Check Beam Cracking Moment

2005 MSJC / 2006 IBC

MSJC Section 3.3.4.2.2.2:

$M_n \geq 1.3M_{cr}$

Table 3.1.8.2.1: tensile stress parallel to bed joints, fully grouted, Type S mortar

$f_r = 200$ psi

$1.3M_{cr} = 1.3S_x f_r$

$= 1.3 \frac{9.625(48)^2}{6} \frac{200}{12(1000)}$

$= 80$ kip-ft

$M_n = 150$ kip-ft /.9 = 167 ≥ 80 <u>OK</u>

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1.2D + 0.5 *S*:

P = (1.2 × 1280 lb + 960 lb) + 0.5 ×1600 lb
= 3488 lb per foot of wall

M = (1.2 × 2.0 in. (1280 lb)/2) + 0.5 × 2.0 in. (1600 lb)
= 4672 lb-in./ft. = 389 lb-ft. per foot of wall

1.2*D* + 1.6*S* + 0.8*W:*

P = 1.2 ×(1280 lb + 960 lb) + 1.6 × (1600 lb)
= 5248 lb per ft of wall

M = (2.0 in./2)×(1.2 × 1280 lb) + 1.6 × 1600 lb. (2.0 in./2) + (0.8 × 21600 lb.)
= 21,376 lb-in. /ft = 1781 lb-ft per ft of wall

1.2*D* + 1.6*W* + 0.5 *S:*

P = 1.2 × (1280 lb + 960 lb) + 0.5 × (1600 lb)
= 3488 lb per ft of wall

M = (2.0 in./2)×(1.2×1280 lb) + 1.6×(21,600lb-in.) + (0.5×1600 lb) (2.0-in./2)
= 36,896 lb-in. /ft = 3,075 lb-ft per ft of wall

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$(1.2 + 0.2\ S_{DS})\ D + \rho Qe$: $S_{DS} = 1.0$ p = 1.0

$P = 1.4 \times (1280 \text{ lb} + 960 \text{ lb}) + 1.0 \times (448 \text{ lb})$

$= 3584$ lb per ft of wall

$M = 1.4 \times (2.0\text{in.}/2\ (1280 \text{ lb})) + 1.0 \times 28{,}572$ lb-in.

$= 30{,}364$ lb-in. /ft = 2530 lb-ft per ft of wall

$0.9D + 1.6W$:

$P = 0.9 \times (1280 \text{ lb} + 960 \text{ lb})$

$= 2016$ lb per foot of wall

$M = (2.0 \text{ in.}/2) \times (0.9 \times 1280 \text{ lb}) + 1.6 \times 21{,}600$ lb

$= 35{,}712$ lb-in. /ft = 2976 lb-ft per ft of wall

$(0.9 - 0.2\ S_{SD})\ D + \rho Qe$

$P = 0.7 \times (1280 \text{ lb}) - 1.0 \times (448 \text{ lb})$

$= 448$ lb per foot of wall.

$M = 0.7 \times (2.0 \text{ in.}/2) \times (1280 \text{ lb}) + 1.0 \times 28{,}572$ lb-in. /ft

$= 29{,}468$ lb-in. /ft = 2456 lb-ft per ft of wall

$\Phi = 0.90$ for reinforced masonry

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Check the maximum reinforcing limits

$$\rho_{\max} = \frac{0.64 f'_m \left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha \varepsilon_y} \right) - \dfrac{P}{bd}}{f_y}$$

Check the P load. $D + 0.75\ L + 0.525\ Q_e$ and $\alpha = 1.5$ for walls loaded out of plane (Code Section 3.3.3.5.1.(d))

$P = (1280 \text{ lb} + 960 \text{ lb}) + 0.525 \times 448 \text{ lb} = 2475 \text{ lb}$

$$\rho_{\max} = \frac{0.64 \times 1500 \text{ psi} \left(\dfrac{0.0025}{0.0025 + 1.5 \times 0.00207} \right) - \dfrac{2475 \text{ lb}}{12 \text{ in.} \times 3.81 \text{ in.}}}{60{,}000 \text{ psi}} = .0062$$

$0.0062 > 0.0038$ = #5 @ 16 in. o.c.

$A_s = 0.31 \text{ in.}^2 (12/16) = 0.23 \text{in}^2 > 0.20 \text{ in}^2$

Check vertical compression stress

$P_u / A_g = 3488 \text{ lb} / 91.5 \text{ in}^2 = 38 \text{ lb/in}^2 < 0.05 f'_m = 75 \text{ lb/in}^2$

Therefore slenderness need not be considered.

