IBC 2000/MSJC 2002
TRIAL DESIGN – STRENGTH

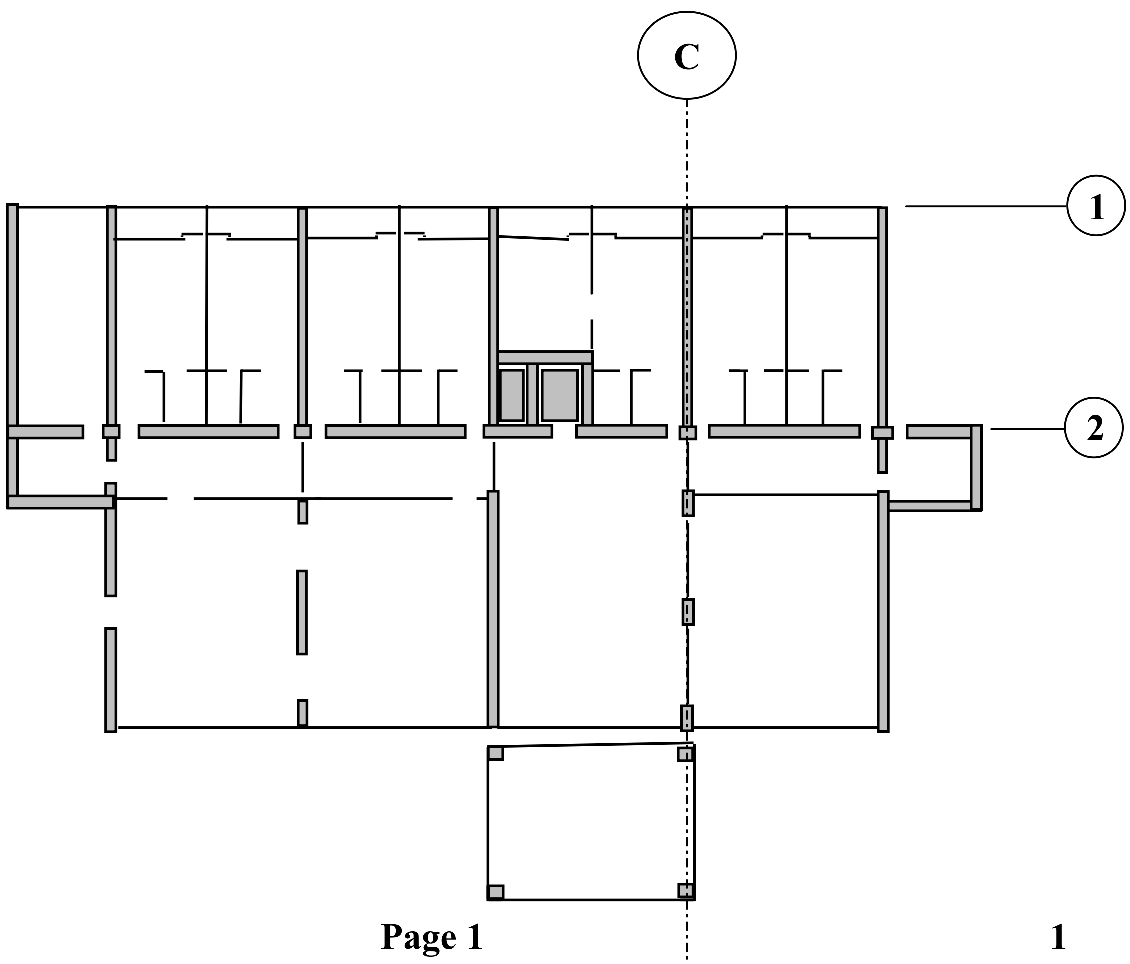
This problem was taken from the Masonry Designers Guide RJC Hotel. It is the wall on grid line C between 3 and 4. The RJC hotel is 4 stories. Two floors were added and a new lateral force analysis conducted to determine appropriate lateral loads.

