DESIGN NOTES FOR MASONRY

INDUSTRIAL - WAREHOUSE BUILDING

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ACKNOWLEDGEMENTS

This design example has been reviewed by Mr. David Laird, P.Eng., of Robert Halsall and Associates. I would like to express my gratitude to David for his valuable comments and assistance provided, especially in interpreting the Masonry Code.

MASONRY DESIGN OF ONE-STOREY INDUSTRIAL BUILDING

Introduction

The information presented in this design example is intended as a guide, and is not to be used in an actual building. The loads, both dead and live, are approximate only.

However, this example demonstrates the procedures and assumptions that may be useful to the designer of in-dustrial buildings. Emphasis is placed only on the design of the masonry parts of the structure.

The design of the masonry elements of the structure is based on CSA Standard S304 1977 "Masonry Design and Construction for Buildings". Although no effort has been spared in an attempt to ensure that all data is factual, the Alberta Masonry Institute does not assume responsibility for errors or oversights resulting from the use of the informa-tion contained herein.

Materials

a) Hollow load-bearing concrete masonry units.

Two core 10 x 8 x 16 inch light weight blocks are used. The masonry units are assumed to satisfy all the requirements of CSA Standard A165.1-1964.

The compressive strength of the material based on the net cross-sectional area is assumed to be 2350 psi, which is considered typical of the concrete blocks available in the Edmonton, Alberta area.

b) Mortar

Type S mortar mixed in proportions by volume in accordance with CSA Standard Al79-1975 "Mortar and Grout for Unit Masonry" is assumed. The proportions are:

1 part normal cement; 1/2 part hydrated lime;
4-1/2 parts sand.

The sand is assumed to satisfy CSA Standard A82.56-1950 "Aggregate for Masonry Mortar". According to this document the grading of the sand shall conform with the following limts.

Sieve Size	Percentages Passing each Sieve
#4	100
#8	95 to 100
#16	60 to 100
#30	35 to 70
#50	15 to 35 0
#100	to 15

The total deleterious substances are not to exceed 3% by weight.

c) Grout

Coarse grout is used to fill cores, as required, and to construct lintel beams.

The grout is mixed in accordance with CSA Standard A179-1975.

The following proportions (by volume) are recommended by the above document.

1 part normal cement; 0 to 1/10 parts hydrated lime or lime putty; 2-1/4 to 3 times the sum of the cementitious materials fine aggregate; and
1 to 2 times the sum of the cementitious materials coarse aggregate. Both fine and coarse aggregate to be measured in damp, loose state.

The compressive strength of the grout depends on its con-sistency when poured, on the size of the void it fills, and the absorptive capacity of the masonry unit it contacts.
As a result of the above no requirement is placed in the above standard in relation to the compressive strength of the grout. When knowledge of its strength is needed for a particular project it should be determined using the pro-cedure given in Appendix B of the standard. It should be noted here that standard cylinder strength test is not acceptable and it can be very misleading.

d) Steel

The reinforcing steel used in the lintel beams and in all grouted cores has yield strength of 60 ksi.

Allowable Stresses

The compressive strength (f'_m) to be used in the design of masonry structures is determined by either of two methods, namely by testing prisms or by testing masonry units and mortar cubes.

The procedures to be followed and correction factors to be applied for slenderness effects are given in CSA Standard S304-1977 "Masopry Design and Construction for Buildings" and in Supplement No. 4 to the National Building Code of Canada, 1975.

For this example the compressive strength of the concrete block units is assumed to be 2350 psi (based on the net section area). If these units are used with type S mortar, the compressive strength (f'_m) of the masonry to be used in the calculations of allowable stress is obtained from Table 3 of CSA S304. The value of (f'_m) is found by inter-polating to be 1490 psi. The allowable stresses resulting from the various types of load that may act on the building are obtained from Table 5 of CSA S304.

Note that for this example the column for units with voids is used and that for earthquake zones reinforced masonry must be used.

Based on the following table the allowable stresses are:

Compressive axial in walls	0.225 (f'_m) = 335 psi
Compressive axial in columns	0.200 (f'_m) = 298 psi

MAXIMUM ALLOWABLE STRESSES AND MODULI FOR

PLAIN CONCRETE BLOCK MASONRY AND STRUCTURAL CLAY TILE MASONRY *

Type of Stress or Modulus	Designation	Maximum Allowable Stress or Modulus, psi	
		Units without Voids or Filled Hollow Units Based on Gross Cross-Sectional Area	Units with Voids Based on Net Cross-Sectional Area
Compressive, axial			
Walls	f_m	$0.20\ f'_m$	$0.225\ f'_m$
Columns	f_m	$0.18\ f'_m$	$0.20\ f'_m$
Compressive, flexural			
Walls	f_m	$0.30\ f'_m$	$0.30\ f'_m$*
Columns	f_m	$0.24\ f'_m$	$0.24\ f'_m$*
Tensile, flexural			
Normal to bed joints			
M or S mortar	f_t	36	23*
N mortar	f_t	28	16*
Parallel to bed joints			
M or S mortar	f_t	72	46*
N mortar	f_t	56	32*
Shear			
M or S mortar	v_m	34	34*
N mortar	v_m	23	23*

* Shear and flexural calculations shall be based on net mortar bedded area.

Compressive flexural in walls	$0.300\ f'_m$	= 447 psi
Compressive flexural in columns	$0.240\ f'_m$	= 357 psi
Bearing on masonry	$0.250\ f'_m$	= 372 psi
Tensile Flexural		
Normal to bed joints		= 23 psi
Parallel to bed joints		= 46 psi
Shear		= 34 psi

Design Loads

An isometric drawing of the example building is shown in Figure 1. Floor plans are shown in Figure 2. The structure consists of an office, warehouse unit with main floor area of 7500 ft^2. The office portion of the building consists of two storeys and is situated at the front part of the structure. For simplicity both parts are of the same height.

The roof system consists of open web steel joists, steel decking and rigid insulation sections through the structure are shown in Figures 3 and 4. For this example the loads acting on the building are assumed as follows:

Roof	15 psf
Joist	3 psf
Sprinkler system	1.2 psf
Framing	2.3 psf
Miscellaneous	1.5 psf
Total Dead Load	23 psf
Wind**	16.5 psf
Roof (snow, rain, ponding)	20 psf
Office	100 psf
Wall self-weight*	50 psf

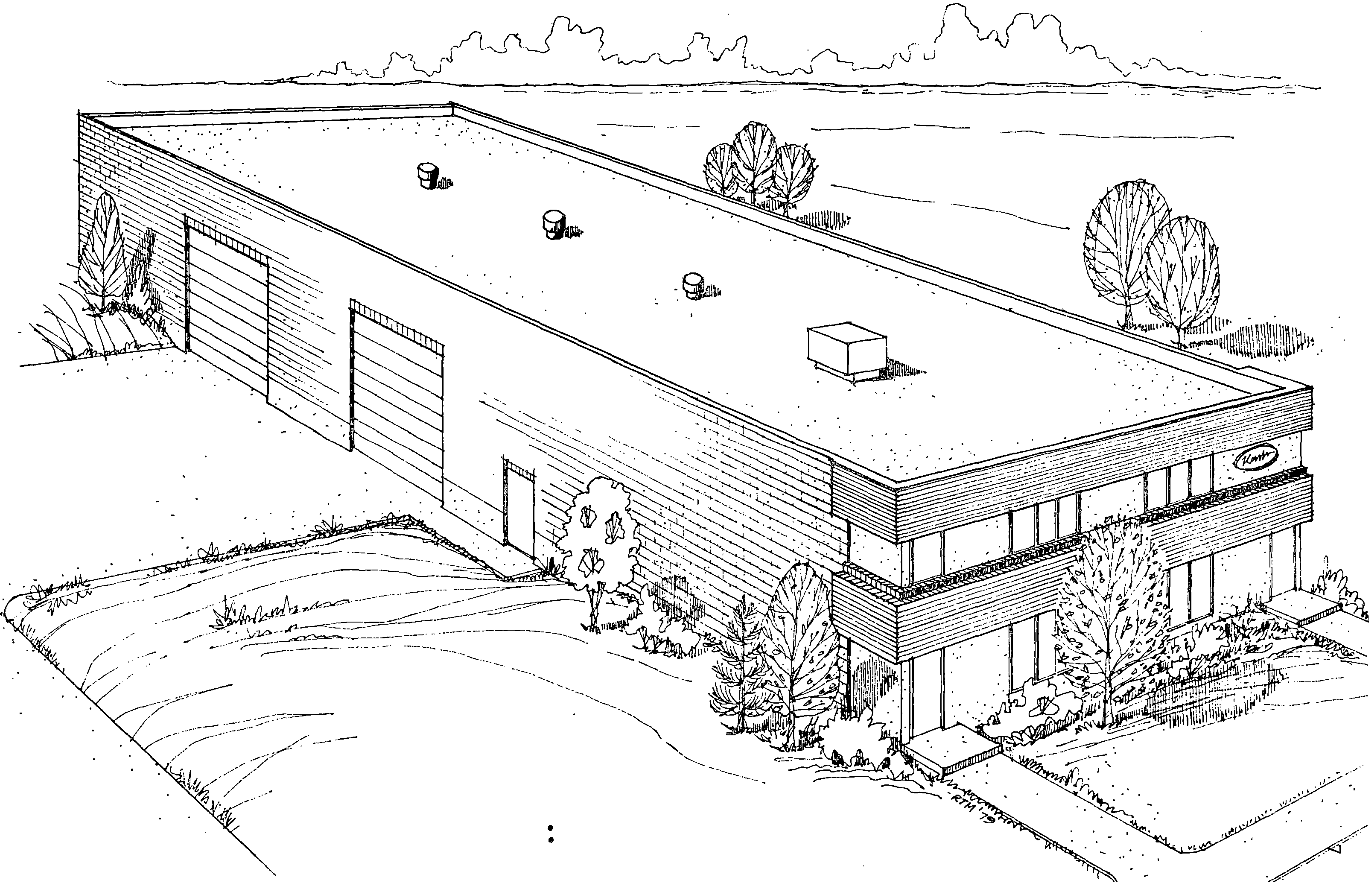
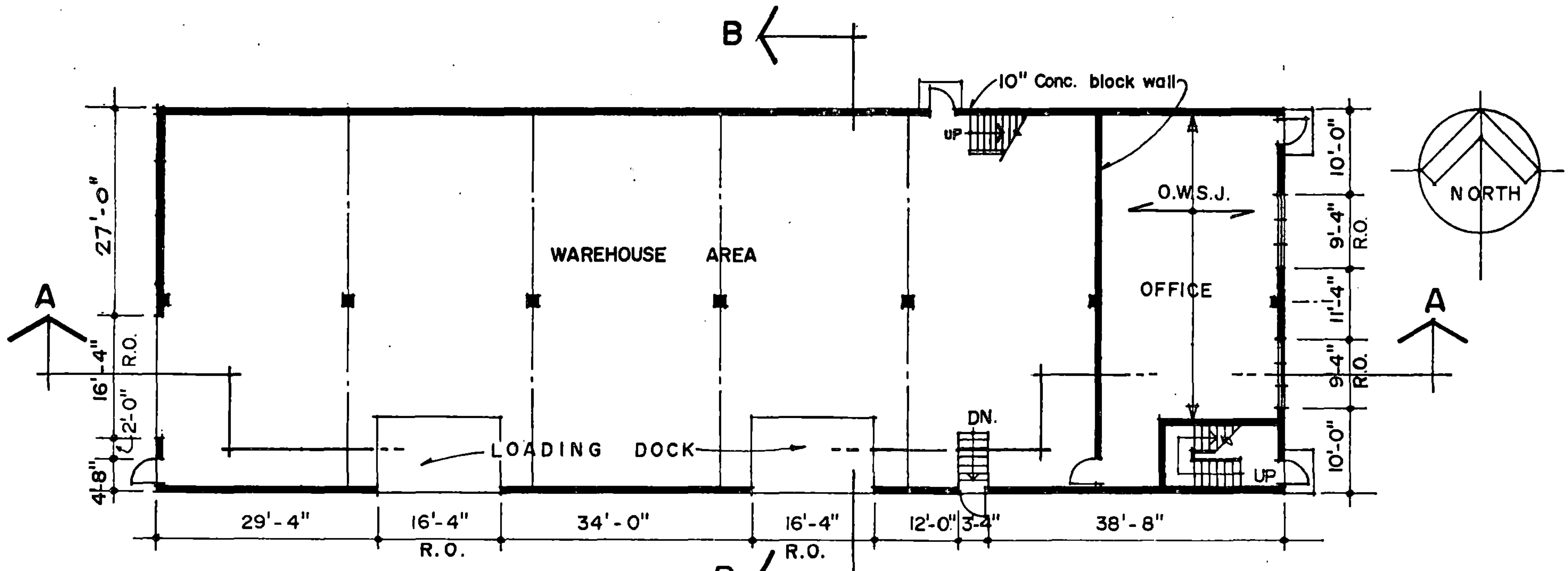


