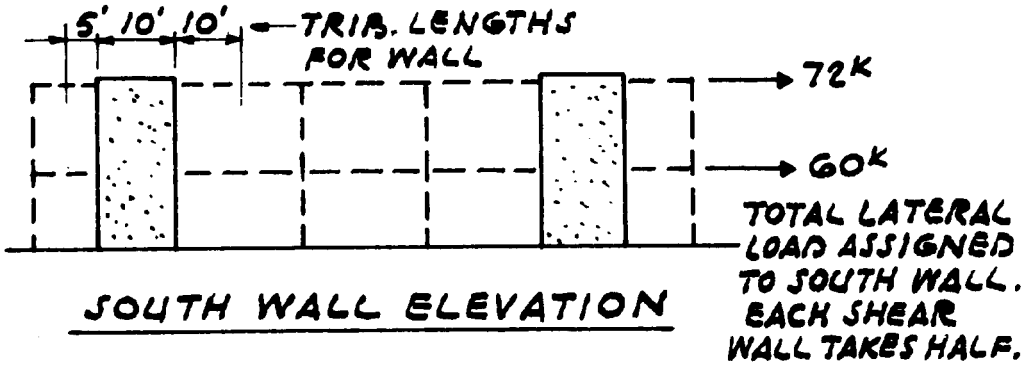
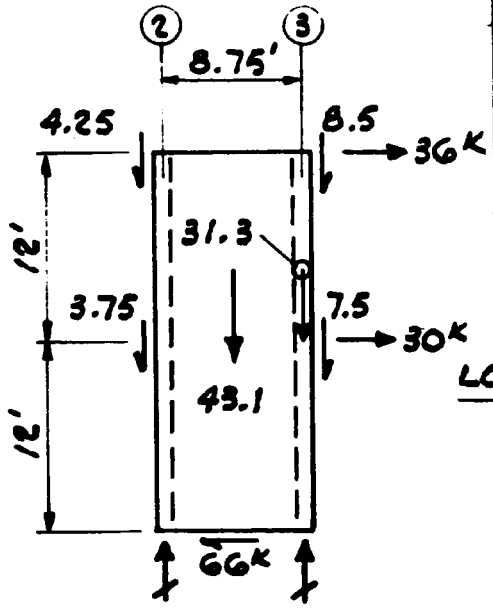
SHEAR WALL ANALYSIS

SHEAR WALLS ARE DESIGNED FOR 100% OF THE TOTAL REQUIRED LATERAL FORCE. DESIGN FORCES FOR THE SOUTH WALL ARE SHOWN.





SOUTH WALL BEAM REACTIONS		
FLAT LOAD, EDGE BEAM & WALL	LEFT 5' TRIB.	RIGHT 10' TRIB.
ROOF:		
DL @ 0.85 K/'	4.25	8.5
LL @ 0.10	0.50	1.0
FLOOR:		
DL @ 0.75 K/'	3.75	7.5
LL @ 0.15	0.75	1.5

LOAD TRIBUTARY DIRECTLY TO WALL

ROOF 0.85×10 = 8.5
FLOOR 0.75×10 = 7.5
WALL 10×24×0.113 = 27.1
TOTAL DL 43.1K
FLOOR LL 0.15×10 = 1.5K

LINE 3 GIRDER REACTION

ROOF + FLOOR DL = 31.3K
FLOOR LL = 6.0K
(FROM TRIB. AREA TO GIRDER C-3 - D-3 CALCULATION NOT SHOWN)

8.0	16.0	
21.6	21.6	
	31.3	
29.6	68.9	VERT. DL
∓140K	±140K	SEISMIC (P. 5)

Figure D-4. Continued.

SHEAR WALL ANALYSIS - CONT'D

OVERTURNING

$$M_{OT} = (36^K \times 24') + (30 \times 12) = 1224^{K'}$$

$$F_{OT} = \pm \frac{1224}{8.75} = \pm 140^K$$

UPLIFT

CALCULATE MAX. NET UPLIFT USING

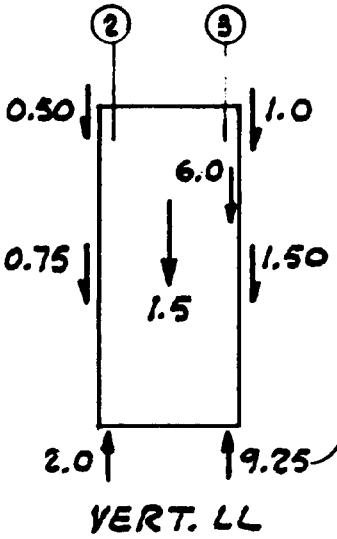
$P = 0.9D - 1.4E$

$= 0.9(29.6) - 1.4\ (140) = -169^K$

THIS COULD BE REDUCED BY INCLUDING FOUNDATION DEAD LOAD, OR BY WIDENING THE WALL (IF ARCHITECTURALLY ACCEPTABLE).

THE NET UPLIFT FORCES COULD BE TAKEN BY DRILLED PIERS OR ROCK ANCHORS IF APPROPRIATE.

MAX. COMPRESSION (LINE 3)



$P = 1.4\ (D + L + E)$

VERT. DL	68.9	P.4
VERT. LL	9.3	P.5
SEISMIC	140.0	P.4
	218.2	

$P = 1.4\ (218.2) = 305^K$

APPLY TO BOUNDARY MEMBER AND FOOTING ON LINE 3

Figure D-4. Continued.

SHEAR WALL DESIGN

WALL SHEAR $V_u = 1.4E$, $\phi = 0.60$

$V_u = 1.4\left(\frac{72+60}{2}\right) = 92.4$ $A_c = 9'' \times 10' \times 12''/_1 = 1080 \text{ IN}^2$

SHEAR CARRIED BY CONC.: $v_u = \frac{92{,}400}{0.60 \times 1080} = 143$ PSI

$v_c = 2\sqrt{f_c'} = 2\sqrt{3000} = 110$ PSI $< 8\sqrt{f_c'}$ OK

$V_c = 0.110 \times 1080 = 119^K$

SHEAR CARRIED BY REINF. $V_u' = \frac{V_u}{\phi} - V_c = \frac{132}{0.85} - 119 = 36.3^K$

$A_v = \frac{V_u' s}{f_{yd}}$ $s = \frac{A_v f_{yd}}{V_u'}$

Try #4 @ 18" CC EACH WAY EACH FACE

REQ'D $s = \frac{2(0.20)(60)(9.25 \times 12)}{36.3} = 73''$

$A_{s\,MIN.} = 0.0025bd = 0.0025 \times 9 \times 12 = 0.27 \text{ IN.}^2/\text{FT.}$

A_s PROVIDED $= \frac{12}{18} \times 2 \times 0.20 = 0.27$ OK

ALLOWABLE SHEARING STRESS $= 2\sqrt{f_c'} + \rho f_y$

$= 110 + (0.0025 \times 60{,}000) = 110 + 150 = 260 > 143$

REQ'D MIN. SPACING $= \frac{d}{3} = \frac{10'}{3}$ OR 18"

OR $3b = 27''$ ACI 11.10.9.5

SHEAR-FRICTION AT CONSTRUCTION JOINT AT BASE

$A_v = \frac{V_u}{\phi f_y \mu} = \frac{92.4}{0.6 \times 60 \times 0.60} = 4.28 \text{ IN.}^2$ ACI 11-7

$4.28 \div 10 = 0.43 \text{ IN}^2/\text{FT.} > 0.27 \text{ IN.}^2/\text{FT}$ N.G.

∴ PROVIDE INTERM. #4 DOWELS @ 18" O.C.

Figure D-4. Continued.

SHEAR WALL DESIGN - CONT'D

VERTICAL BOUNDARY ELEMENT AT LINE 2

Check the requirement for confined boundary elements. Consider, for example, Line 2 of shear wall 2-3 on Line D.

$t = 9'', D = 10'$
$A = tD = 9/12\ (10) = 7.5$ sq. ft.
$S = tD^2/6 = 9/12\ (10)^2/6 = 12.5\ ft^3$
$M = 1.4 \times 1224\ k' = 1714\ k'$
$P = 1.4\ (D + L) = 1.4\ [\ (29.6+68.9) + (2.0+9.25)\ ] = 154^k$
$f_M = M/S = 1714/12.5 = 137$ ksf
$f_R = P/A = 154/7.5 = 20.5$ ksf
$f = f_M + f_R = 157.5$ ksf, or 1094 psi

A confined boundary element is required if $f < 0.2\ f_c'$

$0.2\ f_c' = 0.2\ (3{,}000\ psi) = 600$ psi
$f > 600$, therefore, a boundary element is required.

Design the boundary element per ACI 21.5.3. The need for confinement applies also to the boundary element on Line 3. Special confinement may be discontinued when $f < (0.15\ f_c' = 450$ psi).

Figure D-4. Continued.

SHEAR WALL DESIGN - CONT'D.

VERTICAL BOUNDARY MEMBER - LINE 2

VERTICAL LOADS: 12"x12" COL.

FOR MAX. TENSION,

REQ'D $P_u = 169^k$ (p. 5)

FOR MAX. COMPRESSION

REQ'D $P_u = 1.4(D+L) + 1.4E$

$= 1.4(29.6+2.0) + 1.4(140) = +240^k$

FOR TENSION ON COL. CORE:

$$\frac{P_u}{\phi} = \frac{169}{0.90} = 188^k$$

$$A_s = \frac{188}{60} = 3.13^{\square''}$$

FOR COMPRESSION:

$P_u = 240^k$ $\gamma = \frac{12-4.38}{12} = 0.635$

USE $\gamma = 0.60$

$$\left.\begin{aligned} \frac{P_u}{A_g} &= \frac{240}{12\times12} = 1.67 \\ \frac{e}{h} &= 0.10 \text{ MIN.} \end{aligned}\right\} \text{FROM ACI SP-17A CHART R3-60.60}$$

REQ'D $\rho_g = 0.010$

$A_s = 0.01(12\times12) = 1.44^{\square''} < 3.13$

TENSION CONTROLS:

PROVIDE 4-#8 ($3.16^{\square''}$)

VERTICAL COLUMN CORE REINFORCEMENT

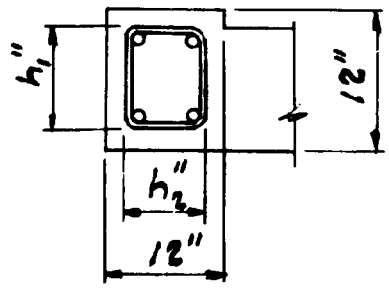
Figure D-4. Continued.

SHEAR WALL DESIGN - CONT'D
VERTICAL BOUNDARY MEMBERS - CONT'D

SPECIAL TRANSVERSE REINFORCEMENT:

FOR #4 HOOPS : $h_{c1} = 12 - 3.0 = 9''$

$A_{SH} = .40 \square''$ $\quad h_{c2} = 12 - 3.0 = 9''$



FROM ACI 21.4.4.1

$$s = \frac{A_{SH}}{0.30\, h_c \frac{f_c'}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1 \right]}$$

$$= \frac{.40}{0.30(9.0)\frac{3}{60}\left[\frac{12 \times 12}{9 \times 9} - 1\right]} = 3.81''$$

OR $$s = \frac{A_{SH}}{0.09 h_c \frac{f_c'}{f_{yh}}} = \frac{.40}{0.09(9)\frac{3}{60}} = 9.88''$$

$S_{MAX} = 3.81''$ $\quad$ ∴ USE #4 HOOPS @3½" O.C. THRU-OUT LENGTH OF COL. CORE

Figure D-4. Continued.

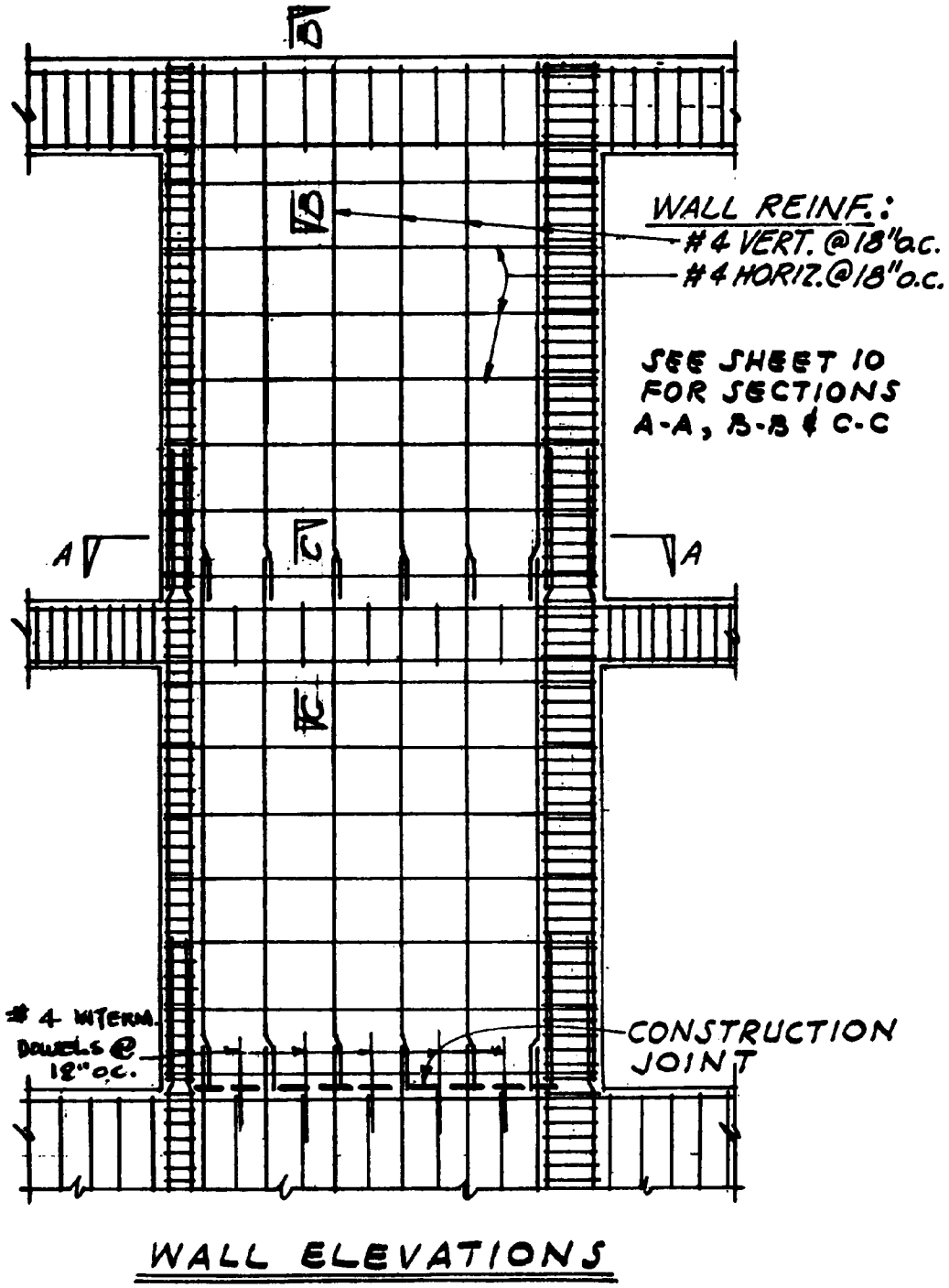


Figure D-4. Continued.