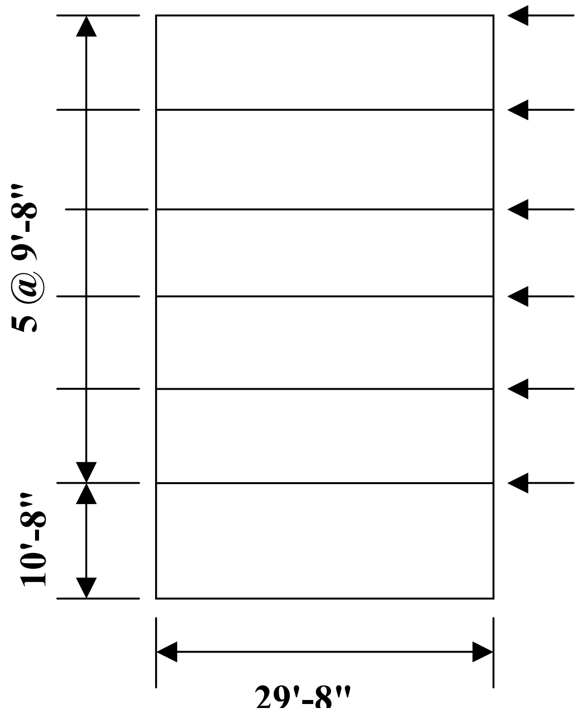
The design seismic performance category is D. The maximum considered earthquake response acceleration at short period, S_s, is 1.5, and at 1 second period, $S_{1,}$ is .4

Design the wall by determining the wall thickness and reinforcement required.

The wall is single wythe CMU masonry.

f'_m = As required
F_y = 60,000 psi (Grade 60)
Units 8x8x16 available in 6,8,10 and 12 inch nominal thickness





Wall at grid line C between 1 and 2

Cumulative Loads (Top of Wall)				
Floor No.	Shear (E) Kip	Moment (E) Kip-Ft	Dead Load Kip/Ft	Live Load Kip/Ft
R	39.0	0	2.85	.6
6	99.2	377	6.83	1.32
5	145.1	1336	10.8	1.81
4	176.5	2739	14.8	2.24
3	177.2	4446	18.8	2.67
2	252.3	6160	22.75	3.10
1	252.3	8893	23.5	3.10

1. Assumes wall weight of 70 psf.
2. Includes live load reduction.

Strength Design

IBC Section 1605.2.1 Load Combinations

1.2D + .5L + E	**Formula 16-5**
.9D + E	**Formula 16-6**
$E = \rho Q_E + .2S_{DS}D$	**Equation 16-28**
$E = \rho Q_E - .2S_{DS}D$	**Equation 16-29**

ρ = Reliability Index = 1
(SDC of D use IBC equations 16-32, value between 1.0 and 1.5, many redundant walls in this building, use 1.0)
Q_E = Effect of Horizontal Forces
D = Dead Load (given)
S_{DS} = 1.0

P_E = .2 x 1.0 x D = .2 D

Design - Estimate Size of Wall

IBC Section 2108.9.2.13 limits the amount of reinforcement in the wall to assure ductile performance. The assumption is a cantilevered wall with a plastic hinge at the base. IBC Section 2106.5.2 also requires that the reinforcement be "uniformly distributed".

From the derivation of Rho Max for CMU and grade 60 steel the following equation was determined for the maximum amount of reinforcement:

$$\rho_{max} = \frac{.124f'_m - \frac{P}{bd}}{53.6}$$

IBC Section 2108.9.2.13.1 also requires that P be the "factored" gravity axial load. The MSJC 2002 Section 3.2.3.5.1 uses "unfactored" axial gravity load.

P = (1.2)*23.5 K/ft * 29.67 ft + .5*3.1 *29.67 = 882.7 Kip

For 8 inch CMU and an f'_m=1500 psi:

ρ_{max}=(.124*1.5 – (882.7/(7.625*356))/53.6 = -.00259

Since a minimum of .0007 is required by IBC Section 2106.5.2 for Seismic Design Category D the 8 inch wall will not work.

Try a 10 inch wall with f'_m=2500 psi:

ρ_{max}=(.124*2.5 – (882.7/(9.625*356))/53.6 = .00098

This is sufficient.

Try a 12 inch wall with f'_m=3000 psi:

ρ_{max}=(.124*3.0 - 882.7/(11.625*356))/53.6 = .0029

Try a 12 inch wall with f'_m=2000 psi:

ρ_{max}=(.124*2.0 - 882.7/(11.625*356))/53.6 = .00064 NG

Try a 12 inch wall with f'_m=2500 psi:

ρ_{max}=(.124*2.5 - 882.7/(11.625*356))/53.6 = .0018 OK

Design

Load Combination .9D + E (Formula 16-6)

P = (.9 - .2)*23.5 K/ft * 29.67 ft = 488.1 Kip
M = 8893 K-ft

Try #5@24, $\rho = .31/(11.625*24) = .0011 < \rho_{max} = .0018$
$f'_m = 2500$ psi
$E_m = 900*f'_m = 2{,}250{,}000$ psi
$E_s = 29{,}000{,}000$ psi
$L_w = 356$ in
b = 11.625 in
d = 356 in (Using a wall length of d is consistent with the assumptions of distributed reinforcement)