FIGURE :I.. perspective
proposed office/warehouse



MAIN FLOOR PLAN

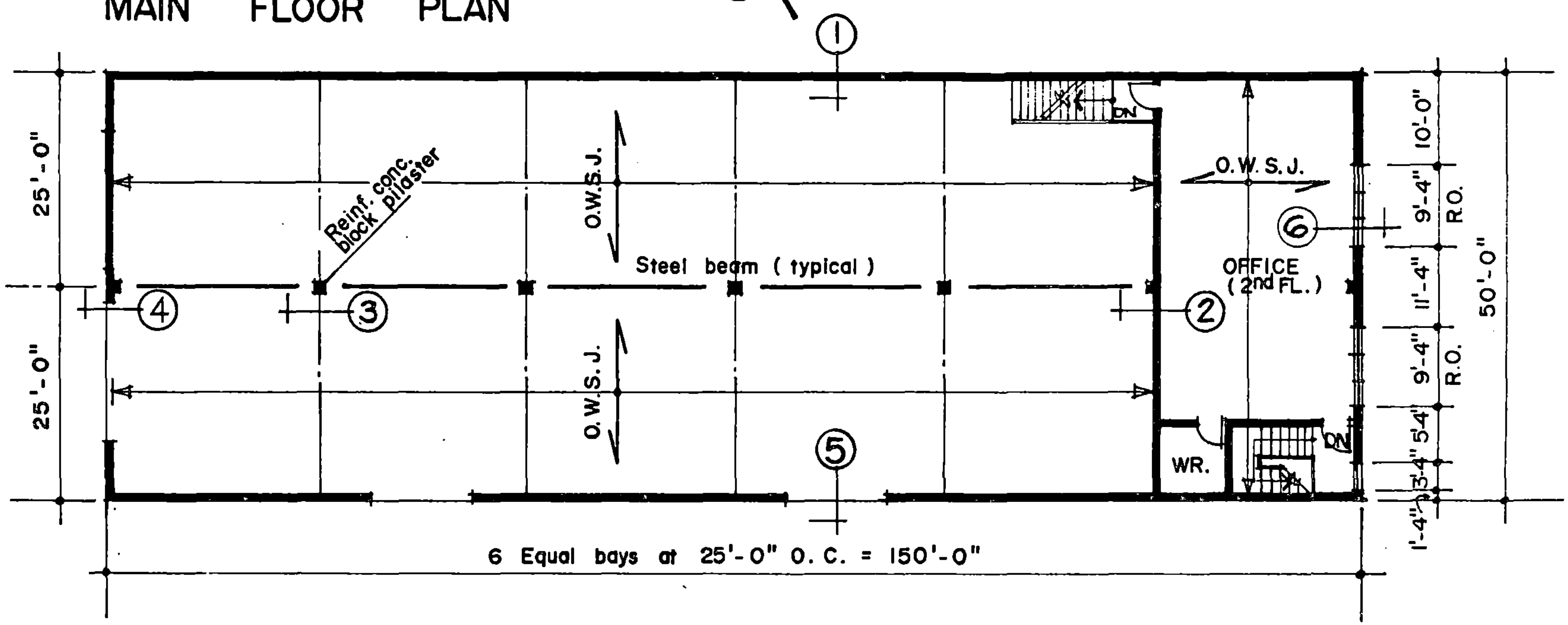
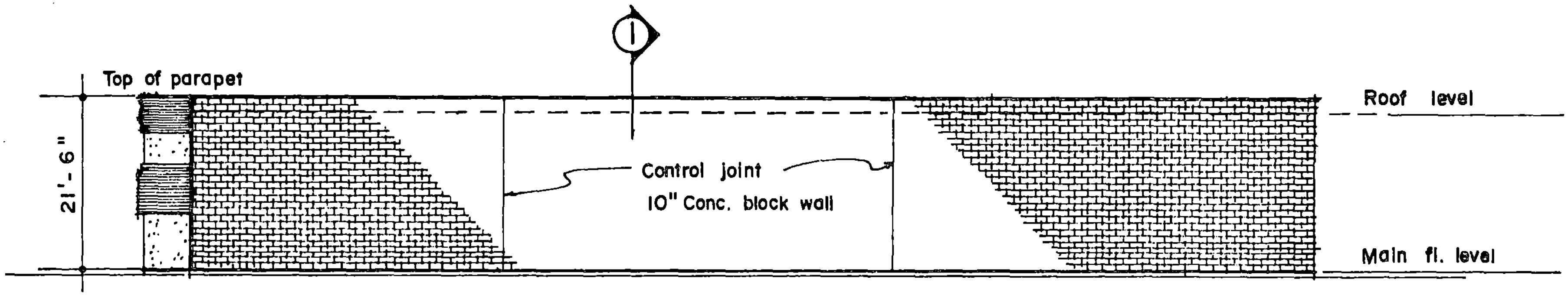
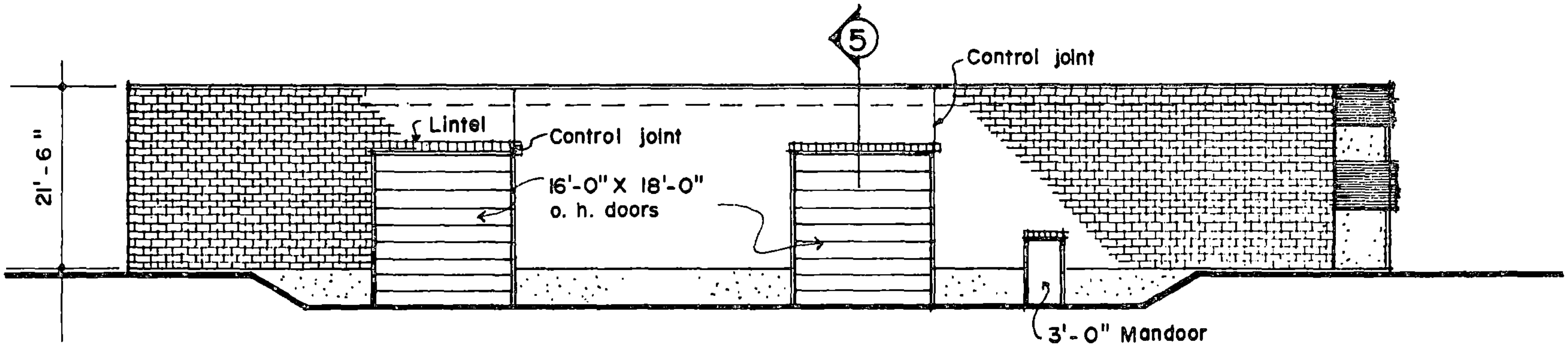


FIGURE 2 SECOND FLOOR & ROOF PLAN



NORTH ELEV.



SOUTH ELEV.

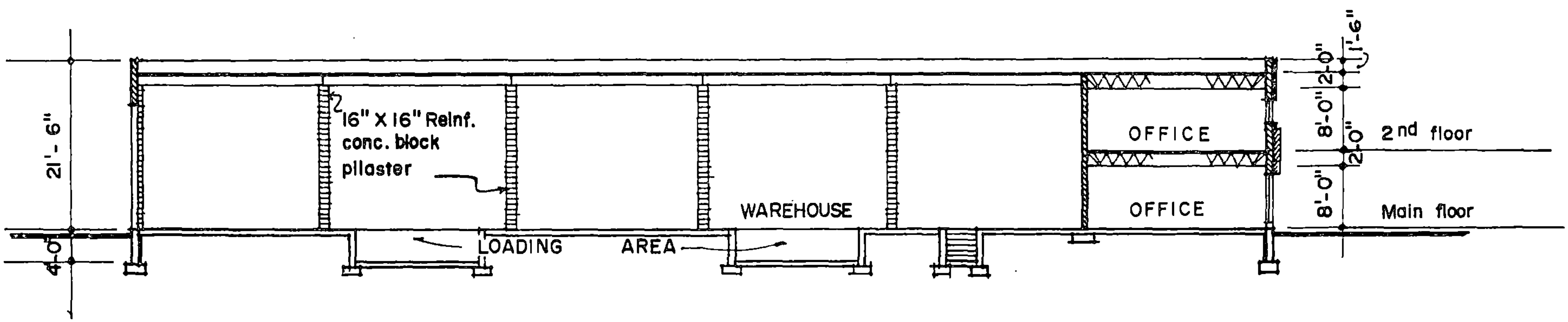
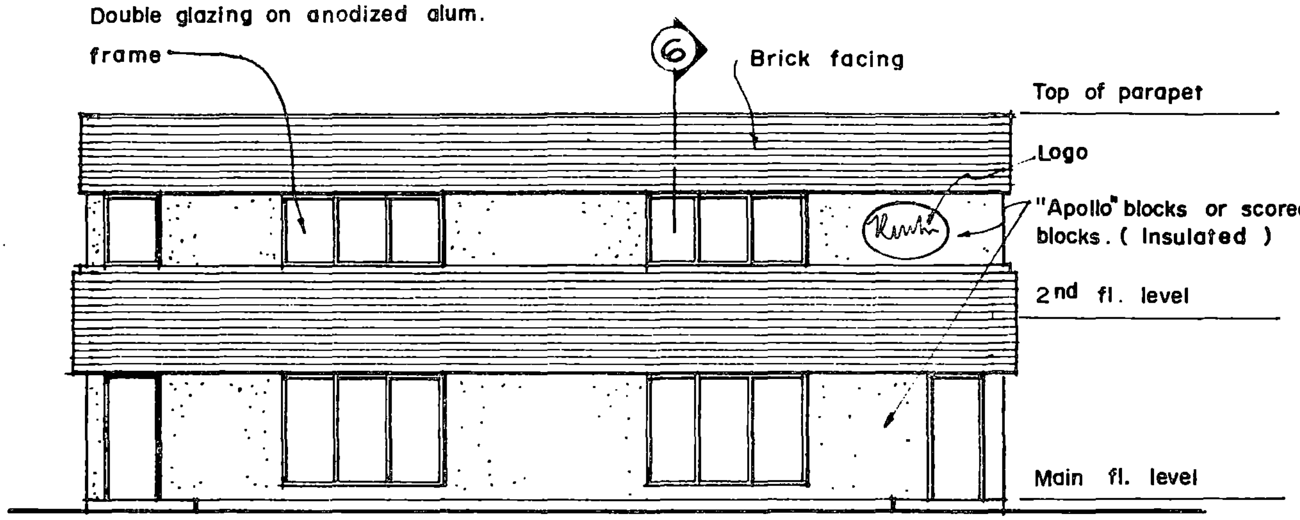
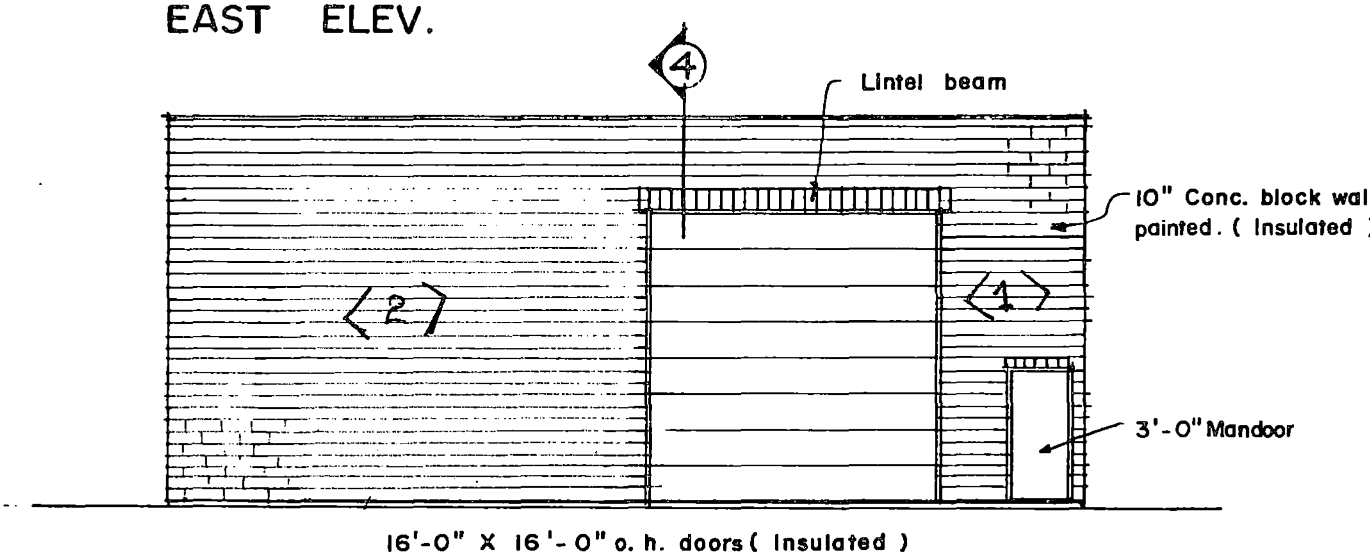


FIGURE 3 LONGITUDINAL SECTION A-A



EAST ELEV.



