APPENDIX A

REFERENCES

Government Publications.

Departments of the Army, Navy, and Air Force.

TM 5-809-1/AFM 88-3, Chap 1	Structural Design Criteria—Loads
TM 5-809-2/AFM 88-3, Chap 2	Structural Design Criteria for Buildings
TM 5-809-3/AFM 88-3, Chap 3	Masonry Structural Design for Buildings
TM 5-809-10-1/NAVFAC P-355.1/ AFM 88-3, Chap 13, Sec A	Seismic Design Guidelines for Essential Buildings
TM 5-809-10-2/NAVFAC P-355.2/ AFM 88-3, Chap 13, Sec B	Seismic Design Guidelines for Upgrading Existing Buildings

Nongovernment Publications.

American Concrete Institute (ACI), P.O. Box 19150, Redford Station, Detroit, MI 48219

ACI 318-89	Building Code Requirements for Reinforced Concrete
ACI 315-86	Details and Detailing of Concrete Reinforcement
ACI SP-17-91, -17A-90	Design Handbooks

American Institute of Steel Construction (AISC), 400 North Michigan Avenue, 8th Floor, Chicago, IL 60611

Manual of Steel Construction, Allowable Stress Design, ninth edition (1989)

Specification for Structural Steel for Buildings (June 1, 1989) with Commentary

American National Standards Institute, 1430 Broadway, New York, NY 10018

ANSI A17.1-1984, Appendix F, National Elevator Standards

American Petroleum Institute, 1220 L Street Northwest, Washington, DC 20005

API Standard 650, Welded Steel Tanks for Oil Storage (1984)

American Water Works Association, 6666 West Quincy Avenue, Denver, CO 80235

ANSI/AWWA D100-84, AWWA Standard for Welded Steel Tanks for Water Storage (1979)

National Fire Protection Association, Batterymarch Park, Quincy, MA 02269

NFPA 13 (1989), Installation of Sprinkler Systems

National Forest Products Association, 1250 Connecticut Avenue Northwest, Washington, DC 20036

National Design Specification for Wood Construction (1986) and Design Values for Wood Construction (1986).

Prestressed Concrete Institute, 20 North Wacker Drive, Chicago, IL 60606

PCI-MNL-120-85, PCI Handbook: Precast and Prestressed Concrete, third edition (1981)

Steel Deck Institute, Inc., P.O. Box 9506, Canton, OH 44711 SDI-02 Diaphragm Design Manual, second edition (1987)

Structural Engineers Association of California, P.O. Box 19440, Sacramento, CA 94819-0440
Recommended Lateral Force Requirements and Commentary (1990)

Appendix B

RECOMMENDED LATERAL FORCE REQUIREMENTS 1990 EDITION

OF

GENERAL REQUIREMENTS FOR THE DESIGN AND CONSTRUCTION OF EARTHQUAKE RESISTIVE STRUCTURES

B-1. General.

a. Every structure and every portion thereof shall, as a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in these requirements. Any jurisdiction may adopt more stringent requirements.

b. Where code prescribed wind design produces greater effects the wind design shall govern but detailing requirements and limitations prescribed in these provisions shall be followed.

c. A continuous load path, or paths, with adequate strength and stiffness shall be provided which will transfer all forces from the place of application to the resisting elements.

d. The basis for the seismic design shall be stated on the structural drawings. The statement shall include: (1) the governing edition of the building code; (2) the total base shear coefficient used for seismic design; and (3) a description of the lateral force resisting system, as defined in these requirements.

e. Calculations may include the results from an electronic digital computer program. The following requirements apply to calculation submittals to a building official which include such computer output.

(1) A drawing of the complete mathematical model used to represent the structure in the computer-generated analysis shall be provided.

(2) A program description (User's Guide) shall be available and contain the information necessary to determine the nature and extent of the analysis, verify the input data, interpret the results, and determine whether the computations comply with these recommendations.

(3) Data provided as computer input shall be clearly distinguished from those computed in the program. The information required in the output shall include date of processing, program identification, identification of structures being analyzed, all input data, units and final results.

(4) The first sheet of each computer run shall be signed by the engineer responsible for the structural design.

B-2. Design criteria. The Structural Engineers Association of California (SEAOC) manual *Recommended Lateral Force Requirements and Commentary 1990 Edition* is an integral part of this manual. It is necessary to have the SEAOC manual to use this Technical Manual.

The SEAOC manual can be obtained by contacting:

Structural Engineers Association of California
P.O. Box 19440
Sacramento, California 94819-0440

APPENDIX C

SEAOC AMENDMENTS TO ACI 318

C-1. The following amendments to ACI 318-89 are compatible with the 1990 SEAOC recommendations.

C-2. Add the following provision:
Plain concrete shall not be used for structural members of buildings in Zones 2, 3, and 4.

C-3. In ACI 21.1, add the following definitions:

a. Confined core. The area within the core defined by h_c.

b. SMRF. Special moment resisting frame conforming to the provisions of ACI 21.1 through 21.7, as they apply to moment resisting frames.

c. Seismic hook. A 135-degree bend with an extension of six bar diameters, but not less than 3 inches, that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

d. Wall pier. A wall segment with a horizontal length-to-thickness ratio between 2.5 and 6, whose clear height is at least two times its horizontal length.

C-4. In ACI 21.1, add the following definition of design load combinations:
The load factors given in ACI equations 9-2 and 9-3 shall be modified to

$U = 1.4(D + L + E)$

$U = 0.9D + 1.4E$

C-5. Add to ACI 21.2.4.2:
In no case shall the compressive strength of lightweight concrete used in design exceed 6000 psi.

C-6. Add to ACI 21.3.3.4:
Stirrups shall have seismic hooks.

C-7. Add to ACI 21.4.1:
The requirements of this section also apply to all members resisting gravity loads by compression.

C-8. Add to ACI 21.4.1:
Any area of a column which, for architectural purposes, extends more than 4 inches beyond the confined core shall have minimum reinforcing as required for nonseismic columns as specified in 21.8.

C-9. Add to ACI 21.4.3.2:
Tension splices shall be proportioned as Class A tension splices in accordance with chapter 12 and shall have transverse reinforcement over the full lap splice length in accordance with 21.3.2.3.

C-10. Add to ACI 21.4.4.4:
Where the calculated point of contraflexure is not within the middle half of the member clear height, provide transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 over the full height of the member.

C-11. Replace the last sentence of ACI 21.4.4.5 with the following:
If the column terminates on a wall footing or mat, transverse reinforcement as specified in 21.4.4.1 or 21.4.4.3 shall extend into the footing or mat either the compressive development length of the largest longitudinal reinforcement or the lead length of a standard hook.

C-12. Add to ACI 21.4.4:
At any section where the ultimate capacity of the column is less than the sum of the shears V_e, computed in accordance with 21.7.1.1 for all the beams framing into the columns above the level under consideration, transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 shall be provided. For beams framing into opposite sides of the columns, the moment components may be assumed to be of opposite sign. For the determination of ultimate capacity of the columns, these moments may be assumed to results from the deformation of the frame in any one principal axis.

C-13. Add to ACI 21.5.2.4:
Splices in horizontal reinforcement shall be staggered. Splices in two curtains, where used, shall not occur in the same location.

C-14. Add to ACI 21.5.2:
Where boundary members are not required by 21.5.3.1, minimum reinforcement parallel to the edges of all shear walls and diaphragms and boundaries of all openings shall consist of twice the cross-sectional area of the minimum shear reinforcement required per lineal foot of wall.

C-15. Add to ACI 21.5.2:
Transverse reinforcement terminating at the edges of shear walls without boundary elements shall have a standard hook engaging the edge reinforcement, or the edge reinforcement shall be enclosed in 'u' stirrups of the same size and spacing and shall be spliced to the transverse reinforcement.

EXCEPTION: Walls with a factored shear, V_u, in the plane of the wall less than $A_{cv} \sqrt{f'_c}$ need not meet these requirements.

C-16. Add to ACI 21.5.3:
Structural steel members conforming to SEAOC Chapter 4 and encased monolithically in the walls at the edges may be used for boundary members provided adequate shear transfer is provided between the steel and the concrete.

C-17. Add to ACI 21.5:
A cast-in-place topping on a precast floor system may serve as the diaphragm provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces.

C-18. Add to ACI 21.5:
Minimum thickness of diaphragms. Diaphragms used to resist prescribed lateral forces shall not be less than 2 inches thick. Topping slabs placed over precast floor and roof elements shall not be less than 2½ inches thick.

C-19. At the end of ACI 21.5, add a new section: Wall piers.

a. Wall piers not designed as part of a SMRF shall have transverse reinforcement designed to satisfy the requirements of paragraph b below.

EXCEPTION 1. Wall piers that satisfy ACI 21.8.

EXCEPTION 2. Wall piers along a line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness at least six times the sum of the stiffnesses of all wall piers along that line.

b. Transverse reinforcement shall be designed to resist the shear forces determined from ACI 21.7.1.2 and shall meet the requirements of ACI 21.7.2.1. When the axial compressive force, including earthquake effects, is less than $A_q f_c'/20$, transverse reinforcement in wall piers may have standard hooks on each end in lieu of hoops. Spacing of transverse reinforcement shall not exceed six inches. The zone of transverse reinforcement shall be extended beyond the wall pier clear height for at least the development length of the largest longitudinal reinforcement in the wall pier.

c. Wall segments with horizontal length-to-thickness ratio less than 2½ shall be designed as columns.

C-20. Add to ACI 21.6.1:
Where longitudinal beam reinforcing bars extend through a joint, the column depth in the direction of loading shall not be less than 20 times the diameter of the largest longitudinal bar.

C-21. In ACI 21.7.1.3, change the specification of load combinations from 9.2 to paragraph C-4.

C-22. Add to 21.7.2.1:
Earthquake-induced shear force is the shear induced by the flexural moment strengths of the beams calculated in accordance with 21.7.1.1.

C-23. Delete ACI 21.7.3.1.

C-24. Revise the title of ACI 21.8 to read:
Frame members not part of the lateral force resisting system.

C-25. Delete ACI 21.8.1 and substitute the following:
All frame members assumed not to be part of the lateral force resisting system shall be shown to be adequate for vertical load carrying capacity with the structure assumed to have deformed laterally in accordance with SEAOC 1H2d. Such members shall satisfy the minimum reinforcement requirements specified in 21.3.2.1 and 21.5.2.1 and chapters 7, 10, and 11.

C-26. In ACI 21.9.3, delete the word "factored" in reference to "gravity loads."

APPENDIX D

DESIGN EXAMPLES—BUILDING SYSTEMS

D-1. Introduction. This appendix gives illustrative examples for designing various types of lateral systems. Generally, the calculations determine earthquake lateral forces and their distribution to the resisting elements of the buildings; some examples covering frames, walls, diaphragms, and foundations are essentially complete. Calculations are not given where ordinarily accepted design procedures are involved, such as sizing and detailing members once forces are determined.

D-2. Use of this appendix. This appendix is purely advisory; it is not intended to place super-restrictions on the manual. This appendix is not a handbook for the inexperienced designer. Neither the manual, nor the manual supplemented by this appendix, can replace good engineering judgment in specific situations. Designers are urged to study the entire manual.

D-3. Commentary.

a. Unless otherwise indicated, all design examples in this appendix are based on Zone 4, where Z = 0.40. But the principles and methods for determining lateral forces are alike for all zones.

b. Examples D-1, D-2, D-3, and D-5 are for the same basic building, using (1) bearing walls, (2) concrete frames, (3) steel frames, and (4) frames in combination with shear walls (a dual bracing system), respectively. These examples tend to illustrate the relationship between architectural features (fenestration and materials of construction) and structural design.

c. A 10-pound-per-square-foot weight is added to the roof for the seismic effect of the upper half of the top-story partitions.

d. It is assumed that stairs are detailed so as not to transmit shears from floor to floor. Also, removable and special partitions (such as utility room walls) will be made flexible or isolated so as not to affect the distribution of lateral loads or to act as shear walls.

e. Metal-deck roofs are considered to form flexible diaphragms, and roof loads are distributed according to tributary area rather than relative rigidity of walls below.

D-4. Design examples.

Fig. No.	*Description of Design Examples*
D-1	*Box System.* A two-story building with bearing walls in concrete using a series of interior, vertical load-carrying columns and girder bents.
D-2	*Concrete Ductile Moment Resisting Space Frame.* A three-story building with a complete ductile moment resisting space frame in concrete without shear walls.
D-3	*Steel Ductile Moment Resisting Space Frame and Steel Braced Frame.* A three-story building with transverse special moment resisting frames and longitudinal frames with chevron bracing.
D-4	*Dual Bracing System.* A two-story building in concrete with a ductile moment resisting space frame and with shear walls.
D-5	*Dual Bracing System.* A three-story building with a ductile moment resisting space frame in structural steel and with shear walls in concrete.
D-6	*Wood Box System.* A two-story wood framed building, using wood floor and roof decks, and wood stud walls with plywood sheathing.
D-7	*Special Configuration.* A one-story building with concrete bearing walls on three sides and open on one side.
D-8	*L-Shaped Building.* A three-story building with bearing walls in concrete, using a series of interior vertical load-carrying columns and girder bents.

DESIGN EXAMPLE D-1

Building With A Bearing Wall:

Description of Structure. A two-story administration building with bearing walls in concrete, using a series of interior, vertical load-carrying column and girder bents. The structural concept is illustrated on Sheets 3 and 4.

Construction Outline.

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

2nd Floor :
- 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 dry-wall, except concrete as structurally required.

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 second 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 second floor forms a rigid diaphragm. The shear walls react to the forces from the diaphragm, therefore the relative rigidities of the various walls and the individual piers must be determined. This is necessary so that a logical and consistent distribution of story shears to each wall and pier can be made. The wall analysis utilizes the Design Curve for Masonry and Concrete Shear Walls on Figure 6- 4.

Discussion. A 10 psf partition load is included in the seismic roof loading but is not included in the vertical design. The stairs are isolated so that they will not transmit shears from floor to floor. The walls along Lines A C 3 & 5 act as vertical cantilever beams joined by struts at the floor lines. The overturning moments are distributed to the individual piers in proportion to the pier stiffnesses. The end wall along Line 7 abuts an existing building, therefore a wall with no openings is provided. The spandrels in wall along Line 1 must be designed to transfer vertical shears due to shear wall action.

Figure D-1. Box system.

Loads.

Roof:

5-ply roofing	=	6.0 p.s.f.
1" insulation	=	1.5
Steel deck	=	2.3
Steel purlins	=	3.7
Steel girders & columns	=	1.2
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load		25.7 p.s.f.
Add for seismic: Partitions		10.0 p.s.f.
Total for seismic		35.7 p.s.f.*
Live Load		20 p.s.f. (no snow)

2nd Floor:

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

Materials.

Structural steel .... F_y = 36 k.s.i.
Concrete .... f'_c = 4,000 p.s.i., $E_c = 3.6 \times 10^6$ psi
Reinforcing steel .... f_y = 40,000 p.s.i.
Allowable soil pressure ... = 3,000 p.s.f. Vertical Load
Allowable soil pressure ... = 4,000 p.s.f. Vertical plus Seismic

*Weight of shear walls are not included here. The weight of the concrete shear walls are calculated on pages 4 and 5. The weights of the exterior windows and architectural wall panels are included in the partition weights.

