength corresponding to this nominal axial load strength is determined from the calculated nominal axial load versus moment curve for the input wall and reinforcing.

The third column of this table is the φ factor associated with the nominal strengths shown in the previous two columns. For shear walls, the φ factor for axial load and moment is always 0.90. This φ factor times the nominal axial load and moment in the previous two columns is the reduced nominal axial load and moment shown in the next two columns.

The reduced nominal axial loads and moments are the nominal axial loads and moments, corresponding to each of the ultimate load combinations, reduced by the strength reduction, φ, factor. The program limits the maximum reduced axial load to 80% of the pure reduced axial load, φP_o, as per 2008 MSJC Section 3.3.4.1.1, equations 3-17 and 3-18, 2011 MSJC Section 3.3.4.1.1, equations 3-18 and 3-19, 2013 MSJC Section 9.3.4.1.1, equations 9-19 and 9-20 or 2016 TMS Section 9.3.4.1.1, equations 9-15 and 9-16. The program checks each ultimate load case against the reduced nominal strength of the wall and gives a demand capacity ratio in the last column. This demand capacity ratio is the moment for the ultimate loads divided by the moment for the reduced nominal strength, see Figure 12 page 77. This ratio is calculated for each load combination.

The next two columns of the table are the calculated ultimate (factored) axial loads and moments for each of the design load combinations.



Shear Wall – Demand Capacity Ratio

Figure 12

The second table is the nominal strength table for the boundary member check for the design load combinations. This table will not be printed for designs in accordance with the 2008/2011/2013 MSJC that do not exceed the maximum reinforcing requirements of 2008/2011 MSJC Section 3.3.3.5, 2013 MSJC Section 9.3.3.5 or 2013 TMS Section 9.3.3.2.

This table first has a section listing the design limitations for the wall as per the 2008/2011 MSJC Sections 3.3.6.5.1 and 3.3.6.5.3, 2013 MSJC Sections 9.3.6.5.1 and 9.3.6.5.3, 2016 TMS Sections 9.3.6.6.1 and 9.3.6.6.3 for walls in single curvature or 2008/2011 MSJC Sections 3.3.6.5.1 and 3.3.6.5.4, 2013 MSJC Sections 9.3.6.5.1 and 9.3.6.5.4, 2016 TMS Sections 9.3.6.6.1 and 9.3.6.6.4 for other walls. For walls in single curvature with the input reinforcing exceeding the maximum permitted per 2008/2011 MSJC Section 3.3.3.5/2013 MSJC Section 9.3.3.5/2016 TMS Section 9.3.3.2, the program will request information to determine the deflection of the wall. The user may either input a deflection or the program will calculate the deflection based on the formula, $\Delta = \left(V_u h^3 / 3E_m I_g\right) + \left(1.2V_u h / A_g G_m\right)$, for solid cantilever walls. The height used to calculate the deflection and the '*c*' distance limitation for single curvature wall is either input by the user or assumed to be the braced height of the wall input on the input screen. For designs where the input reinforcing exceeds the maximum permitted per 2008/2011 MSJC Section 3.3.3.5/2013 MSJC Section 9.3.3.5/2016 TMS Section 9.3.3.2, but are not in single curvature, this part of the table lists the maximum permitted stress in the masonry due to bending plus axial load per 2008/2011 MSJC Section 3.3.6.5.4, 2013 MSJC Section 9.3.6.5.4 or 2016 TMS Section 9.3.6.6.4. This part of the table is followed by a typical listing for each of the design load combinations.

The first three columns are the ultimate axial, moment and shear forces on the wall for each of the design load combinations. The next column is the ultimate $M_u/V_u l_w$ ratio for each of the load combinations. The next column is different depending on the

characteristic of the shear wall. If the shear walls is in single curvature as per 2008/2011 MSJC Section 3.3.6.5.3, 2013 MSJC Section 9.3.6.5.3 or 2016 TMS Section 9.3.6.6.3, then the column shows the distance to the neutral axis, '*c*', for each load combination. This '*c*' distance is based on the nominal moment, M_n, corresponding to the ultimate axial load, P_u, on the wall. If the shear wall is not in single curvature, the table prints the maximum stress corresponding to $P_u/A_g + M_u/S_g$. The last column indicates whether boundary members are required for a given load combination. (2008/2011 MSJC Sections 3.3.6.5.1 through 3.3.6.5.5, 2013 MSJC Sections 9.3.6.5.1 through 9.3.6.5.5 or 2016 TMS Sections 9.3.6.6.1 through 9.3.6.6.5).