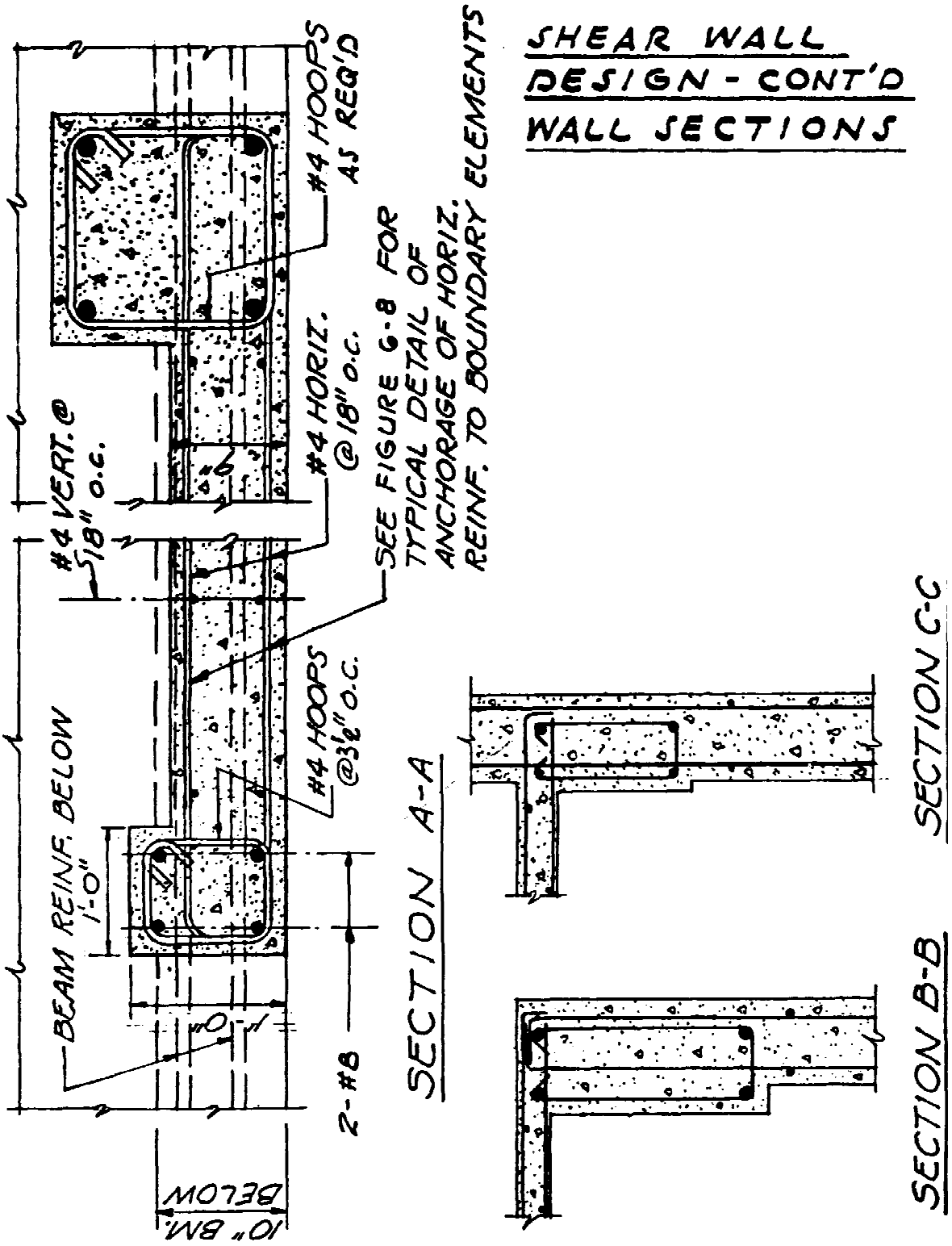
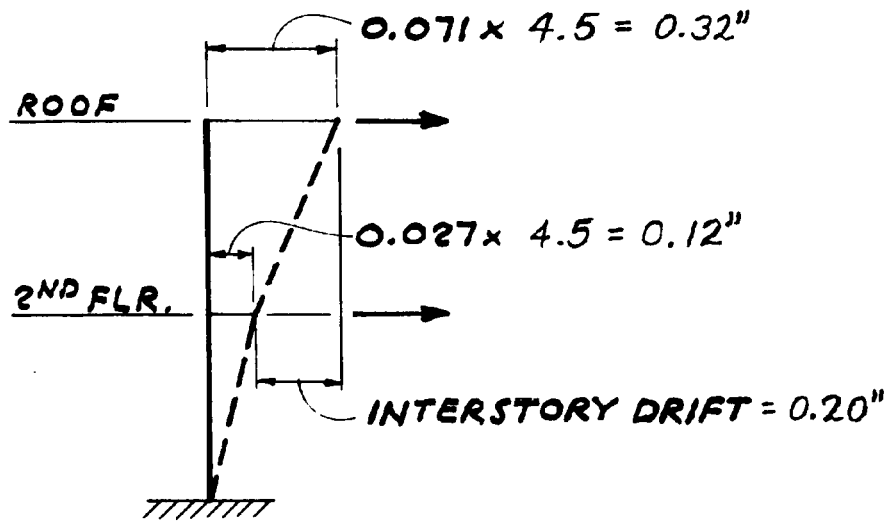


Figure D-4. Continued.

Deformation Compatibility, (3Rw/8) Times Deflection. In this example, the shear walls (with vertical boundary members) on Lines A and D and the frames on Lines B and C are designed to resist the seismic forces. The framing members on Line A and D (other than the shear wall vertical boundary members) are not part of the lateral force resisting system; therefore, they will be investigated for deformation compatibility (SEAOC 1H2d). When the lateral forces shown on page 4 are applied to the structure, the lateral displacement is 0.071 inch at the roof and 0.027 inch at the floor level. The framing members on Lines A and D must be investigated for 3(12)/8 = 4.5. times these displacements. Refer to SEAOC Commentary, p. 42-C. Also, see Design Example D-7, p. 9 and 10.



The resulting member forces are combined with the forces due to vertical gravity loads. In this example, the resulting stresses are within the strength of the members and the P-Δ effects are negligible. Therefore, the requirements for deformation compatibility are satisfied.

Figure D-4. Continued.

DESIGN EXAMPLE D-5

Dual Bracing System With Steel Frame

Description of Structure. A three-story Administration Building in Zone 4 with dual bracing system consisting of a special moment resisting frame in structural steel and concrete shear walls. The structural concept is illustrated on Sheets 2, 3, and 4.

Construction Outline

Roof:
- Built-up, 5-ply
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

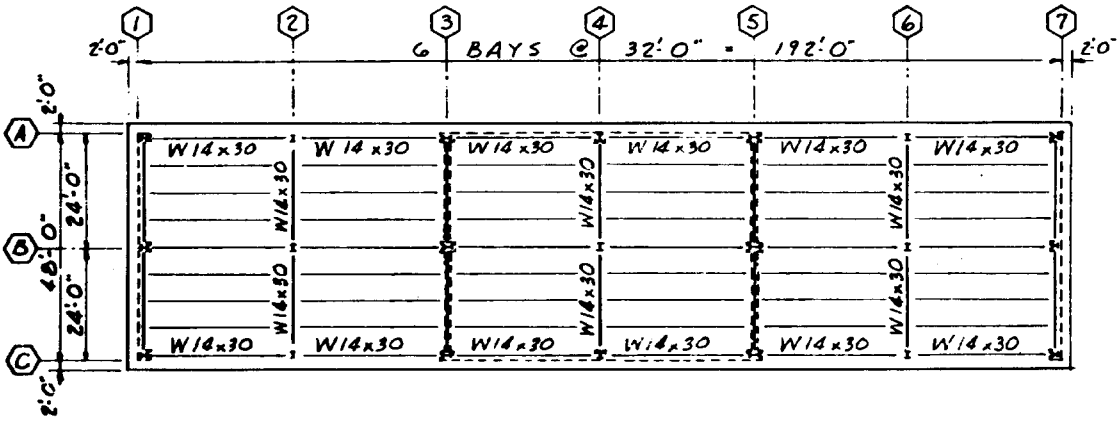
Exterior Walls:
- Bearing Walls in concrete and non-bearing, non-shear insulated metal panels.

Partitions:
- Non-structural removable drywall, except concrete as structurally required.

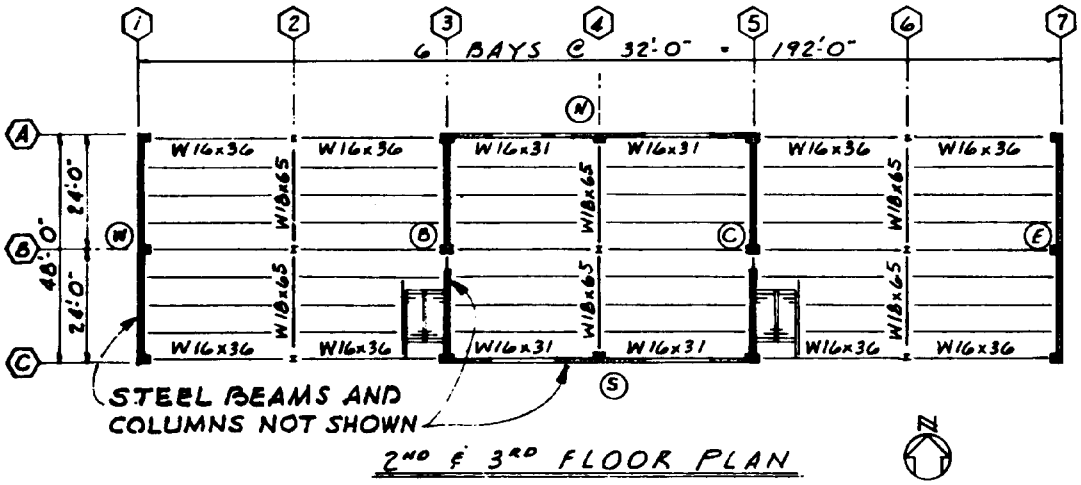
Design Concept. The building has a complete steel space frame capable of carrying all gravity loads. Lateral loads are resisted by a dual system. Concrete shear walls are designed to carry 100% of the prescribed lateral loads. (The transverse walls are on lines B, C, E and W; longitudinal walls are on N and S.) Steel moment frames are designed to carry their share of the lateral loads when acting together with the shear walls, and at least 25% of the prescribed lateral loads when acting alone. (The transverse frames are on Lines 2, 3, and 6; the longitudinal frames are on Lines A and C, with moment connections to the columns on Lines 1, 3, 5, and 7.) For this system, $R_W = 12$. At the roof, the metal deck system forms a flexible diaphragm and the lateral loads are distributed by tributary area; at the floor the concrete-filled metal deck system forms a rigid diaphragm and the loads are distributed by relative rigidities.

Discussion. Calculations are given for the amount of shear to each floor for 100% of the total base shear to the shear walls and 25% of the total base shear to the frames. The distribution of the base shear to the shear walls is not given here; it would follow the procedures of example A-1. The 25% requirement for the frames governs the design of the frames because they have negligible rigidity compared to the walls. The exterior concrete at the shear walls is exposed; the other portions of the exterior walls are covered with insulated steel sandwich panels.

Figure D-5. Dual bracing system.



ROOF PLAN



2ND & 3RD FLOOR PLAN

LOADS.

ROOF:

5 PLY ROOFING	6.0 #/□'
1" INSULATION	1.5
STEEL DECK	2.3
STEEL PURLINS	3.7
STEEL GIRDERS AND COLUMNS	1.2
CEILING	10.0
MISCELLANEOUS	1.0
DEAD LOAD	25.7
ADD FOR SEISMIC LOAD:	
PARTITIONS	10.0
TOTAL FOR SEISMIC =	35.7 #/□'

2ND & 3RD FLOORS:

FINISH	1.0 #/□'
STEEL DECK	3.1
CONCRETE FILL	32.0
STEEL BEAMS	5.9
STEEL GIRDERS AND COLUMNS	1.5
PARTITION	20.0
CEILING	10.0
MISCELLANEOUS	1.0
DEAD LOAD	74.5 #/□'
LIVE LOAD	50.0 #/□'

Figure D-5. Continued.

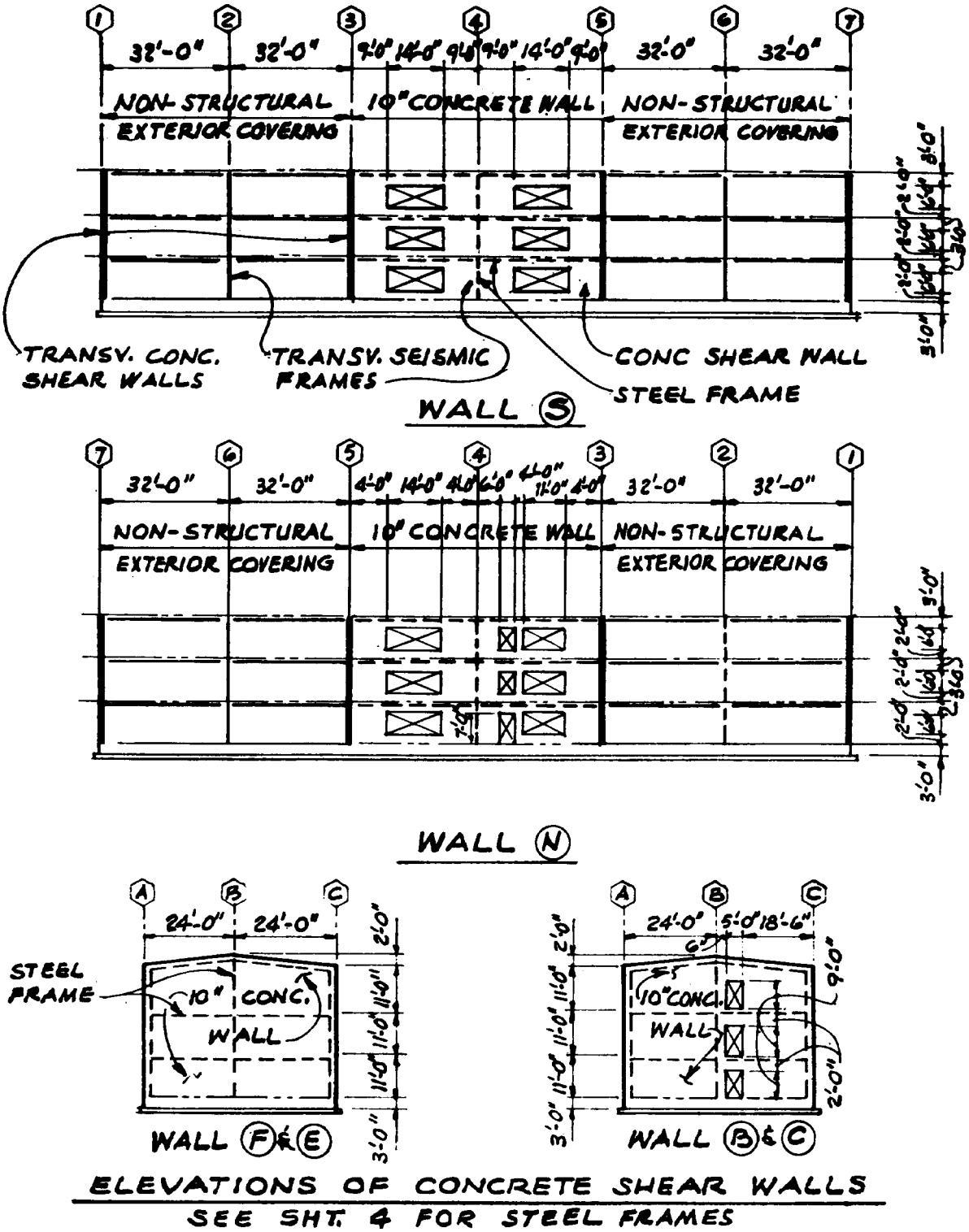
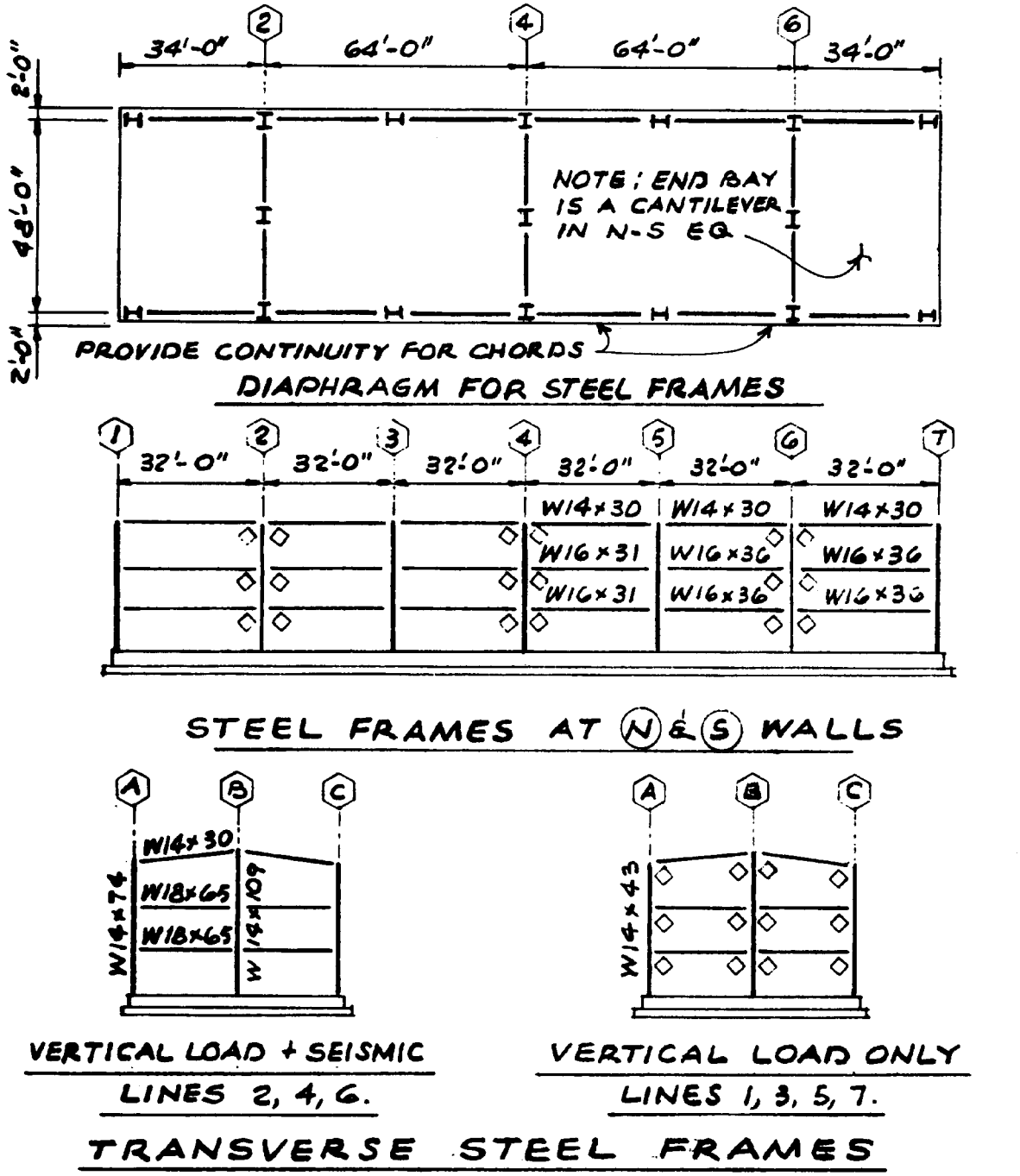