WEST ELEV.

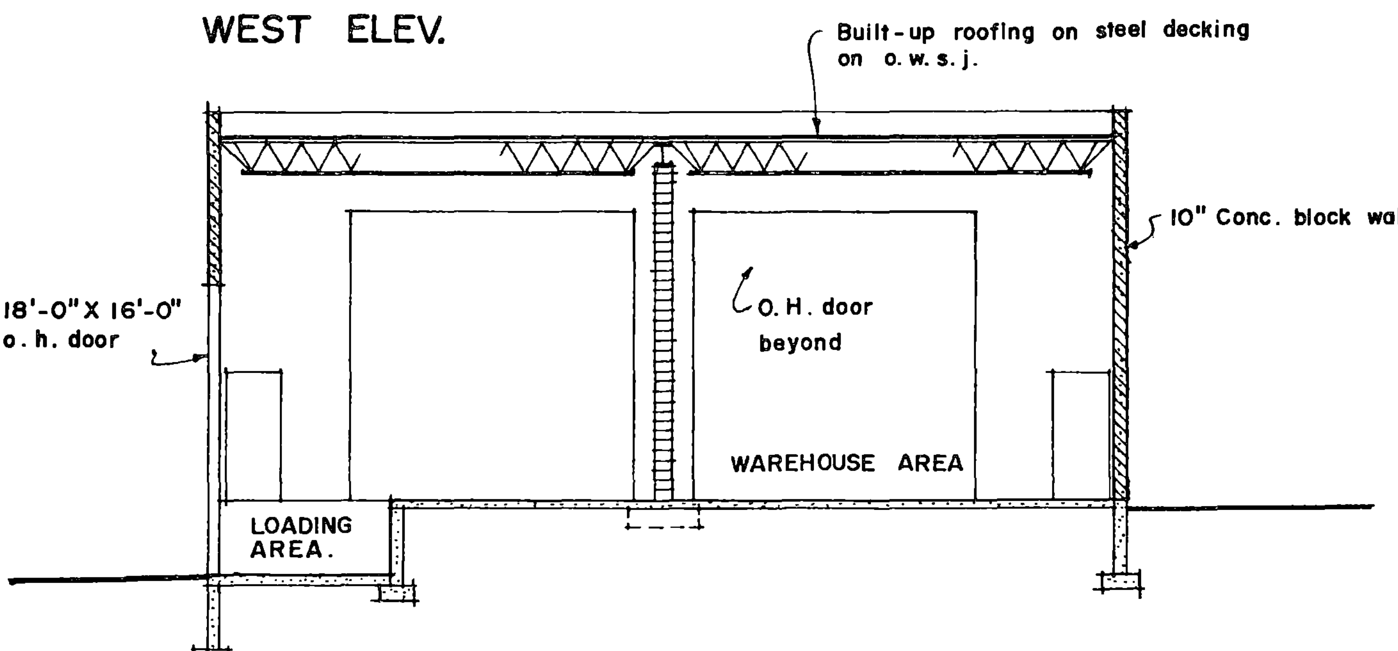


FIGURE 4 CROSS SECTION B - B

* Approximate weight accounting for 1 core filled every 48" and 10" two core block.

** The wind load is calculated on the probability of being exceeded in any one year of 1 in 10 for wall design.

$$= q\ C_e\ C_g\ C_p + q\ C_e\ C_{pi}$$

$$= 6.7 \times 1 \times 2.5 \times 0.7 + 6.7 \times 1 \times 0.7 = 16.42$$

$$\simeq 16.5 \text{ psf}$$

($\frac{1}{30}$ for the building, $\frac{1}{100}$ for special buildings)

For more information the reader is referred to the NBC 1975 subsection 4.1.8 "Effects of Wind".

<u>Design of Masonry Elements</u>

a) Type of Construction

The two basic types of construction for concrete block masonry walls are stack bond and running bond. In stack bond type of construction the mortar joints line up both in the horizontal and vertical direction. In running bond there is an overlapping of blocks by 50% and the vertical joints are in alignment in every second course. For two core blocks and running bond type of construction the cross webs do not line up, and as a result this type of construction should only be carried out with face mortar bedding.

For the construction of load bearing walls and especially for walls subjected to lateral loads running bond is recommended. This type of construction provides for better resistance to lateral loads by the interlocking of blocks where stack bond has weak lines along the vertical mortar joints. Also vertical loads applied at the top of the walls are more effectively spread in running bond than in stack bond.

b) Design of Load Bearing Wall Section 1, Figure 5 Loads:

Live load	20 x 12.5	=	250.0 lb/ft.
Dead load	23 x 12.5	=	287.5 lb/ft.
Selfweight at midheight and parapet	50 (10+2.0)	=	600 lb/ft.

Note that 10" block is used in all walls, the weight of the block material is assumed to be 105 lb/ft^3 and allowance has been made for partial grouting.

Wind induced moment

$$M_W = Wh^2/8 = \frac{16.5 \times 20^2}{8} = 825.0 \text{ ft-lb.}$$

Assuming 1.5" eccentricity of the vertical load the moment at mid-height assuming simple support at the bottom of the wall is:

$$(250 + 287.5) + \frac{1.5}{12} \simeq 68 \text{ ft-lb.}$$

The total moment at this section is

$$825 + 68 = 893 \text{ ft-lb.}$$

Check for [dead plus live]; and for 0.75 (dead+wind+live load).

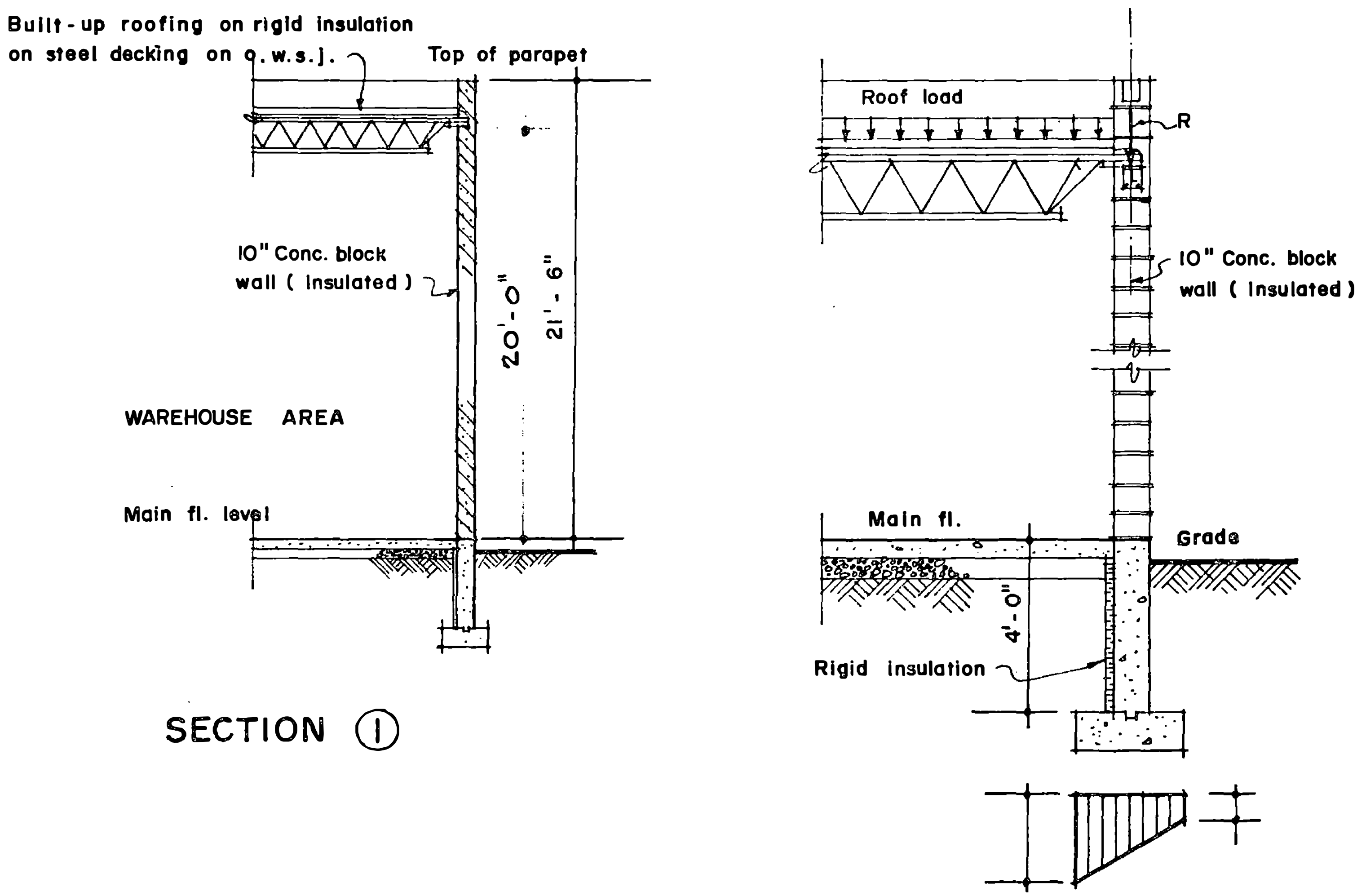


FIGURE 5 LOAD BEARING WALL

The total vertical load on the wall at this section is

$$250 + 287.5 + 600 = 1137.5 \text{ lb.}$$

check for other load combinations.

The virtual eccentricity at mid-height is

$$e = \frac{M}{P} = \frac{893}{1137.5} = 0.79 \text{ or } 9.42 \text{ inches}$$

since $e > t/3 = \frac{9.625}{3}$ the tensile stress must be checked.

$$f_t = -\frac{P}{A_m} + \frac{My}{I_m} = -\frac{1137.5}{(1.5 \times 2)12} + \frac{893 \times 9.625 \times 12}{2 \times 2 \times (1.5 \times 12) \times 4.0^2}$$

$$= -31.6 + 89.50 \simeq 58.0 \text{ psi (tension)}$$

Note that t/3 applies to solid sections only but is used here as an indicator of stress conditions since this stress is larger than the 23 psi allowable stress, reinforcement is required. Note that in the above calculation the mortar bedded area was used in calculating the compressive stress and the flexural stresses.

Since the tensile stress exceeds the allowable, rein-forcement must be provided to resist the wind induced moment. The distance between reinforced cores must be limited by the ability of the masonry to span between rein-forcement and the capacity of the reinforced section. In computing flexural stresses in walls where reinforcement occurs the effective width shall not be greater than 4 times the wall thickness.

Consider placing the reinforcement at 6 feet center to center. The wind induced moment between grouted cores is:

$$16.5 \times 6^2/8 = 74.25 \text{ ft-lb. or } 891.0 \text{ in-lb.}$$

The stress due to this moment is:

$$f_t \simeq \frac{891 \times 9.625}{2 \times 2(1.5 \times 12)(40)^2} \simeq 7.4 \text{ psi} < 46 \text{ psi} \quad \text{ok}$$

The wind load to be resisted by the reinforced cores is

$$6 \times 16.5 = 99 \text{ lb/ft.}$$

Check the effect of wind at the openings.