Figure D-1. Continued

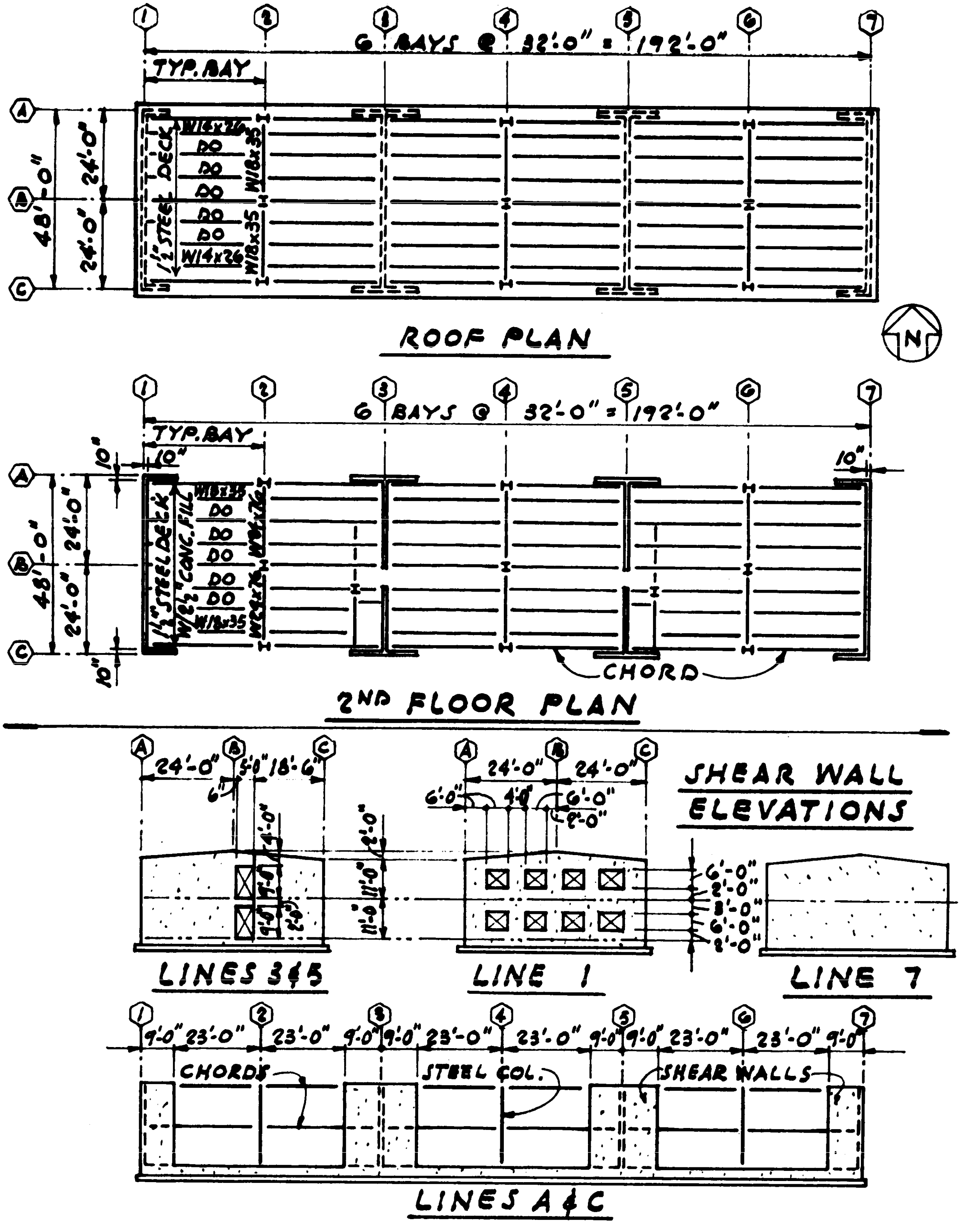
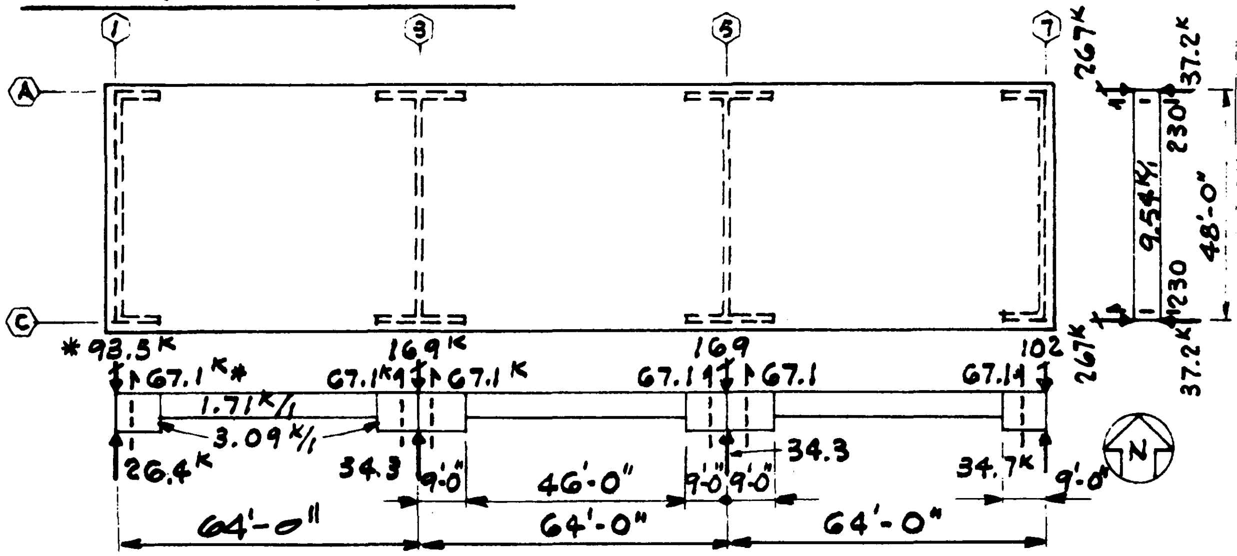


Figure D–1. Continued

DISTRIBUTION OF BUILDING WEIGHT TO ROOF DIAPHRAGM @ 100 % G



10" CONC. WALLS 0.125 KSF

SIDEWALLS A & C: 5.5'x0.125=0.688K/l, x2 = 1.38 K/l

CROSSWALLS 1,3,5&7: 6.0'x0.125=0.75K/l, x(2x.91+.76+1.0)=2.69K/l

WALL ON 1 : .76 x 0.75 x 46.33' = 26.4 (76 % SOLID)
WALL ON 7 : 1.0 x 0.75 x 46.33' = 34.7 (100 % SOLID)
WALL ON 3 & 5 : .91 x 0.75 x 46.33' = 31.6 (91 % SOLID)

WALLS ON A & C:

9' WALLS	0.688 x 9'	= 6.2K	x 2 = 12.4K
18' WALLS		= 12.4K	x 2 = 24.8K
			37.2K

	(P.2)	E-W LOADS	N-S LOADS
ROOF	0.0357 KSF	x 192' = 6.85 K/l	x 48' = 1.71 K/l
WALLS		2.69	1.38
		9.54 K/l	3.09 K/l

*SAMPLE CALCULATION OF WALL $\boxed{1}$

DISTRIBUTION OF DIAPH. WT TO WALL $\boxed{1}$ = 1.71 K/l x 64'/2 = 54.7 }
" " SIDEWALL " " " = 1.38 x 9' = 12.4 } 67.1K

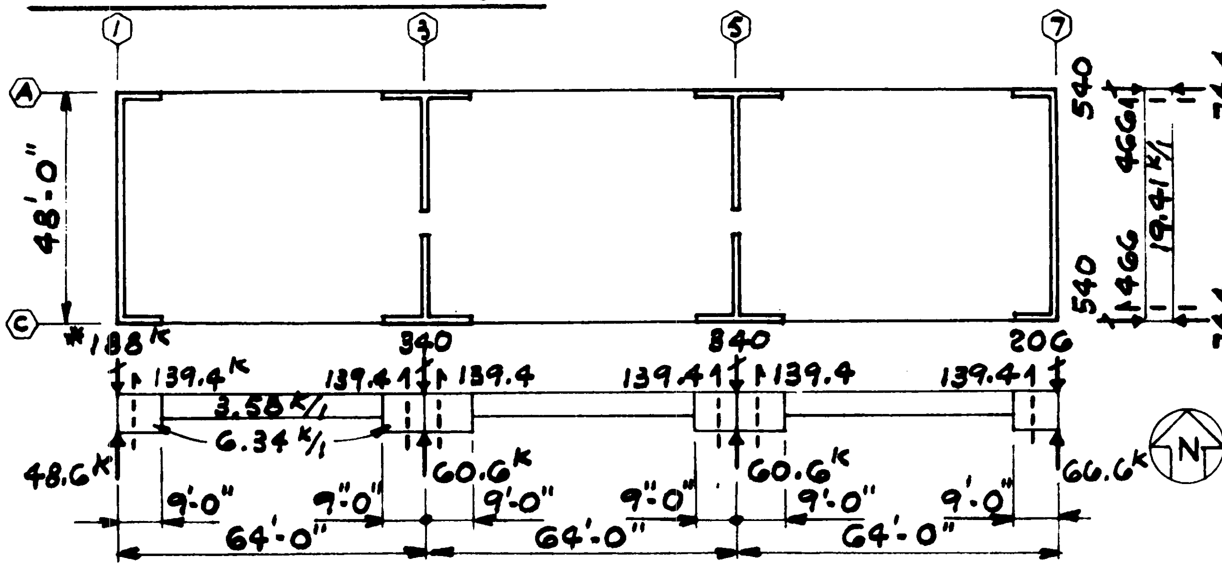
THE WT CONTRIBUTION FROM IN-PLANE SHEAR WALL $\boxed{1}$ (SEE ABOVE) 26.4

THE TOTAL WT CONTRIBUTION TO SHEAR WALL $\boxed{1}$ 93.5K

TOTAL TRIB. WT OF ROOF & ALL WALLS W_R = 584K @ 100 % G

Figure D–1. Continued.

DISTRIBUTION OF BUILDING WEIGHT AT 2ND FLOOR DIAPHRAGM @ 100 % G



10" CONC. WALLS = 0.125 KSF

WALL ON 1 : .73 × 0.125 × 11.5' × 46.33 = 48.6 K

WALL ON 7 : 1.0 × 0.125 × 11.5 × 46.33 = 66.6 K

WALLS ON 3 & 5 : .91 × 0.125 × 11.5 × 46.33 = 60.6 K

WALLS ON A & C :

9' WALLS 0.125 × 11' × 9' = 12.4 K × 2 = 24.8 K

18' WALLS = 24.8 K × 2 = 49.6 K

74.4 K

	(P.2)	E-W LOADS	N-S LOADS
FLOOR	0.0745 KSF	× 192' = 14.30 K/l	× 48' = 3.58 K/l
WALL	0.73 × 0.125 KSF × 11.5' =	1.05	2 × 0.125 × 11' = 2.76
	1.0 × 0.125 KSF × 11.5' =	1.44	6.34 K/l
	2 × .91 × 0.125 KSF × 11.5' =	2.62	
		19.41 K/l	

*SAMPLE CALCULATION FOR WALL $\boxed{1}$

DISTRIBUTION OF DIAPH. WT TO WALL $\boxed{1}$ = 3.58 K/l × 64/2 = 114.6 } = 139.4 K

" " SIDEWALL " " " = 2.76 × 9' = 24.8 }

THE WT CONTRIBUTION FROM THE IN-PLANE SHEAR WALL $\boxed{1}$ (SEE ABOVE) = 48.6

THE TOTAL WT CONTRIBUTION TO SHEAR WALL $\boxed{1}$ = 188.0 K

TOTAL TRIB WT OF 2ND FLR DIAPH & 2ND FLR TRIB WALLS W_x = 1080 K @ 100% G

Figure D-1. Continued.

LATERAL FORCES (BOTH DIRECTIONS)

$V = \dfrac{ZIC}{R_w} W$ SEAOC FORMULA 1-1

$Z = 0.40$ (ZONE 4) TABLE 1-A

$I = 1.0$ TABLE 1-C

$R_w = 6$ (BEARING WALL SYSTEM CONCRETE SHEAR WALLS) TABLE 1-G

$S = 1.5$ (ASSUME SOIL S_3) TABLE 1-B

$h_n = 22$ FT. (AVG.)

$C_t = 0.020$

$T = C_t (h_n)^{3/4} = 0.020 (22)^{3/4} = 0.203$ FORMULA 1-3

$C = 1.25S/(T)^{2/3}$ FORMULA 1-2

$= 1.25 \times 1.5 \div (0.203)^{2/3} = 5.43$

BUT NEED NOT EXCEED 2.75

$V = \dfrac{0.40 \times 1.0 \times 2.75}{6} W = 0.183W$

$= 0.183 \times 1614 = 295^K$, SAY 300^K

LEVEL	h FT.	Δh FT.	w K	Σw K	wh K·FT.	wh/Σwh	F_x K	V K	ΔM_{OT} K·FT.	M_{OT} K·FT.
R	22		534		11,748	.50	150			
		11		534				150	1650	1650
2	11		1080		11,880	.50	150			
		11		1614				300	3300	4950
GRD	0									
Σ			1614		23,628	1.00	300		4950	

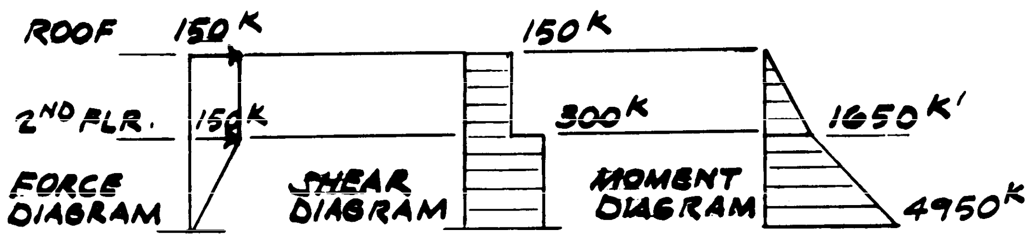
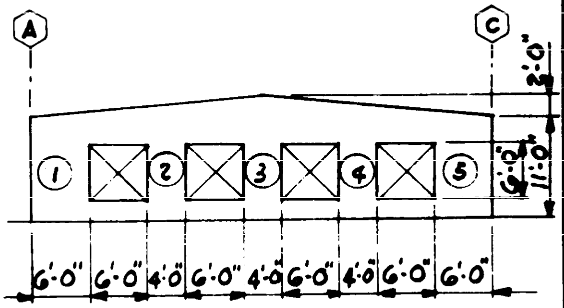
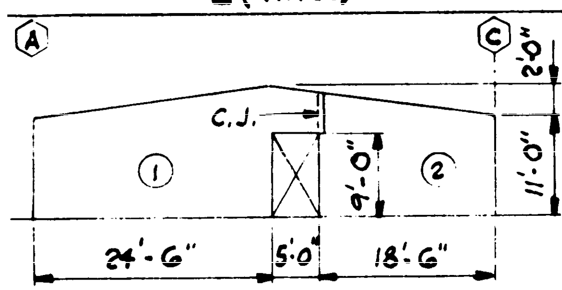


Figure D–1. Continued.

RELATIVE RIGIDITIES
2ND STORY WALLS



WALLS	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
	1 & 5 (CORNER FIXED)	6'	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
	2,3,4 (RECT. FIXED)	6'	4'	1.5	0.22	4.5 × 3 PIERS = 13.5	
						ΣR=35.7	
WALL 1 - 2ND STORY					Δ(1-5) = 1/R = 0.028		
	SOLID WALL (CORNER CANT)	12' AVE	48'	0.25	0.018		
Δ(WALL) = 0.018 - 0.008 + 0.028 = 0.038	SUBTRACT BAND @ WINDOW (CORNER CANT)	6'	48'	0.125	⟨0.008⟩		
R(WALL) = $\frac{1}{\Delta(\text{WALL})}$ = 26.3					ΣΔ(WALL) = 0.038		ΣR = $\frac{1}{\Sigma\Delta W}$ = 26.3



WALLS	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
	1 (CORNER CANT.)	12' AVE	24.5	0.49	0.044	22.7	
WALL 3 - 2ND STORY (WALL 5 SIM.)	2 (CORNER CANT.)	12'	18.5	0.65	0.065	15.4	
FOR THIS EXAMPLE, CONTROL JT. IS PROVIDED TO MAKE WALL MORE FLEXIBLE, THEREBY DISTRIBUTING MORE LOAD TO WALL 7						Σ 38.1	ΣR = 38.1

NOTE: SINCE ALL WALLS ARE THE SAME THICKNESS (i.e. 10") THE VALUES FROM FIG. 6-4 FOR 12" WALLS MAY BE USED FOR RELATIVE RIGIDITIES WITHOUT ADJUSTMENT.

Figure D-1. Continued.

RELATIVE RIGIDITIES
2ND STORY

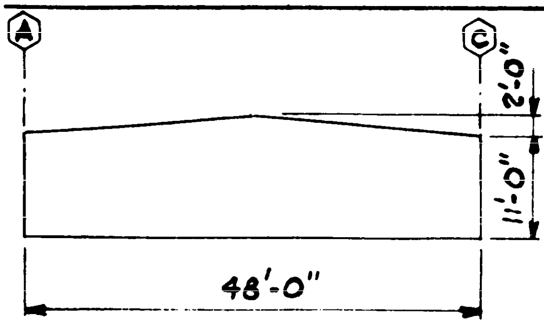
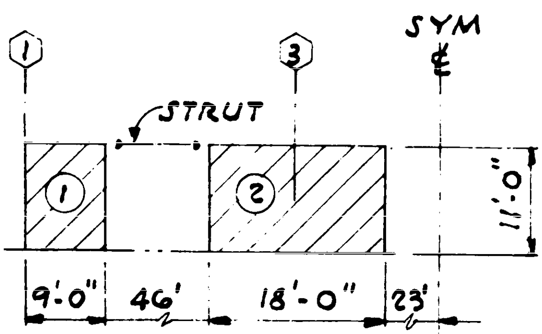
WALL	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
 WALL 7 - 2ND STORY	1 CORNER CANT	12 AVE	48	0.25	0.018	55.5	55.5
 WALL C - 2ND STORY (WALL A - SIM.)	1, 4 CORNER CANT	11	9	1.22	0.22	4.5 × 2 PIERS 9.1	
	2, 3 RECT. CANT	11	18	0.61	0.075	13.3X 2 PIERS 26.6	ΣR=35.6

Figure D-1. Continued.