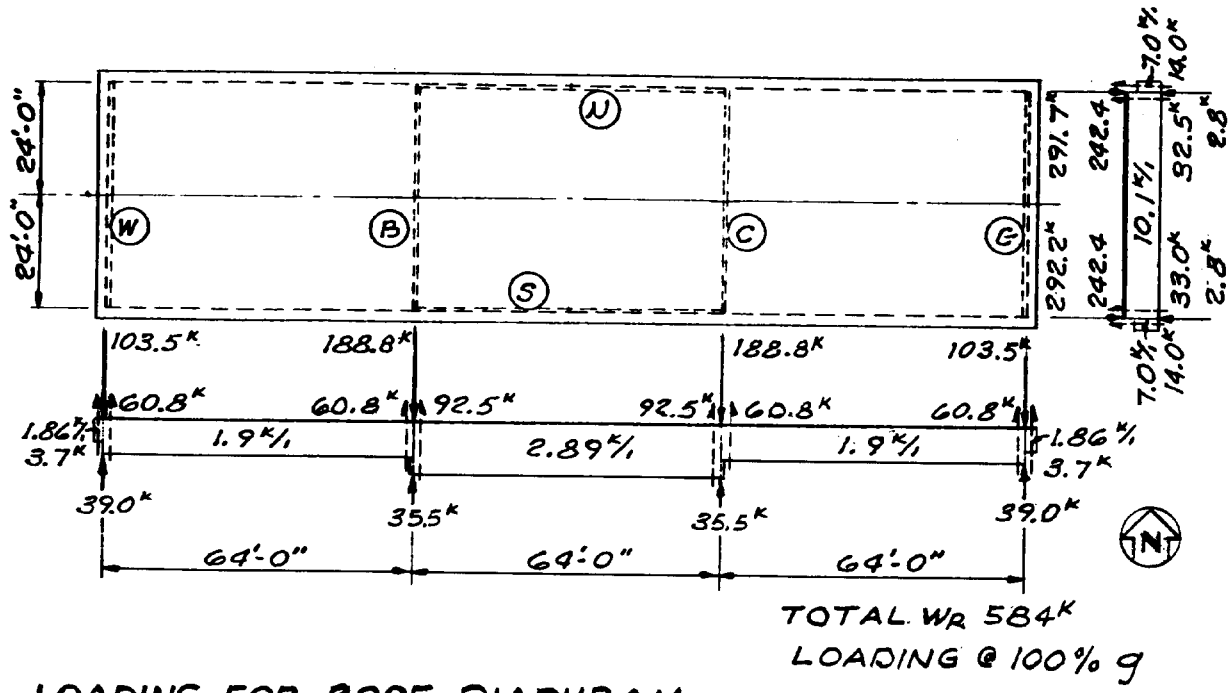
Figure D-5. Continued.



◇ DENOTES FRAMING CONNECTION FOR SHEAR & CHORD FORCES; OTHER CONNECTIONS TO DEVELOP FLEXURAL CAPACITY FOR FRAME ACTION AS WELL AS SHEAR AND CHORD FORCES.

BEAMS IF EMBEDDED IN SHEAR WALLS SHALL BE DESIGNED TO CARRY THE WEIGHT OF CONCRETE IN THE STORY ABOVE.

Figure D-5. Continued.



LOADING FOR ROOF DIAPHRAM

EXTERIOR WALLS (ACCOUNT FOR PERCENTAGE OF SOLID WALL - WINDOWS OUT)

METAL PANEL
WALL WT. 4 PSF × 5.5' = 22#/' × 62.8 = 1381.6# × 2 = 2,760#

10" CONC. WALLS { E & W WALLS: .833 × 6.5 × 150 = 813#/'
N & S WALLS: .833 × 5.5 × 150 = 687

W = 813 × 1.0 = 813 × 48 = 39,024#
FRACTION SOLID

E = 813 × 1.0 = $\frac{813}{1626\#/'}$ × 48 = 39,024#

N = 687 × .75 = 522 × 63.2 = 32,548#

S = 687 × .76 = $\frac{522}{1037\#/'}$ × 63.2 = 32,900#

B OR C = 813 × 91% SOLID = 740#/' × 48.00 LENGTH = 35,520#

E-W LOADS

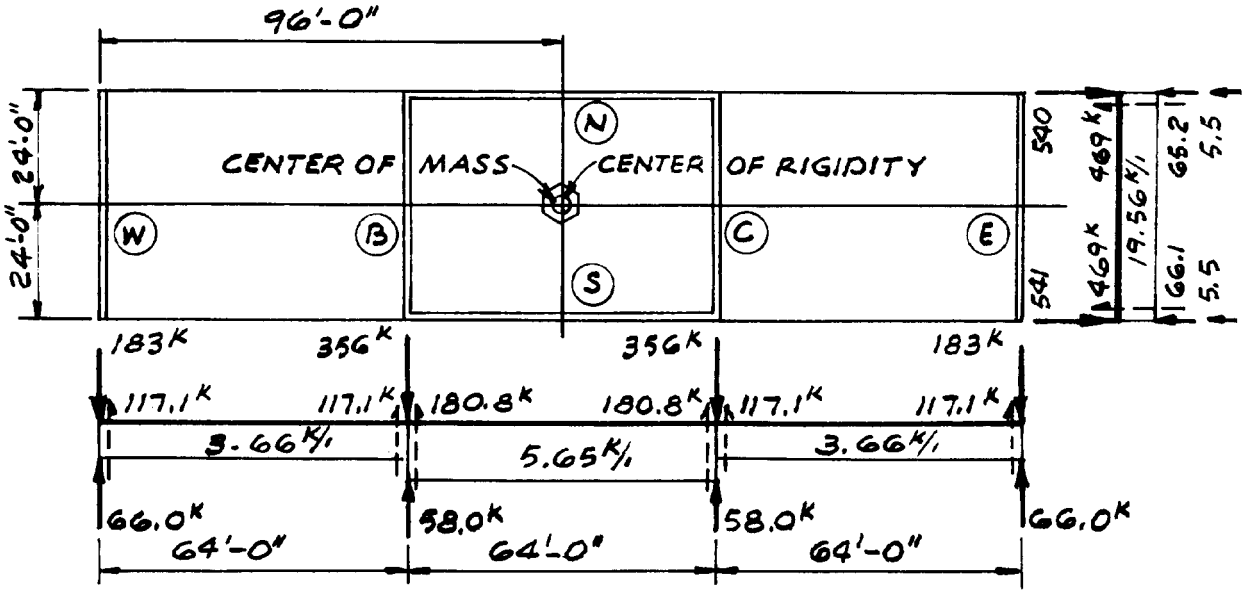
ROOF = 35.7 × 196 = 6997#/'
WALLS E & W = 1626
WALLS B & C 2 × 740 = 1480
10,103#/'

N-S LOADS

ROOF = 35.7 × 52' = 1856
WALLS N & S = 1037
CENTER BAY = 2893#/'

ROOF 1856
EXT. WALL 2 × 22 = 44
END BAYS = 1900#/'

Figure D-5. Continued.



TOTAL $W_3 = 1078^K$

LOADING @ 100% q

LOADING FOR 3RD FLOOR DIAPHRAGM (2ND FLOOR SAME)

EXTERIOR WALLS (ACCOUNT FOR PERCENTAGE OF SOLID WALL - WINDOW OUT)

METAL PANEL

WALL WT. 4 P.S.F. × 11' = 44 × 62.8' = 2.76K × 2 = 5,520#

10" CONC. WALL = .833 × 11' × 150 = 1375#/'

(W) = 1375 × 1.0 = 1375 × 48 = 66,000#

(E) = 1375 × 1.0 = 1375 / 2750#/' × 48 = 66,000#

(N) = 1375 × 0.75 = 1031 × 63.3 = 65,262#

(S) = 1375 × 0.75 = 1045 / 2076#/' × 63.3 = 66,149#

(B) OR (C) = 1375 × 0.91 = 1251#/' × 46.33 = 57,970#

N-S LOADS

CENTER BAY

FLOOR 74.5 × 48 = 3576
WALLS (N) & (S) = 2076
5652#/'

END BAYS

FLOOR 74.5 × 48 = 3576
EXT. WALL 2 × 44 = 88
3664#/'

E-W LOADS

FLOOR 74.5 × 192 = 14,304
WALLS (E) & (W) = 2750
WALLS (B) & (C)
2 × 1251 = 2,502
19,556#/'

Figure D-5. Continued.

LATERAL FORCES

$$V = \frac{ZIC}{R_W} W$$

$$Z = 0.4,\ I = 1.0,\ R_W = 12,\ S = 1.5$$

$$T = 0.020\,(h_n)^{3/4},\quad h_n = 34'$$

$$= 0.020\,(34)^{3/4} = 0.282 \text{ SEC.}$$

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.282)^{2/3}} = 4.36 \text{ USE } C_{MAX} = 2.75$$

$$V = \frac{0.4 \times 1.0 \times 2.75}{12} W = 0.092\,W$$

LEVEL	h_x	Δh	W_x	$W_x h_x$	$\frac{Wh}{\Sigma Wh}$	F	V	ΔM_{OT}	M_{OT}
ROOF	33'	11	584^k	19,272	.35	88^k	88^k	968	968
3RD	22'	11	1078	23,716	.43	108	196^k	2156	3124
2ND	11'	11	1078	11,858	.22	56	252^k	2772	5896
			W = 2740^k	54,846	1.0	252			
V = 0.092 x 2740 = 252^k $F_T = 0$ SINCE T < 0.7 SEC.									

STORY FORCES FOR DESIGN

LEVEL	SHEAR WALL: F_x		STEEL FRAME $0.25F_x$	
ROOF	88^k	DISTRIBUTE TO CONC. SHEAR WALLS IN PROPORTION TO THEIR RELATIVE RIGIDITIES. INCLUDE ACCIDENTAL TORSION. SIM. TO FIG D-1	22^k	DISTRIBUTE TO STEEL FRAMES IN PROPORTION TO RELATIVE RIGIDITIES INCLUDE ACCIDENTAL TORSION. SIM. TO FIG D-3
3RD	108^k		27^k	
2ND	56^k		14^k	
	252^k		63^k	

Figure D-5. Continued.

DESIGN EXAMPLE D-6

Wood Shear Panel System:

Description of Structure. A two-story wood framed classroom building in Zone 3, using wood floor and roof decks and wood stud walls. Girders and columns on centerline of building support roof rafters and floor joists. The structural concept is illustrated on Sheets 2 and 3.

Construction Outline.

Roof:
- Composition & gravel.
- 1" diagonal sheathing.
- Wood rafters, wood girders, and columns.
- Ceiling (drywall + acoustic tile).

2nd Floor:
- 3/4" plywood sheathing.
- Asphalt tile.
- Wood floor joists, steel girders & columns.
- Ceiling (drywall + acoustic tile).

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Wood stud bearing walls with exterior and interior plaster.

Partitions:
- The stair enclosure walls are wood stud with plywood sheathing on one. Other interior walls are removable drywall.

Design Concept. There is a line of columns and girders on the centerline of the building, but the exterior walls are bearing walls. Thus the structure does not have a complete vertical load-carrying space frame and is a Wood Box System with a R_W-factor of 8. The diagonally-sheathed roof acts as a diaphragm spanning between exterior walls. This is a very flexible diaphragm incapable of transferring significant rotational forces. The plywood sheathed second floor is a flexible diaphragm. This second floor diaphragm is interrupted by a stairwell. The permanent stair enclosure walls running in a north-south direction are therefore used as shear walls.

Discussion. The accompanying computations show the load diagrams and distribution of horizontal forces to the various shear walls and the unit shear and chord stresses in the diaphragm. Attention is called to the two second-floor struts which must transfer diaphragm shears to the shear walls on each side of the stairs. Double joists are used for these struts. Plywood sheathing is given for one of the stair walls. As this wall is short, it will be provided with special tie-down fastenings. Shear in piers of each wall are computed as proportional to the solid space between openings.

Figure D-6. Wood box system.

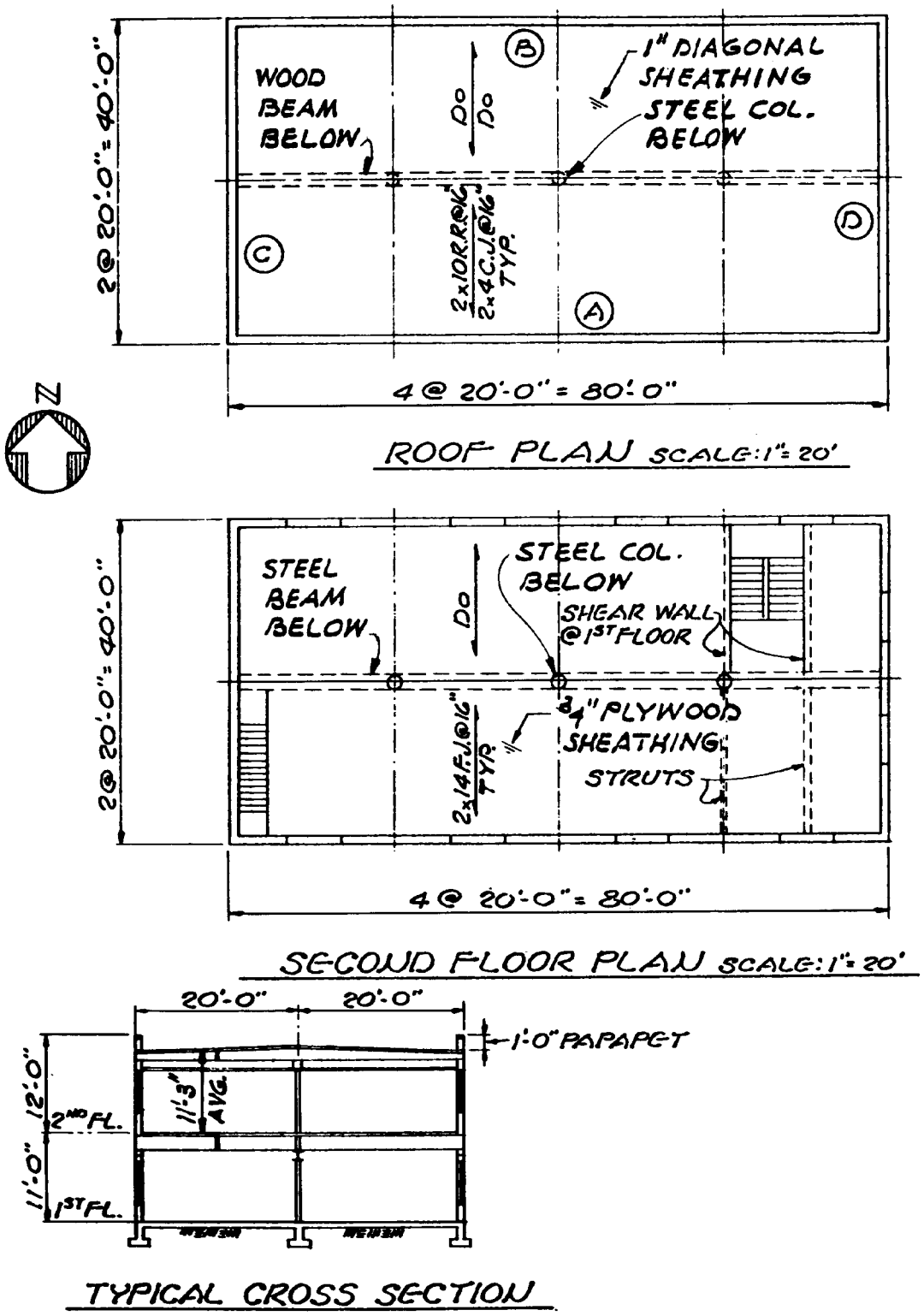


Figure D-6. Continued.