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First iteration for maximum moment

$\Delta = 0$ in. Eccentricity of the wall from ledger supporting the roof joists

Check $P - \Delta$ effects $M_s = 21,600$ in-lb + 1,280 in-lb = 22,880 in-lb

$M_{cr} < M_u < M_n$ = 18,908 in-lb < 30,284 in-lb <39,850 in-lb

Maximum moments at mid height of wall

$\Delta_u = (5\, M_{cr}\, h^2 / 48\, E_m\, I_g) + (5\, (M_u - M_{cr})\, h^2 / 48\, E_m\, I_{cr}$

= 5(18,908 in-lb) (288 in)^2 / 48 (1,350,000 psi) (443 in^4)

+ 5 (30,284 in-lb – 18, 908in-lb) (288 in)^2/ 48 (1,350,000 psi) 46.7 in^4

= 0.273 + 1.56 = 1.84 in

$M_u = w\, h^2 / 8$ ft + $P_u\, (e/2) + P_u \Delta_u$

M_u = 25 psf(288 in)^2 / (12) 8 + 2336 lb (2.0 in / 2) + 3488 lb (1.84 in) = 30,354 lb-in

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Through Iteration

Δ_s = 1.841 in

M_u = 30,358 in-lb

Find the nominal moment in the wall

Φ= 0.90

$M_n = (A_s f_y + P_u)\ (d - (a/2))$ = 39,850 in - lb

ΦM_n = 35,865 lb in > M_u = 30,358 lb- in

Therefore the design is adequate

Check deflection

Through Iteration

$M_s = w_s\, h_r^2 / 8$ ft. + $P_s\, (e/2) + P_w\, \Delta_s$

= 15.6 psf (288 in)^2 / 8 + 2880 lb (2.0 in/2) + 960 lb (1.0 in) = 17,318 lb-in

$\Delta_s = 5\, M_s h^2 / 48\, E_m\, I_g$

= 5(17,318 in-lb) (288 in)^2 / 48 (1,350,000 psi) 443 in^4 = 0.250 in

Δ = 0.250 in < 0.007 (24 ft) 12 in/ft = 2.02 in allowable

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Determine the area of jamb steel for the IBC - continued

A_s = 1.20 in^2, $A_{s\ eff}$ = 3,029 / 32,000 = 0.09 in^2
ρ = 1.29 / (7.625 x 40) = 0.0042
$n\rho$ = 21.5 (0.0042) = 0.0912
$k = [(n\rho)^2 + 2n\rho]^{1/2} - n\rho = 0.345$
j = 1-k/3 = 1 - 0.345 / 3 = 0.885 $\sum$M about C = 0
M - P[(L/2-kd/3] - $\sum A_s f_s$ jd = 0
{M - P[(L/2-kd/3] } / $\sum A_s f_s$jd =
{90,000(12) - 3,029 (24-0.345(40)/3) } / $\sum A_s f_s$jd
f_s = 1,021,300 / 1.29 (0.885) 40 = 22,287 psi < 32,000 psi <u>ok</u>

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Determine the area of jamb steel for the IBC - continued

Since $\sum A_s$, which included P, was used to find the neutral axis location, the corresponding tension force is also based on the $\sum A_s$.
T + P = 1.29 (22,287) = 28,854 lbs
T = 28,854 – 3,029 = 25,825 lbs
C = T + P = 28,854 lbs
f_b = 2 (C) / (t) (kd) = 548 psi < 658 psi <u>ok</u>
This amount reinforcement and axial load results in both tension and compression stresses which are less than allowables.

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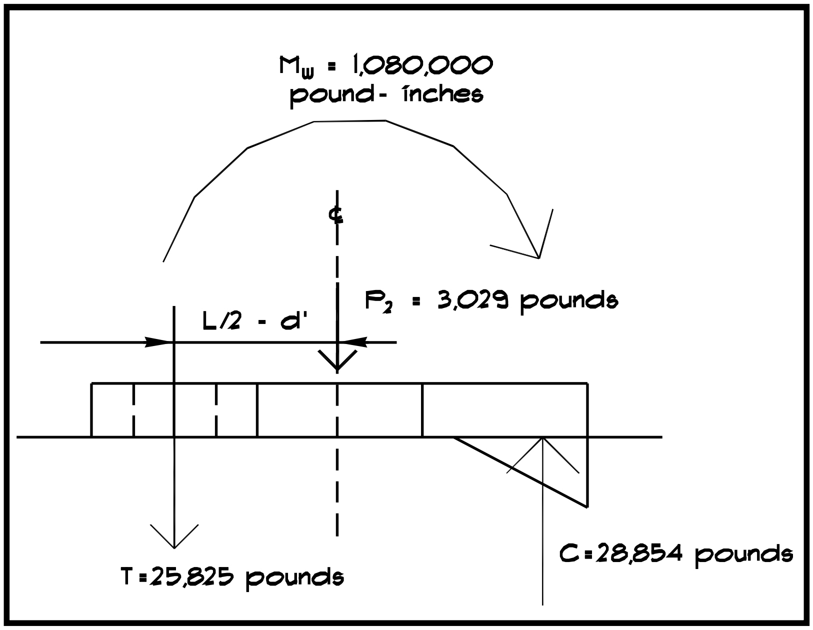