RELATIVE RIGIDITIES

1ST. STORY WALLS

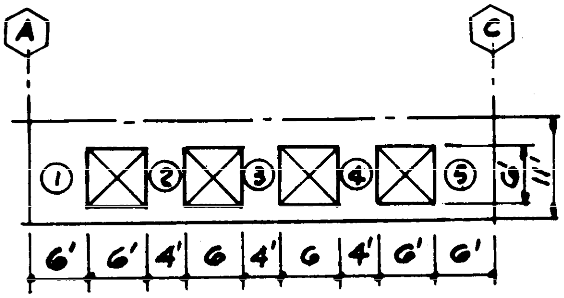
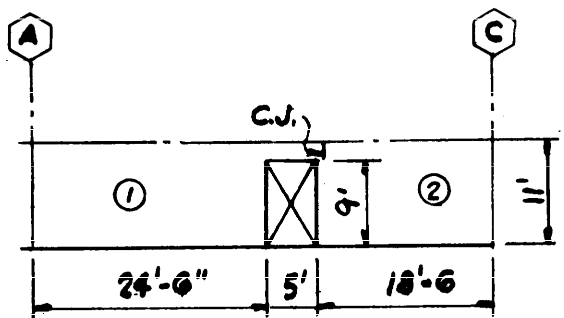
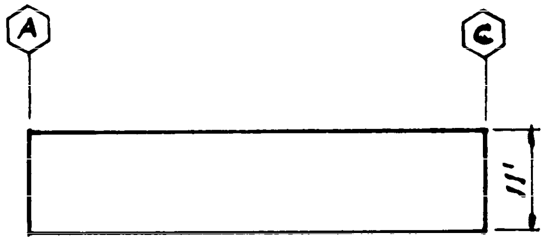
WALL	PIER	H	D	H/D	Δ FIG.6-4	R	ΣR WALL
	1 & 5 (CORNER CANT.)	6	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
	2,3,4	6	4	1.5	0.22	4.5 × 3 PIERS = 13.5	
WALL 1 1ST STORY					Δ(1-5) = $\frac{1}{R}$ = 0.028	ΣR = 35.7	
	SOLID WALL (CORNER CANT.)	11'	48'	.23	0.017		
Δ(WALL) = 0.017 − 0.008 + 0.028 = 0.037	SUBTRACT BAND @ 6' WINDOW (CORNER CANT.)		48'	0.125	⟨0.008⟩		
$\Sigma R_{WALL} = \frac{1}{0.037} = 27$					ΣΔ(WALL) = .037		$\Sigma R = \frac{1}{\Sigma \Delta W}$ = 27
	1 (CORNER CANT)	11'	24.5	0.45	0.037	27.0	
WALL 3 - 1ST STORY (WALL 5 - SIM.)	2 (CORNER CANT.)	11'	18.5	0.59	0.056	17.9	
						44.9	ΣR = 44.9

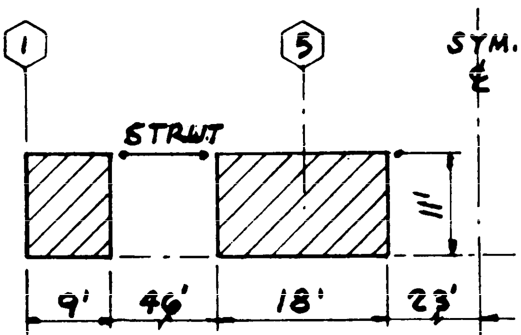
Figure D-1. Continued.

RELATIVE RIGIDITIES

1ST STORY WALLS



WALL 7 1ST. STORY

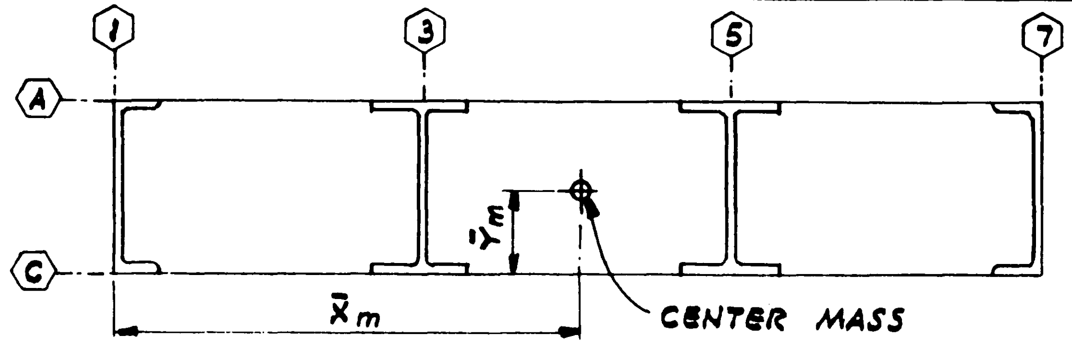


WALL C - 1ST STORY
(WALL A SIM.)

WALL	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
WALL 7 1ST. STORY	1 (CORNER CANT.)	11'	48'	0.23	.017	58.8	58.8
WALL C - 1ST STORY (WALL A SIM.)	1,4 (CORNER CANT.)	11'	9'	1.22	0.22	4.5× 2 PIERS = 9.0	
	2,3 (RECT. CANT.)	11'	18'	0.61	0.075	13.3× 2 PIERS = 26.6	ΣR = 35.6

Figure D–1. Continued.

CENTER OF MASS AND CENTER OF RIGIDITY
ROOF DIAPHRAGM



			CENTER OF MASS			CENTER OF RIGIDITY		
	X	Y	W (P.4)	W·Xm	W·Ym	RIGIDITY R	R·Xr	R·Yr
WALL 1	0.42'		93.5	39				
WALL 3	64		169	10816				
WALL 5	128		"	21632				
WALL 7	191.58		102	19541				
			533.5	52028				
WALL A		47.58	267		12704			
WALL C		0.42	"		112			
			534		12816			

CENTER OF MASS OF ROOF DIAPHRAGM;

$$\bar{X}_m = \frac{52028}{533.5} = 97.5 \qquad \bar{Y}_m = \frac{12816}{534} = 24'$$

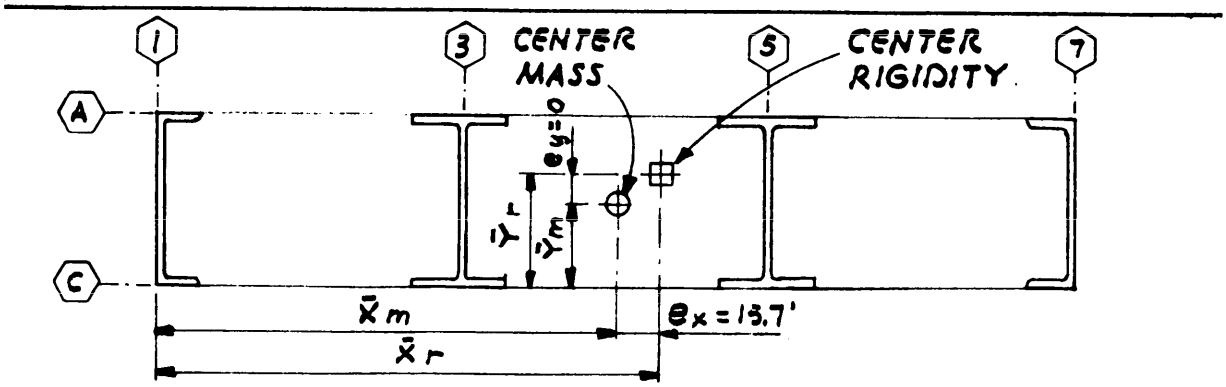
CENTER OF RIGIDITY:

CALCULATIONS NOT REQUIRED SINCE ROOF DIAPHRAGM IS FLEXIBLE AND SEISMIC FORCES ARE DISTRIBUTED BY TRIBUTARY AREA.

CENTER OF MASS OF ROOF DIAPHRAGM IS REQUIRED SINCE THE ECCENTRICITY OF THIS MASS EFFECTS THE TORSIONAL FORCE ON THE RIGID 2ND FLOOR DIAPHRAGM BELOW.

Figure D-1. Continued.

CENTER OF MASS AND CENTER OF RIGIDITY
2ND FLOOR DIAPHRAGM



			CENTER OF MASS			CENTER OF RIGIDITY		
	X	Y	W (P.5)	W·Xm	W·Ym	RIGIDITY R (P.10)	R·Xr	R·Yr
WALL 1	0.42		188	79		27	11	
WALL 3	64		340	21760		44.9	2874	
WALL 5	128		"	43520		"	5747	
WALL 7	191.58		206	39465		58.8	11265	
			*1074	104824		175.6	19897	
WALL A		47.58	540		25693	35.6	NONE	1694
WALL C		0.42	"		227	"	NONE	15
			*1080		25920	71.2		1709

* TOTAL WTS. DO NOT CORRESPOND DUE TO ROUNDING OFF.

CENTER MASS: $\bar{X}_m = \frac{104824}{1074} = 97.6'$ CENTER RIGIDITY: $\bar{X}_r = \frac{19897}{175.6} = 113.3'$

$\bar{Y}_m = \frac{25920}{1080} = 24'$ $\bar{Y}_r = \frac{1709}{71.2} = 24'$

ECCENTRICITY OF ROOF MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

$e_x = 113.3 - 97.5 = 15.8'$ $e_y = 24' - 24' = 0$

(P.11)

ECCENTRICITY OF 2ND FLR MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

$e_x = 113.3' - 97.6' = 15.7'$ $e_y = 24' - 24' = 0$

Figure D–1. Continued.

DISTRIBUTION OF SEISMIC FORCES

FROM ROOF DIAPHRAGM TO WALLS BELOW

THE ROOF DIAPHRAGM IS FLEXIBLE; THEREFORE THE FORCES ARE OBTAINED BY THE TRIBUTARY AREA METHOD. THE 100%g FORCES OF P.4 ARE SCALED IN PROPORTION TO THE 150^K ROOF FORCE (P.6).

WALL	100%g	$F_R = 150$
1	94	26.4
3	169	47.5
5	169	47.5
7	102	28.6
	534^K	150^K

WALL	100%g	$F_R = 150$
A	267	75
C	267	75

FROM SECOND FLOOR DIAPHRAGM TO WALLS BELOW

THE SECOND FLOOR DIAPHRAGM IS RIGID; THEREFORE IT REDISTRIBUTES FORCES FROM THE SECOND FLOOR AND ABOVE.

N-S DIRECTION

P.6 P.12 P.6 P.12

$$M_T = \Sigma F_x e_x = \underset{ROOF}{150^K} \times 15.8' + \underset{2ND}{150} \times 15.7 = 4725 K'$$

$$M_A = 300^K \times (0.05 \times 192') = \qquad 2880 K'$$

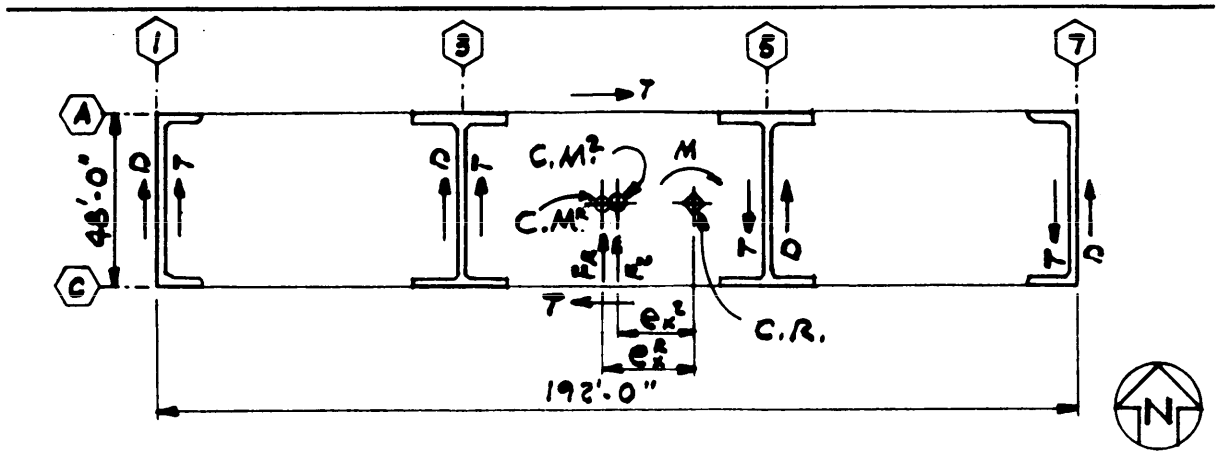
E-W DIRECTION

$$M_T = \qquad 0$$

$$M_A = 300 \times (0.05 \times 48) = \qquad 720 K'$$

Figure D-1. Continued.

DIAGRAM SHOWING DIRECT SHEAR AND TORSIONAL SHEAR FORCES IN PLAN



NOTES:

C.R. = CENTER OF RIGIDITY
C.M. = CENTER OF MASS
T = TORSIONAL SHEAR FORCE
D = DIRECT SHEAR FORCE
F_R = FORCE FROM ROOF DIAPH (N-S)
F_2 = FORCE FROM 2ND DIAPH (N-S)
e_x^R = ECCENTRICITY OF ROOF MASS W/ RESPECT TO 2ND FLR C.R
e_x^2 = " " 2ND FLR " " " " " "
M_T = TORSIONAL MOMENT
M_A = ACCIDENTAL TORSION
$M = M_T + M_A$

$$V_D = \frac{K}{\sum K} V$$

$$V_T = \frac{Kd^2}{\sum Kd^2} \cdot \frac{M_T}{d} = \frac{Kd}{\sum Kd^2} \cdot M_T$$

$$V_A = \frac{Kd^2}{\sum Kd^2} \cdot \frac{M_A}{d} = \frac{Kd}{\sum Kd^2} \cdot M_A$$

Figure D-1. Continued.

DISTRIBUTION FROM SECOND FLOOR DIAPHRAGM TO WALLS BELOW — CONTINUED

N-S DIRECTION										
WALL	K	$\frac{K}{\Sigma K}$	V_D	d	d^2	Kd^2	$\frac{Kd}{\Sigma Kd^2}$	V_T	V_A	V_W
1	27.0	.154	46.2	112.9	12,746	344,142	.00353	16.7	10.2	73.1
3	44.9	.256	76.8	49.3	2,430	109,107	.00256	12.1	7.4	96.3
5	44.9	.256	76.8	14.7	216	9,698	.00076	- 3.6	2.2	75.4
7	58.8	.334	100.2	78.3	6,131	360,503	.00533	-25.2	15.4	90.4
	175.6		300							
A	35.6	.500	—	23.6	557	19,829	.00097	4.6	2.8	7.4
C	35.6	.500	—	23.6	557	19,829	.00097	4.6	2.8	7.4
	71.2					863,108				

E-W DIRECTION										
1			—					—	2.5	2.5
3			—					—	1.8	1.8
5			—					—	0.5	0.5
7			—					—	3.8	3.8
A			150					—	0.7	150.7
C			150					—	0.7	150.7
			300							

Figure D-1. Continued.