The calculated moment is:

	Force	Depth	Moment
Plastic compression steel	-5001.4	3.26	-16301.0
Elastic compression steel	-12003.3	16.95	-203436.2
Elastic tension steel	12003.3	58.67	704202.1
Plastic tension steel	220126.3	212.55	46787520.5
Compression in the masonry	-703224.9	15.12	-10634953.1
Applied compression	488100.0	178.00	86881800.0
			(inch-lbs)
Mn	0		123,518,832.39
Phi Mn			111,166,949.15

M_n= 10293 Kip-Ft
ϕM_n= 9263 Kip-Ft Ok

The moment at first yield is 8,298 Kip-ft, which is close to the applied moment of 8,893 Kip-ft. The cracking moment is 7,529 Kip-Ft

Load Combination 1.2D + .5L + E (Formula 16-7)

P = (1.2 + .2)*23.5 K/ft * 29.67 ft + .5*3.1 *29.67 = 1,022 Kip
M = 8893 Kip Ft

The calculated moment is:

	Force	Depth	Moment
Plastic compression steel	-8509.1	5.55	-47184.9
Elastic compresssion steel	-20421.9	28.84	-588867.9
Elastic tension steel	20421.9	99.81	2038389.0
Plastic tension steel	182944.2	236.78	43317399.2
Compression in the masonry	-1196435.1	25.73	-30784020.6
Applied compression	1022000.0	178.00	181916000.0

M_n= 16,320 Kip-Ft
ϕM_n= 14,688 Kip-Ft Ok

Shear Design:

IBC Section 2108.9.3.5.2 Nominal Shear strength.

M/Vd = 8893/(252.3 x 29.67) = 1.18 > 1.0 use 1.0

MSJC Section 3.1.3: "The design shear strength, ϕV_n ,shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n of the member, except that the nominal shear strength, V_n, need not exceed 2.5 times the required shear strength, V_u."

Shear factor = 1.25 M_n/M = 1.25*10,293/8893 = 1.45 < 2.5

Nominal shear strength provided by the masonry is:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right]A_n\sqrt{f'_m} + .25P$$

V_n = (4.0-1.75*1.0)*11.625*356*√2500 +.25*488,100 = 587,600 lb
ϕ V_n = .8*587,600 = 470,000 lb > 252,000 *1.45 = 365,400 lbs

No shear reinforcement required.

Check for maximum shear allowed:

$$V_{n\,max} = 4A_n\sqrt{f'_m}$$

V_{mmax} = 4*11.625*356*√2500 = 827,700 lb
ϕ V_m = .8*827,700 = 662,000 lb OK

Seismic Design Category D (IBC Section 2106.5)

IBC Section 2106.5.3 Masonry Shear Walls

Minimum reinforcement for "Special reinforced masonry shear walls"

$$A_{sv} + A_{sh} = .002*A_{gross} = .002*11.625*12 = .279 \text{ in}^2/\text{ft}$$

and

$$A_{sv} \geq .0007*11.625*12 = .097 \text{ in}^2/\text{ft}$$

or

$$A_{sh} \geq .0007*7.625*12 = .097 \text{ in}^2/\text{ft}$$

Vertical reinforcement provided for moment:

$$A_{sv} = .31 \text{ in}^2/2 = .155 \text{ in}^2/\text{ft OK}$$

Minimum horizontal reinforcement .124 in^2/ft

Try 2#5 @ 48" O.C.

$$A_{sh} = 2*.31/4 = .155 \text{ in}^2/\text{ft}$$

IBC Section 2106.5.3.2 Shear Wall Shear Strength:

Defines the nominal shear strength as:

Try 2#5 @ 48" O.C.

$$V_n = A_n \rho_n f_y = 11.625*356*(.62/(48*11.625))*60{,}000 = 275{,}900 \text{ Lbs}$$

$$\phi V_n = .8*275{,}900 = 220{,}720 \text{ lb NG, Need } 365{,}400 \text{ lbs}$$

Try 2#5 @ 24" O.C.

$$V_n = A_n \rho_n f_y = 11.625*356*(.62/(24*11.625))*60{,}000 = 551{,}800 \text{ Lbs}$$

$$\phi V_n = .8*551{,}800 = 441{,}400 \text{ lb OK, } > 365{,}400 \text{ lbs}$$

IBC Section 2106.5.3.1 Minimum vertical reinforcement is at least 1/3 of the shear reinforcement.

$A_{sv\ min}$ = .31/3 = .103 in^2/ft
Try #5 @ 24" O.C.
A_{sv} = .31/2 = .155 in^2/ft OK

Solution for Strength Design