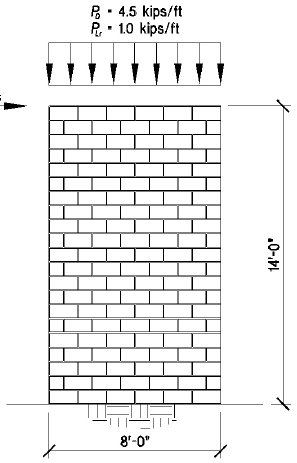
The last table is a table of the shear loads and demand capacity ratios for the design load combinations. The first column is the nominal moment at the ultimate axial load for that load combination. The second column is the ultimate shear demand corresponding to the nominal moment shown in the first column. This is the shear strength corresponding to the development of the nominal flexural strength of the wall. This provision is only applicable to *Special Reinforced Masonry Shear Walls*, see 2008 MSJC Section 1.17.3.2.6.1.1, 2001 MSJC Section 1.18.3.2.6.1.1 or 2013 MSJC/2016 TMS Section 7.3.2.6.1.1. The third column is the calculated nominal shear strength of the wall. The fourth column is the phi, φ, factor for the nominal strength. For shear walls, the phi factor is always 0.80. The next column is phi times the nominal shear strength of the wall. The sixth column is the ultimate shear loads for each of the design load combinations. The last column is the demand capacity ratio of ultimate shear load to reduced shear capacity.

Example Problem

Following is an example problem including the output from the template. The example problem is taken from the CMACN book 2006 Design of Reinforced Masonry Structures, by Gregg E. Brandow, Chukwuma G. Ekwueme and Gary C. Hart, copyright 2007. This problem is a strength design problem (Example 5.4.1).

Problem 7.1

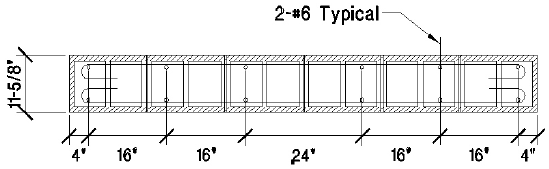
Design of a reinforced concrete masonry shear wall subjected to axial load and bending moment. Strength design procedure. See also Figure 14 page 79.



Problem 7.1
Figure 13

Given: Wall Length = 8'-0"
12" Nominal CMU, solid grouted
f'_m = 2,500 psi
f_y = 60,000 psi
P_w = 124 psf
P_D = 4.5 klf ×8' + 0.124 ksf ×8'×14'
= 49.88 kips
P_{Lr} = 8.00 kips = 1.0 klf × 8'
V_E = 63 kips
M_E = 882 kip-ft = 63 kips × 14'
Wall Reinforcing - see Figure 14, page 79
Vertical reinforcing: 3 sets of 2 #6 @ 16" o.c. starting at 4" from each end of wall (6 sets total).
Horizontal reinforcing: 2 #4@ 16"o.c.

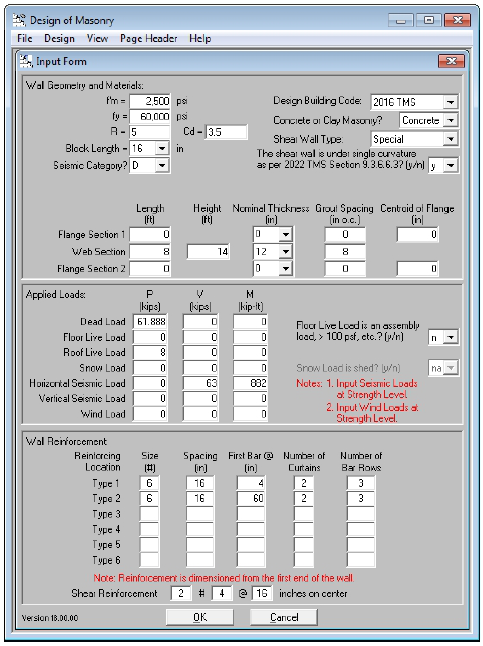
The input is shown on page 80 and the output is shown on page 81.