DISTRIBUTION OF SEISMIC FORCES & OVERTURNING MOMENTS

NORTH - SOUTH DIRECTION

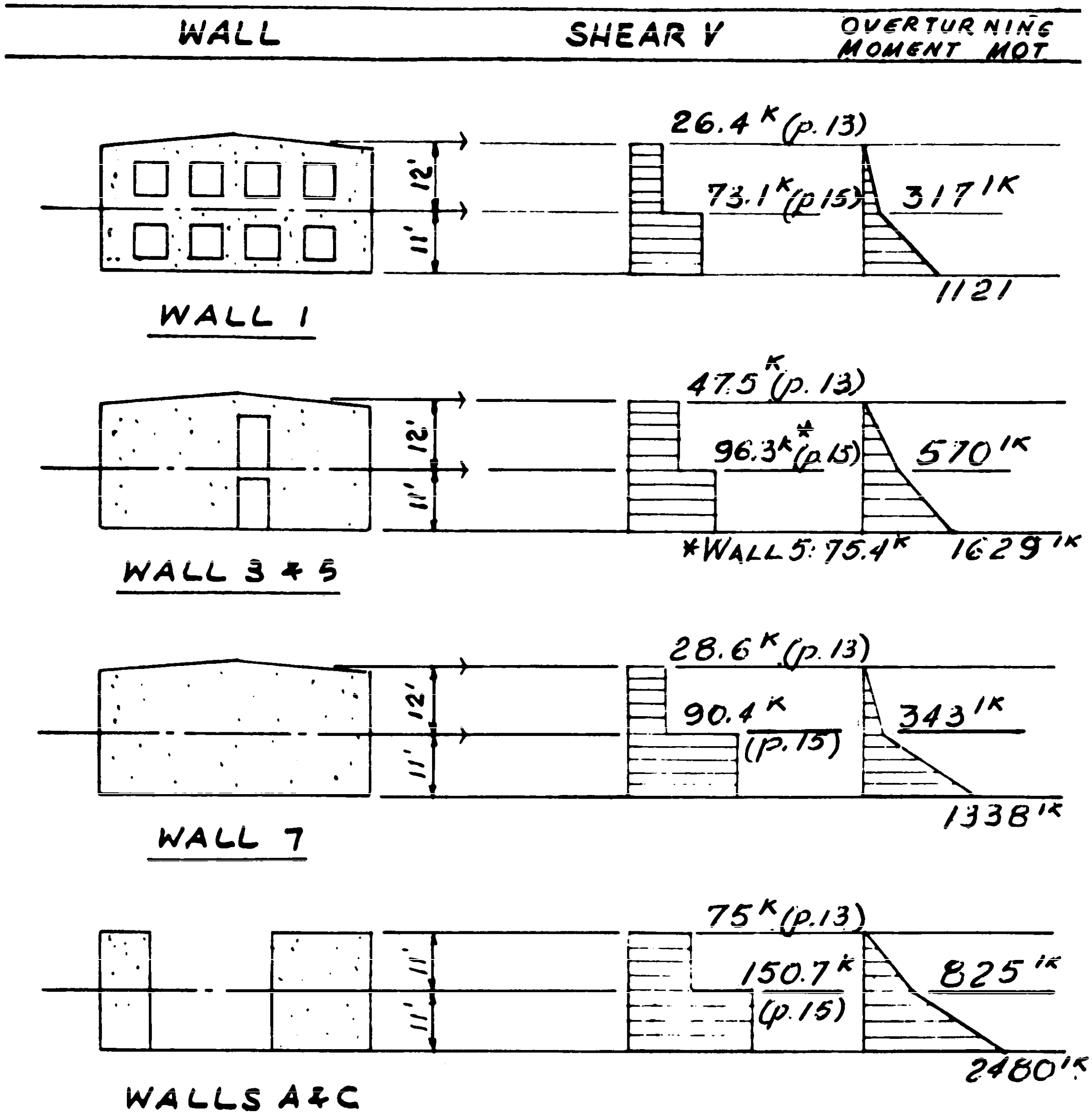


Figure D-1. Continued.

VERTICAL LOAD DESIGN

THESE CALCULATIONS ARE EXTRACTED FROM THE VERTICAL LOAD CALCULATION WHICH ARE REQUIRED FOR COMBINING WITH LATERAL LOADS.

WALL	DEAD LOAD	LIVE LOAD
1	(p. 2) ROOF 24.5 #/□' x 16' = 391 #/' (LESS GIRDER + COL.) WALL 125 #/□' x 12' AVG = 1500 LESS WALL OPEN'G $4 \times 6' \times 6' \times \frac{125}{48'}$ #/□' = <-375> 1516 #/' ABOVE 2ND 2ND FLR. 73 #/□' x 16' = 1168 (LESS GIRDER + COL.) WALL 125 #/□' x 11 = 1375 LESS OPEN'G = <-375> 3684 #/' FDN. WALL 125# x 1.5' = 188 FTG. (ASSUME 2.5 WIDE) = 563 TOTAL DEAD = 4435 #/'	ROOF 20# x 16' = 320 #/' 2ND FLR. 50# x 16' = 800 TOTAL LIVE = 1120 #/'

FOOTING WIDTH REQ'D = $\frac{4435 + 1120}{3000 \text{ PSF}}$ = 1.85 TRY 2'-6" x 18" CONT. FTG.

NOTE: FOOTING WIDTH TO BE CHECKED FOR SEISMIC LOAD

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
7	ROOF (p. 2) 24.5 #/□' x 16' = 391 #/' (LESS GIRDER + COL.) WALL 125 #/□' x 12' AVG. = 1500 #/' 1891 #/'	ROOF 20# x 16' = 320 #/'
	2ND FLR 73 #/□' x 16' = 1168 #/' (LESS GIRDER + COL.) WALL 125 #/□' x 11' = 1375 #/' FDN WALL 125 #/□' x 1.5' = 188 #/' FTG. (ASSUMED 2.75' WIDE = 619 #/' TOTAL DEAD = 5241 #/'	2ND FLR. 50# x 16' = 800 #/' TOTAL LIVE = 1120 #/'

$$\text{FOOTING WIDTH REQ'D} = \frac{5241 + 1120}{3000\ PSF} = 2.12'$$

TRY, 2'-9" x 18" CONT. FTG.

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
3 (WALL 5 SIM)	(p. 2) ROOF 24.5#/□' x 32' = 784#/' WALL 125#/□' x 12' AVG. = 1500 LESS WALL OP'N'G 5' x 9' x 125/40' = <-117> 2167 (p. 2) 2ND FLR 73#/□' x 32' = 2336 WALL 125#/□' x 11' = 1375 LESS OPEN'G = <-117> 5761#/' FDN WALL 125#/□' x 1.5' = 188#/' FTG. (ASSUME 3' WIDE) = 675#/' TOTAL DEAD = 6624#/'	ROOF 20#/□' x 32' = 640#/' 2ND FLR 50#/□' x 32' = 1600#/' TOTAL LIVE = 2240#/'

$$\text{FOOTING WIDTH REQ'D} = \frac{6624 + 2240}{3000\,psf} = 2.96'$$

TRY 3'x0" x 18" CONT. FTG.

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
A (9' PIER) (WALL C SIM.)	(p. 2) ROOF 24.5#/□' x 16' x 3' = 1176# WALL 125#/□' x 9' x 11' = 12375# 13551# WT/FT = $\frac{13551}{9}$ = 1506#/' 2ND FLR 73.0#/□' x 16' x 3' = 3504# WALL = 12375# 29430# WT/FT = $\frac{29430}{9'}$ = 3270#/' TOTAL DEAD (EXCL. FTG.) = 29430#	ROOF 20#/□' x 16' x 3' = 960# 2ND FLR 50#/□' x 16' x 3' = 2400# WT/FT = $\frac{2400}{9'}$ = 270#/' TOTAL LIVE = 3360#
	ALLOW SOIL PRESS. = 3000 psf − (300 psf WT FTG) = 2700 psf AREA REQ'D = $\frac{29430+3360}{2700}$ = 12.1□'	TRY, 8'-0" x 20'-0" FTG. REQ'D FOR SEISMIC OVERTURNING. A = 160□'
18' PIER	ROOF 24.5#/□' x 32' x 3' = 2352# WALL 125#/□' x 18' x 11' = 24750# 27102# WT/FT = $\frac{27102}{18}$ = 1506#/' 2ND FLR 73#/□' x 32' x 3' = 7008# WALL = 24750# 58860# WT/FT = $\frac{58860}{18}$ = 3270#/' TOTAL DEAD (EXCL. FTG) = 58860# ALLOW SOIL PRESS. = 2700 PSF AREA REQ'D = $\frac{58860+6720}{2700}$ = 24.3□'	ROOF 20#/□' x 32' x 3' 1920# 2ND FLR 50#/□' x 32' x 3' = 4800# TOTAL LIVE = 6720# TRY, 8'-0" x 31'-0" FTG. REQ'D FOR SEISMIC OVERTURNING A = 248□'

Figure D-1. Continued.

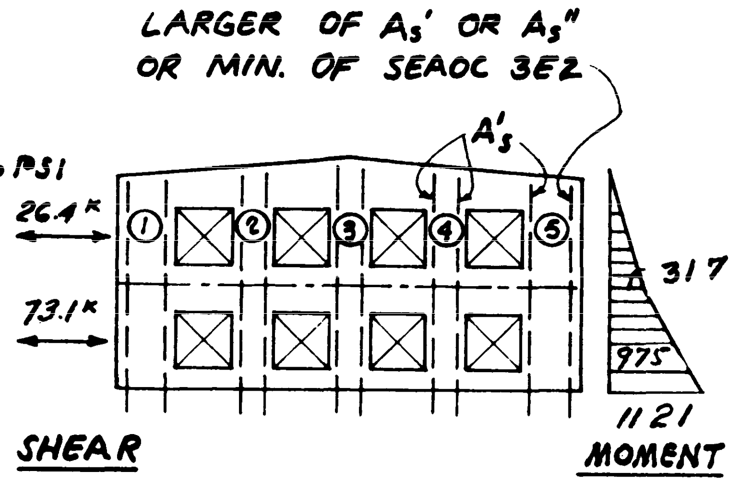
WALL DESIGN

WALL 1

$U = 1.4(D+L+E)$

$U = 0.9D + 1.4E$

$v = 2\sqrt{f'_c} = 126$ PSI



THE INDIVIDUAL PIERS IN WALL 1 SHOULD BE DESIGNED FOR THE BENDING MOMENT DUE TO THE LATERAL LOAD ($M = V \times h/2$) PLUS THE AXIAL LOADS FROM DEAD AND LIVE LOADS (EXCEPT ROOF L.L.) PLUS THE AXIAL LOAD DUE TO WALL OVERTURNING. UNLESS AXIAL LOADS ARE EXCEPTIONALLY LARGE IT IS USUALLY CONSERVATIVE TO NEGLECT AXIAL LOADS. THIS PROBLEM PROCEEDS ON THIS SIMPLIFYING ASSUMPTION.

SEE NEXT SHT. FOR SAMPLE CALCULATION FIRST STORY

	PIER	WIDTH	R (P. 9)	V	A_c	$v = \frac{1.4V}{\phi A_c}$	$M = V\frac{h}{2}$	A_s'	$1.33A_s'$	REINF.
1ST STORY	1	72"	11.1	22.7	720 $^{\square''}$	74	68.1	0.46 $^{\square''}$	0.61	2-#5
	2	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	3	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	4	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	5	72"	11.1	22.7 K	720	74	68.1	0.46	0.61	2-#5
			Σ=35.7	Σ=73.1K (P.15)		φ=0.6	h/2=3'		NOTE 1	

NOTE: 1. A_s' INCREASED BY 1/3 PER ACI ¶ 10.5.2
2. MIN. REINF. 2-#5 PER FIG. 6-6
3. REINF. FOR 1ST STORY PIERS IS EXTENDED TO THE 2ND STORY

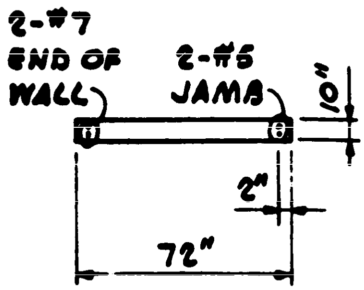
Figure D-1. Continued.

WALL DESIGN (CONT.)

WALL 1

SAMPLE CALCULATION FOR SHT. 21:

1ST STORY - PIER 1



PLAN - PIER 1

SHEAR IN PIER 1

$V_1 = \frac{R}{\Sigma R} \times V_{WALL} = \frac{11.1}{35.7} \times 73.1^K = 22.7^K$ (P.9) (P.15)

$V_u = 1.4 \times V_1 = 31.8^K$

$v = \frac{V_u}{\phi A_c} = \frac{31.8}{0.60 \times 720} = 74 \text{ PSI} < 126 \text{ PSI}$ OK

MOMENT IN PIER 1 DUE TO PIER SHEAR

$M = V_1 \times \frac{h}{2} = 22.7 \times \frac{6'}{2} = 68.1$

$M_u = 1.4M = 1.4 \times 68.1 = 95.3'^K$

REQ'D $A_s' = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{95.3 \times 12}{0.9 \times 40 \times (70 - \frac{1.0}{2})} = 0.46 \,\square''$

WHERE a IS ASSUMED AS 1.0

CHECK ASSUMPTION:

$a = \frac{A_s f_y}{0.85 f_c' b_w} = \frac{0.40 \times 40}{.85 \times 4 \times 10} = 0.47 < 1.0$ ASSUMED

$\therefore$ OK

SEE SHT. 23 FOR BOUNDARY MEMBERS

Figure D-1. Continued.

WALL DESIGN

WALL 1 (CONT.)

BOUNDARY MEMBER FOR ENTIRE WALL

GRAVITY LOADS (P. 17)

$W_D = 3684 \#/' \times 48' = 177^K$

$W_L = 800 \#/' \times 48' = 38^K$ (2ND FLOOR ONLY)

OVERTURNING MOMENT (P.16)

$M_{OT} = 1121$

USE LOAD COMBINATION 0.9D − 1.4E

LOAD AT EACH END =

$$F = C_D - T_M = \frac{0.9 W_D}{2} - \frac{1.4 M_{OT}}{0.9D}$$
$$= \frac{0.9(177)}{2} - \frac{1.4(1121)}{0.9(48')} = 79.7 - 36.3$$

SINCE $C_D > T_M$ THERE IS NO TENSION.

PROVIDE TRIM REINF. =

$$A_s'' = \frac{T_M}{\phi f_y} = \frac{36.3}{0.9 \times 40} = 1.01 \text{ IN}^2$$

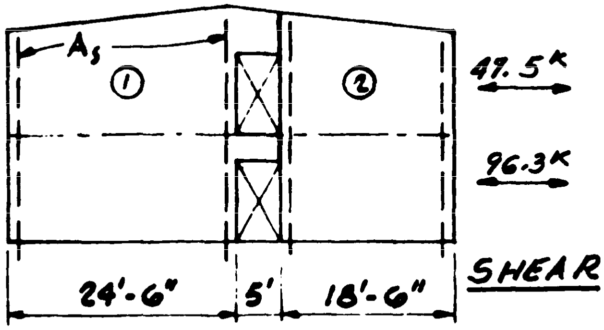
2 − #7 AT EACH END OF WALL

Figure D-1. Continued.

WALL DESIGN

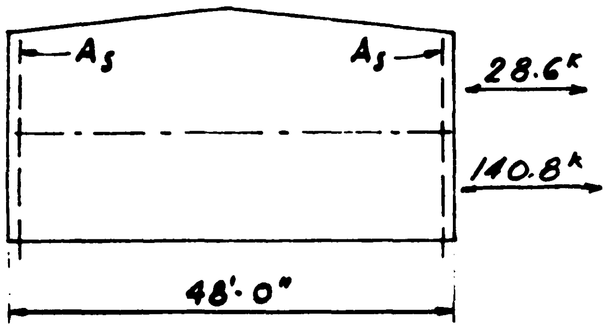
WALL 3 (WALL 5 SIM.)

ASSUME THAT PIERS ACT AS SERIES OF VERTICAL CANTILEVER BEAMS STRUTTED AT ROOF & 2ND FLR. LINE & FIXED AT 1ST FLR.



	PIER	WIDTH	R (P. 9)	V	A_c	$v = \frac{1.4V}{\phi A_c}$	M_{OT}	A_s *	REINF.
1ST STORY	1	24.5'	27	57.9 K	2940 □"	46 PSI	980 'K	2.07 □"	2-#9
	2	18.5'	17.9	38.4	2220	40	649	1.81	2-#9
			Σ=44.9	Σ=96.3 K (P.16)		ϕ=0.6	Σ=1629 'K (P.16)		

WALL 7



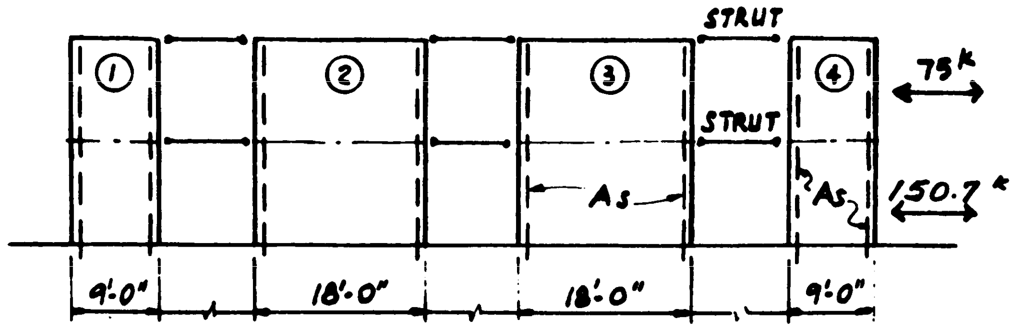
	PIER	WIDTH	R (P.10)	V	A_c	$v = \frac{1.4V}{\phi A_c}$	M_{OT}	A_s *	REINF
1ST STORY	1	48'	58.8	140.8 K	5760	57 PSI	1892 (P.16)	2.04	2-#9
						ϕ=0.6			

* INCREASED BY 1/3 PER ACI ¶ 10.5.2

Figure D-1. Continued.