For the wall to crack at the bottom due to the wind load, the resistance due to self-weight and bond must be overcome. Not taking into account the roof dead load, and the bond, in order to account for possible uplift, the wall self-weight moment resistance is

$$(50 \times 21.5) \times \frac{1}{2}\, 9.625 \times \frac{1}{12} \times 6 = 2586 \text{ ft-lb.}$$

The wind induced moment assuming fixed support is

$$M = \frac{WL^2}{8} = 99 \times \frac{20^2}{8} = 4950 \text{ ft-lb.}$$

Therefore at the bottom reinforcing must be provided to resist

$$4950 - 2586 = 2364 \text{ ft-lb.}$$

The maximum moment to be resisted along the height is

$$M = \frac{9}{128} WL^2 = \frac{9}{128} \times 99 \times 20^2 = 2784 \text{ ft-lb.}$$

and occurs at 12.5 feet above the floor level. Note that self-weight is neglected here for simplification. Since the moment at this point is larger than the moment to be resisted at the floor level, reinforcing will be designed for this location and carried through the whole wall.

The total moment at this location is

$$2784 + \frac{1}{2}\,68 \times 6.0 = 2784 + 204 \simeq 3000 \text{ ft-lb.}$$

The wall is now forced into double curvature. The virtual eccentricity for the positive moment is

$$+\,e_1 = \frac{+M}{P} = 13.34"$$

where $M = 3000$ ft-lb.

$P = (7.5 + 1.5) \times 50 \times 6 = 2700$ lb.

The virtual eccentricity at the ground level is

$$-\,e_2 = \frac{M}{P} = \frac{2364}{21.5 \times 50 \times 6} = -\,4.4 \text{ inches}$$

$$\frac{e_2}{e_1} = -\,\frac{4.40}{13.34} = -\,0.33$$

Since the virtual eccentricity (e_1 or e_2) exceeds t/3 the wall must be designed by the use of a transformed section, and the resulting stresses must be modified by the slender-ness coefficient (C_s). See Figure 6 for stress diagram.

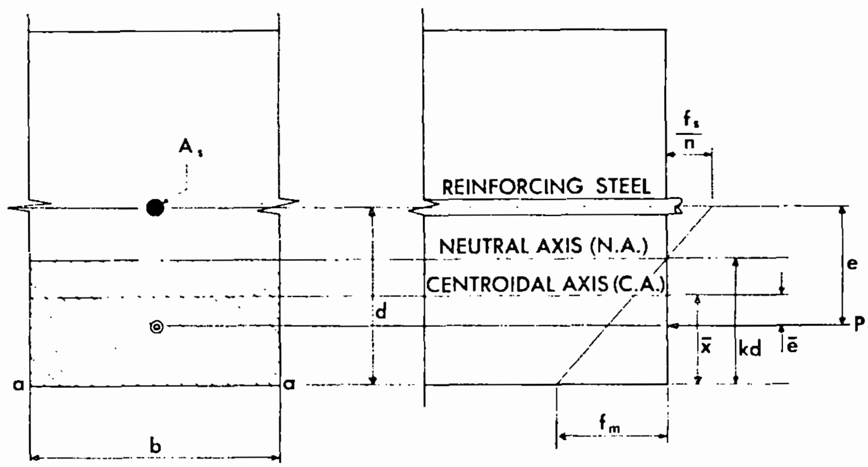


FIGURE 6 STRESS DIAGRAM FOR REINFORCED WALL SECTION

for $\frac{h}{t} = \frac{20 \times 12}{9.625} \simeq 25.0$ and $\frac{e_2}{e_1} = -0.33$

$C_s = 0.60$ (from Table 8 S304).

$f_m = 0.3\ f'_m = 447$ psi

$f_s = 24000$ psi (article 4.5.2.16 S304).

$\eta = \frac{E_s}{E_m} = 10$ (assumed)

$A_s = 1 - 15M = 0.31\ in^2$

$M = 3000$ ft-lb

$P = (2700 + 1500 + 1725) = 5925$ lb.

$=$ load over the 6' between rebars

$e = \frac{3000}{5925} = 0.506$ ft $= 6.08$ in.

Note that the vertical load is considered over the total 6 feet between bars but only 4 x tis active in conjunction with the steel to resist flexural stresses. For stack bond only the width of the unit should be used.

$d = \frac{1}{2}t \simeq 4.75$ in (assuming steel centered in wall)

Assume

$kd = 1.75$ in (by trial and error)

$\eta A_s = 10 \times 0.31 = 3.1\ in^2$

$b(kd) = 4t \times 1.75 = 67.38\ in^2$

$A_t = 70.48\ in^2$

$b(kd)(\frac{kd}{2}) = 4 \times 9.625 \times 1.75 \times 0.5 \times 1.75 = 58.95$

$\eta A_s d = 3.1 \times 4.75 = 14.73$

$A_t\ \bar{x} = 58.95 + 14.75 = 73.67$

$$\overline{X} = \frac{A_t\overline{X}}{A_t} = \frac{73.67}{71.78} = 1.02 \text{ in}$$

$$\frac{1}{3}\, b(kd)^3 = \frac{1}{3}\, 4 \times 9.625\ (1.75)^3 = 68.78 \text{ in}^4$$

$$\eta A_s d^2 = 10 \times 0.31 \times 4.75^2 = 69.95 \text{ in}^4$$

$$I_a = 68.78 + 69.95 = 138.7 \text{ in}^4$$

$$I_{cA} = I_a - A_t\,(\overline{X})^2 = 138.7 - 70.48 \times 1.02^2 = 65.4 \text{ in}^4$$

$$\overline{e} = 1.02 - 4.75 + 6.08 = 2.35 \text{ in}$$

$$d - \overline{X} = 4.75 - 1.02 = 3.73 \text{ in}$$

$$f_m = \frac{P}{A_t} + \frac{P\overline{e}\overline{X}}{I_{cA}} = \frac{0.53 \times 5925}{70.48} + \frac{5925 \times 2.35 \times 1.02}{65.4}$$

$$= 44.55 + 217.15 = 261.7 \text{ psi}$$

note that

$$0.53 = \frac{1}{6}\ [4t]$$

$$\frac{f_s}{\eta} = \frac{P}{A_t} - \frac{P\overline{e}(d-\overline{X})}{I_{ca}} = 44.55 - \frac{13922\ (4.75 - 1.02)}{65.4}$$

$$= 43.75 - 794 = -750$$

check kd

$$kd = \left[\frac{f_m}{f_m + \frac{f_s}{\eta}}\right] d = \left[\frac{261.72}{261.72 + 750}\right] 4.75 = 1.23$$

$$\frac{1}{C_s}\, f_m = \frac{1}{0.60} \times 261.7$$

$$= 435 < 447 \text{ psi} \therefore \text{ ok}$$

$$\frac{1}{C_s}\left(f_s \times \frac{1}{n}\right) n = \frac{1}{0.60} \times 750 \times 10 = 12500 < 24000 \text{ psi}$$

Note that k_d was assumed as 1.75 and 1.23 was calculated. One can repeat the process for refinement.

Grout 1 core every 6 feet with one 1 - 15 M bar at the center.

b) Design of Load Bearing Wall, Section 2 Figure 7.

Loads on wall at second floor

Live load	20 x 15.1	=	302.0 lb/ft
Dead load	23 x 15.1	=	347.0 lb/ft
Midheight self weight	50 x 5.0	=	250 lb/ft
Bottom self weight	50 x 10	=	500 lb/ft

Loads on wall at top of first floor

From above 1149 lb/ft

Live load 12.1 x 100 = 1210.0 lb/ft

Dead load 23 x 12.1 = 278.0

2" concrete toping

25 x 12.1 = 302

Mid height self weight = 250

Bottom self weight = 500

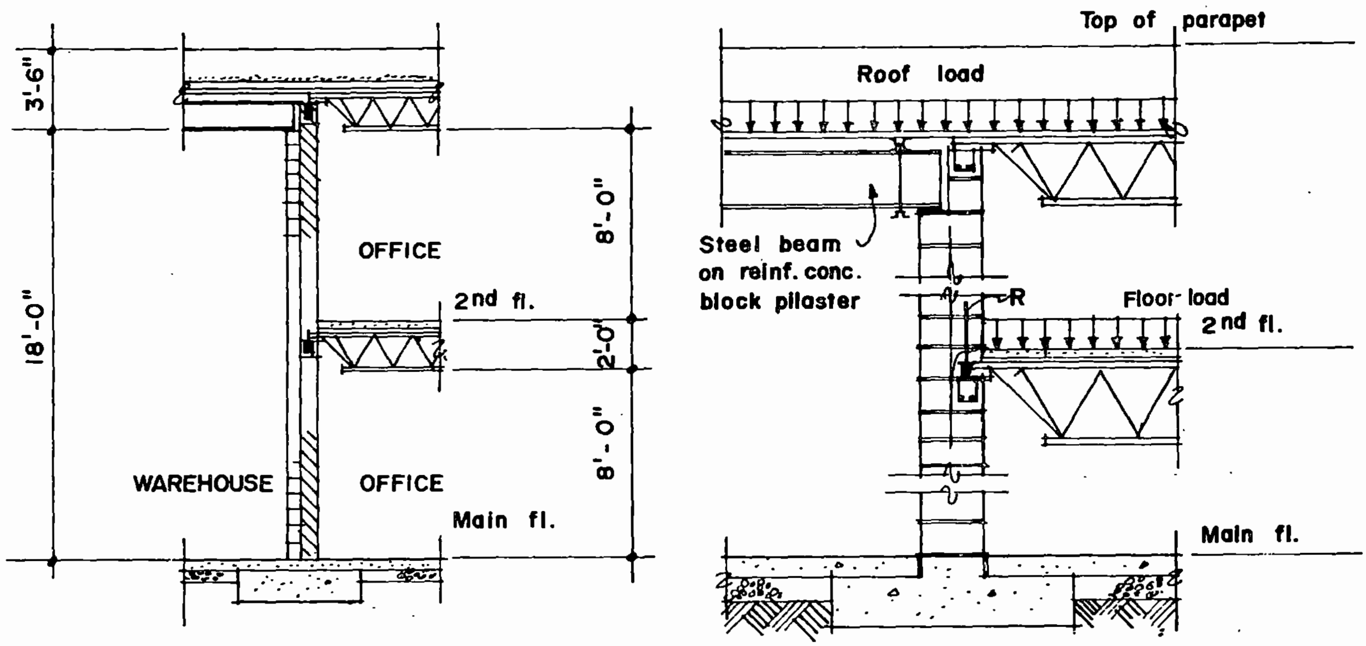
1) Second floor wall

Assume $e_{top} = 1 = e_{bottom}$

Most critical loading condition when full load on roof and floor.

At the second floor level

$$M = \frac{1}{12}(1210 + 278 + 302) = 149.0 \text{ ft-lb.}$$



SECTION ②

FIGURE 7 LOAD BEARING WALL LOCATED AT THE TWO STOREY PART OF THE STRUCTURE

Assuming that the wall is uncracked this moment is resisted by the wall above and below the second floor. Thus the virtual eccentricity at this level is $-\frac{1}{2}$" (the wall is forced into double curvature.)

Wind Moment

$$M = \frac{WL^2}{8} = 16.5 \times \frac{10^2}{8} = 206 \text{ ft-lb.}$$

$$e = \frac{206}{(302 + 347 + 250)} = 0.23' = 2.75 < \frac{t}{3} = 3.2 \text{ in}$$

Since this eccentricity is less than t/3 the wall can be designed using the relation

$$P_{all} = C_s \, C_e \, f_m \, A_n$$

For

$$\frac{h}{t} = \frac{10 \times 12}{9.625} = 12.5 \quad \text{and} \quad \frac{e_1}{e_2} = -0.5$$

from Table 8 of CSA S304

$$C_s = 0.91.$$

From Table 9 the eccentricity coefficient for

$$\frac{e}{t} = \frac{2.75}{9.625} = 0.29 \quad C_s \simeq 0.58.$$

$$A_n = 4.8 \times 12 = 57.6 \text{ in}^2$$

$$f_m = 0.225 \, f'_m = 335 \text{ psi}$$

$$P_{all} = 0.91 \times 0.58 \times 335 \times 57.6 = 10184 \text{ lb/ft.}$$

This capacity is more than adequate for the design loads. However, to avoid confusion, grout one core every 6 1 with 1 - 15M at the center.