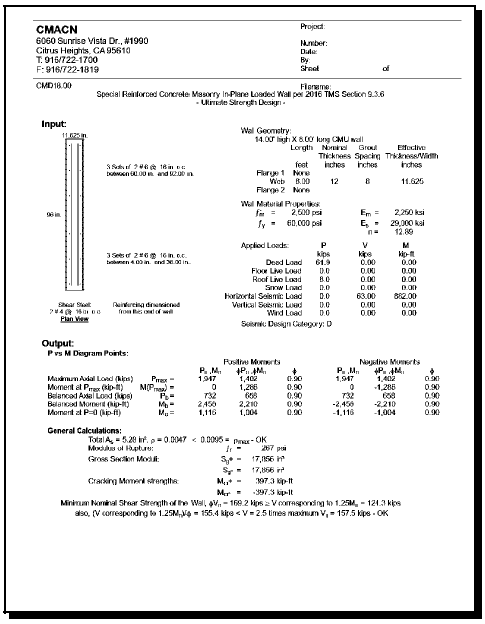
Location of Vertical Reinforcing Steel

Problem 7.1
Figure 14

Example Problem Input



Example Problem Output



CMACN
6060 Sunrise Vista Dr., #1990
Citrus Heights, CA 95610
T: 916/722-1700
F: 916/722-1819

Project:
Number:
Date:
By:
Sheet of

CMD18.00
Filename:
Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 TMS Section 9.3.6
- Ultimate Strength Design -

Input:

1.625 in.
96 in.
3 Sets of 2 #6 @ 16 in. o.c. between 80.00 in. and 82.00 in.
3 Sets of 2 #6 @ 16 in. o.c. between 4.00 in. and 36.00 in.
Shear Shell 2 #4 @ 16 in. o.c.
Reinforcing dimensioned from the end of wall
Plan View

Wall Geometry:
14.00' high X 8.00' long CMU wall

	Length feet	Nominal Thickness inches	Grout Spacing inches	Effective Thickness/Width inches
Flange 1	None			
Web	8.00	12	8	11.625
Flange 2	None			

Wall Material Properties:
f'_m = 2,500 psi
f_y = 60,000 psi
E_m = 2,250 ksi
E_s = 29,000 ksi
n = 12.89

Applied Loads:

	P kips	V kips	M kip-ft
Dead Load	64.9	0.00	0.00
Floor Live Load	0.0	0.00	0.00
Roof Live Load	0.0	0.00	0.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	0.0	63.00	882.00
Vertical Seismic Load	0.0	0.00	0.00
Wind Load	0.0	0.00	0.00

Seismic Design Category: D

Output:

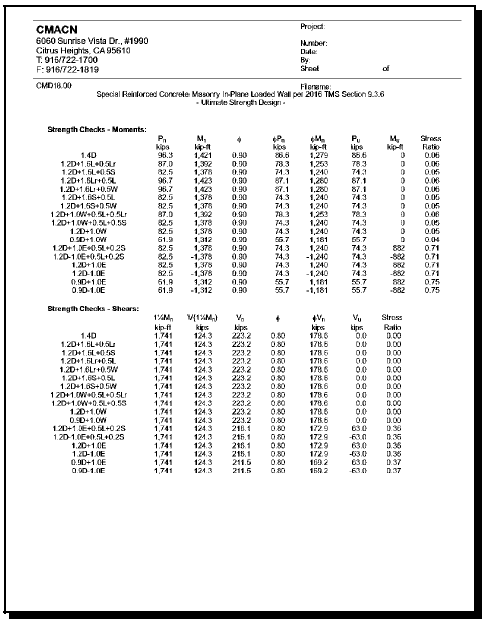
P vs M Diagram Points:

		Positive Moments P_n, M_n	ϕP_n, ϕM_n	ϕ	Negative Moments P_n, M_n	ϕP_n, ϕM_n	ϕ
Maximum Axial Load (kips)	P_{max} =	1,947	1,402	0.90	1,947	1,402	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	0	1,266	0.90	0	-1,266	0.90
Balanced Axial Load (kips)	P_b =	732	659	0.90	732	659	0.90
Balanced Moment (kip-ft)	M_b =	2,456	2,210	0.90	-2,456	-2,210	0.90
Moment at P=0 (kip-ft)	M_0 =	1,116	1,004	0.90	-1,116	-1,004	0.90