WALL DESIGN

WALL A (WALL C SIM.)



	PIER	WIDTH	R (p/10)	V	Ac	$\tau = \frac{1.4V}{\phi Ac}$	M_{OT}	As	
1ST. STORY	1	9'	4.5	19.0^k	1080	41 psi	314	1.82	2-#9
	2	18'	13.3	56.3	2160	61	928	2.67	3-#9
	3	18'	13.3	56.3	2160	61	928	2.67	3-#9
	4	9'	4.5	19.0	1080	41	314	1.82	2-#9
			Σ 35.6	Σ 150.7^K (p/6)		φ = 0.6	Σ 2483'^K (p/6)		

SAMPLE CALCULATION FOR PIER (1) :

SHEAR $V = \frac{R}{\Sigma R} V_{WALL} = \frac{4.5}{35.6} \times 150.7 = 19.0^K$

$V_u = 1.4V = 1.4 \times 19.0^K = 26.6^K$

$\tau = \frac{1.4V}{\phi Ac} = \frac{V_u}{\phi Ac} = \frac{26.6}{0.6 \times 1080} = 41 \text{ psi} < 126 \text{ psi OK}$

MOMENT $M_1 = \frac{R}{\Sigma R} M_{OT} = \frac{4.5}{35.6} \times 2483'^K = 314$

$M_u = 1.4 M_1 = 1.4 \times 314 = 440'^K$

$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{440 \times 12}{0.9 \times 40 \times (108'' - \frac{3}{2})} = 1.37^{\square''}$

$A_s = 1.33 \times 1.38 = 1.82^{\square''}$ INCREASE PER ACI # 10.5.2

Figure D–1. Continued.

WALL DESIGN

WALL A – CONT. (WALL C SIM.)
BOUNDARY MEMBER – PIER 1

$M_u = 440^{k'}$ (p. 25)

$W_D = 29.4^k, \quad W_L = 2.4^k$ (p. 20)

$C_D = \frac{0.9 W_D}{2} \quad \frac{0.9 \times 29.4}{2} = 13.2$

$T_M = \frac{M_u}{0.9d} = \frac{440}{0.9(9)} = 54.3$

$T = T_M - C_D = 54.3 - 13.2 = 41.1 = \phi A_s'' f_y$

$A_s'' = \frac{41.1}{0.9 \times 40} = 1.14 \text{ IN.}^2 < A_s$ p. 25

CHECK TRANSV. REINF. (ACI 21.5.3.1)

WALL $A = 10' \times 9/12 = 7.5 \text{ ft.}^2$

$S = td^2/6 = \frac{10}{12} \frac{(9)^2}{6} = 11.2 \text{ ft.}^3$

$M = M_u = 440^{k'}$

$P = 1.4 (W_D + W_L) = 1.4 (29.4 + 2.4) = 44.5^k$

$f = \frac{P}{A} + \frac{M}{S} = \frac{44.5}{7.5} + \frac{440}{11.2} = 45.2 \text{ ksf}$

OR 314 psi $< (0.2 f_c' = 800 \text{ psi})$

NO TRANSV. REINF. REQ'D

Figure D-1. Continued.

WALL DESIGN

WALL A-CONT. (WALL C SIM.)

BOUNDARY MEMBER - PIER 2

$M_h = 928, \quad M_u = 1.4 \times 928 = 1299$

$W_D = 58.9 \quad W_L = 4.8$

$$C_D = \frac{0.9 W_D}{2} = \frac{0.9 \times 58.9}{2} = 26.5$$

$$T_M = \frac{M_u}{0.9d} = \frac{1299}{0.9 \times 18'} = 80.2$$

$$T = 80.2 - 26.5 = 53.7$$

$$A_s'' = \frac{53.7}{0.9 \times 40} = 1.49 < A_s$$

CHECK TRANSV. REINF.

$$\text{WALL } A = 18' \times \frac{10}{12} = 15 \text{ ft.}^2$$

$$S = \frac{10}{12} \times \frac{(18)^2}{6} = 45 \text{ ft.}^3$$

$$P = 1.4\,(58.9 + 4.8) = 89.2^k$$

$$M_u = 1299$$

$$f = \frac{89.2}{15} + \frac{1299}{45} = 6.0 + 28.9 = 34.9 \text{ ksf}$$

OR 242 psi < 800

NO TRANSV. REINF. REQ'D

Figure D-1. Continued.

WALL DESIGN - SEISMIC FORCES NORMAL TO WALL

$$F_p = Z I C_p W_p$$

WHERE $C_p = 0.75$ (SEAOC TABLE 1-H)

$W_p = 125^{\#}/\square'$ (10" CONC)

$Z = 0.4 \quad I = 1.0$

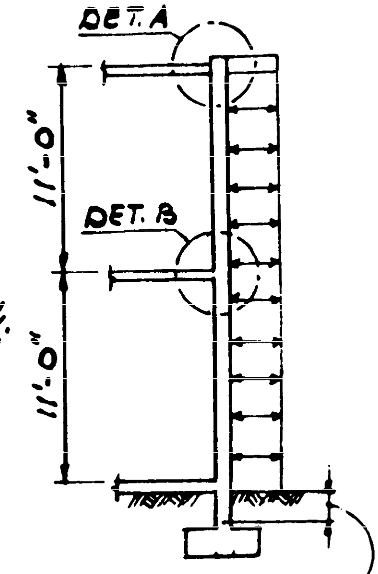
$F_p = F_p = 0.4 \times 1.0 \times 0.75 \times 125 = 37.5^{\#}/\square'$

REACTION @ ROOF $= 37.5^{\#}/' \times 11/2 = 206^{\#}/'$

CONT. SPAN

REACTION @ 2ND FLR $= 37.5^{\#}/' \times 11' \times \frac{10}{8} = 516^{\#}/'$

MIN. LOAD $= 200^{\#}/'$ (SEAOC 1H2h)



NOTE: WHEN FLOATING SLAB IS USED, ASSUME PT. OF FIXITY 2' BELOW GRADE LINE FOR DESIGN OF WALL

MAX. WALL BENDING @ 2ND FLR LINE

16' TRIB TO LINES 1 OR 7

$$M = \frac{wl^2}{8} + \frac{M_{ecc}}{2}$$

$$= \frac{37.5^{\#} \times 11^2 \times 12}{8} + \frac{(73^{\#} + 50^{\#}) \times 16 \times 5'' ecc}{2}$$

$$= 6806 + 4920 = 11{,}726^{\#''}$$

ASSUME $a = 0.1''$

$$A_s = \frac{M}{\phi f_y (d - \frac{a}{2})} = \frac{11.73^{''K}}{0.9 \times 40 (8.5 - \frac{0.1}{2})}$$

$$= 0.039^{\square''}$$

CHECK ASSUMED $a = 0.1$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.039 \times 40}{0.85 \times 4 \times 10} = 0.05 < 0.1 \text{ OK}$$

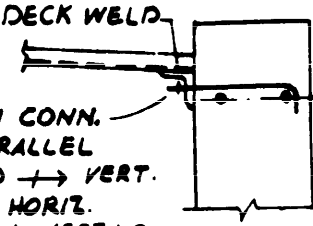
MIN. $A_s = .0025\, bd$

$= .0025 \times 10 \times 8.5 = 0.21^{\square''}$

USE #4 @ 16" O.C. E.F.

$A_s = 0.30$

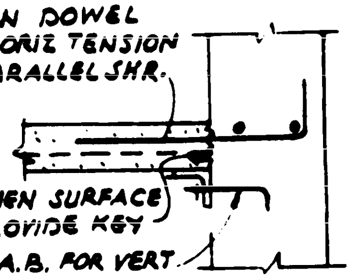
DECK WELD



DESIGN CONN. FOR PARALLEL SHR LD ⟷ VERT. LD OR HORIZ. TENSION + VERT. LD.

DETAIL A

DESIGN DOWEL FOR HORIZ TENSION OR PARALLEL SHR.



ROUGHEN SURFACE OR PROVIDE KEY

DESIGN A.B. FOR VERT. LD. (IF THE SHR. TRANSFER IS THRU THE DECK WELDS, THEN BOLTS SHALL BE DESIGNED FOR VERT ⟷ SHR)

DETAIL B

Figure D–1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

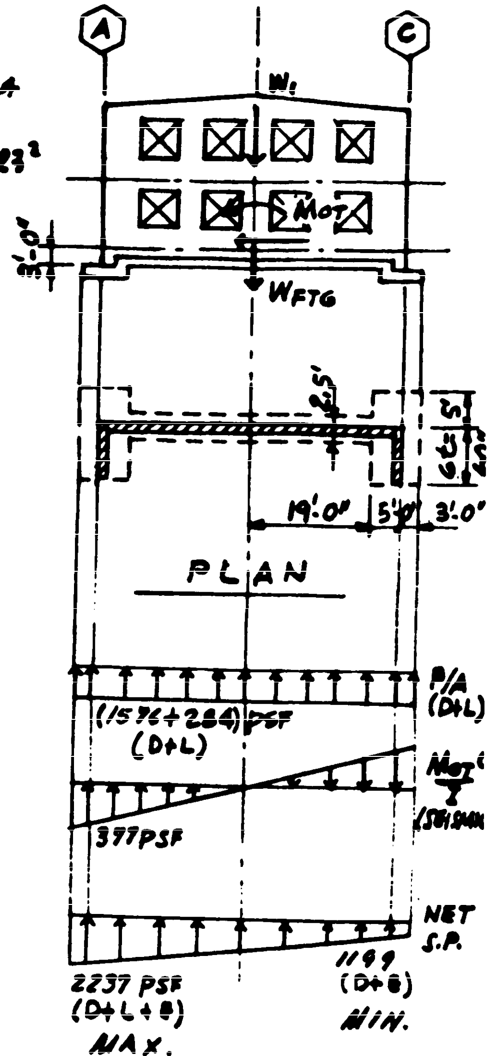
WALL 1

MOMENT OF INERTIA OF FTG.:

FOR VERTICAL LOAD $I_1 = 2.5' \times \frac{48^3}{12} = 23040\ FT^4$

FOR SEISMIC LOAD $I_2 = \frac{2.5' \times 38^3}{12} + 2 \times 8' \times 10' \times 23^2$
INCLUDE RETURN WALL FTGS. $= 96072\ FT^4$

AREA OF FTG. $= 2.5' \times 54' = 135\ \square'$



WEIGHT (p. 17)

$W_1 = (3684^{\#}/') \times 48' = 176832^{\#}$ (DEAD)

$W_l = (800^{\#}/') \times 48' = 38400^{\#}$ (LIVE W/O ROOF L.L.)

$W_{FTG} = (751^{\#}/' \times 48' = \underline{36000}$

ΣW (DEAD) $= 212832$

ΣW (LIVE) $= 38400$

OVERTURNING MOMENT @ BASE OF FTG.

$M_{OT} = \underset{\text{℄ (p.21)}}{1121'^{K}} + \underset{\text{℄ (p.21)}}{73.7^{K}} \times 3' = 1340'^{K}$

SOIL PRESSURE		MAX.	MIN.
P/A (DEAD)	$\frac{212800}{135}$	+ 1576	+ 1576
P/A (LIVE)	$\frac{38400}{135}$	+ 284	
$M_{OT}C$	$\frac{1340 \times 27'}{96072}$	+ 377	− 377
		2237 < 4000	1199 NO UPLIFT

NOTE: THE SOIL PRESSURE UNDER THE RETURN WALLS DUE TO OVERTURNING (377 PSF) IS ADDED TO THE SOIL PRESSURE UNDER THE RETURN WALL WHICH IS VERY LOW ∴ OK

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 7

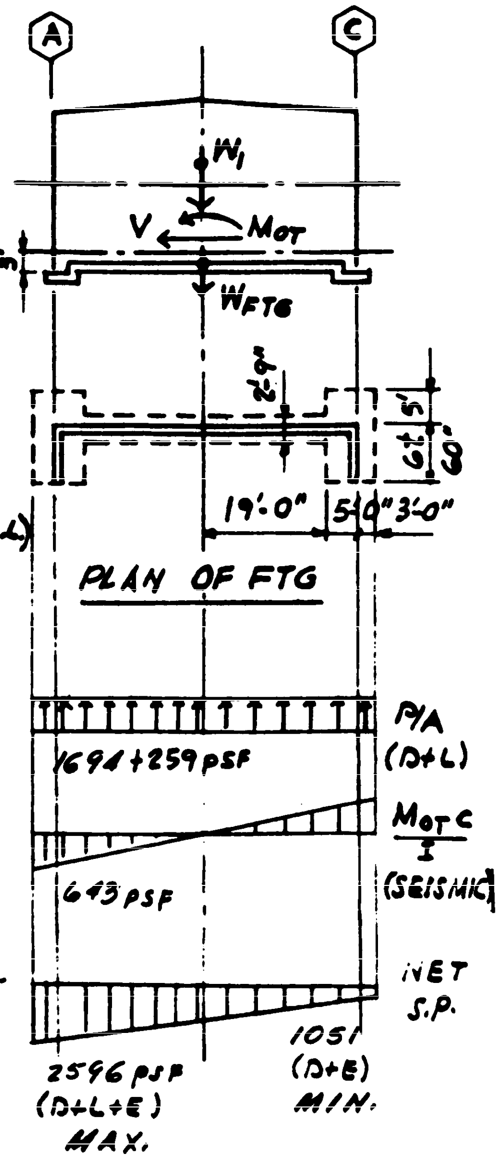
MOMENT OF INERTIA OF FTG:

FOR VERTICAL Ld $I_1 = \dfrac{2.75 \times 48^3}{12} = 25344\ FT^4$

FOR SEISMIC Ld $I_2 = \dfrac{2.75 \times 38^3}{12} + 2 \times 8' \times 10' \times 23^2$

$= 97215\ FT^4$

AREA OF FTG $= 2.75' \times 54' = 148.5^{\square'}$



WEIGHT (p.16)

$W_1 = (4434^{\#}/' \times 48' = 212\,832^{\#}$ (DEAD)

$W_1 = (800^{\#}/' \times 48' = 38400$ (LIVE W/o ROOF LL)

$W_{FTG} = (806^{\#}/' \times 48' = 38736$

ΣW (DEAD) $= 251\,568^{\#}$

ΣW (LIVE) $= 38400$

OVERTURNING MOMENT AT BASE OF FTG.

(p.16)

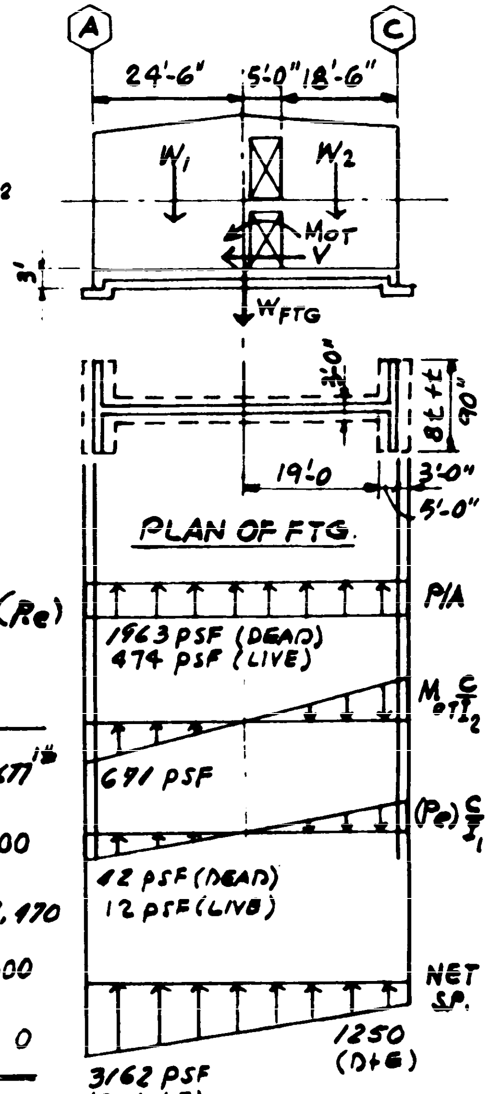
$M_{OT} = 1892 + 140.8^{k} \times 3' = 2314'^{k}$

SOIL PRESSURE		MAX	MIN.
P/A (DEAD)	$\dfrac{160788}{148.5}$	+1694	+1694
P/A (LIVE)	$\dfrac{38400}{148.5}$	+259	
$\dfrac{M_{OT}C}{I_2}$	$\dfrac{2314 \times 27}{86212}$	+643	−643
		+2596	+1051
		< 4000	NO UPLIFT

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (WALL 5 SIM.)