2) First floor wall

At mid-height the loads on the wall are

(302 + 347 + 500 + 1210 + 278 + 302 + 250) = 3189 lb.

$$\frac{e_1}{e_2} = \frac{0}{0.5} = 0 \quad \text{and} \quad \frac{h}{t} = 12.5$$

wind moment $M = 206$ ft-lb.

$$e = \frac{206}{3189} = 0.78 \text{ in.}$$

$$\frac{e}{t} = \frac{0.78}{9.625} = 0.081$$

$$C_s = 0.87 \quad C_e \simeq .94 \quad A_n = 57.6 \text{ in}^2$$

$$f_m = 335 \text{ psi}$$

$$P_{all} = 0.87 \times 0.94 \times 335 \times 57.6 = 15780 \text{ lb/ft.}$$

The wall is more than adequate.

Design of Concrete Block Pilaster

Section 3 Figure 8 shows a typical interior rein-forced concrete pilaster. The tributary area on this pilaster is

$$25 \times 25 = 625 \text{ ft}^2$$

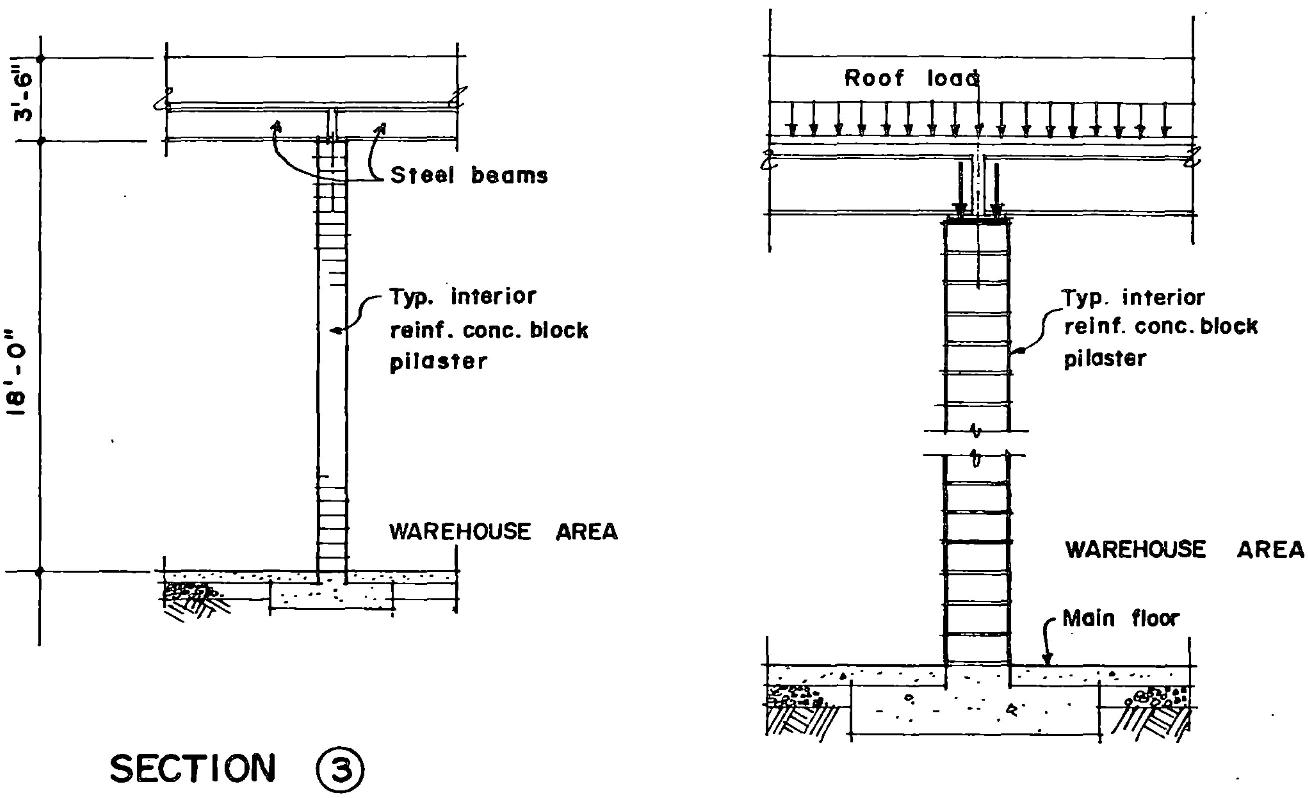


FIGURE 8 INTERIOR CONCRETE BLOCK PILASTER

The total dead plus live load acting on this pilaster is:

$$625 \times (23 + 20) = 28036 \text{ lb.}$$

Assuming a 16 x 16 pilaster, the self weight at mid-height is:

$$(16 \times 16 \times 9 \times \frac{1}{144})\ 150 = 2400 \text{ lb.}$$

and thus the total load at mid-height is

$$28036 + 2400 = 30436 \text{ lb.}$$

The allowable load on this pilaster is obtained using the following relation

$$P = C_s\, C_e\, (f_m + 0.8\, \rho_n\, f_s)\, A_n$$

(article 4.6.7.5 of S304).

Assuming that the load is applied axially ($\frac{e_1}{e_2} = 0$), the slenderness coefficient is $C_s = 0.83$ (obtained from Table 8 of S304 by entering with $h/t = \frac{18 \times 12}{15.625} = 13.82 \simeq 14.0$ and $e_1/e_2 = 0.$)

Note that the limiting h/t ratio for $e_1/e_2 = 0$ is 20.0.

The eccentricity coefficient (C_e) is obtained from Table 9 by entering the table with e/t and e_1/e_2. This factor for this example is 1.0.

The maximum amount of reinforcement which can be used is 4 per cent and the minimum 0.5 per cent. (Article 4.6.8.3.1 S304). The strength of the pilaster is now checked providing minimum reinforcement.

$$A_s = 0.005 \times 16 \times 16 = 1.28 \text{ in}$$

$$4 + 20M = 0.44 \times 4 = 1.76 \text{ in}^2$$

$$P = 0.83 \times 1.0\ (0.20 \times 1350 + 0.8 \times 0.0069 \times 24000)15.625^2$$

$$= 0.83\ [270 + 132.4]\ 15.625^2 = 81557 \text{ lb.}$$

Note that since the column is stressed to less than one half of its capacity the code allows a further reduction of the reinforcing steel to 0.27 per cent of the cross-sectional area. Use 4-15M

Lateral ties are provided in accordance with articles 4.6.8.32 and 4.6.8.33. The ties are placed in contact with the vertical steel.

Figure 9 shows details of end pilaster. The design of this pilaster, provided that the load is applied axially, is similar to the previous one.

<u>Lintel Beams</u>

1) General

In the construction of masonry buildings one of the most frequent problems is the construction of beams over openings such as doors and windows. These members, commonly referred to as lintels, function as beams in supporting the weight of the wall and other loads over the openings. These loads are transmitted in the form of reactions to the

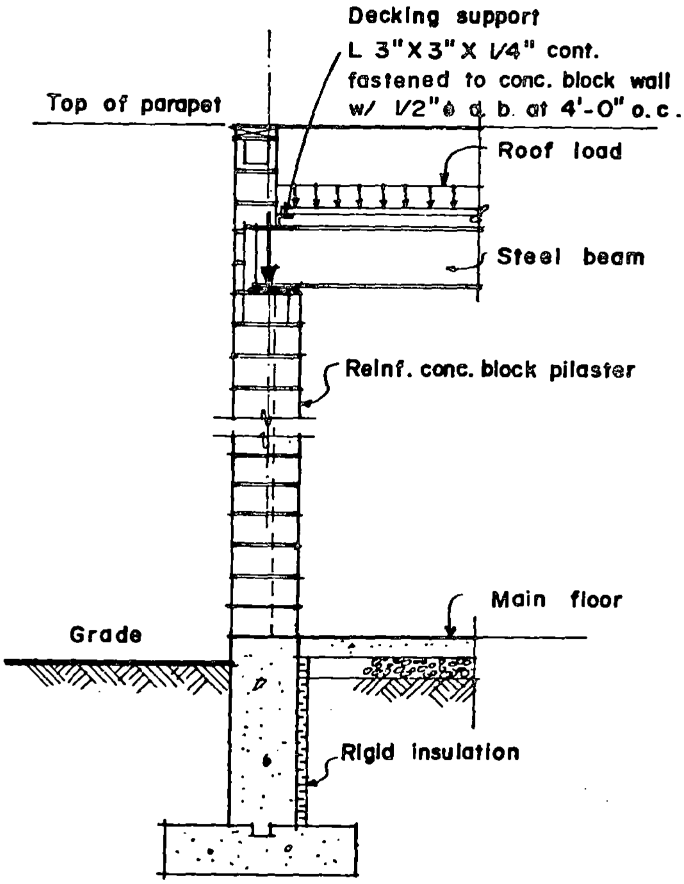


FIGURE 9 END PILASTER SUPPORTING STEEL BEAMS

adjacent masonry. Lintels can be constructed with a number of different materials and combinations of materials.
Lintels can be of reinforced concrete masonry construction, of precast or cast-in-place concrete, or of structural steel. Only masonry lintels are discussed here because of their advantages for use in masonry buildings. These advantages include a) Efficient use of material(reinforcing steel);
b) Lower maintenance (painting is eliminated); c) Fire proofing; d) Elimination of cracking due to differential movements; e) Better bond between lintel and masonry support.

Reinforced concrete masonry lintels may be built with lintel blocks (special units), bond beam masonry units or standard units with depressed, cut out or grooved webs. Lintels may also be cast-in-place using reinforced concrete techniques. However, shrinkage and surface texture characteristics make this particular application not very attractive.

Lintels may also be prefabricated and placed over the openings. These lintels may be of concrete masonry construction of prefabricated cast concrete, with spectal provisions to achieve a) continuity; b) similar surface texture and appearance.

2) Loads

The amount of reinforcements required to resist the superimposed loads is dependent on the type of loading and the position of these loads.
Two types of loads are encountered:
a) uniformly distributed loads or loads that can be assumed uniformly distributed.

b) Concentrated loads.

The first category includes the dead weight of the lintel, the masonry above the lintel, and the roof load, if directly supported on the masonry. The second category includes loads from beams, joists, and rafters. However, if the distance from the lintel to the bearing of rafter or joists is more than 1/3 of the joist or rafter spacing then these loads are assured uniformly distributed. Once the loads are established, the procedure to be used in designing the reinforcement required is demonstrated in the following example.

3} Design of lintel section 4 Figure 10

Loads :

roof	43 x 3	= 129 lb/ft
self-weight*	= 235 + 170	= 405 lb/ft
Total	534 lb/ft.	

Moment

$$M = \frac{WL^2}{8} = 534 \times \frac{17^2}{8} = 19290 \text{ ft-lb.}$$

Note: effective length = clear space + bearing (center to center bearing)

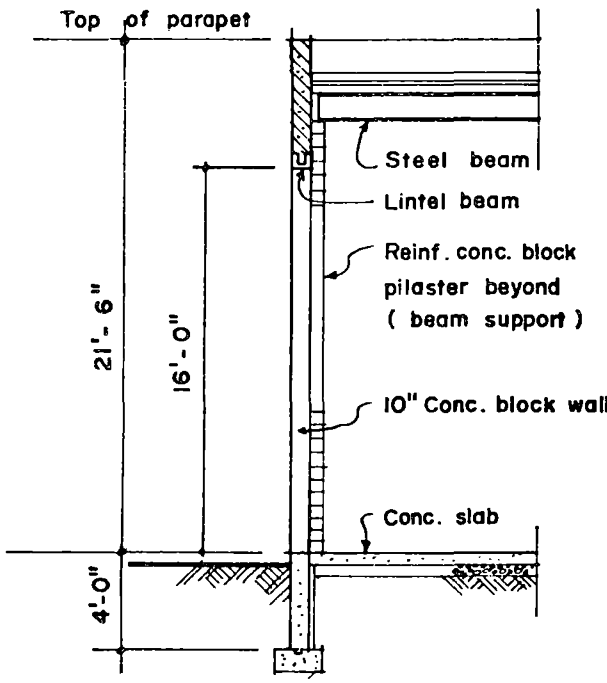
For balance conditions

$$K = \frac{1}{1 + \frac{f_s}{\eta f_m}} = \frac{1}{1 + \frac{24000}{30 \times 0.33 \times 1350}} = 0.3574$$

$$j = 1 - \frac{k}{3} = 0.8809$$

$$K = \frac{1}{2} f_m k_j = \frac{1}{2} \times 0.33 \times 1350 \times 0.3574 \times 0.8809 = 70.05$$

* based on 3 blocks filled with concrete and the rest unfilled.