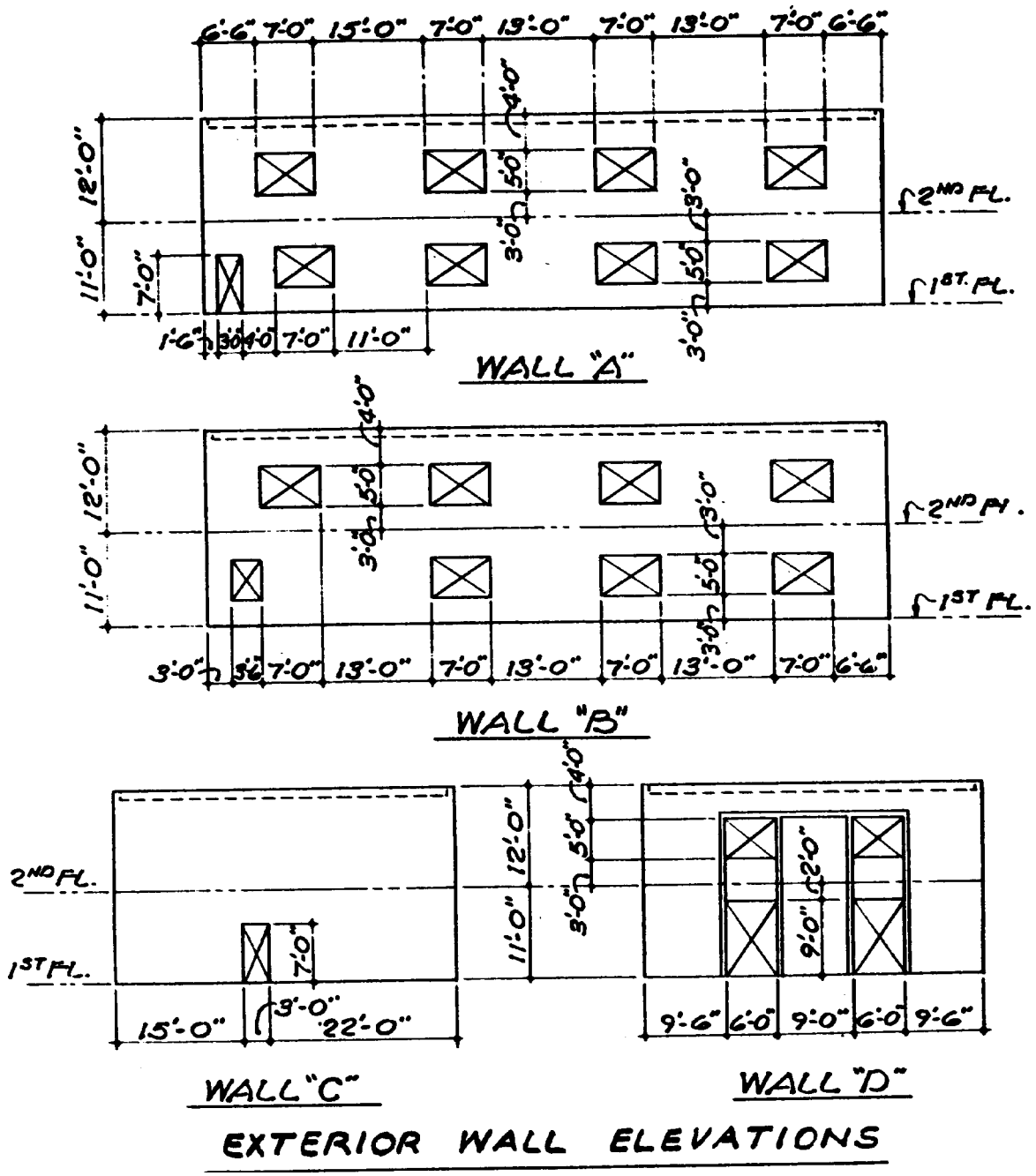


Figure D-6. Continued.

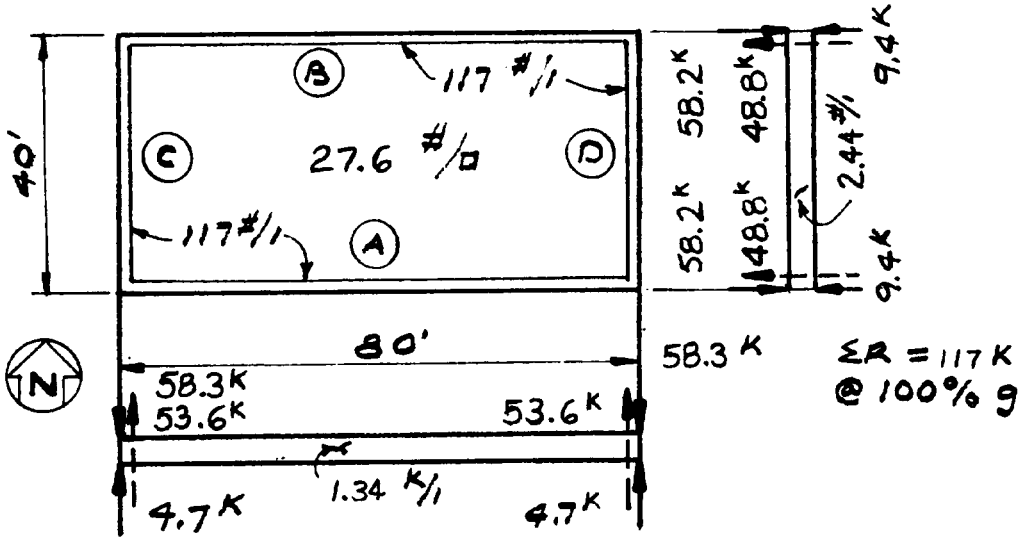
LOADS FOR ROOF DIAPHRAGM

ROOF

COMPO & GRAVEL ROOFING	= 6.6
1" DIAG. SHEATHING	= 1.5
RAFTERS & CEILING JOISTS	= 3.5
CEILING (DRYWALL + AC. TILE)	= 5.0
MISCELLANEOUS	= 1.0
DL	= 17.0
ADD PARTITIONS FOR SEISMIC	10.0
	27.6 PSF FOR SEISMIC

WALLS 11' HIGH & 1 FT. PARAPET
STUDS & PLASTER 18 PSF × 6.5' = 117.0 #/'

LOADING DIAGRAM - ROOF DIAPHRAGM



(N-S) LOADS

27.6 #/□' × 40'	= 1104 #/'
117 #/' × 2	= 234
	1338

WALL C OR D 117 × 40 = 4680#

(E-W) LOADS

27.6 × 80	= 2208
117 × 2	= 234
	2442

WALL A OR B 117 × 80 = 9360 #

Figure D-6. Continued.

LOADS FOR 2ND FLOOR DIAPHRAGM

FLOOR

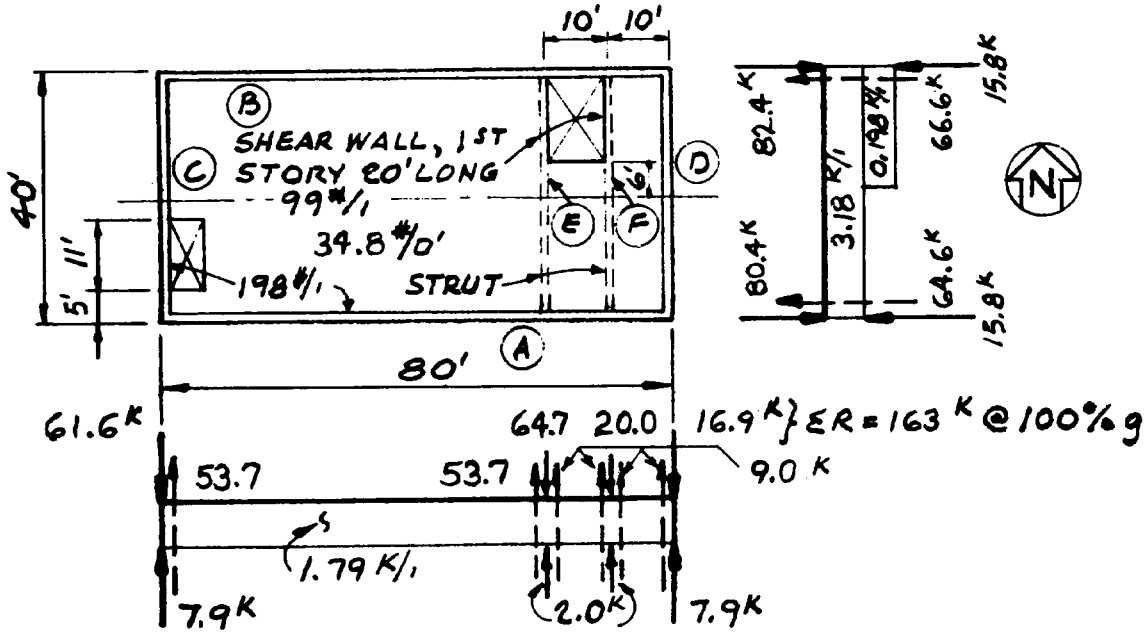
ASPHALT TILE	=	1.0 #/□'
¾" PLYWOOD SHEATHING	=	2.3
FLOOR JOISTS	=	4.6
STEEL BEAMS & COLUMNS	=	1.0
CEILING & MECHANICAL	=	5.9
PARTITIONS	=	20.0
FLOOR DEAD LOAD	=	34.8 #/□'

WALLS

EXTERIOR: 18 #/□' × 11' = 198 #/'

INTERIOR: 18 × 5.5 = 99

LOADING DIAGRAM - 2ND FLOOR DIAPHRAGM



(N-S) LOADS

34.8 × 40 = 1392

198 × 2 = 396

1788 #/'

WALL C OR D = 198 × 40 = 7920 #

WALL E OR F = 99 × 20 = 1980 #

(E-W) LOADS

34.8 × 80 = 2784

198 × 2 = 396

3180

WALL E & F = 99 × 2 = 198 #/'

WALL A OR B = 198 × 80 = 15,840 #

Figure D-6. Continued.

LATERAL FORCES

$$V = \frac{ZIC}{R_w} W$$

$Z = 0.30$, $I = 1.0$, $R_w = 8$, $S = 1.5$

$T = 0.020\,(h_n)^{3/4}$, $h_n = 22.25'$

$= 0.020\,(22.25)^{3/4} = 0.205$ SEC

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.50}{(0.205)^{2/3}} = 5.39, \text{ USE } 2.75$$

$$V = \frac{0.30 \times 1.0 \times 2.75}{8} W = 0.103\,W$$

$$\text{STORY FORCE } F_x = \frac{wh}{\Sigma wh}(V)$$

LEVEL	W_x	h	wh	$\frac{wh}{\Sigma wh}$	F_x
ROOF	117^k	$22.25'$	2603	0.59	17.0
2ND FLR.	163^k	$11.0'$	1793	0.41	11.8
$W = 280$			4396	1.0	28.8

$V = 0.103 \times 280^k = 28.8^k$

Figure D-6. Continued.

ROOF DIAPHRAGM

LATERAL FORCE

STORY FORCE = 17.0^K

DIAPHRAGM FORCE

$F_{px} = \frac{\Sigma F_i}{\Sigma W_i} W_{px} = \frac{17}{117} W_{px} = 0.145\ W_{px}$ ← GOVERNS

MIN. $F_{px} = 0.35\ ZIW_{px} = 0.35 \times .3 \times 1 \times W_{px} = 0.105\ W_{px}$

MAX. $F_{px} = 0.75\ ZIW_{px} = 0.75 \times .3 \times 1 \times W_{px} = 0.225\ W_{px}$

DIAPHRAGM STRESSES

$W = 0.145\ W_{px}$

	BENDING M $= WL^2/8$	CHORD FORCE $= M/D$	SHEAR V $= WL/2$	$v = V/D$
N-S	$(1.34 \times 0.145) \times 80^2/8 = 155^k$	$\div 40' = 3.88^k$	7.76^k	$\div 40' = 0.19^k/FT$
E-W	$(2.44 \times 0.145) \times 40^2/8 = 71^k$	$\div 80' = 0.89^k$	7.07^k	$\div 80' = 0.09^k/FT$

SHEATHING

$v_{MAX.} = 190\#/1$

1x DIAGONAL SHEATHING - DOUGLAS FIR

VERY FLEXIBLE DIAPHRAGM WEB: F = 250 (TABLE 5-1 & 5-2)
ALLOWED DIAPHRAGM LENGTH = 2 × WIDTH (TABLE 5-1)
ALLOWED SHEAR = 300 #/FT. (TABLE 5-2)

CONNECTIONS

CHORD SPLICE NEAR MIDSPAN OVER WALL A OR B P = 3880#

TOP PLATE OF STUD WALL IS CHORD. LAP PLATES AND CONNECT WITH 3-¾"ϕ BOLTS EACH SIDE OF SPLICE. CAPACITY IN SINGLE SHEAR IN 1½" MEMBERS = 3 × 1350# × 1.33 = 5.40^K.

CHORD SPLICE NEAR MIDSPAN OVER WALL C OR D P = 890#

EDGE ROOF RAFTER IS CHORD. PROVIDE 7-16d NAILS EACH SIDE OF SPLICE. CAPACITY = 7 × 107# × 1.33 = 996#

DIAPHRAGM CONNECTION WALL A OR B v = 90#/1

BLOCKING TO BLOCKING & BLOCKING TO PLATE (Sect. A, FIGURE 5-23). PROVIDE 2-16d (OR METAL FRAMING ANCHORS) BETWEEN RAFTERS. CAPACITY = (2 × 107 × 1.33) ÷ 1.33^{FT} = 214#/FT

2 of 4

DIAPHRAGM CONNECTION TO WALL C OR D v = 190#/1

RAFTER TO BLOCKING & BLOCKING TO TOP PLATE (Sect. C, FIGURE 5-23) USE 16d @ 8" O.C. CAPACITY = (107# × 1.33 ÷ 0.67 = 212#/FT.

2 of 4

Figure D-6. Continued.

2ND FLOOR DIAPHRAGM

LATERAL FORCE STORY FORCE = 11.8^k $\frac{11.8}{163} = 0.0724$

DIAPHRAGM FORCE

$$F_{px} = \frac{28.8}{280} W_{px} = 0.103 \; W_{px} \longleftarrow \text{GOVERNS}$$

MIN. $F_{px} = 0.103 \; W_{px}$

DIAPHRAGM STRESSES

	BENDING M $WL^2/8$	CHORD FORCE $= M/D$	SHEAR V $= WL/2$
N-S	$(1.79 \times 0.103) \times \overline{60}^2/8 = 83.0^{k'}$	$\div 40' = 2.08^k$	5.53^k
E-W	$(3.18 \times 0.103) \; \overline{40}^2/8 = 65.5$	$\div 80' = 0.82^k$	6.55^k

WALL	L	V	v	CASE *	BOUNDARY NAILS	PANEL NAILS	ALLOWED v
A	80'	6550#	82#/'	3	6" C.C.	6" C.C	215#/'
B, WEST OF STAIR	60'	"	109				
C, NORTH OF STAIR	24'	5530'	230	1	6" C.C.	6" C.C.	285#/'
E, PLUS STRUT	40'	"	138				

* SEE TABLE 5-6:
UNBLOCKED DIAPHRAGM, C-C EXT-APA PLYWOOD.
USE VALUES FOR 5/8" PLYWOOD, 10d NAILS, 2×MEMBERS.

FLEXIBILITY

USE L = 60' (DIAPH. SPANING FROM WALL C TO E)
FORMULA 5-33: $q_{AVE} = 230$ $q_d = 285$

$$F = \frac{33{,}000 \, q_{AVE}}{q_d^{\,2}} = \frac{33{,}000 \times 230}{(285)^2} = 93$$

TABLE 5-1: DIAPH. IS "FLEXIBLE". MAX SPAN = 100' > 60' OK
WITH FLEXIBLE WALLS AND NO CALCULATED TORSION IN
THE DIAPH. THE MAX. SPAN = 3 × DEPTH OR 3×40'
= 120' > 60' OK

Figure D-6. Continued.