MOMENT OF INERTIA OF FTG:

FOR VERTICAL LD $I_1 = \frac{3 \times 48^3}{12} = 27648\ FT^4$

FOR SEISMIC LD $I_2 = \frac{3 \times 38^3}{12} + 2 \times 8' \times 7.5 \times 23^2$

$= 77200\ FT^4$

AREA OF FTG. $3' \times 54' = 162^{\square'}$

OVERTURN'G MOMENT = $1629^{'K}$ (p. 16)

SHEAR V = 96.3^K (p. 16)

OVERTURN'G MOMENT @ BASE OF FTG.

$M_{OT} = 1629^{'K} + 96.3^K \times 3 = 1918^{'K}$

CALCULATION OF ECCENTRIC MOMENT (P_e) OF WALL MASS RESPECT TO N.A.

WEIGHTS (p. 19)		X	DIST. TO N.A. FTG	= WX
W_1 (DEAD) = 5761 #/' (24.5' + 2.5')	= 155 547#	x	-11.75 =	-1,827,677 '#
W_1 (LIVE) = 1600 #/' (24.5' + 2.5')	= 43200	x	-11.75 =	-507600
W_2 (DEAD) = 5761 #/' (18.5' + 2.5')	= 120 981	x	+14.75 =	1,784,470
W_2 (LIVE) = 1600 #/' (18.5' + 2.5')	= 33600	x	+14.75 =	495600
W_{FTG} = 863 #/' x 48'	41400	x	0 =	0

ΣW (DEAD) = 317,928# ΣW(DEAD) - 43 207#

ΣW (LIVE) = 76800# ΣW(LIVE) - 12000#

THESE SOIL PRESSURES ARE CALCULATED ON THE NEXT PAGE

ECCENTRICITY $e = \frac{-43207^{\#'}}{317928^{\#}} = -0.14'$ (DEAD)

$e = \frac{-12000}{76800} = -0.156'$ (LIVE)

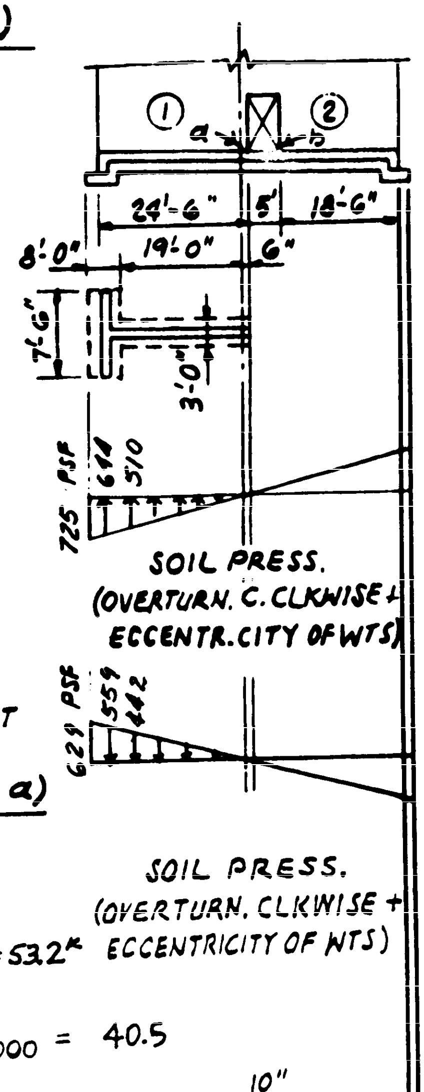
NOTE: THESE ARE SMALL, THE EFFECTS COULD BE NEGLECTED.

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.) (WALL 5 SIM.)

SOIL PRESS. PSF	MAX.	MIN.
$\frac{P}{A}$ (DEAD) $\frac{317,928}{162}$	+1963	+1963
$\frac{P}{A}$ (LIVE) $\frac{76800}{162}$	+474	
$M_{OT}\ \frac{C_2}{I_2}\ \frac{1918 \times 27}{77200}$	+671	−671
$M_{ecc}\ \frac{C_1}{I_1}$ (DEAD) $\frac{43207 \times 27}{27648}$	+42	−42
$M_{ecc}\ \frac{C_1}{I_1}$ (LIVE) $\frac{12000 \times 27}{27648}$	+12	
	3162 < 4000	+1250 NO UPLIFT



CHECK SHEAR IN FDN. WALL @ DOOR OP'N'G (Pt a)

DL + LL $\quad W = 5' \times 3'\ (1.963 + 0.474) = 36.6^K$

$V = 36.6/2 = 18.3^K$

SEISMIC (CCW OVERTURNING)

$V = (510 \times \frac{19'}{2} \times 3') + (644 \times 8' \times 7.5') = 53.2^K$

$V_u/\varphi = \frac{1}{0.60}(18.3 + 53.2) = 119^K$

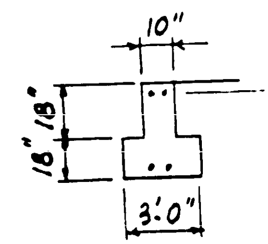
$V_c = 2\sqrt{f_c'}\ bd = 2\sqrt{4000} \times 10 \times 32/1000 = 40.5$

$V_s = 119 - 40.5 = 78.5$

$V_s < 8\sqrt{f_c'}\ bd = 162$ OK

PROVIDE #4 ⊓ @ 6" OK

$V_s = \frac{0.40 \times 40 \times 32}{6} = 85.3 > 78.5$



SECTION

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.)

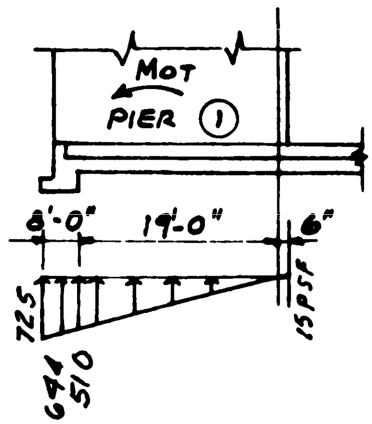
CHECK MOMENT IN FDN WALL @ DOOR OP'NG (Pt. a)

$M_{D+L} = \frac{WL}{10} = \frac{36.6 \times 5}{10} = 18.3'^{K}$

M_{OT} (PIER 1) $= \Sigma M_{OT}$

$= \frac{27}{44.9} \times 1918'^{K} = 1153'^{K}$

RIGIDITIES, P.24



MOMENT AT Pt. a (SEISMIC)

$M_a = 1153'^{K} - \left[(510 \times \frac{19}{2} \times 3'\text{WIDE}) \times (19' \times \frac{2}{3} + 0.5')\right]$

$- \left[644 \times 7.5' \times 8' \times 23'\right] = 72.9^{K}$

$M_u = 1.4(18.3 + 72.9) = 128'^{K}$

$F = \frac{bd^2}{12000} = \frac{10 \times 32^2}{12000} = 0.86$

$K = \frac{M_u}{F} = \frac{128}{0.86} = 149$

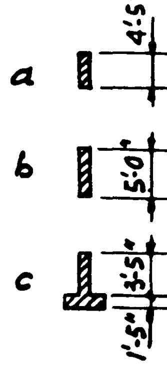
$a_u = 2.92$ (ACI-SP.17 FLEX 1-2)

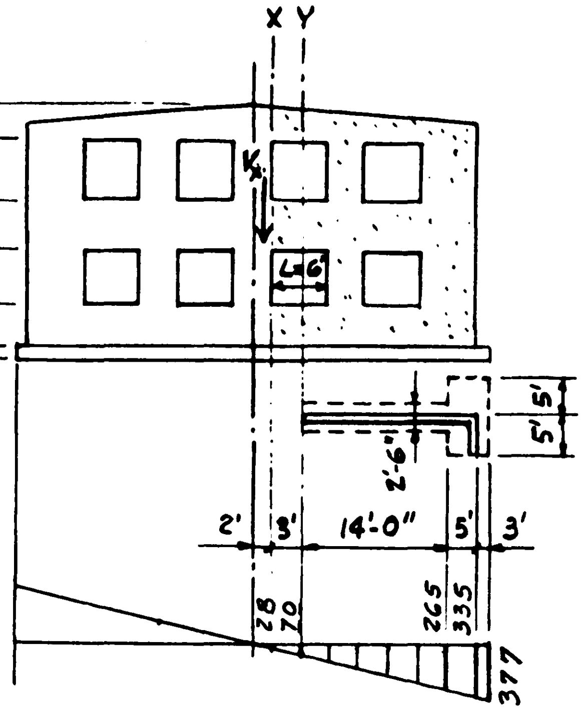
$A_s = \frac{M_u}{a_u d} = \frac{128}{2.96 \times 32} = 1.40 \square''$ **2-#8 TOP & BOTT**

Figure D-1. Continued.

SPANDREL DESIGN

WALL 1





AS A SIMPLYFYING APPROXIMATION ASSUME THAT SPANDRELS a & b AND FOUNDATION WALL c SHARE V_X IN PROPORTION TO THEIR RELATIVE RIGIDITIES

$V_X = \frac{265+28}{2} \times 17' \times 2.5$ WIDE

$+ 335 \times 10' \times 8' = 33000^{\#}$ SHEAR DUE TO SEISMIC OVERTURNING

SOIL PRESSURE (SEISMIC)
(p.29)

$V_y = \frac{265+70}{2} \times 14 \times 2.5 + 335 \times 8 \times 70 = 32700^{\#}$

	$I(FT^4)$	L	$R=I/L$	$R/\Sigma R$	V_X	$v=\frac{V}{\phi A_c}$	V_y	$M=\frac{VL}{2}$	W_{D+L}	$M'=\frac{wl^2}{12}$	$M+M'$
4.5'	6.3	6'	1.05	0.22	7.3^K	22 psi	7.2	21.6 'K	954#/'	2.86'K	24.5 'K
5'	8.68	6'	1.45	0.31	10.2	28	10.1	30.3	2593#/'	7.78'K	38.1 'K
5' 2.5'	13.31	6'	2.22	0.47	15.5	43	15.4	46.2	1178#/'	3.5'K	49.7 'K
			4.72	1.0	33.0^K		32.7^K				

DESIGN SPANDREL FOR MAX. MOMENT (M + M')

SAMPLE CALCULATION: $V_X = 33.0^K \times R/\Sigma R = 33.0^K \times .22 = 7.3^K$

$W_{D+L} = (391^{\#}/') + 4.5' \times 125^{\#} = 954^{\#}/'$ UNIT WT. ON SPANDREL
℄ (p.18)

NOTE: ROOF LL NEGLECTED WHEN COMBINED WITH SEISMIC (SEAOC SEC. 1H1 a.)

Figure D-1. Continued.

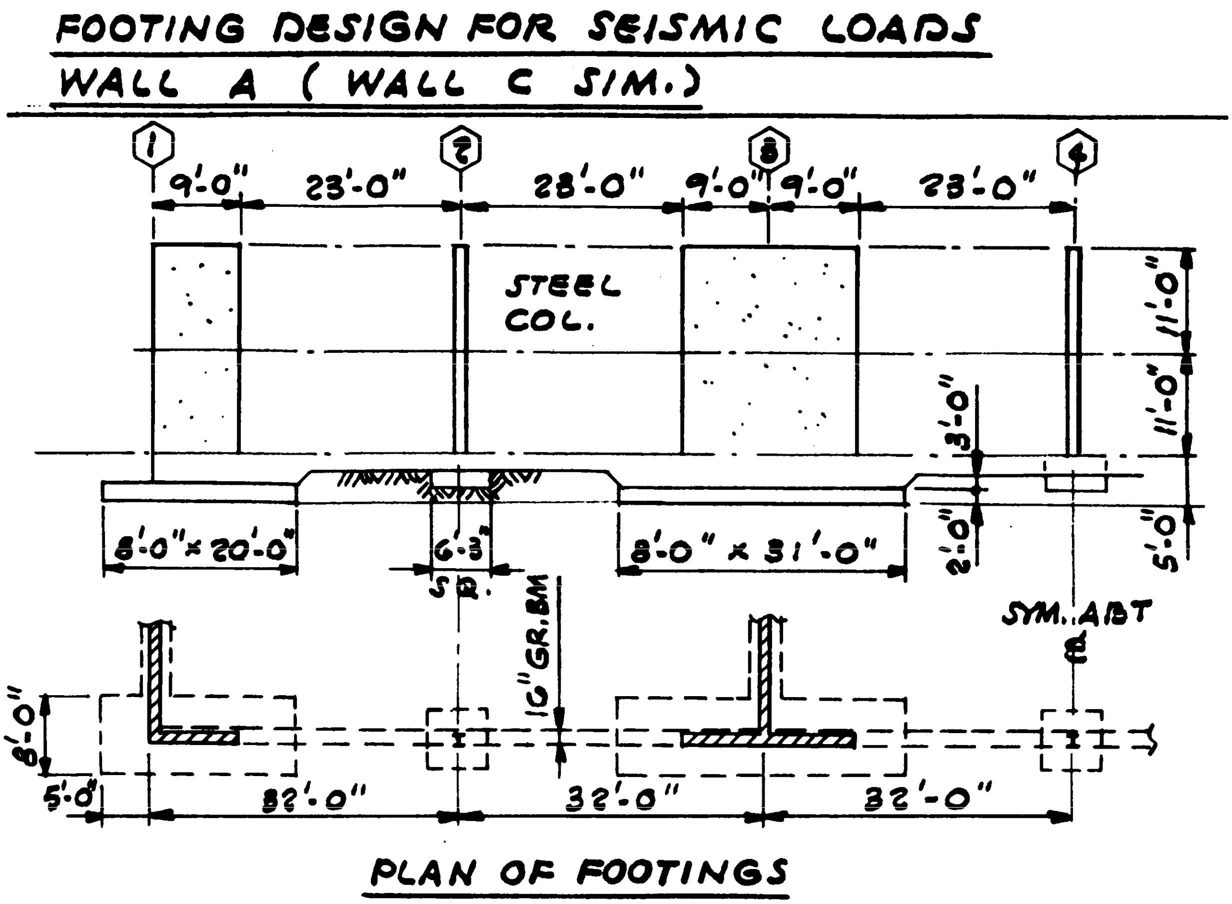


Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM.)

18' PIER

TOTAL WALL OVERTURN'G MOMENT = 2480'K (P.16)

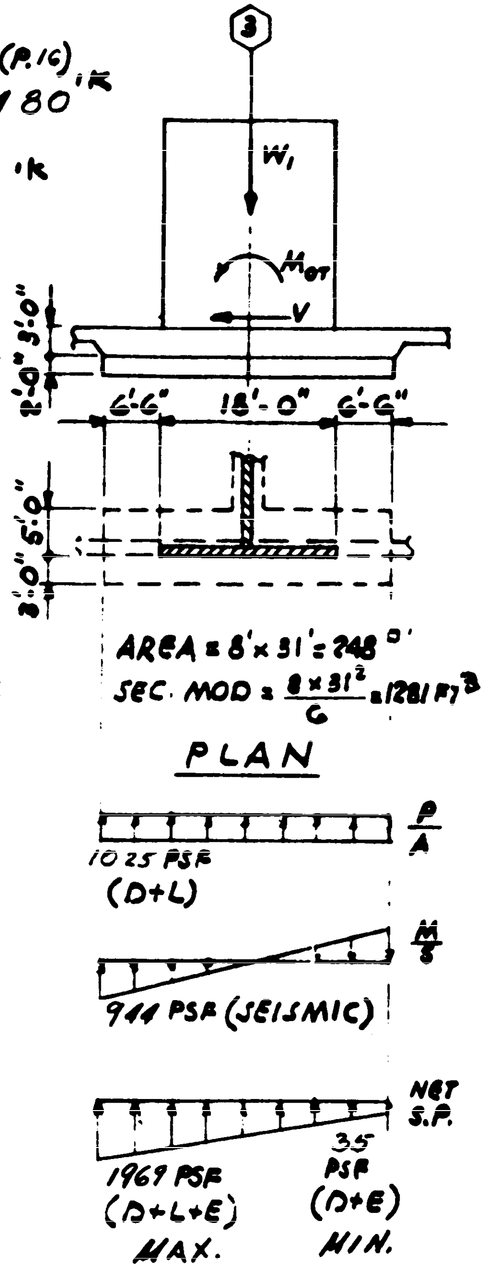
OVERTURN'G MOMENT TO 18' PIER =

$\frac{R}{\Sigma R} \times 2480'^{K} = \frac{13.3}{35.6} \times 2480 = 927'^{K}$ (P.10)

SHEAR $V = 56.3^{K}$ (p.25)

OVERTURN'G MOMENT @ BASE OF FTG.:

$M_{OT} = 927'^{K} + 56.3^{K} \times 5' = 1209'^{K}$



WEIGHTS

W_1 = 58860# (DEAD) (P.20)

W_1 = 4800# (LIVE EXCL. ROOF L.L.) (P.20)

W_1 = 5761 #/' × 4.17' TRIB = 24023# (DEAD) } CROSS WALLS (P.19)

W_1 = 1600 #/' × 4.17' TRIB = 6672# (LIVE) }

ΣW_1 (DEAD) = 82883#

ΣW_1 (LIVE) = 11472#

W_{FTG} = 2' × 150 PCF = 300 PSF

W_{SOIL} = 3' × 115 PCF = 345 PSF

SOIL PRESSURE	MAX.	MIN.
P/A (FTG + SOIL)	+645 PSF	+645 PSF
P/A (DEAD) $\frac{82883}{248}$	+334	+334
P/A (LIVE) $\frac{11472}{248}$	+46	
M/S (SEISMIC) $\frac{1209{,}000}{1281}$	+944	-944
	+1969 PSF	+35 PSF
		NO UPLIFT

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM.)