SECTION ④

FIGURE 10 LINTEL BEAM FOR OVERHEAD DOOR OPENING

$$d = \sqrt{\frac{M}{Kt}} = \sqrt{\frac{231489}{70.05 \times 9.625}} = 18.5 \text{ in}$$

Three blocks must be filled

$$A_s = \frac{M}{f_s\, jd} = \frac{19290 \times 12}{24000 \times 0.8809 \times 20} = 0.55 \text{ in}^2$$

use 2 - 15M bars (60 ksi)

Note that the depth calculated should be increased by 4.5 inches for cover. (2.5 inches grout cover plus the block thickness).

Minimum steel required

$$A_s = \frac{80}{f_y}\, bd \; = \text{Design for Shear} \; \frac{80}{60000}\, bd = 0.32 \text{ in}^2 < 0.55 \text{ in}^2 \;\therefore\; \text{ok}$$

Shear at face of support (assuming 8" bearing length)

$$V = \frac{1}{2} WL - \frac{W}{3} = \frac{1}{2}\, 534 \times 17 - \frac{1}{3}\, 534 = 4361 \text{ lb.}$$

$$\text{allowable shear} = 0.02\, f'_m\, bd = 0.02 \times 1490 \times 9.625 \times 20$$

$$= 5736 > 4361 \text{ lb.}$$

Therefore no stirrups required. If stirrups were needed they should be designed to carry the total shear.

Design of Lintel Through Section 5 Figure 11

The loads on this beam result from the reactions of two steel joist positioned approximately at third point. These reactions are 3225 lb/joist. The weight of the lintel beam plus the masonry above is estimated at 405 lb/ft.

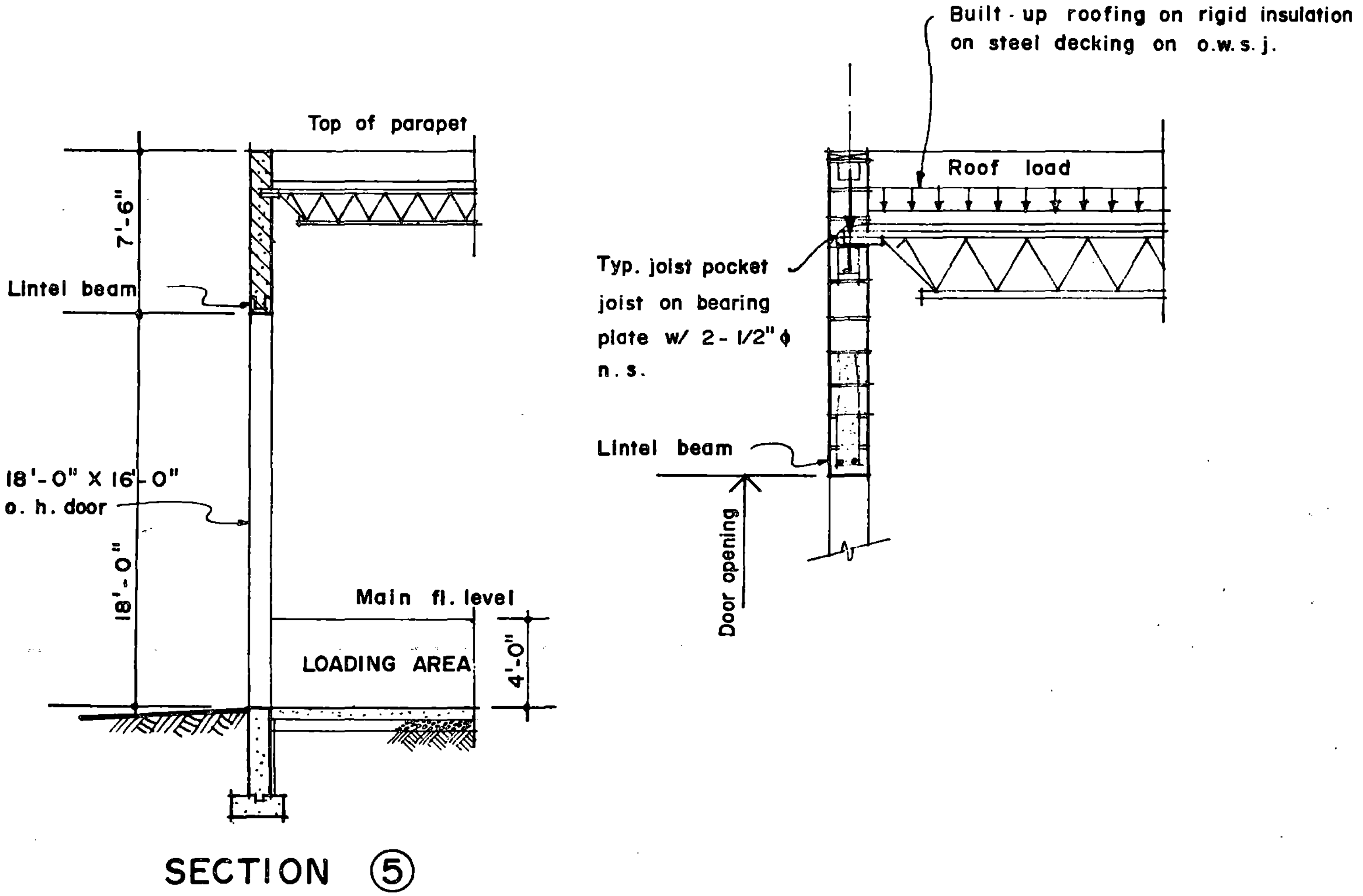


FIGURE 11 LINTEL BEAM DETAIL

The joists are framed 4.30 feet above the lintel opening which is more than 1/3 of the joist spacing. As a result these loads can be assumed as uniformly distributed. However, for this example the analysis will be made with

these loads as point loads. Arching is not considered because of the relatively shallow beam.

$$[(2 \times 4.3 \cos 45) < 16.5"]$$

Moment

$$M = \frac{WL^2}{8} + \frac{1}{3} P L = 405 \times \frac{17^2}{8} + \frac{1}{3} \times 322517$$

$$= 14630 + 18275 = 32905 \text{ ft-lb.}$$

For balance conditions

$$k = 0.3574 \quad j = 0.8809 \quad K = 70.05$$

$$d = \sqrt{\frac{M}{Kt}} = \sqrt{\frac{32905 \times 12}{70.05 \times 9.625}} = 24.2 \text{ in.}$$

In this case four blocks must be filled as grout

$$A_s = \frac{M}{f_s jd} = \frac{32905 \times 12}{24000 \times 0.8809 \times 28} = 0.67 \text{ in}^2$$

Use 2 - 20M bars

Design for shear

$$V = \frac{1}{2} WL - \frac{w}{3} + P = \frac{1}{2} 405 \times 17 - \frac{1}{3} (405) + 3225$$

$$= 6532 \text{ lb.}$$

Allowable shear $= 0.02 \ f'_m \ bd = 0.02 \times 1490 \times 9.625 \times 28 = 8031$

> 6532 therefore no stirrups required

Details for lintel beams located at the two storey part of the building are shown in Figure 12.

4) Check for overstressing at openings. Assume that one core is filled on each side of the opening and reinforced

with 2 - 15M re-bars.

The reaction from the lintel beam is:

$$\frac{1}{2}\,WL = \frac{1}{2}\,534 \times 16 = 4272 \text{ lb.}$$

The allowable bearing load is

$$0.25\, f'_m \times 9.625 \times 8.0 = 0.25 \times 1490 \times 9.625 \times 8.0$$
$$= 28682 \text{ lb} > 4272 \text{ lb.}$$

Similar checks should be made in all openings.

Bearing Stresses

Where concentrated loads are applied a check should be made to ensure that the masonry is not overstressed.

Such conditions exist at the support of beams, joists, etc. The load resulting from the joist bearing on the wall is 3225 lb. To avoid overstressing the masonry this load must

be spread over an area of $3225/0.25\, f_m' = 9.56 \text{ in}^2$ Note that

the allowable bearing stress is different than the compressive or flexural allowable stress.

Similar checks should be carried out for the pilasters. Typical connection of O.W.S.j. on concrete block walls is shown in Figure 13. Figure 14 shows a typical connection of O.W.S.j beams on steel beams.