General Calculations:

Total A_s = 5.28 in², ρ = 0.0047 < 0.0095 = ρ_{max} - OK
Modulus of Rupture: f_r = 267 psi
Gross Section Modult: S_g = 17,856 in³
S_{gr} = 17,856 in³
Cracking Moment strengths: M_{cr} = 397.3 kip-ft
M_{cr} = -397.3 kip-ft

Minimum Nominal Shear Strength of the Wall, ϕV_n = 168.2 kips ≥ V corresponding to 1.25M_n = 124.3 kips
also, (V corresponding to 1.25M_n)/ϕ = 155.4 kips < V = 2.5 times maximum V_u = 157.5 kips - OK



CMACN
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Special Reinforced Concrete Masonry In-Plane Loaded Wall per 2016 TMS Section 9.3.6
- Ultimate Strength Design -

Strength Checks - Moments:

	P_u kips	M_u kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	90.9	1,421	0.90	86.6	1,279	86.6	0	0.06
1.2D+1.6L+0.5Lr	87.9	1,392	0.90	78.3	1,253	78.3	0	0.06
1.2D+1.6L+0.5S	82.5	1,378	0.90	74.3	1,240	74.3	0	0.06
1.2D+1.6Lr+0.5L	96.7	1,423	0.90	87.1	1,280	87.1	0	0.06
1.2D+1.6Lr+0.5W	96.7	1,423	0.90	87.1	1,280	87.1	0	0.06
1.2D+1.6S+0.5L	82.5	1,378	0.90	74.3	1,240	74.3	0	0.06
1.2D+1.6S+0.5W	82.5	1,378	0.90	74.3	1,240	74.3	0	0.05
1.2D+1.0W+0.5L+0.5Lr	87.9	1,392	0.90	78.3	1,253	78.3	0	0.06
1.2D+1.0W+0.5L+0.5S	82.5	1,378	0.90	74.3	1,240	74.3	0	0.05
1.2D+1.0W	82.5	1,378	0.90	74.3	1,240	74.3	0	0.05
0.9D+1.0W	61.9	1,312	0.90	55.7	1,181	55.7	0	0.04
1.2D+1.0E+0.5L+0.2S	82.5	1,378	0.90	74.3	1,240	74.3	882	0.71
1.2D+1.0E+0.5L+0.2S	82.5	-1,378	0.90	74.3	-1,240	74.3	-882	0.71
1.2D+1.0E	82.5	1,378	0.90	74.3	1,240	74.3	882	0.71
1.2D-1.0E	82.5	-1,378	0.90	74.3	-1,240	74.3	-882	0.71
0.9D+1.0E	61.9	1,312	0.90	55.7	1,181	55.7	882	0.75
0.9D-1.0E	61.9	-1,312	0.90	55.7	-1,181	55.7	-882	0.75

Strength Checks - Shears:

	1/4M_n kip-ft	V_t(1/4M_n) kips	V_n kips	ϕ	ϕV_n kips	V_u kips	Stress Ratio
1.4D	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6L+0.5Lr	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6L+0.5S	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6Lr+0.5L	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6Lr+0.5W	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6S+0.5L	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.6S+0.5W	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.0W+0.5L+0.5Lr	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.0W+0.5L+0.5S	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.0W	1,741	124.3	223.2	0.80	178.6	0.0	0.00
0.9D+1.0W	1,741	124.3	223.2	0.80	178.6	0.0	0.00
1.2D+1.0E+0.5L+0.2S	1,741	124.3	215.1	0.80	172.0	63.0	0.36
1.2D+1.0E+0.5L+0.2S	1,741	124.3	215.1	0.80	172.0	-63.0	0.36
1.2D+1.0E	1,741	124.3	215.1	0.80	172.0	63.0	0.36
1.2D-1.0E	1,741	124.3	215.1	0.80	172.0	-63.0	0.36
0.9D+1.0E	1,741	124.3	211.5	0.80	169.2	63.0	0.37
0.9D-1.0E	1,741	124.3	211.5	0.80	169.2	-63.0	0.37