9' PIER

(P.17)

TOTAL WALL OVERTURN MOMENT = 2480'K

OVERTURN MOM. TO 9' PIER

$\frac{R}{\Sigma R} \times 2480 = \frac{4.5}{35.6} \times 2480 = 313$ 'K

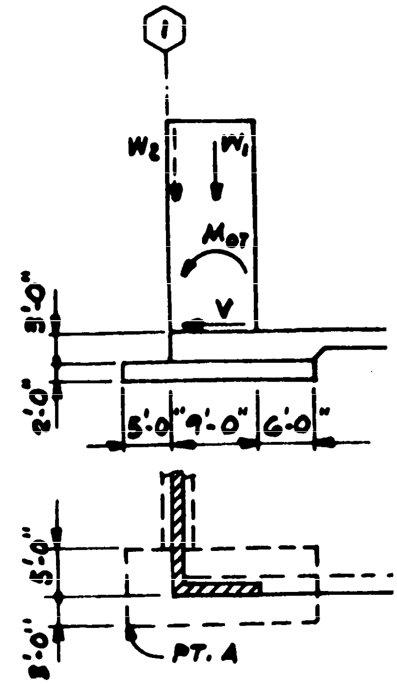
SHEAR V = 19.0K (P.24)

OVERTURN MOM. @ BASE OF FTG.

$M_{OT} = 313$'K + 19.0K × 3' = 408 'K

AREA OF FTG. = 8' × 20' = 160 □'

SECTION MODULUS = $\frac{8 \times 20^2}{6}$ = 533 FT³



PLAN

WEIGHTS	×	DIST. TO PT. A	=	Wd
W_1 (DEAD) = 29430# (P.20)	×	9.5'	=	279585
W_1 (LIVE) = 2400# (P.20)	×	9.5'	=	22800
W_2 (DEAD) = 3684 #/' × 4.17' TRIB. = 15362#	×	5.42'	=	83264
W_2 (LIVE) = 800 #/' × 4.17' TRIB. = 3336#	×	5.42'	=	18081
$W_{FTG.}$ 2' × 150# × 8' × 20' = 48000#	×	10'	=	480000
W_{SOIL} 3' × 115# × 8' × 20' = 55200#	×	10'	=	552000

Σ W (DEAD) = 147992#   Σ Wd (DEAD) = 1394849

Σ W (LIVE) = 5736#   Σ Wd (LIVE) = 40881

ECCENTRICITY e (DEAD) = $\frac{1394849}{147992} - 10' = -.57'$

e (LIVE) = $\frac{40881}{5736} - 10' = 2.87'$

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM)

9' PIER (CONT.)

SOIL PRESSURE		MAX	MIN.
P/A (FTG + SOIL)	300# + 345# =	+645 PSF	+645
P/A (DEAD)	$\frac{29430 + 17514}{160}$	= +293	+293
P/A (LIVE)	$\frac{2400 + 3336}{160}$	= + 36	
Pe/S (DEAD)	$\frac{150144 \times 0.63'}{533}$	= + 177	−177
Pe/S (LIVE)	$\frac{5736 \times 2.87'}{533}$	= + 31	
M_{OT}/S (SEISMIC)	$\frac{408,000}{533}$	= +765	−765
		1947 PSF < 3000 × 1⅓ OK	− 4 UPLIFT.

SMALL UPLIFT CONSIDERED OK, SINCE GR. BM. CAN OFFER SOME RESISTANCE

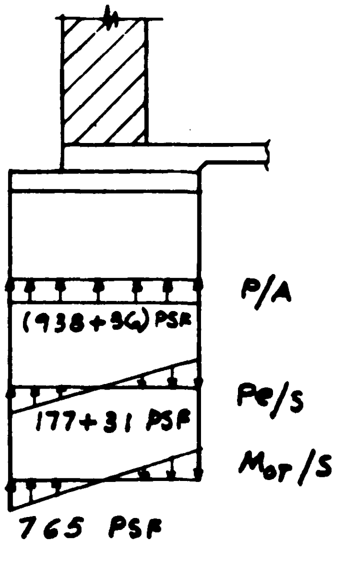
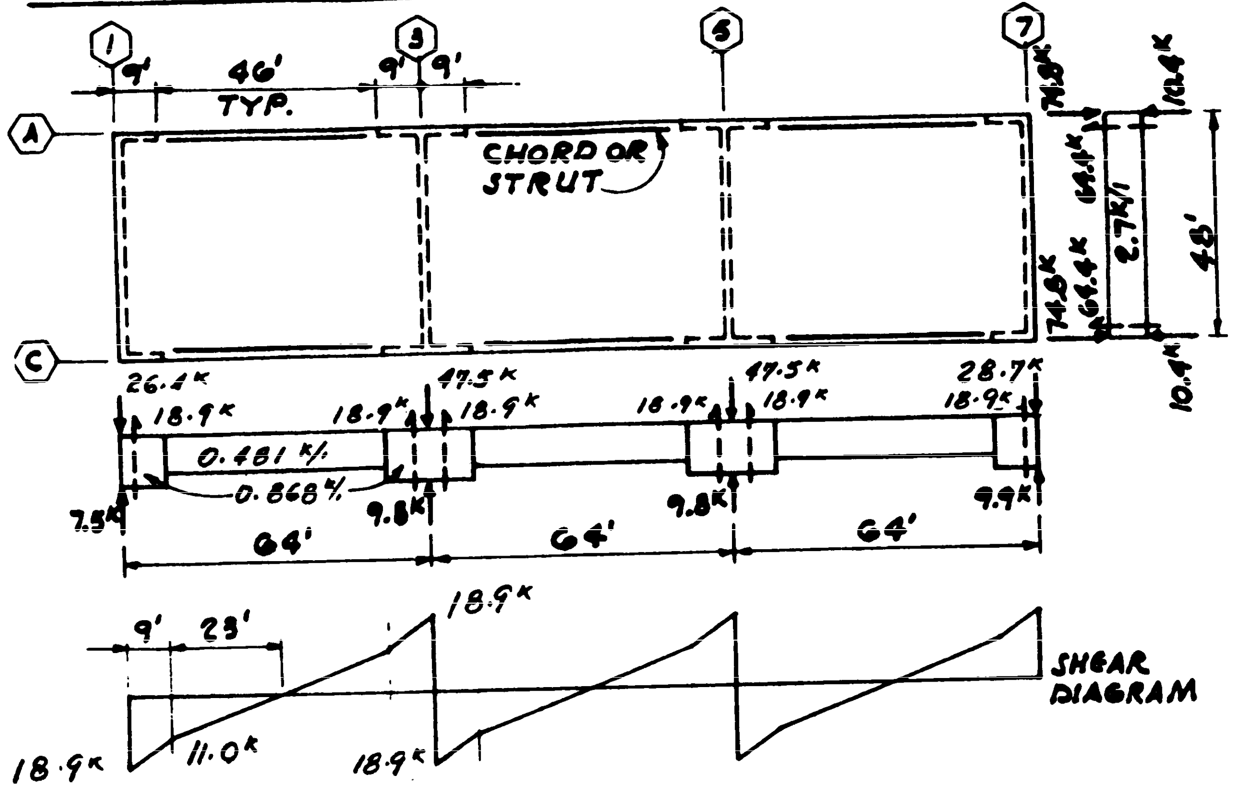


Figure D-1. Continued.

DESIGN OF ROOF DIAPHRAGM



SEAOC FORMULA (1-11): $F_{px} = \left(\dfrac{F_t + \Sigma F_i}{\Sigma w_i}\right) w_{px}$

$= \left(\dfrac{0+150}{534}\right) w_{px} = 0.281 w_{px}$

MIN. $F_{px} = 0.35 Z I w_{px} = 0.14 w_{px}$ WHERE $Z = 0.4$

MAX. F_{px} NEED NOT EXCEED $0.75 Z I w_{px}$ $I = 1.0$

FROM DIAGRAM p.4, MULTIPLY ALL VALUES SHOWN @ 100% G BY $C_p = 0.281$

MAX. AVE. DIAPH. SHEAR: (N-S) $\dfrac{18{,}900\#}{48'} = 394\#/l$

(E-W) $\dfrac{64400\#}{192'} = 335\#/l$

USE 1½" STEEL DECK 20 GA. SPAN 6'-0" ALLOW SHR = 470#/l
5 END WELDS & BUTTON PUNCH @ 24" O.C. (FIG. 5-19)

Figure D-1. Continued.

DESIGN OF ROOF DIAPHRAGM - CONT.

MAXIMUM MOMENT $= \left(\frac{18.9 + 11.0}{2}\right) 9' + \left(11.0 \times \frac{23}{2}\right) = 261'^{K}$

CHORD STRESS (N-S) $= \frac{M}{D} = \frac{261}{47.2'} = 5.5^{K}$

CHORD STRESS (E-W) $= \frac{VL}{4D} = \frac{64.4 \times 48}{4 \times 191.58} = 4^{K}$

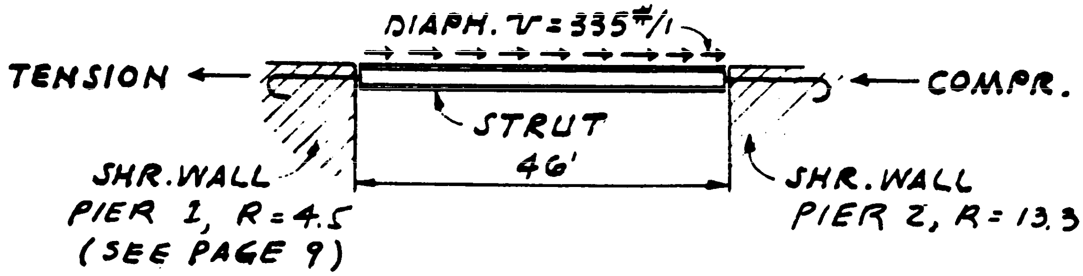
DESIGN CHORD FOR TENSION OR COMPR OF 5.5^{K}

DESIGN FOR CHORD REBAR IN WALLS 1 & 7

$A_s = \frac{1.4T}{\phi f_y} = \frac{1.4 \times 4^{K}}{0.9 \times 40} = 0.16^{\square''}$

USE 2-#5

STRUT DESIGN (E-W): IN THE EAST-WEST DIRECTION, THE CHORD BEAMS ALONG A & C ACT AS COLLECTOR OR DRAG STRUTS. BECAUSE OF WALL RIGIDITIES, DIAPH. SHR. IS TAKEN THRU THE STRUT IN TENSION & COMPR. IN PROPORTION TO THE WALL RIGIDITIES.



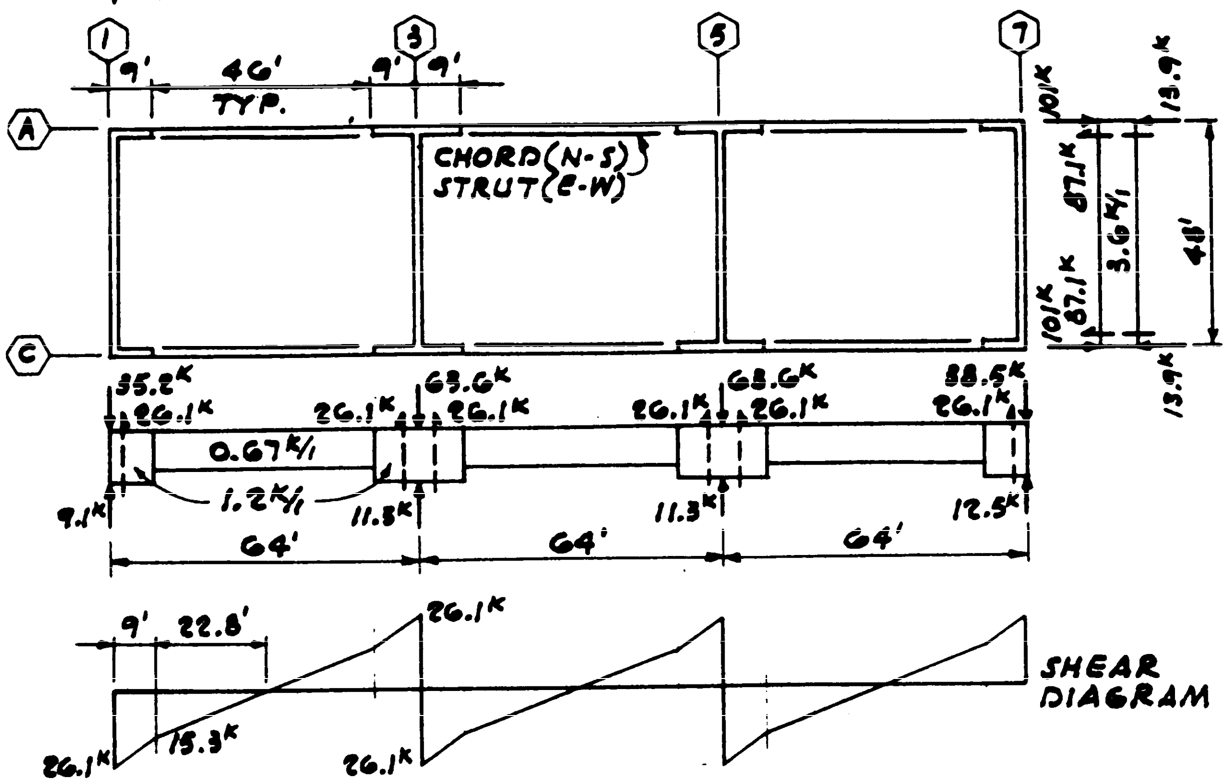
∴ DESIGN STRUT FOR:

$C = T = 335^{\#}/_{1} \times 46'\ \text{LENGTH} \times \frac{13.3}{17.8} = 11{,}514^{\#}$

Figure D-1. Continued.

DESIGN OF 2ND FLOOR DIAPHRAGM

THE CONCRETE DIAPHRAGM SHALL BE DESIGNED TO SATISFY SEAOC FORM. (1-11), BUT SHALL NOT BE LESS THAN THAT REQUIRED TO TRANSFER SHEAR ON P. 15.



SEAOC (1-11) : $F_{px} = \left(\frac{F_t + \Sigma F_i}{\Sigma w_i}\right) w_{px}$

$= \left(\frac{0+300}{1614}\right) w_{px} = 0.186\ w_{px}$ (NORTH-SOUTH & EAST-WEST)

(P. 6)

MIN. $F_{px} = 0.14\ w_{px}$

FROM DIAGRAM P. 5, MULTIPLY ALL VALUES SHOWN @ 100 G BY $C_p = 0.186$.

MAX. AVG. DIAPH. SHEAR : (N-S) $\frac{26100^{\#}}{48'} = 544^{\#}/'$

(E-W) $\frac{87100^{\#}}{192} = 454^{\#}/'$

USE 2½" CONC. ON 16-18 GA. STEEL DECK ALLOW. $q = 2760^{\#}/'$
(EQ. 5-27 & FIG 5-22, 3 of 3) $f_c' = 3{,}000$ PSI

Figure D-1. Continued.