2ND FLOOR DIAPHRAGM - CONT'D.

CONNECTIONS

CHORD SPLICE AT WALL A OR B $P = 2080^{\#}$

SIMILAR TO ROOF. USE 2-5/8"ϕ BOLTS EACH SIDE.

CAPACITY $= 2 \times 1000^{\#} \times 1.33 = 2660^{\#}$

CHORD SPLICE AT WALL C OR D $P = 820^{\#}$

SIMILAR TO ROOF. USE 6-16d NAILS EACH SIDE.

CAPACITY $= 6 \times 107^{\#} \times 1.33 = 854^{\#}$

DIAPHRAGM AT WALL A

WALL ABOVE, SOLE PLATE TO BLOCKING, 2-16d BETWEEN RAFTERS FOR 90#/FT, AS AT TOP PLATE.

BLOCKING TO BLOCKING AND BLOCKING TO TOP PLATE:

SHEAR FROM ROOF & FLOOR $= 90 + 82 = 172^{\#}/FT$

USE 2-16d BETWEEN RAFTERS, SIMILAR TO ROOF.

CAPACITY $= 214^{\#}/FT$.

DIAPHRAGM AT WALL B

SIMILAR TO WALL A. SHEAR $= 90 + 109 = 199^{\#}/FT$., USE 2-16d

DIAPHRAM AT WALL C

WALL ABOVE, SOLE PLATE TO EDGE RAFTER.

USE 16d @ 8" C.C. FOR 190#/FT, AS AT ROOF.

RAFTER TO BLOCKING AND BLOCKING TO TOP PLATE:

SHEAR FROM ROOF & FLOOR $= 190 + 230 = 420^{\#}/FT$

USE 16d @ 4" C.C.

CAPACITY $= 107^{\#} \times 1.33 \div 0.33' = 431^{\#}/FT$

DIAPHRAGM AT WALL E

NO WALL ABOVE.

STRUT IS DOUBLE JOIST EXTENDING OVER WALL E, SIMILAR TO PLAN A, FIG. 5-23

$$\text{STRUT FORCE} = \left(\frac{20'}{40'} \times 0.183^{K}/FT \times \frac{60'}{2}\right) + \left(\frac{20'}{26'} \times 0.183 \times \frac{10'}{2}\right)$$

$$= 3.45^{K}$$

USE 2-3/4"ϕ BOLTS, DOUBLE JOIST TO L4×4×1/4 AND 2-3/4"ϕ BOLTS, ANGLE TO DOUBLE TOP PLATE OF WALL E.

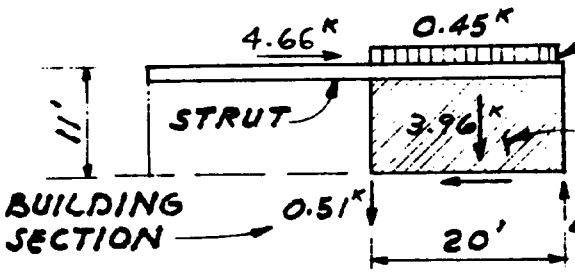
CAPACITY IN SINGLE SHEAR IN 3" OF WOOD WITH METAL SIDE PLATE $= 2 \times (1.25 \times 1470^{\#}) \times 1.33 = 4888^{\#}$

Figure D-6. Continued.

SHEAR WALLS

WALL E

2ND STORY FORCE = 0.072 × FLOOR WEIGHT



2ND FLR. DL = $\frac{1.33'}{2}$ × 20' × 34.8 PSF = 463#

WT. OF WALL = 20' × 11' × 18 PSF = 3960#

TOTAL DL = 4410#

SEISMIC LOADS (0.072 × FLOOR WT.)

DIAPH. LOADING DIAGRAM

FROM THE WEST 0.072 × 53.7 K = 3.87 K

FROM THE EAST × 9.0 = 0.66

FROM THE WALL × 2.0 = 0.14

WALL V = 4.66 K

WALL SHEAR $V = \frac{4660\#}{20'} = 233\#/1$

USE 5/16" STRUCT. I EXT. - APA PLYWOOD ONE SIDE 6d @ 4" AT PANEL EDGES, 6d @ 12" AT INTERMEDIATE SUPP'TS. ALLOWABLE SHEAR = 300#/1 (FIGURE 6-18)

OVERTURNING M = 4660# × 11' = 51,260 #'

WALL REACTIONS = $\frac{4410\#}{2} \pm \frac{51{,}260\#'}{19'}$ = 2205 ± 2698

DOWN LOAD = 2205 + 2698 = 4903

UP LOAD = 0.85 (2205) − 2698 = −824 (SEAOC 1H1b)

TIE DOWN (FIG. 6-20)

POST BOLTS TO ANGLE: 2-5/8" ϕ SINGLE SHEAR 2 1/2" NET, WITH METAL SIDE PLATES: ALLOW 2 × (1.25 × 1020#) × 1.33 = 3392# > 824#. ANCHOR BOLT: 5/8 ϕ ALLOW 1.33 × 3000 = 4000#. L4 × 4 × 5/8 × 0'-3 1/2": S = (3 1/2 − 7/8") (5/8")²/6 = .173 IN³ M = 511# × (2 1/2 − 11 = 767#" F = 767/.173 = 4333 PSI

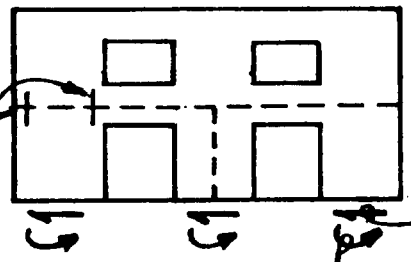
Figure D-6. Continued.

SHEAR WALLS CONT'D.

WALL (D)

TREAT AS 3 EQUAL CANTILEVER PIERS
ASSUME NO MOMENT DEVELOPED IN SPANDRELS.

SPLICE FOR VERTICAL CONTINUITY TYPICAL AT ALL PIERS.



$.145 \times 58.3^k = 8.45^k \times 22.3 = 189$

$.072 \times 16.9^k = 1.22^k \times 11.0 = 13$

$9.67^k \quad M_{OT}\ 202^{k'}$

$V = \frac{9.67}{3} = 3.22^k$

$M = \frac{202}{3} = 67.3^{k'}$

WALL SHEAR
1ST. STORY:

$v = \frac{9670}{3 \times 9} = 358\ \#/FT$

WALL HT/WIDTH

$= \frac{11.3}{9} = 1.25 < 2$ OK

EXT. LATH & PLASTER - 200
INT. " " " - 100
1" DIAG. SHEATING - - - 300
ALLOW. v = 600#/FT

OVERTURNING:	2ND STORY	1ST STORY
WT. OF WALL	$9' \times 12' \times 18$ psf. = 1944#	$9' \times 23' \times 18$ = 3613#
DL OF ROOF	$15' \times \frac{8}{12} \times 17$ psf = 170	170
DL OF FLOOR		$15 \times \frac{8}{12} \times 34$ = 340
TOTAL DL	2114#	4123#
O.T. MOMENT	$\frac{8.45^k \times 11.3}{3} = 31.8^{k'}$/PIER	$\frac{202}{3} = 67.3^{k'}$/PIER
O.T. FORCE	$\frac{31.8}{8.5'} = 3.75^k$	$\frac{67.3}{8.5'} = 7.92^k$
REACTIONS	$\frac{2114}{2} \pm 3750$	$\frac{4123}{2} \pm 7920$
DOWN LOAD	1057 + 3750 = 4807	2062 + 7920 = 9982
UPLIFT	0.85(1057) - 3750 = -2852	0.85(2062) - 7920 = -6167

Figure D-6. Continued.

SHEAR WALLS - CONT'D

WALL (D) CONT'D

UPLIFT AT 2ND FLOOR $F = 2852^{\#}$

PROVIDE VERTICAL CONTINUITY WITH METAL SPLICE PLATE 3/16" x 2 1/2" WITH 2-5/8" BOLTS EACH END. ALLOW $2 \times (1.25 \times \frac{2030}{2}) \times 1.33 = 3375^{\#} > 2852$

TIE DOWNS AT 1ST FLOOR $F = 6167^{\#}$

USE STIFFENED ANGLE (FIG. 6-20) 3-3/4" ϕ BOLTS TO 4x4 POST, SINGLE SHEAR IN 2 1/2" NET WITH METAL SIDE PLATE. ALLOW $3 \times (1.25 \times \frac{2870^{\#}}{2}) \times 1.33 = 7157^{\#}$

7/8" ϕ ANCHOR BOLTS WITH 3" x 3" WASHER

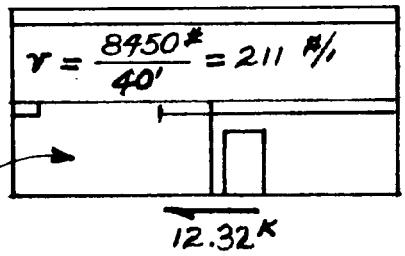
NET AREA OF WASHER = 8 IN.^{2}
ALLOW 600 PSI IN B'R'G: $600 \text{ PSI} \times 8 \times 1.33 = 6400^{\#}$

NOTE THAT FOOTING MUST BE REINFORCED SO THAT THE BOLT CAN PICK UP ABOUT 1.5 CU. YDS OF CONCRETE.

Figure D-6. Continued.

SHEAR WALLS - CONT'D

WALL (C)



$.145 \times 58.3^k = 8.45^k \times 22.3 = 189^k$

$.072 \times 53.7^k = 3.87^k \times 11.0 = 43$

$12.32^k \quad M_{OT} = 232^{k'}$

$v = \frac{12,320}{15' + 22'} = 333\ \#/'$

WT. OF WALL = 18 psf × 23
= 414 #/'

RESISTING MOMENT
$M_R = 0.414^{k}/' \times 40' \times 20'$
$= 331 > 232^{k'}$
∴ NO UPLIFT

SHEATHING:
2ND STORY:
EXT. + INT. LATH & PLASTER
200 + 100 = 300 #/' > 212
1ST STORY:
ADD 1" DIAG. SHEATHING
300 + 300 = 600 #/' > 335

WALL (A) ((B) SIMILAR)

	F	V	NET L	v
ROOF 0.145 × 58.2 =	8.44	8.44	52'	162 #/'
FLOOR 0.072 × 80.4 =	5.79	14.23	47.5'	300

SHEATHING:

2ND STORY EXT. + INT. L + P 300 #/' > 162

1ST STORY ADD 4 LET-IN BRACES

$300\ \#/' + \frac{4 \times 1000^{\#}}{47.5} = 384\ \#/' > 300$

Figure D-6. Continued.

DESIGN EXAMPLE D-7

Special Configuration:

Description of Structure. A one-story industrial garage building in Seismic Zone 3. The north, east, and west walls are concrete bearing walls. The south wall is largely open for drive-in access and has concrete columns and concrete beams over the openings. The roof is concrete slab and beams. The structural concept is illustrated on Sheets 2 and 3.

Design Concept. The roof is a reinforced concrete beam and slab system forming a relatively rigid diaphragm, even with a 6 to 1 length-width ratio. The north, east, and west walls are concrete bearing walls. The south wall is a rigid frame. The lateral forces are resisted by shear walls. The building is a Box System with $R_w = 6$.

Discussion. An estimate of the relative deflections and stiffnesses of the north wall versus the south wall rigid frame indicates that practically all of the east-west forces would be carried by the north wall. The resulting rotation is resisted by the east and west walls. A computation of the deflection of the roof diaphragm in resisting north-south forces is shown. The transverse bents formed by the south wall columns, the transverse roof beams, and a portion of the north wall are checked to see if these bents are adequate for the vertical load carrying capacity and the induced moment due to $3R_w/8$ times the deflection resulting from the lateral forces. The vertical load stresses in the south wall beams will be combined with chord stresses of the roof diaphragm.

LATERAL FORCES

$$V = \frac{ZIC}{R_w} W$$

$$Z = 0.3,\ I = 1.0,\ R_w = 6,\ S = 1.5$$

$$T = 0.020\,(h_n)^{3/4} \qquad h_n = 16.0'$$

$$= 0.020\,(16.0)^{3/4} = 0.16 \text{ SEC.}$$

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.16)^{2/3}} = 6.36,\ \text{USE } 2.75$$

$$V = \frac{0.3 \times 1.0 \times 2.75}{6} W = 0.138\,W$$

Figure D-7. Special configuration.

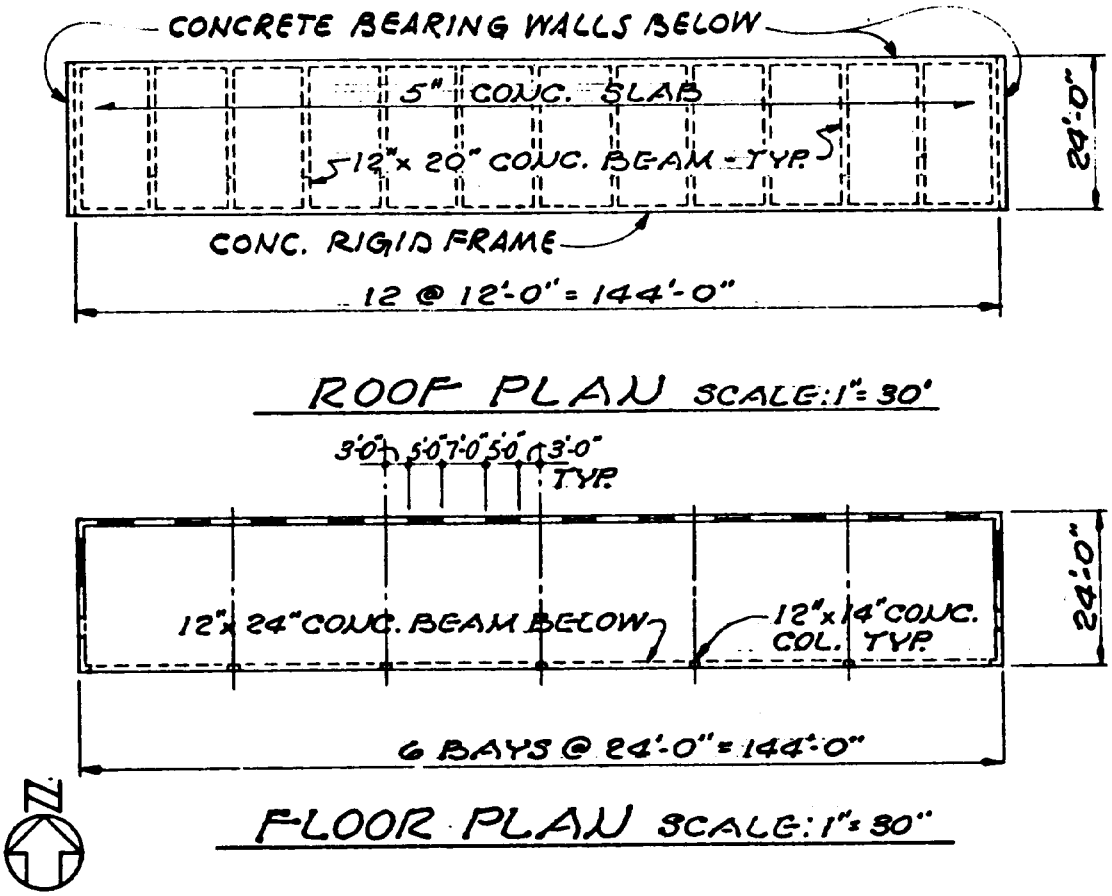


Figure D-7. Continued.