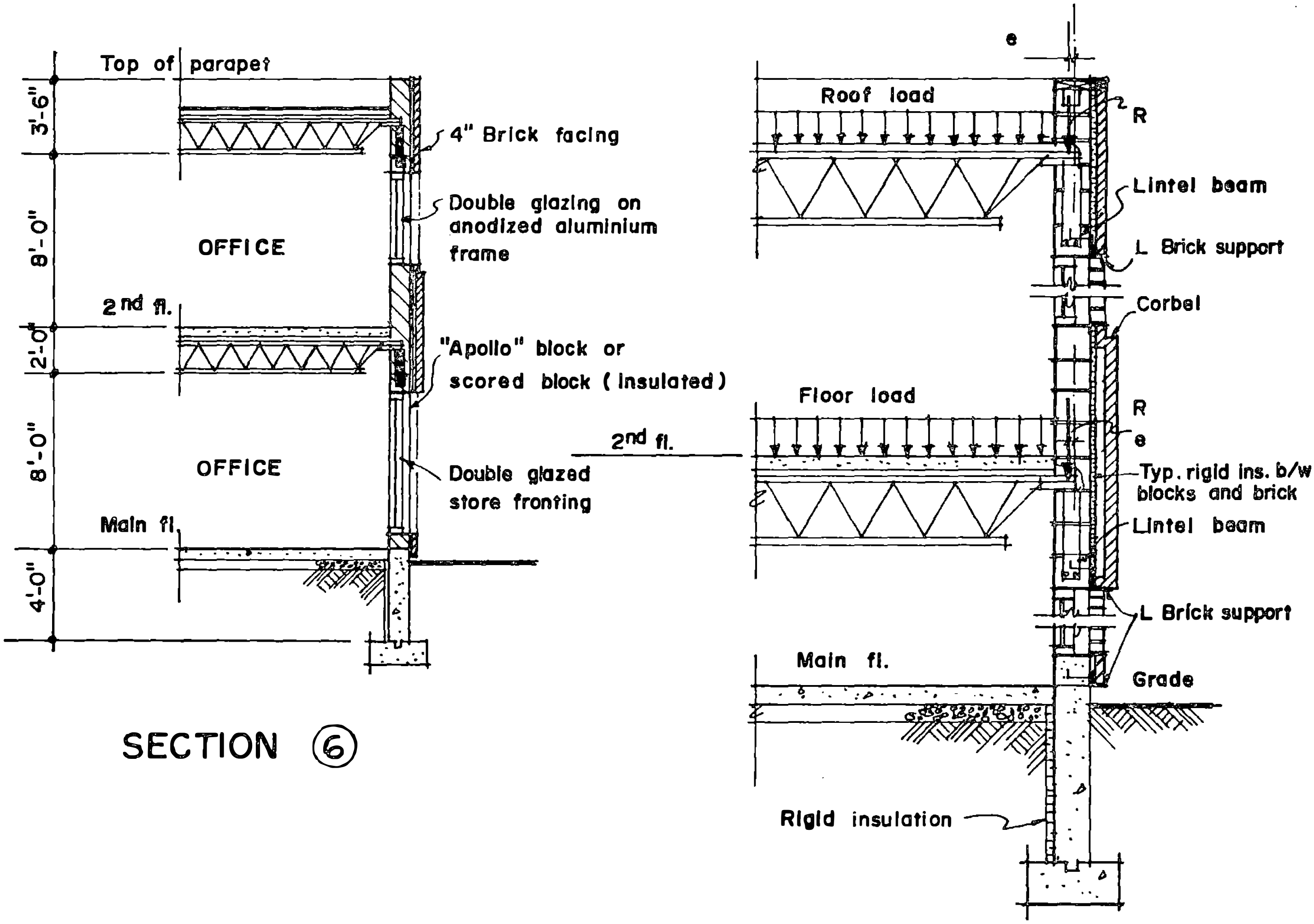


FIGURE 12 TYPICAL DETAILS OF OPENINGS AT THE TWO STOREY PORTION OF THE BUILDING

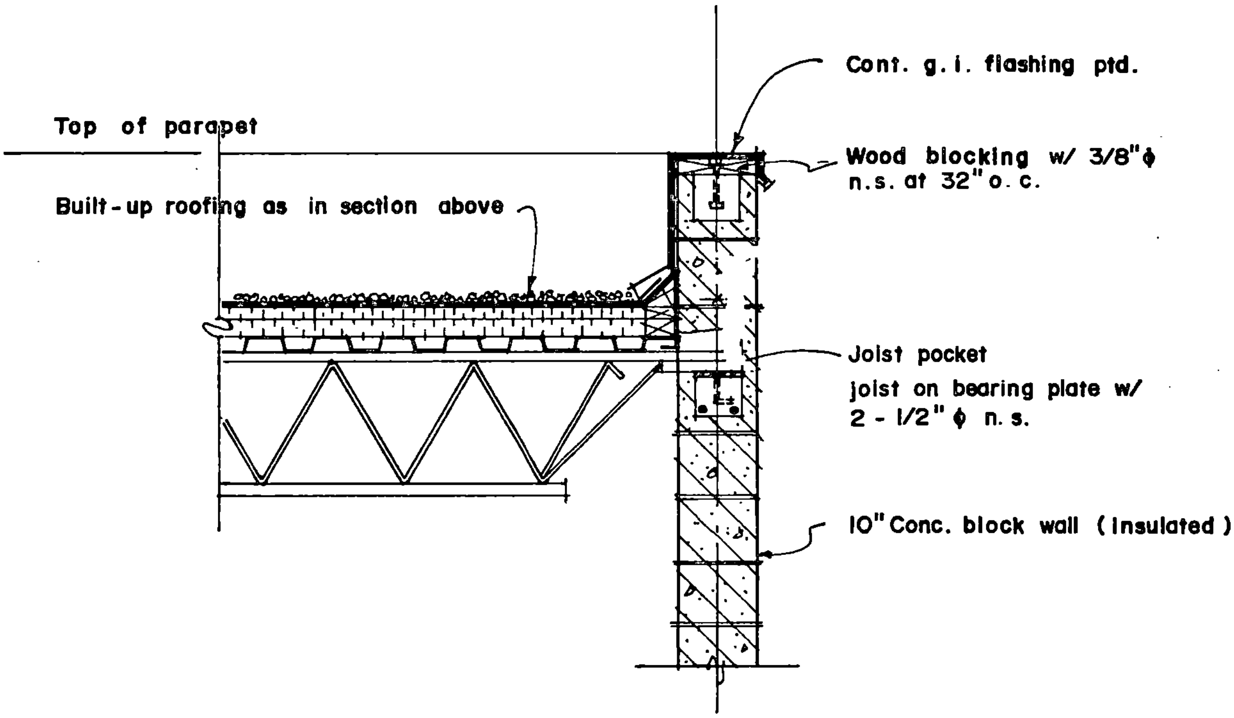


FIGURE 13 TYP. CONNECTION
O.W.S.J. BEARING ON CONC. BLOCK WALL

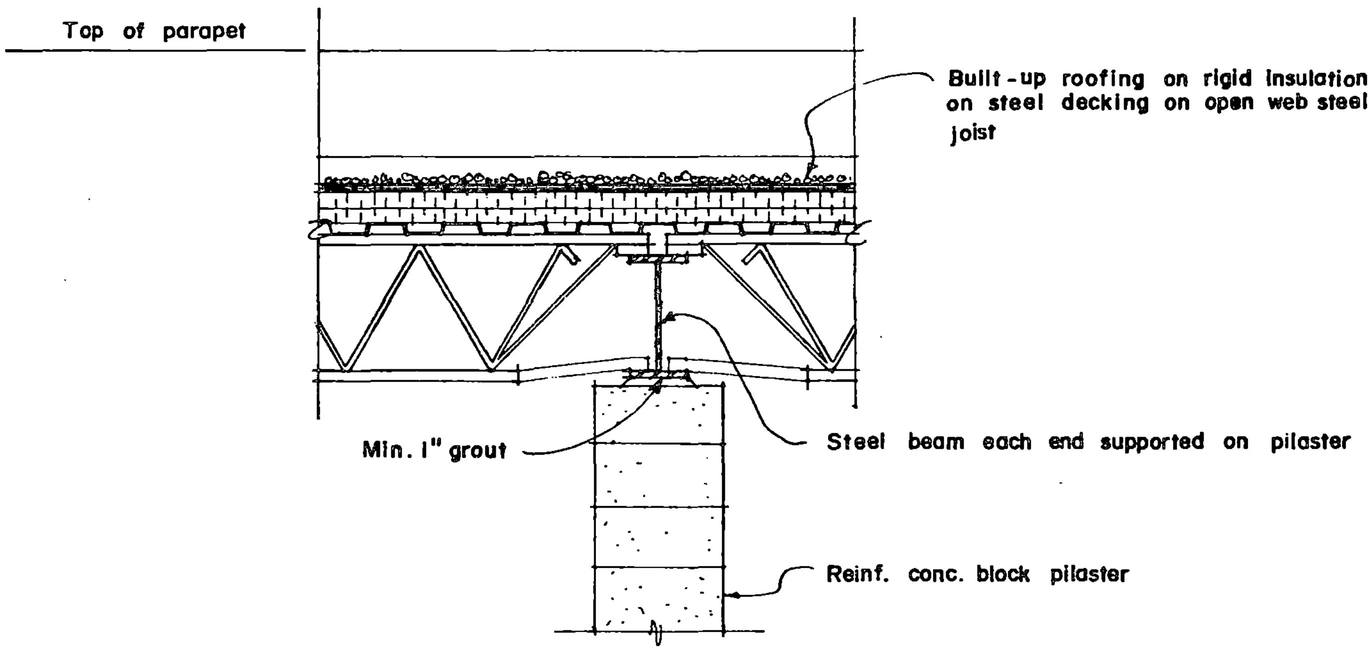


FIGURE 14 TYP. CONNECTION
O.W.S.J. BEARING ON STEEL BEAM

Design for Lateral Loads

a) Wind

The total lateral load resulting from the wind pres-sure is evaluated by multiplying the wind pressure per square foot with the largest exposed surface of the building. The larger area exposed is

$$150 \times 21.5 = 3225 \text{ ft}^2$$

The wind pressure per square foot is

$$P_w = q\ C_e\ C_g\ C_p = 8.3 \times 1 \times 2\ (0.7 + 0.5) = 20 \text{ lb-ft}^2$$

The total wind load is

$$P = 3225 \times 20.0 = 64500 \text{ lb.}$$

If it is assumed that the building is supported only at the ends then the wind induced moment at the roof level is

$$M = \frac{WL^2}{8} = (200 \times \frac{h}{2})\ 150^2 \times \frac{1}{8} = (20.0 \times 11.5) \times 150^2 \times \frac{1}{8}$$

$$= 646873 \text{ ft-lb.}$$

This action is illustrated in Figure 15.

The force required to resist this moment is

$$F = \frac{M}{d} = \frac{646873}{54} = 11979 \text{ lb.}$$

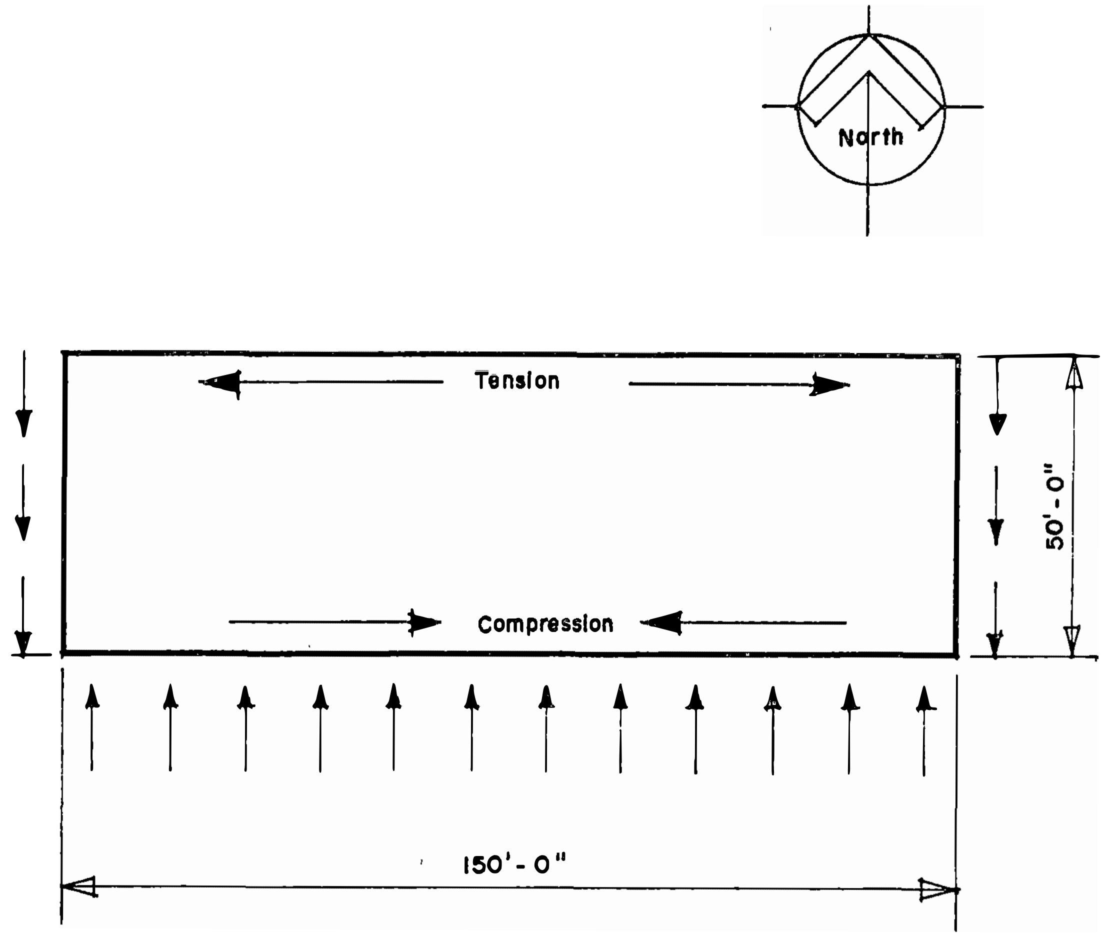


FIGURE 15 IDEALIZED DIAPHRAMOUS ACTION

Tension steel must be provided at the bond beams to develop this force

$$A_s = \frac{T}{f_s} = \frac{11979}{24000} = 0.50 \text{ in}^2$$

Use 2 - 15M bars at the bond beam. The area required to resist the compressive load is

$$A_\eta = \frac{11979}{0.225 f_m} = \frac{11979}{335} \simeq 36 \text{ in}^2$$

therefore fill one block.

b) Earthquake Force

Assume that the building is in zone 2

$$V = ASKIFW$$

$$A = 0.04$$

$$S = \frac{0.5}{\sqrt[3]{T}}$$

$$T = \frac{0.5h}{\sqrt{D}}$$

$$K = 1.3$$

$$I = 1.0$$

$$F = 1.3$$

W = weight of structure

$= (300 + 100 + 50)\ 21.5 \times 50 + 50 \times 150 \times 23 + 50 \times 50 \times 48$

$= 483750 + 172500 + 120000$

$= 776250$ lb.

Seismic force acting on building

$$V = 0.04 \left(\frac{0.5}{\sqrt[3]{0.12}}\right) (1.3) \times 1 \times 1.3 \times 776250$$

$$= 52474 \text{ lb} < \text{wind load}$$

Therefore wind governs.

*Note that if the building is located in an earthquake zone minimum reinforcement is required.

Distribution of Lateral Load to Shear Walls

For the purpose of this example it is assumed that the roof diaphram is semirigid and we further assume that torsional forces are distributed to the shear walls in direct proportion to their rigidities and their distance from the center of rigidity.

The shear and moment deflection for cantilever pier is given by

$$\Delta_c = \frac{P}{E_{mt}} \left[4 \left(\frac{H}{d}\right)^3 + 3\left(\frac{H}{d}\right)\right]$$

For fixed piers the shear and moment deflection is given by

$$\Delta_F = \frac{P}{E_{mt}} \left[\left(\frac{H}{d}\right)^3 + 3\left(\frac{H}{d}\right)\right]$$

Assume that the wind load is applied in the north direction. The load will be resisted by three main walls. Neglecting the effect of the openings in the rigidity of these elements, then their relative stiffness is the same. The center of rigidity of the building is:

$$\bar{X} = \frac{1}{3}(125 + 150) \simeq 92.0 \text{ ft}$$ from the west wall.

(see Figure 16).