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Assumed neutral axis depth = 58.99

	A (in^2)	d (in)	A_t (in^2)	At (kd-d)
Masonry	450	29.50	450	13,269
Bar 1	0.31	68	6.66	-60
Bar 2	0.31	116	6.66	-380
Peff	2.45	116	52.61	-2,999
Bar 3	0.31	164	6.66	-699
Jamb R/f	2.64	220	56.71	-9,131
Σ =				0.00

k = 0.268

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Check Strain Compatibility to See if Steel Stress Controls the Section

$f_b = \varepsilon_s (1/1\text{-}k) E_m =$ 546

$f_{b\,max} =$ 650

Since f_b is less than $f_{b\,max}$ the steel stress does control the section.

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Take Moments About the NA to Determine the Allowable Moment

$C = f_b\,t\,k\,d / 2 =$ 122,766 $\quad M_m = C(kd - kd/3) =$ 4,828,352

	A (in^2)	d (in)	L -d'- d (in)	f_s	T + P	M_s
Bar 1	0.31	68	152	1,790	555	4,997
Bar 2	0.31	116	104	11,330	3,512	200,218
Peff	2.45	116	104	11,330	27,749	1,581,861
Bar 3	0.31	164	56	20,870	6,470	679,352
Jamb R/f	2.64	220	0	32,000	84,480	13,601,742
Σ =					122,766	16,068,171

$\Sigma M_m + M_s =$ 4,828,352 + 16,068,171 = 20,896,523

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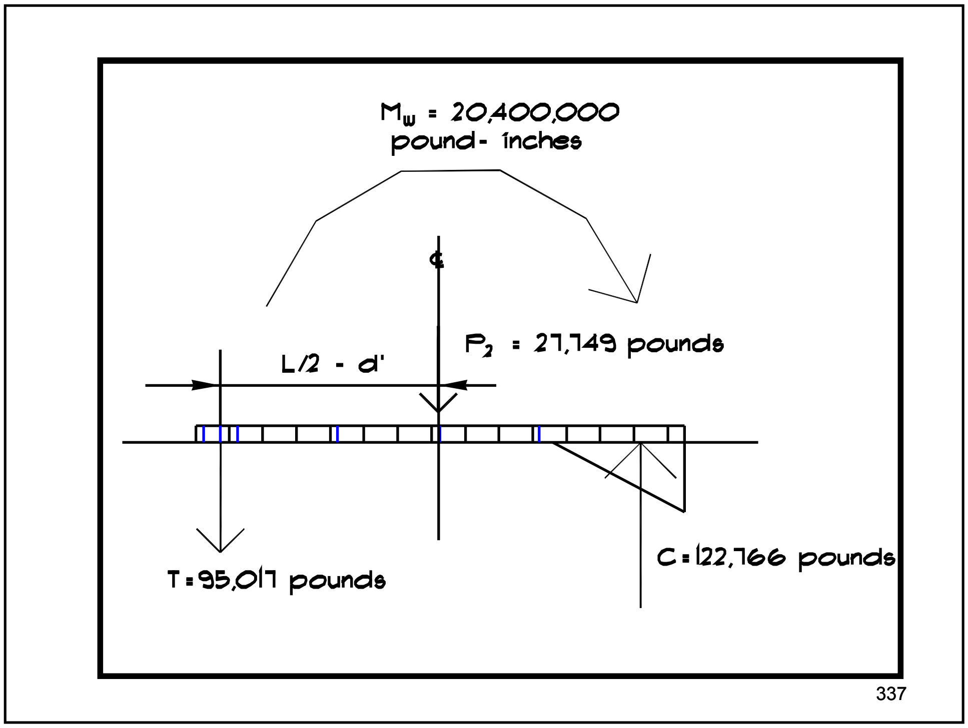

M_w = 20,400,000
pound- inches
℄
P_2 = 27,749 pounds
L/2 - d'
T = 95,017 pounds
C = 122,766 pounds
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