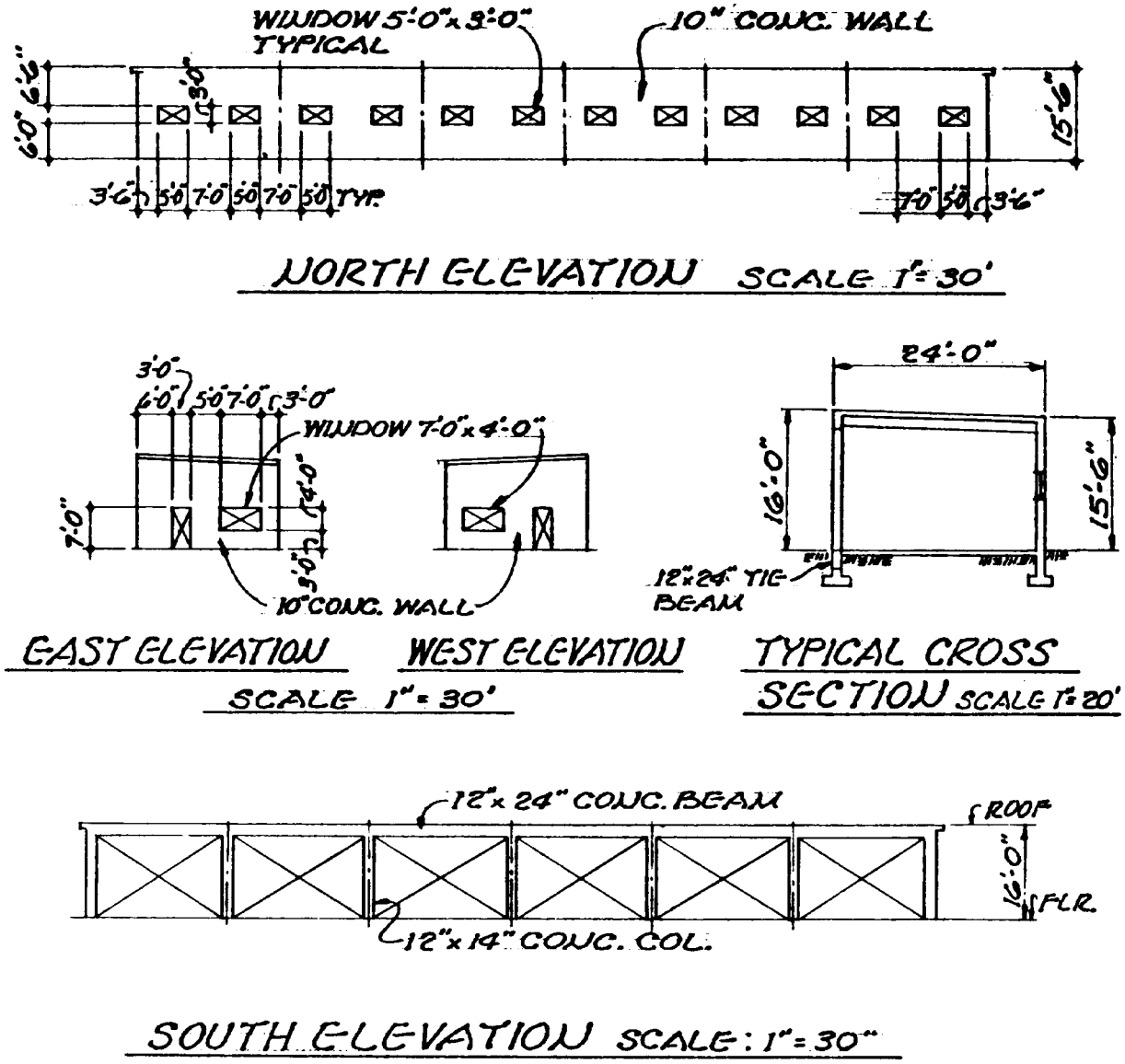


Figure D-7. Continued.

ROOF D.L.

COMPO & GRAVEL ROOF 7.0
5" CONC. SLAB 63.0
BEAMS 16.0
86.0 #/□'

12" x 24" CONC. BEAM = 150 x 1.58 = 237 #/,
COLUMNS = 1 x 1.17 x 150 x $\frac{14}{2}$ = 1228 #

EXTERIOR WALLS

N. WALL 10" CONC. 125 x 7.55 = 945 #/,
E. & W. END WALLS 125 x 7.88 = 985 #/,

SEISMIC N-S

ROOF 86 x 24 = 2060
N. WALL = 945
BEAM = 237
COLS. = $\frac{5 \times 1228}{144}$ = 43
DOOR OR COVER 10 x 7 = 70
.138 x 3355 = 463 #/,

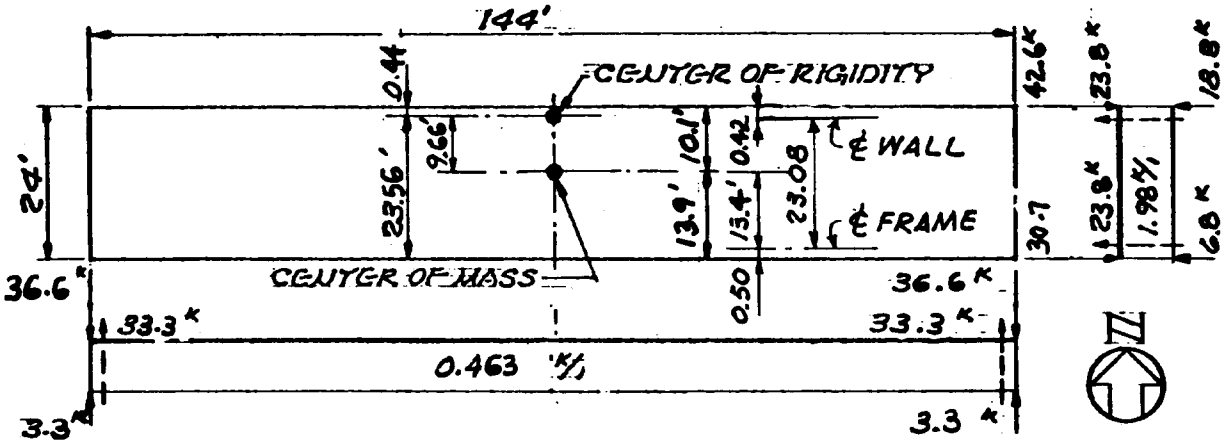
END WALL 985 x 24 = 23,600 x .138 = 3.3 K

SUMMARY

2 END WALLS @ 3.3 = 6.6
DIAPHRAGM 0.463 x 144 = 66.7
TOTAL SEISMIC WEIGHT = 73.3 K

SEISMIC E-W

ROOF 86 x 144 = 12,400
WALLS 2 x 985 = 1,970
.138 x 14,370 = 1980 #/,
N. WALL 945 x 144 = 136,000 x .138 = 18.8 K
S. WALL
BEAM 237 x 144 = 84,200
COLS. 1228 x 5 = 6,140
COVER 60 x 144 = 8,650
.138 x 48,990 = 6.8 K



SEISMIC LOADS

Figure D-7. Continued.

RELATIVE RIGIDITIES

NORTH WALL

USING CHART FOR DEFLECTION (FIG. 6-4) DEFLECTIONS TABULATED BELOW FOR 10" WALLS, ARE 12/10 TIMES THE CHART VALUE WHICH ARE FOR 12" WALLS.

PIER	h	d	h/d	Δ	K
1 CF	3	3.5	.86	.0852	11.75
2 RF TO 12 INCL	3	7.0	.43	.0458	21.82 x 11 = 240.0
13 CF	3	3.5	.86	.0852	11.75
					263.5
WALL RC	15.5	144	.1077	.0108	—
BAND RF @ WIND.	3	144	.0208	.0024	—

Δ PIERS (1-13) = 1/263.5 = 0.0038

Δ WALL = .0108 - .0024 + 0.0038 = 0.0122

K (WALL) = 1/0.0122 = 82.6

NOTE:
CF INDICATES CORNER PIER, FIXED CONDITION
RC INDICATES RECTANGULAR PIER CANTILEVER CONDITION

SOUTH WALL (RIGID FRAME)

DEFLECTION OF PIERS - BEAM FIXED

PIER	h	d	h/d	Δ	K
1 CF & 7 CF	14	1.17	12	32.8	.0305 x 2 = .061
2-6 RF INCL.	14	1.17	12	49	.0204 x 5 = .1020
WALL RC	16	144	.111	.0095	—
@ OPNGS RC	14	144	.097	.0085	—

$\Sigma\Delta = 12 \times .0694 + 12^3 \times .0185 = 32.8$

$\Sigma\Delta = 12 \times .08334 + 12^3 \times .0278 = 49.0$

$\Delta_{1-7} = \dfrac{1}{.061 + .1020} = 6.14$

Δ WALL = .0095 - .0085 + 6.14 = 6.141

DEFLECTION DUE TO ROTATION OF BEAM

$\theta = \dfrac{M_o}{EI}\left(a - \dfrac{a^2}{L} - \dfrac{L}{3}\right)$ @ CENTER OF BEAM $a = \dfrac{L}{2}$

$\dfrac{M_o}{EI}\left(-\dfrac{L}{12}\right)$ USE $P = 1000^K$, $M = 1.000 \times 14 = 14.000^{K'}$

$\theta = \dfrac{14{,}000{,}000 \times 12}{3{,}000{,}000 \times 13824}\left(\dfrac{-24' \times 12''/'}{12}\right) = 0.0972$

$\theta h = 0.0972 \times 14 \times 12 = 16.33''$

TOTAL DEFLECTION = 6.14 + 16.33 = 22.47'

$K\,(\text{WALL}) = \dfrac{1}{22.47} = 0.0445$

Figure D-7. Continued.

RELATIVE RIGIDITIES (CONT.)

EAST & WEST WALLS

PIER	h	d	h/d	Δ	K
1^{CF}	7	6	1.17	.132	7.6
2^{RF}	4	5	.8	.0966	10.35
3^{CF}	4	3	1.33	.168	5.96
4_a^{CF}	4	15	.267	.0222	
4^{CF}	7	15	.466	.0408	
5_a^{CF}	7	24	.292	.0246	
5^{CC}	15.75	24	.656	.0804	

$\Delta_{2-3} = \frac{1}{10.35+5.96} = 0.0613$

$\Delta_{1-4} = \frac{1}{7.6+12.5} = .0498$

$\Delta_4 = .0408 - .0222 + .0613 = .0799 \quad K_4 = \frac{1}{.0799} = 12.5$

$\Delta_{WALL} = .0804 - .0246 + .0498 = .1056$ (FOR $P = 1{,}000^k$)

$K_{WALL} = \frac{1}{.1056} = 9.47 \quad \Delta_{WALL}$ FOR $P = 35.6^k = \frac{35.6}{1000}(.1056)$

$= 0.0038''$

CENTER OF RIGIDITY

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $K = 82.6 \times 23.08 = 1906 \quad \bar{x} = \frac{1906}{82.64} = 23.06$

SOUTH WALL $K = \frac{0.0445}{82.64} \times 0 = \frac{0}{1906}$

AT N. WALL

CENTER OF MASS

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $R = 42.6 \times 23.08 = 983.2 \quad \bar{x} = \frac{983.2}{73.3} = 13.4'$

SOUTH WALL $R = \frac{30.7}{73.3^k} \times 0 = \frac{0}{983.2}$

Figure D-7. Continued.

DISTRIBUTION OF FORCES

WALL SHEARS FOR N-S FORCES

WALL	K	$\frac{K}{\Sigma K}$	V_D	d	d^2	Kd^2	$\frac{Kd}{\Sigma Kd^2}$	V_T	V_A	V_W
N	82.6	—	—	0.02	~	~	~	0	0	0
S	0.04	—	—	23.08	533	21.3	9.5×10^{-6}	0	.005	.005
	82.64									
E	9.47	0.5	36.7	71.6	5127	48,600	.00697	0	3.7	40.4
W	9.47	0.5	36.7	71.6	5127	48,600	.00697	0	3.7	40.4
	18.94					97,220				

$V = 73.3^K$

ECCENTRICITY = 0, $M_T = 0$

ACCIDENTAL TORSION =

$M_A = V(0.05L) = 73.3\,(0.05 \times 144) = 528^{K'}$

DIRECT SHEAR, $V_D = \frac{K}{\Sigma K} V$

TORSIONAL SHEAR, $V_T = \frac{Kd}{\Sigma Kd^2} \cdot M$

TOTAL SHEAR, $V_W = V_D + V_T + V_A$

WALL SHEARS FOR E-W FORCES

N		1.0	73.3					0	0	73.3
S		4.8×10^{-4}	0.035					.007	.001	.043
E		—	—					4.93	0.61	5.54
W		—	—					4.93	0.61	5.54

$V = 73.3$

ECC. = 9.66 $M_T = 73.3 \times 9.66 = 708^{K'}$

$M_A = 73.3\,(0.05 \times 24) = 88^{K'}$

Figure D-7. Continued

E-W EARTHQUAKE

NORTH WALL

FACTORED DESIGN LOAD = 1.4 x 73.3 = 103K

$$v_u = \frac{V_u}{\varphi A_c} = \frac{103{,}000}{0.60 \times (144' - 60')12''/_1 \times 10''} = 17 \text{ PSI}$$

SOUTH WALL

RIGIDITY ANALYSIS FINDS NEGLIGIBLE DESIGN FORCE FOR THE SOUTH WALL. ∴ DESIGN THE FRAME FOR VERTICAL LOAD PLUS INDUCED MOMENTS DUE TO $3R_w/8$ TIMES THE DISTORTION RESULTING FROM THE LATERAL FORCE.

$$\Delta = \frac{3(6)}{8}(\Delta_{N.WALL} + \Delta_{DIAPH.})$$

IN THE CASE WHERE THE DIAPHRAGM FLEXIBILITY WOULD PERMIT THE FRAME TO DEFLECT SIGNIFICANTLY MORE THAN THE NORTH SHEAR WALL, A DUCTILE FRAME WOULD BE PROVIDED.

N-S EARTHQUAKE

DIAPHRAGM $M = \frac{0.463 \times 144^2}{8} = 1200^{K'}$

CHORD $F = \frac{1200^{K'}}{23'} = 52.2^K$; $A_s = \frac{52.2 \times 1.4}{40 \times 0.9} = 2.03^{\square''}$

CONT. CHORD BARS 2-#9

END WALLS: DESIGN FORCE, F = 40.4K

FACTORED DESIGN FORCE FOR OVERTURNING = 1.4 x 40.4 = 56.6K; M_{OT} = 56.6 x 15.8' = 894 K'

FACTORED DESIGN FORCE FOR SHEAR = 1.4 x 40.4 = 56.6 K

Figure D-7. Continued.

TRANSVERSE FRAMES

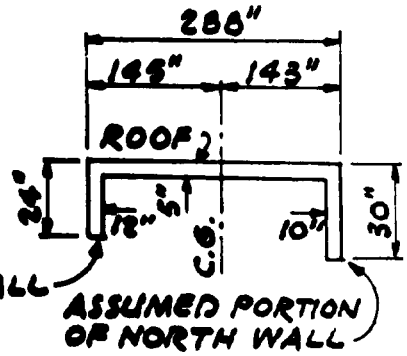
THESE WERE NEGLECTED IN THE RIGIDITY ANALYSIS. CHECK THAT THEY CAN TAKE 3Rw/8 TIMES THE DEFLECTION CALCULATED FOR THE ROOF DIAPHRAGM ACTING WITHOUT THE FRAMES.

DIAPHRAGM DEFLECTION

FLEXURAL DEFLECTION

ASSUMED SECTION

$I = 19{,}120{,}000 \text{ IN}^4$



$$\Delta_f = \frac{5wL^4 \times 1728}{384EI}$$

$$\Delta_f = \frac{5 \times 463 \times 148.17^4 \times 1728}{384 \times 3 \times 10^6 \times 19.12 \times 10^6} = 0.076$$

SHEARING DEFLECTION OF WEB

$$\Delta_w = \frac{q_{AVE} \times L \times F}{10^6} \qquad \text{WHERE } q_{AVE} = \frac{33{,}300}{2 \times 24} = 694 \text{ K/'}$$

$$F = \frac{10^6}{8.5 \times 5 \times 150^{1.5} \sqrt{3000}} = 0.234$$

$$\Delta_w = \frac{694 \times 72 \times .234}{10^6} = 0.0117''$$

TOTAL DEFLECTION OF DIAPHRAGM BETWEEN END WALLS

$$\Delta_D = \Delta_F + \Delta_w = 0.076 + 0.0117 = 0.088 \text{ IN}$$

DEFLECTION OF END WALL

$$\Delta = (0.1056/1000) \times 33.3 = 0.00352$$

DEFLECTION OF FRAME BEAM WITH RESPECT TO GROUND

$$\Delta_B = 0.088 + 0.0035 = 0.092$$

REQUIRED FRAME DEFLECTION

$$\Delta = (3R_w/8)\Delta_B = 3(6)/8 \times 0.092 = 0.207''$$

Figure D-7. Continued.

TRANSVERSE FRAMES (CONT.)