The torsional moment is

$$(92 - \frac{1}{2}150)\ \frac{1}{2}(64500) = 1870500 \text{ ft-lb.}$$

Note that if the load is applied at the center of rigidity all three walls would carry the same shear equal to 1/3 of the total wind load (applied to the diaphram which is 1/2 of the total wind' load). The direct shear on each wall is

$$\frac{1}{3} \times \frac{1}{2} \times 64500 = 10750$$

The additional shear resulting from the torsional moment is evaluated as follows

$$F_B = \frac{33}{92} F_A = 0.36\ F_A$$

$$F_C = \frac{58}{92} F_A = 0.63\ F_A$$

$$(0.36\ F_A)\ 33 + (0.63\ F_A) \times 58 + F_A\ 92 = 1870500 \text{ ft-lb.}$$

$$11.88\ F_A + 36.54\ F_A + 92\ F_A = 1870500 \text{ ft-lb.}$$

$$104.42\ F_A = 1870500$$

$$F_A = 17913 \text{ lb}$$

$$F_B = 6448 \text{ lb}$$

$$F_C = 11285 \text{ lb}$$

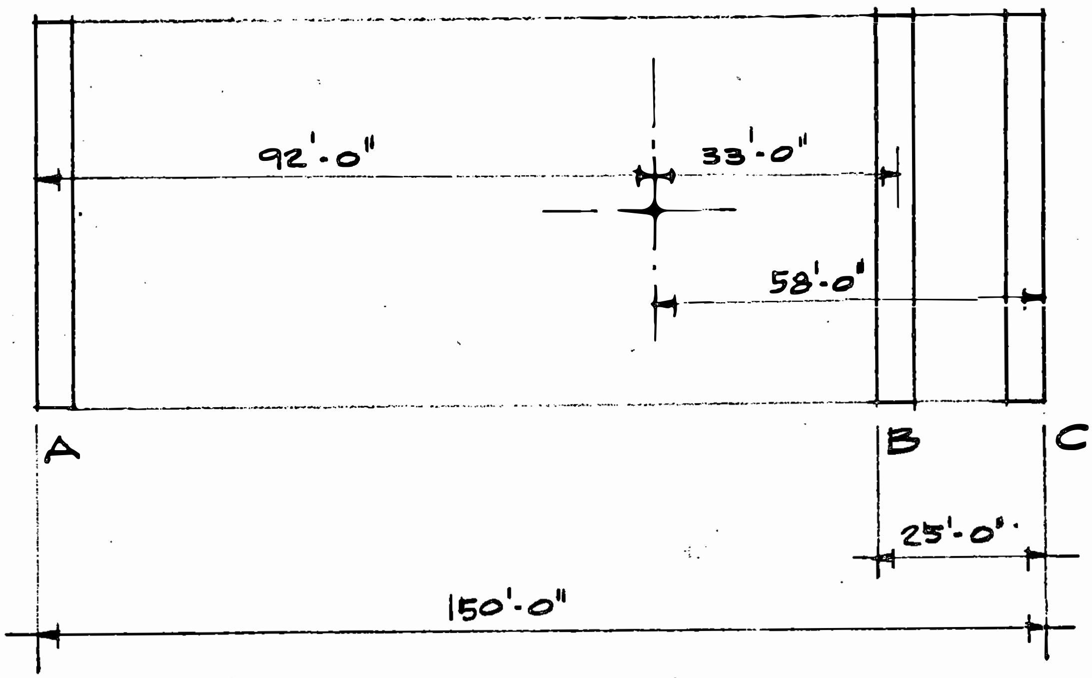


FIGURE 16 CENTER OF RIGIDITY OF THE STRUCTURE

The total shear on the walls is:

$$*F_{At} = 10750 + 17913 = 28663 \text{ lb}$$

$$F_{Bt} = 10750 + 6448 = 17198 \text{ lb.}$$

$$F_{Ct} = 10750 + 11285 = 22035 \text{ lb.}$$

Note that all torsional forces are added to the direct shear in order to account for load application from either direction.

Now that we have found the approximate shear loads on these walls we must check for capacity at the application point and also at the openings.

Consider Wall A (Figure 16). This wall, as can be seen from Figure 4 (west elevation), consists of three main elements. The element above the overhead opening and two piers.

The shear capacity at the roof level is evaluated using the following relation:

$$V = V_m + 0.3 \text{ fcs}$$

where V_m = allowable shear stress = 34 psi

fcs = compressive stress

Considering only self weight at the critical section fcs is:

$$\text{fcs} = \frac{3 \times 35 + 60}{2 \times 1.5 \times 12} = 4.6 \text{ psi}$$

The total capacity at this level is

*Note that this shear must be resisted by the diaphram and heavier gauge of roof membrane might be needed.

$$V_t = (34 + 0.3 \times 4.6)\ A_m = 35.4\ (2 \times 1.5 \times 50 \times 12)$$

$$= 63720 \text{ lb} > 28663 \text{ lb.}$$

At the openings the lateral load will be distributed to the piers in accordance with relative stiffness for fixed conditions.

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{H}{d}\right)^3 + 3\left(\frac{H}{d}\right)\right]$$

For pier 1 $\frac{H}{d} = \frac{16.0}{6.75} = 2.37$

2 $\frac{H}{d} = \frac{16}{27} = 0.59$

$$\Delta F_1 = \frac{P}{E_m t}\ [13.31 + 7.11] = \frac{P}{E_m t} \times 20.47$$

$$\Delta F_2 = \frac{P}{E_m t}\ [0.21+1.77] = \frac{P}{E_m t} \times 1.77$$

$$P_1 + P_2 = 28663 \text{ lb.}$$

$$P_1 = \frac{1.77}{22.19}\ P_2 = 0.0798\ P_2$$

Therefore

$$P_2 = 28663 \times \frac{1}{1.0798} = 26544 \text{ lb.}$$

$$P_1 = 2119 \text{ lb.}$$

The capacity of Pier 2 at the level of the lintel beams is

$$V_t = (34 + 0.3\ fcs)\ [(2 \times 1.3)\ 27 \times 12 + (4 \times 9.625 \times 15.625)]$$

for $fcs = 7.5$ psi

$$V_t = (36.3)(972 + 600) = 57063 \text{ lb} > 28.663 \quad \text{ok}$$

Note that the blocks which are filled with concrete at the support of the lintel and the wall are taken into account to increase the shear resisting area.

For Pier 1 the capacity is

$$V_t = 36.3 \; [(2 \times 1.5 \times 6.7 \times 12) + 2(9.625 \times 15.625)]$$

$$= 36.3 \; [364 + 300] = 24163 > 2119$$

therefore adequate.

Control Joints and Joint Reinforcement

It is well known that building materials are subject to movement.

These movements are the result of changes in tempera-ture, changes in the moisture content, contraction due to carbonation and movements of other parts of the structure
(foundation settlements, deflections of beams, elastic and creep, deformation of columns, etc.).

When masonry units are bonded together by mortar to form a wall, any restraint that will prevent the wall from expanding or contracting freely will set up stresses within the wall.

If a concrete block wall is restrained against expansion it will result in low compressive stresses (in relation to the compressive strength) and rarely causes damage to the walls.

This expansion is offset by shrinkage from carbona-tion and drying of the joints. As a result expansion joints are not necessary in concrete masonry except where required by the configuration of the building. This is true where there is not brick facing attached to the concrete wall.
In the case where concrete block wall is used as backup for clay brick then expansion joints are required to accommodate the brick movements.

When contraction of the concrete masonry units is prevented, tensile stresses build up gradually within the wall. When the tensile stresses exceed the tensile strength of the unit, or the bond between the unit and the mortar joint, cracks will occur and the stresses will be relieved.

These types of cracks usually disfigure the wall and cannot be easily concealed. The stability of the wall is also affected and caulking is required to avoid water penetration.

Such cracks can be prevented by control joints.

Control joints are continuous, vertically weakened sections built into the wall. If the stresses or wall move-ments are sufficient to crack the wall, the cracks will occur at the control joint and thus be inconspicuous. A control joint must permit ready movement of the wall in a longitudinal direction and be sealed against vision, sound and weather.

The control joint must also, in addition to the above, be required at times to stabilize the wall laterally across the joint by means of shear.

Location of Control Joints

Various rules for locating control joints have been developed from experience and will probably continue to be refined. Since there are many possible layouts of walls and partitions with their openings for doors, windows, and ducts, some judgment must be used in determining where the joints should be built.

ACI recommendations for spacing of control joints are given in Table 1. In earthquake regions the U.S. Army, Navy, and Air Force limit the maximum spacing between control joints in reinforced masonry construction to 50 ft and the distance of a joint from a corner to 25 ft.

Control joints should also be located at the following points of weakness or high stress concentrations:

1. At all abrupt changes in wall height.
2. At all changes in wall thickness, such as those at pipe or duct chases and those adjacent to columns or pilasters.
3. Above joints in foundations and floors.
4. Below joints in roofs and floors that bear on the wall.
5. At a distance of not over one-half the allowable joint spacing from bonded intersections or corners.
6. At one or both sides of all door and window openings unless other crack control measures are used, such as joint reinforcement or bond beams.

All large openings in walls should be recognized as natural and desirable joint locations. Although some adjustment in the established joint pattern may be required, it is effective to use vertical sides of wall openings as part of the control joint layout. Under windows the joints usually are in line with the sides of the openings. Above doors and windows the joints must be offset to the end of the lintels. To permit movement, the bearing of at least one end of the lintel should be built to slide.

When a concrete masonry wall is reduced in thickness across the face of a column, a control joint should be placed along one or both sides of the column. Thin concrete masonry across the column face should be tied to the column by means of dovetail anchors or another suitable device.

Where bond beams are provided only for crack control, control joints should extend through them. If there is a structural reason for a bond beam, a dummy groove or raked joint should be provided to control the location of the anticipated crack.

A concrete masonry or cast-in-place concrete founda-tion having both sides backfilled does not usually require control joints. However, long concrete masonry basement walls may require control joints, continuous metal ties (joint reinforcement), or reinforcing bars.

Where concrete masonry units are used as a backup for another material with masonry bond, the control joints should extend through the facing. Control joints need not extend through the facing when using flexible bonds (metal ties) .

Control joints should extend through plaster applied directly to concrete masonry units. Plaster applied on lath that is furred out from concrete masonry requires control joints over previous joints in the base.

Joint Reinforcement

Although concrete masonry walls can be built essenti-ally free of cracks, it is the infrequent crack for which joint reinforcement (continuous metal ties) is provided.

The function of joint reinforcement is not to eliminate cracking in concrete masonry walls but merely to prevent the formation of conspicuous shrinkage cracks. Joint rein-forcement does not become effective until the concrete masonry begins to crack. At this time the stresses are transferred to and redistributed by the steel. The result is evenly distributed, very fine cracks hardly visible to the naked eye.

The effectiveness of joint reinforcement depends on the type of mortar and the bond between the mortar and the longitudinal wires. The better the bond strength, the more efficient the reinforcement in arresting any cracks. In-service experience has shown that only Types M, S, and N mortar should be considered for use with joint reinforcement.

TABLE 1 MAXIMUM SPACING OF CONTROL JOINTS

Maximum spacing of joint reinforcement, in.	Maximum spacing of control joints	
	Panel length/height	Panel length, ft.
None	2	40
24	2.5	45
16	3	50
8	4	60

Note that for stack bond joint,reinforcement should be provided every 16" on centre.

After the joint reinforcement is placed on top of the bare masonry course, the mortar is applied to cover the face shells and joint reinforcement. Minimum recom-mended mortar cover of the wire is 5/8 in. for the exterior wall face and 1/2 in. for the interior face.

Prefabricated or job-fabricated corner and T-type joint reinforcement should be used around corners and to anchor abutting walls and partitions. Prefabricated corners and tees are considered superior because they are more accurately formed, fully welded, and easier to install.

A 6-in. lapping of side wires at splices is essential to the continuity of the reinforcement so that tensile stress will be transmitted.

As can be seen in Table 1 the vertical spacing of joint reinforcement is interdependent with the spacing of control joints. In addition, joint reinforcement should be located as follows:

1. In the first and second bed joints immediately above and below wall openings. The reinforcement should extend not less than 24 in. past either side of the opening or to the end of the panel, whichever is less.
2. In the first two or three bed joints above floor level, below roof level, and near the top of the wall.

Joint reinforcement need not be located closer to a bond beam than 24 in. It should not extend through control joints unless specifically called for and detailed in the plans.

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