STIFFNESS OF FRAME

$I_{AB} = \frac{14 \times 12^3}{12} = 2020 \text{ IN}^4$

$K = \frac{2020}{15 \times 12} = 11.2$

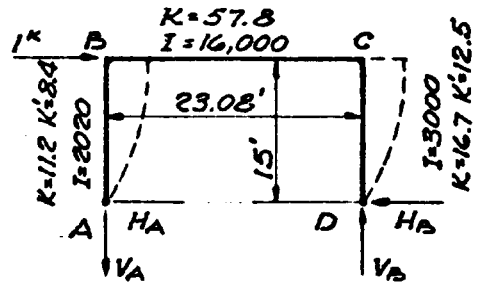
FOR T-BEAM

$I_{BC} = \frac{2 \times 12 \times 20^3}{12} = 16{,}000 \text{ IN}^4$

$K = \frac{16{,}000}{23.08 \times 12} = 57.8$

$I_{CD} = \frac{36 \times 10^3}{12} = 3000 \text{ IN}^4$

$K = \frac{3000}{15 \times 12} = 16.7$



DEFLECTION OF FRAME FOR 1^K LOAD. FIXED JOINTS @ B & C

FIXED END MOMENTS OF COLUMNS $= \frac{-3EK\Delta}{L}$

JOINT	B		C	
	BA	BC	CB	CD
D.F.	.127	.873	.822	.178
F.E.M.	-.187			-.278
	+.024	+.163 +.114	+.229 +.081	+.049
	-.014	-.100 -.033	-.067 -.050	-.014
	+.004	+.029 +.020	+.041 +.014	+.009
	-.003	-.017 -.005	-.011 -.008	-.003
	-.001	+.004	+.007	+.001
	$-.175E\Delta$		$-.236E\Delta$	

FIXED-END MOMENTS:

$M^F_{BA} = \frac{-3E\,11.2\,\Delta}{15 \times 12} = 0.187E\Delta$

$M^F_{CD} = \frac{-3E\,16.7\,\Delta}{15 \times 12} = 0.278E\Delta$

FINAL MOMENTS FOR $P = 1^K$:

$M_{BA} = 0.175E\Delta = 0.175 \times 438$
$= 76.7$ K-IN (6.39 K')

$M_{CD} = 0.236 \times 438$
$= 103$ K-IN (8.61 K')

$\Sigma \text{SHEARS} = 1^K - \frac{.175E\Delta}{15 \times 12} - \frac{.236E\Delta}{15 \times 12} = 0 \longrightarrow E\Delta = 438$

$\Delta = \frac{438}{3 \times 10^3} = 0.146''$

FRAME DEFORMATION COMPATIBILITY

FOR REQ'D FRAME DEFLECTION OF 0.207",

$M_{BA} = \frac{6.39 K'}{0.146 \text{ IN.}} \times 0.207 = 9.1 K'$

$M_{CD} = \frac{8.61}{0.146} \times 0.207 = 12.2 K'$

WHEN COMBINED WITH GRAVITY LOADS THE RESULTING STRESSES ARE WITHIN THE ELASTIC LIMIT; P-Δ IS SMALL. ∴ OK.

Figure D-7. Continued.

DESIGN EXAMPLE D-8

L-Shaped Building:

Description of Structure. A three-story L-shaped Administration Building in Zone 3 with bearing walls in concrete, using a series of interior vertical load-carrying column and girder bents. The structural concept is illustrated on Sheets 2, 3, and 4.

Construction Outline.

Roof:
- Built-up, 5 ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete furred with GWB finish.

Partitions:
- Non-structural removable drywall.

Design Concept. Since the structure is without a complete load-carrying space frame, the R_W-factor is 6. The metal deck roof system forms a flexible diaphragm, therefore the roof loads are distributed to the shear walls by tributary area rather than by third story wall stiffnesses. The roof diaphragm, being flexible, will not transmit accidental torsion to the shear walls. The metal deck with concrete fill system for the floors form rigid diaphragms. The walls act as a series of vertical cantilever beams connected together by struts at the floor lines. The wall analysis utilizes the Design Curve for Masonry and Concrete Shear Walls on Figure 6.4.

Loads.

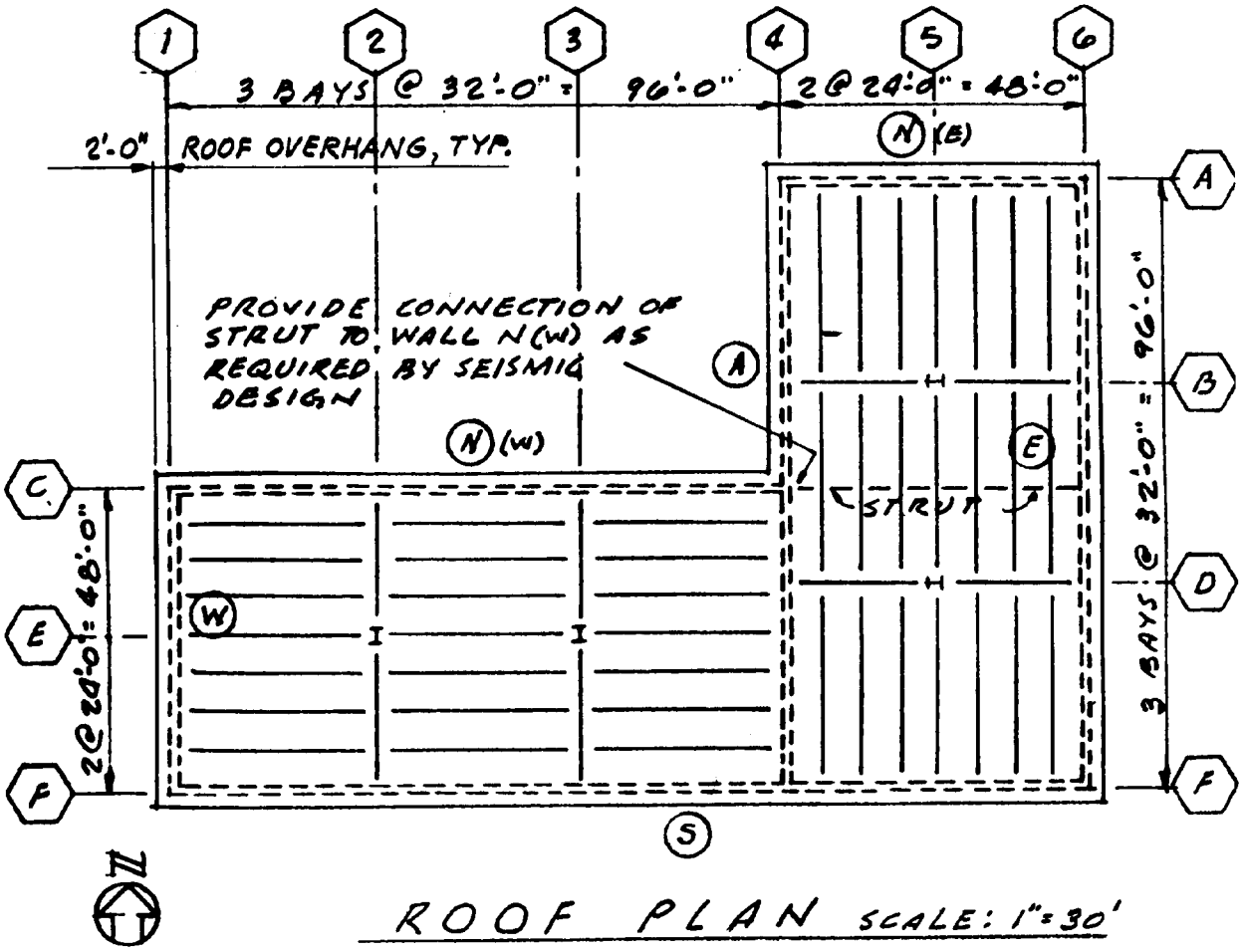
Roof:

5-ply roofing	=	6.0 p.s.f.
1" insulation	=	1.5
Steel decks	=	2.3
Steel purlins	=	3.7
Steel girders and columns	=	1.2
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	25.7 p.s.f.
Add for seismic: Partitions		10.0
Total for seismic	=	35.7 p.s.f.

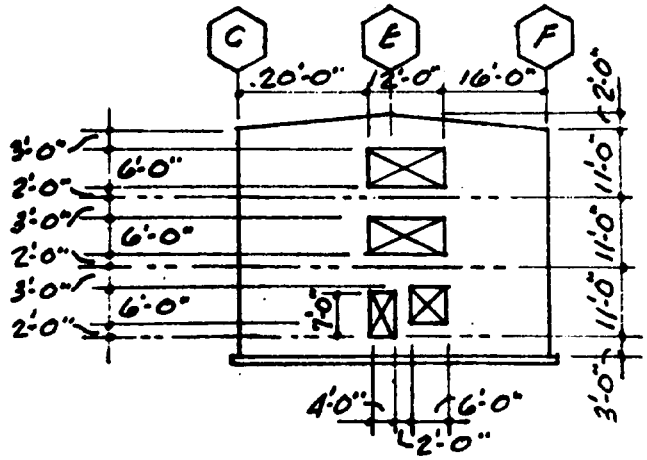
2nd & 3rd Floors:

Finish	=	1.0 p.s.f.
Steel deck	=	3.1
Concrete fill	=	32.0
Steel beams	=	5.9
Steel girders and columns	=	1.5
Partitions	=	20.0
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	74.5 p.s.f.
Live Load	=	50.0 p.s.f.

Figure D-8. L-shaped building.



ROOF PLAN SCALE: 1" = 30'

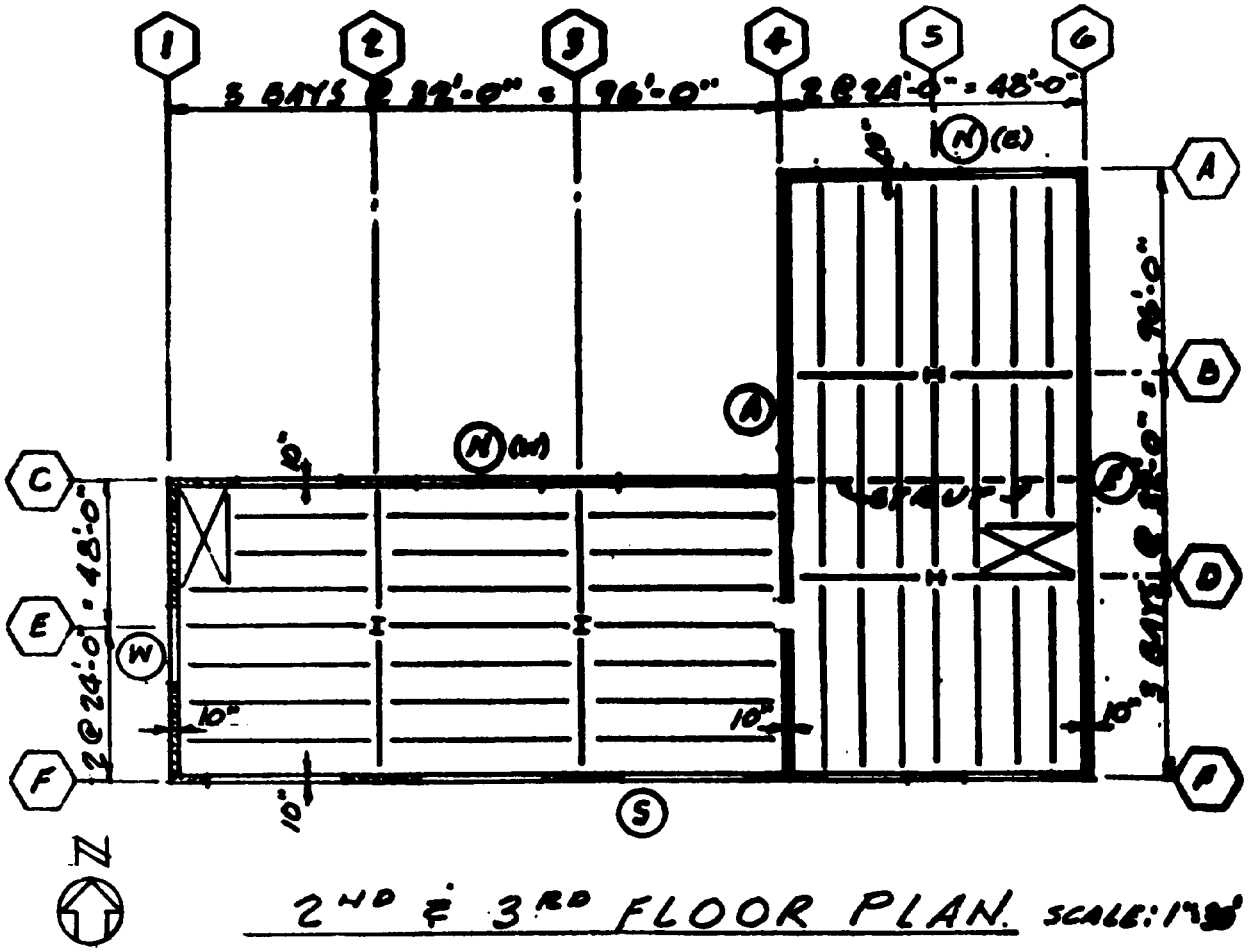


WALL (W)

NOTE:

WALL (N) (W) IS A SUPPORT FOR THE DIAPHRAGM EAST OF LINE 4 FOR E-W FORCES. THE STRUT ON LINE C IS DESIGNED FOR FORCES COLLECTED IN THE DIAPHRAGM. THE STRUT FORCE IS THEN TRANSFERRED TO THE WALL BY A SUITABLE CONNECTION AT THE LOCATION INDICATED.

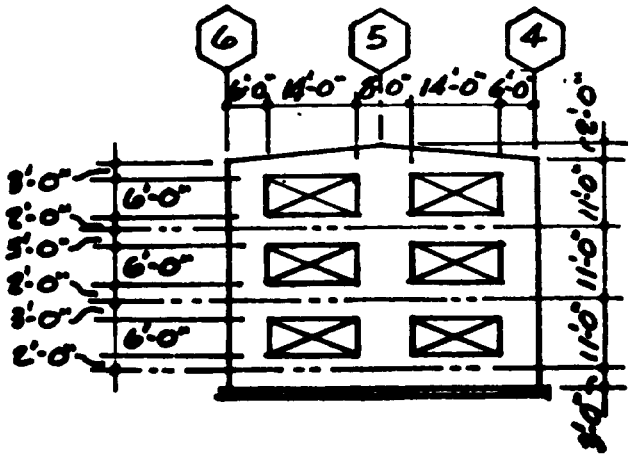
Figure D-8. Continued.



2ND & 3RD FLOOR PLAN. SCALE: 1"=30'

NOTE:

PROVIDE STRUT AND CONNECTION TO WALL ON LINE C SIMILAR TO ROOF.



WALL N (E)

Figure D-8. Continued.

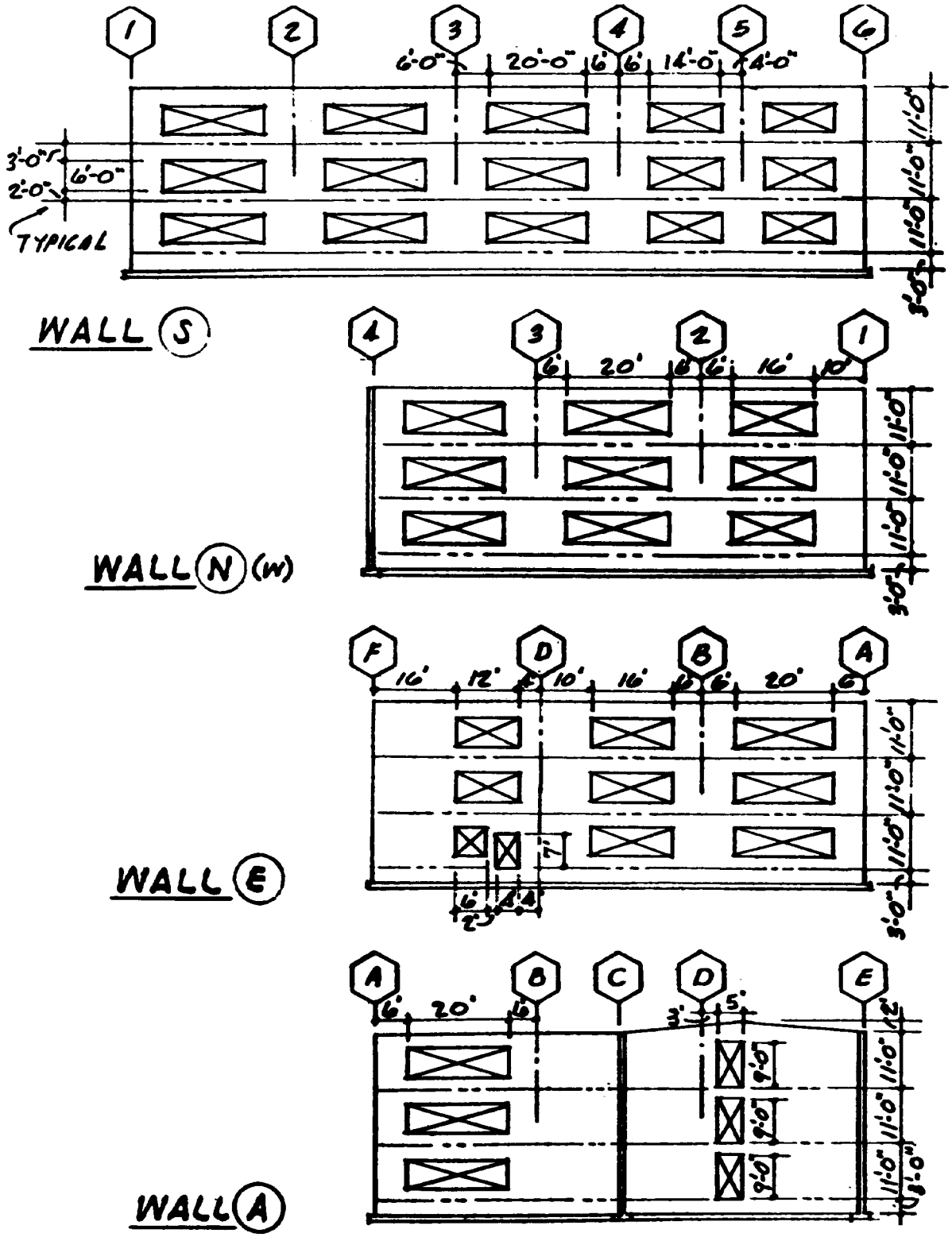
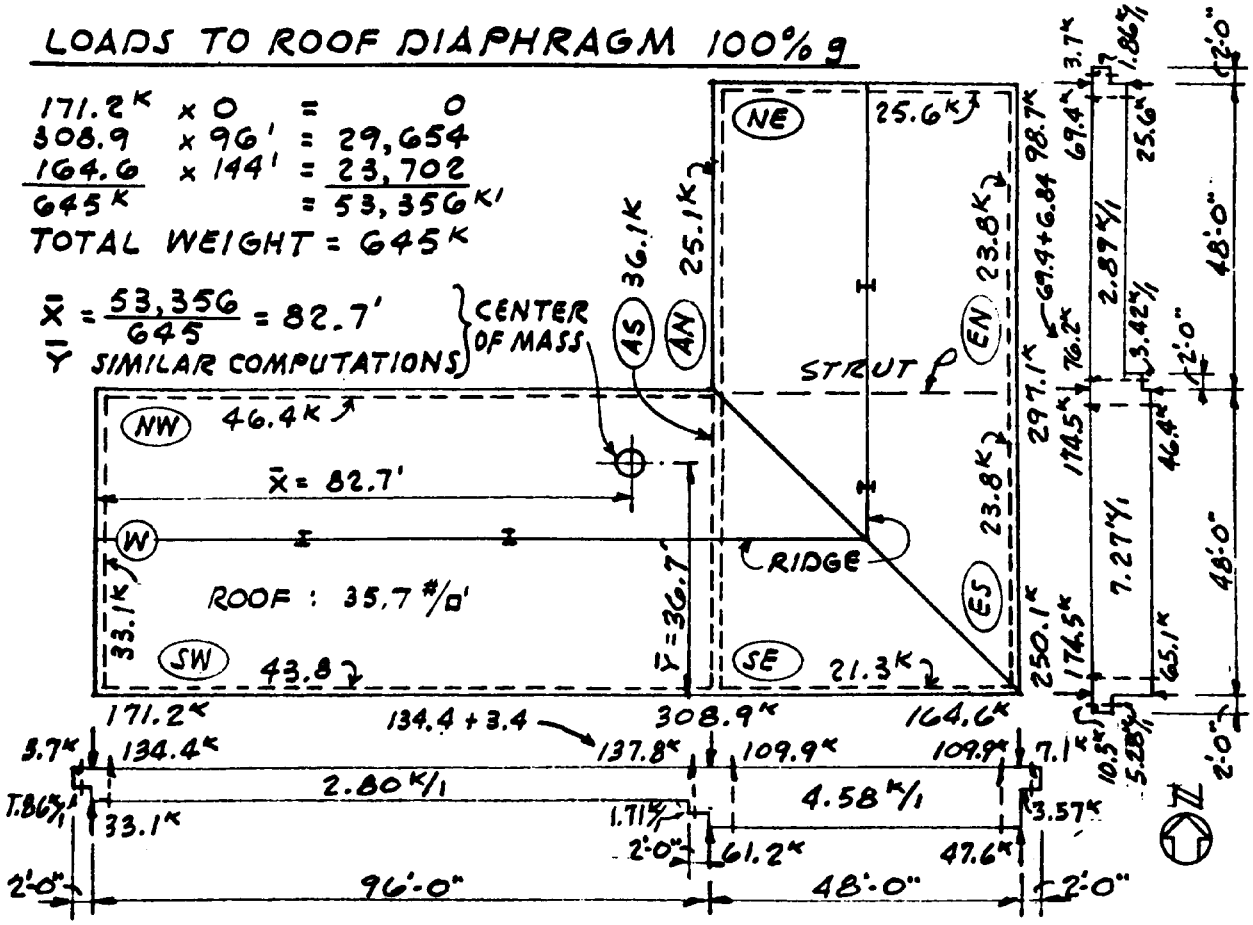


Figure D-8. Continued.



WALL	SOLID WT. #/'	% TOTAL AREA	NET WT. #/'	L'	W K	N-S WEST	N-S EAST	E-W SOUTH	E-W NORTH
NW	687	.71	488	95.17	46.4	488			
NE	818	.68	553	46.33	25.6		553		
SW	687	.67	460	95.17	43.8	460			
SE	687	.67	460	46.33	21.3		460		
W	813	.88	715	46.33	33.1			715	
EN	687	.73	502	47.17	23.8				502
ES	687	.73	502	47.17	23.8			502	
AN	687	.77	529	47.17	25.1				529
AS	813	.94	764	47.17	36.1			764	
WALLS #/'						948	1013	1981	1031
ROOF WIDTH						52'	100'	148'	52'
ROOF WEIGHT #/'						1856	3570	5284	1856
TOTAL #/'						2804	4583	7265	2887
K/'						2.80	4.58	7.27	2.89

10" WALL WT.
125 #/□ x 6.5' = 813 #/' *
125 x 5.5' = 687 #/'
*GREATER HT. TO RIDGE

Figure D-8. Continued.

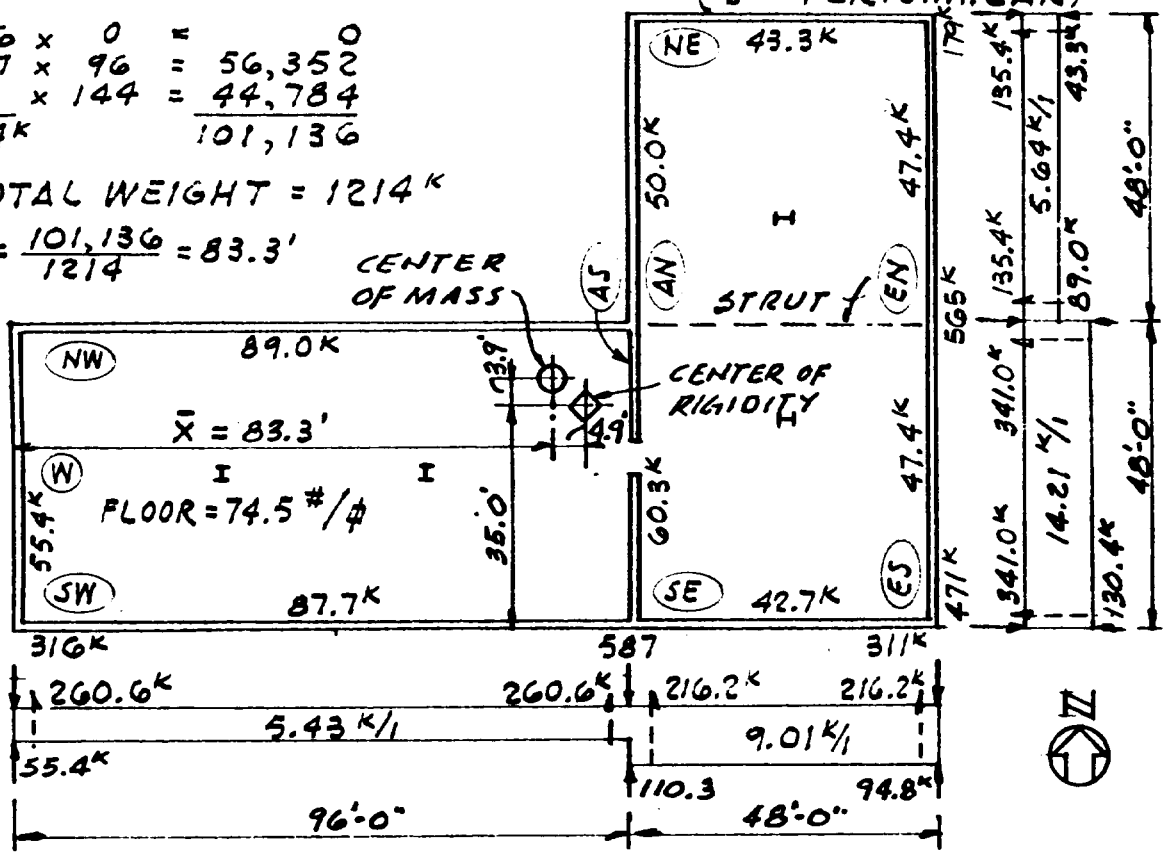
LOAD TO FLOOR DIAPHRAGM 100% 3RD FLR.

(2ND FLR. SIMILAR)

316 x 0 = 0
587 x 96 = 56,352
311 x 144 = 44,784
1214K 101,136

TOTAL WEIGHT = 1214K

$\bar{X} = \frac{101,136}{1214} = 83.3'$



WALL	SOLID WT. #/1	%TOTAL AREA	NET WT. #/1	L'	W'	N-S WEST	N-S EAST	E-W SOUTH	E-W NORTH
NW	1375	.68	935	95.17	89.0	935			
NE	"	.68	935	46.33	43.3		935		
SW	"	.67	921	95.17	87.7	921			
SE	"	.67	921	46.33	42.7		921		
W	"	.87	1196	46.33	55.4			1196	
EN	"	.73	1004	47.17	47.4				1004
ES	"	.73	1004	47.17	47.4			1004	
AN	"	.77	1059	47.17	50.0				1059
AS	"	.93	1279	47.17	60.3			1279	
10" WALL WT. 125 #/□ x 11' = 1375 #/1				WALLS #/1		1856	1856	3479	2063
				FLOOR WIDTH		48'	96'	144'	48'
				FLOOR WEIGHT		3578	7152	10728	3576
				TOTAL #/1		5434	9008	14,207	5639
				K/1		5.43	9.01	14.21	5.64

Figure D-8. Continued.

LATERAL LOADS

$$V = \frac{ZIC}{R_w} W$$

$$Z = 0.3, \quad I = 1.0, \quad R_w = 6, \quad S = 1.5$$

$$T = 0.020\,(h_n)^{3/4}, \quad h_n = 34'$$

$$= 0.020\,(34)^{3/4} = 0.282 \text{ SEC}$$

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.282)^{2/3}} = 4.36; \quad \text{USE} = 2.75$$

$$V = \frac{0.3 \times 1.0 \times 2.75}{6} W = 0.138\,W, \quad \text{Say } 0.14\,W$$

NOTE: THIS BUILDING HAS PLAN IRREGULARITY TYPE B PER SEAOC TABLE 1F. THIS INVOKES SEAOC 1H2j(4) AND 1H2j(5). SEE 1H2j(4) FOR RESTRICTIONS ON ALLOWABLE STRESSES; 1H2j(5) WILL BE MET BY PROCEDURE SHOWN ON P. 8.

VERTICAL DISTRIBUTION OF LATERAL FORCES AND OVERTURNING MOMENTS

$$F_x = \frac{(V - F_t)\,w_x h_x}{\sum w_i h_i}; \quad \text{SINCE } T < 0.7 \text{ SEC.}, \; F_t = 0$$

$$F_x = V \frac{w_x h_x}{\sum w_i h_i}$$

	h_x	Δh	w_x	$w_x h_x$	$\frac{w_x h_x}{\sum w_x h_x}$	F_x	V_x	$V_x h = \Delta M_x$	M_v
ROOF	33	11	645	21285	.347	149	149	1639	1639
3RD FLR.	22	11	1214	26708	.435	187	336	3696	5335
2ND FLR.	11	11	1214	13354	.218	94	430	4730	
TOTAL	-	-	3073	61347	1.000	430^K			10065

$$V = 0.14 \times 3073 = 430^K$$

Figure D-8. Continued.

ROOF DIAPHRAGM

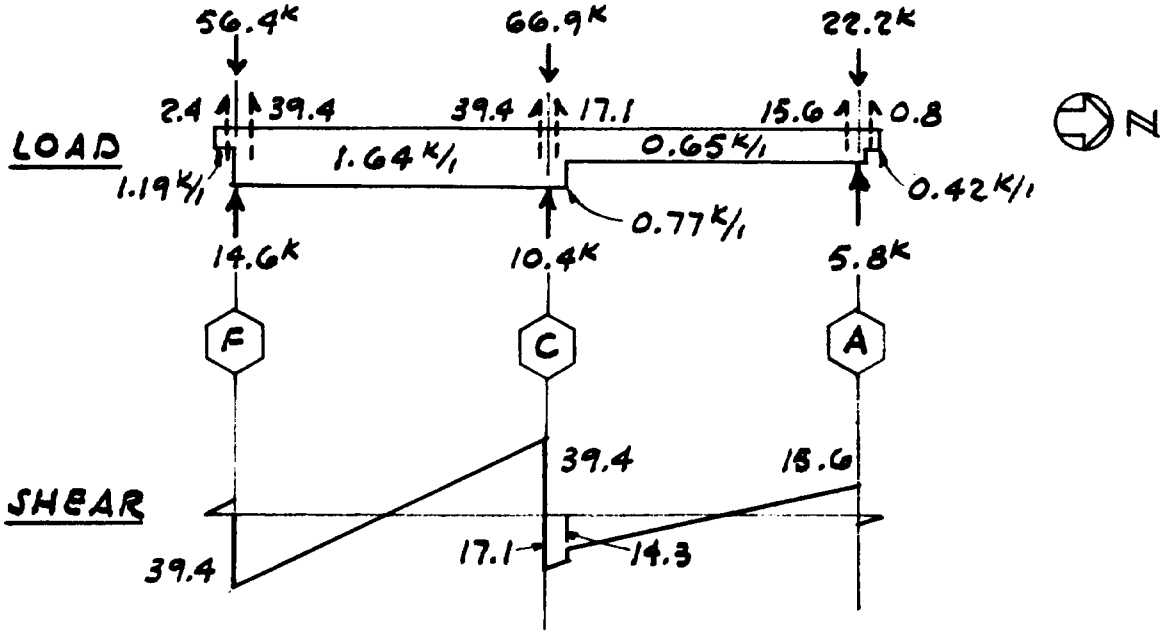
STORY FORCE = 149.5^K $\quad \frac{F}{W} = \frac{149.5}{645} = 0.232$

DIAPH. FORCE PER SEAOC 1-11

$$F_{px} = \left(\frac{\Sigma F_i}{\Sigma w_i}\right) w_{px} = \frac{149.5}{645} w_{px} = 0.232\, w_{px}$$

$$\text{MAX } F_{px} = 0.75\, Z I\, w_{px} = (0.75 \times 0.3 \times 1.0)\, w_{px} = 0.225\, w_{px} \leftarrow \text{GOVERNS}$$

EAST-WEST EQ. MULT. LOAD DIAG., p.5, BY 0.225
$\Sigma R = 145.5^K$



STRUT COLLECTS SHEAR FORCE BETWEEN LINES 4 & 6:

NORTH OF STRUT = $V = 17.1 - 2(0.77) = 15.6$
SOUTH OF STRUT = $V = \frac{48'}{144'} \times 39.4 = 13.1$
} 28.7^K

THE STRUT IS IN TENSION FOR EASTWARD FORCES, COMPRESSION FOR WESTWARD. SUITABLE CONNECTIONS MUST BE PROVIDED ACROSS EACH BEAM AS WELL AS AN END CONNECTION AT THE WALL.

NOTE: DIAPH. CALC. BECOMES MORE COMPLEX AT LOWER FLOORS BECAUSE THEY DISTRIBUTE FLOOR FORCES PLUS LOADS FROM SHEAR WALLS ABOVE ACCORDING TO RELATIVE RIGIDITIES OF SHEAR WALLS BELOW.

Figure D-8. Continued.