DESIGN OF 2ND FLOOR DIAPHRAGM (CONT.)

$$MAX.\ MOMENT = \left(\frac{26.3^K + 15.5^K}{2}\right) 9' + \left(15.5^K \times \frac{23'}{2}\right) = 366'^K$$

$$CHORD\ STRESS\ (N\text{-}S) = \frac{M}{D} = \frac{366'^K}{47.2} = 7.8^K$$ DESIGN CHORD FOR TENSION OR COMPR. OF 7.8^K

$$CHORD\ STRESS\ (E\text{-}W) = \frac{VL}{4D} = \frac{84.5^K \times 48'}{4 \times 191.58} = 5.3^K$$ DESIGN FOR CHORD REBAR IN WALLS 1 & 7

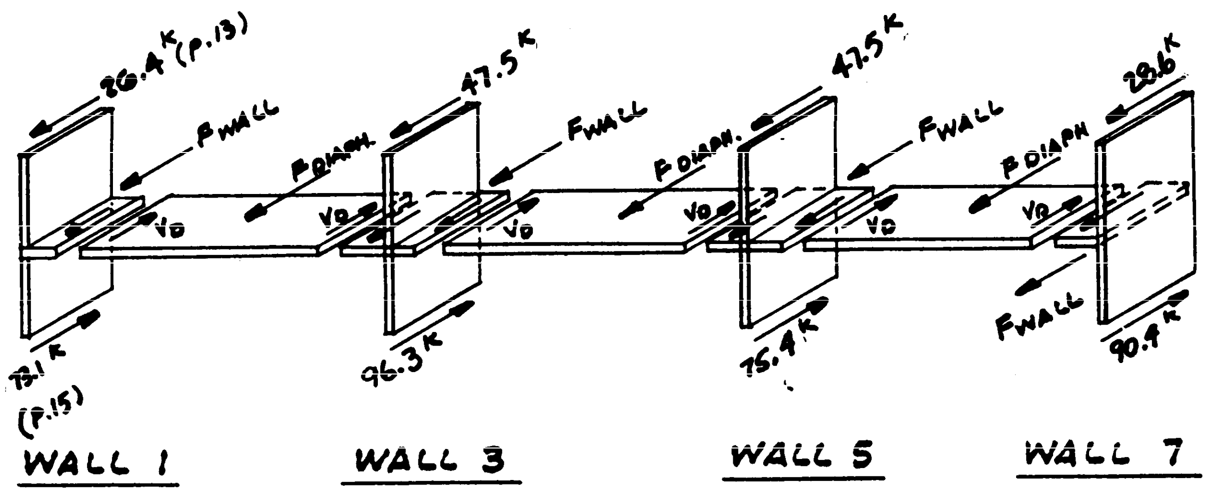
$$A_S = \frac{1.4T}{\phi f_y} = \frac{1.4 \times 5.3}{0.9 \times 40} = 0.21\ \square''$$

USE 2-#5

STRUT DESIGN (E-W):

DESIGN STRUT FOR TENSION & COMP. OF $T = C = 366^{\#}/' \times \frac{46'}{2} = 8420^{\#}$

CHECK STRESSES FOR SHEAR P.16



V_D = DIAPHRAGM SHEAR

F_{WALL} = SEISMIC FORCE FROM WT OF TRIBUTARY WALL

F_{DIAPH} = SEISMIC FORCE FROM WT OF DIAPHRAGM

Figure D–1. Continued.

DESIGN OF 2ND FLOOR DIAPH. (CONT)

NORTH-SOUTH

WALL 1

$F_{WALL} = 0.183 \times \text{TRIB. WALL WT.}$
$= \underset{\text{(p.6)}}{0.183} \times \underset{\text{(p.5)}}{48.6^K} = 9^K$

p.42 { SHEAR IN WALL ABOVE DIAPH. = 26.4^K
SHEAR IN WALL BELOW DIAPH. = 73.1^K }

DIAPH. SHR $V_D = 73.1^K - 9^K - 26.4^K = 37.7^K$

SHEAR STRESS $v = \frac{37,700}{48'} = 785^{\#/'}$ OK

WALL 3

$F_{WALL} = 0.183 \times \overset{\text{(p.5)}}{60.6^K} = 11.1^K$

SHEAR IN WALL ABOVE DIAPH. = 47.5^K

SHEAR IN WALL BELOW DIAPH. = 96.3^K

$F_{DIAPH} = 0.183 \left[3.58^{K/'} \times 46' + 6.34^{K/'} \times 18' \right] = 51.0^K$ (p. 5)

DIAPH SHEAR V_D (WEST) = $51.0^K - 37.7^K = 13.3^K$

DIAPH SHEAR V_D (EAST) = $96.3^K - 13.3^K - 47.5^K - 11.1^K = 24.4^K$

SHEAR STRESS = $\frac{24,400^{\#}}{(48)} = 508^{\#/'}$ OK

WALL 7

$F_{WALL} = 0.183 \times \overset{\text{(p.5)}}{66.6^K} = 12.2^K$

SHEAR IN WALL ABOVE DIAPH. = 28.6^K

SHEAR IN WALL BELOW DIAPH. = 90.4^K

DIAPH SHR $V_D = 90.4^K - 28.6^K - 12.2^K = 49.6^K$

SHEAR STRESS = $\frac{49600^{\#}}{48'} = 1033^{\#/'}$ OK

Figure D-1. Continued.

DESIGN OF 2ND. FLOOR DIAPH. (CONT.)

NORTH-SOUTH

WALL 5

$F_{WALL} = 0.183 \times 60.6^{K} = 11.1^{K}$

SHEAR IN WALL ABOVE DIAPH. $= 47.5^{K}$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 75.4^{K}$ (p.16)

$F_{DIAPH} = 0.183\left[3.58^{K/'} \times 46' + 6.34^{K/'} \times 18'\right] = 51.0^{K}$

DIAPH SHR $V_D(EAST) = 51.0^{K} - 49.6^{K} = 1.4^{K}$

DIAPH SHR $V_D(WEST) = 75.4^{K} - 1.4^{K} - 11.1^{K} - 47.5^{K} - 51.0^{K}$

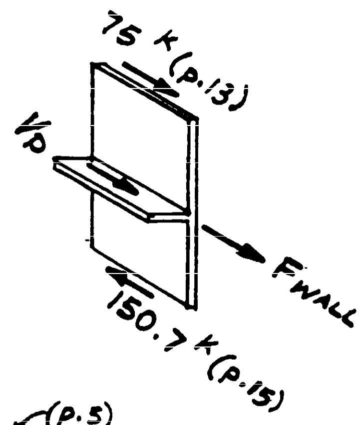
$= {}^{-}35.6^{K}$

$$\text{SHEAR STRESS} = \frac{35{,}600}{(48' - 12')} = 989\ \#/' \quad \text{OK}$$

STAIR OP'G.

EAST-WEST

WALL A & C



(p.6) (p.5)

$F_{WALL} = 0.183 \times 74.4^{K} = 13.6^{K}$

SHEAR IN WALL ABOVE DIAPH. $= 75^{K}$

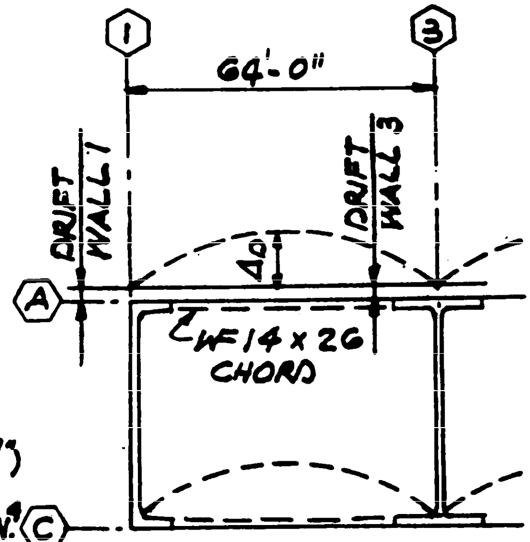
SHEAR IN WALL BELOW DIAPH. $= 150.7^{K}$

DIAPH SHR $V_D = 150.7 - 75^{K} - 13.6^{K} = 62.1$

$$\text{SHEAR STRESS} = \frac{62{,}100}{192} = 323\ \#/' \quad \text{OK}$$

Figure D-1. Continued.

DIAPHRAGM DEFLECTION



CHECK DEFLECTION OF ROOF DIAPH. BETWEEN GRID 1 & 3

$\Delta_D = \Delta_{\text{BENDING OF FLANGE}} + \Delta_{\text{SHEAR IN WEB}}$

ASSUME Δ_B IS DEFLECTION OF A SIMPLY SUPPORTED DIAPH.

$$\Delta_B = \frac{5}{384} \cdot \frac{wl^4}{EI}$$

WHERE I IS ASSUMED TO BE BASED ON WF14 x 26 CHORD (A = 7.67 □")

$$I = 2 \times 7.67\ \square'' \times \left(\frac{47.2'' \times 12''/'}{2}\right)^2 = 1,230,300\ \text{IN.}^4$$

(p. 39)

$$\Delta_B = \frac{5 \times 481\ \#/' \times 64^4 \times 1728}{384 \times 29 \times 10^6 \times 1230300} = 0.005''$$

(p. 39)

AVG. SHEAR/F OF DIAPH $q_{AVG.} = \frac{18.9 + 0}{2 \times 48} = 0.197\ K/'$

FLEXIBILITY $F = 16 + 26.8R$ (SEE FIG. 5-19)

WHERE $R = 6/18 = 0.33$

$F = 16 + 26.8(.33) = 24.8$

DIAPH DEFLECTION FROM SHEAR IN WEB:

$$\Delta_W = \frac{q_{AVG.} L_1 F}{10^6} = \frac{197 \times 32 \times 24.8}{10^6} = 0.156''\ \text{(Equ. 5-2)}$$

TOTAL DIAPH DEFLECTION $\Delta_D = \Delta_B + \Delta_W$

$= 0.005 + .156 = 0.161''$

DRIFT OF SHEAR WALL 1 $\Delta_1 = 0.037'' \times \frac{10''}{12''} \times \frac{26.4^K}{1000^K} \times \frac{3}{3.6} = 0.00070''$

(p. 9) (p. 13)

(ASSUMES FIXED BASE) ADJUSTMENT TO FIG 6-4 FOR THICKNESS, FORCE & MODU. ELAS.

DRIFT OF SHEAR WALL 3 $\Delta_3 = \left(\frac{1}{38.1}\right) \times \frac{10''}{12} \times \frac{47.5}{1000} \times \frac{3}{3.6} = 0.00087''$

ALLOWABLE DRIFT $= 0.005H = 0.005 \times 12'_{AVG} \times 12 = 0.72'' > 0.00087$

$0.04/R_w = 0.04/6 = 0.0067$ (SEAOC 1E8a)

Figure D-1. Continued.

DIAPHRAGM DEFLECTION

THE AVERAGE DRIFT OF WALL (1) & (3) $= \frac{0.00070 + 0.00084}{2} = 0.0008''$

TOTAL RELATIVE DISPLACEMENT OF ROOF DIAPH W/RESPECT TO THE 2ND FLOOR $= 0.161'' + 0.0008$
$= 0.162''$

THE WALL ELEMENT MUST BE DESIGNED TO ACCOMODATE THIS RELATIVE DISPLACEMENT. IN THIS EXAMPLE PROBLEM, THE WALL ELEMENT IS A RELATIVELY FLEXIBLE CURTAIN WALL WHICH PRESENTS NO PROBLEM. THE DEFLECTION CACULATIONS HAVE BEEN INCLUDED PRIMARILY TO ILLUSTRATE THE PROCEDURE IN CASES WHERE BRITTLE WALLS (MASONRY OR CONC.) OCCUR.

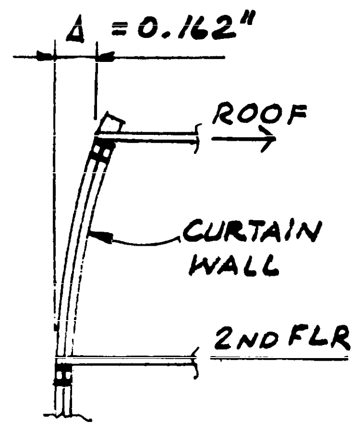


Figure D-1. Continued.

DESIGN EXAMPLE D-2

Concrete Special Moment Resisting Frame

Description of Structure. A three-story Administration Building with a ductile moment resisting space frame in reinforced concrete without shear walls, using non-bearing, non-shear, exterior walls (skin) of flexible insulated metal panels. The structural concept is illustrated on Sheet 3.

Construction Outline.

Roof:
- Built-up 5-ply.
- Concrete joists and girders.
- Suspended ceiling.

2nd & 3rd Floors:
- Concrete joists and girders.
- Asphalt tile.
- Suspended ceiling.

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

Exterior Walls:
- Non-bearing, non-shear, insulated metal panels.

Partitions:
- Non-structural removable drywall.

Design Concept. Since the structure is a ductile moment resisting space frame with the capacity to resist the total required lateral force, the R_w-factor is 12. Seismic Zone 4.

Discussion. Inasmuch as the design requirements for concrete ductile moment-resisting frames are complex, a detailed design procedure is given on p. 2 of the example.

Loads.

Roof:

5-ply roofing	6.0
1" insulation	1.5
Conc. frame	115.0
Ceiling	5.0
Miscellaneous	3.5
Dead Load	131 psf
Add for seismic loading:	
Partitions	10
	141 psf
Live Load	20 psf

Floors:

Floor covering	1
Conc. frame	129
Partitions	20
Ceiling	5
Mech. & Elect.	5
Miscellaneous	4
Dead Load	164 psf
Live Load	50 psf
Exterior Wall	11 psf

Materials.

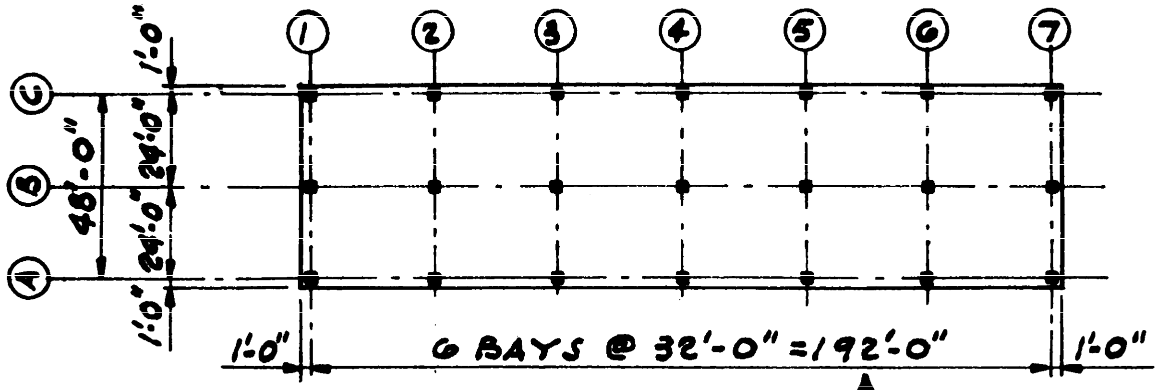
Concrete: $f'_c = 4$ ksi $\quad E_c = 3.6 \times 10^6$ psi

Steel: $f_y = 60{,}000$ psi

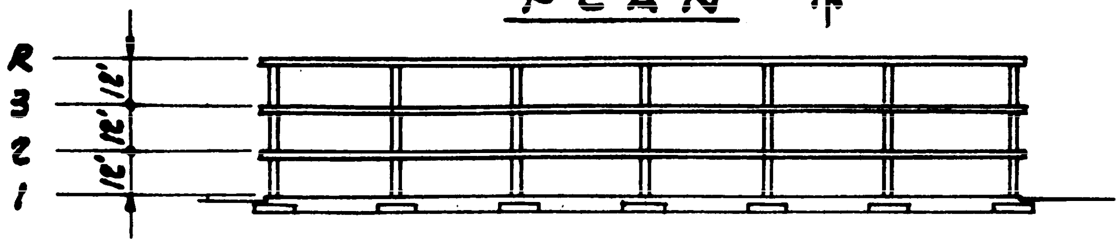
Figure D-2. Concrete ductile moment resisting space frame.

DESIGN PROCEDURE

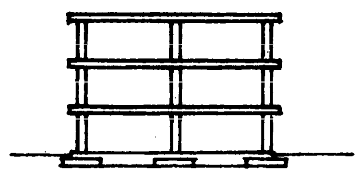
Figure D-2. Continued.



PLAN



NORTH & SOUTH ELEVATIONS
(SECTION AT LINE B SIMILAR)



EAST & WEST ELEVATIONS
(TRANSVERSE SECTIONS SIMILAR)

SCOPE OF DESIGN EXAMPLE

1. CALCULATION OF LATERAL FORCES ON BLDG. AND DISTRIBUTION TO FRAME
2. TABULATION OF THE RESULTS OF ANALYSIS OF THE FRAME ON LINE B
3. DESIGN OF SECOND FLOOR BEAMS BETWEEN 1 AND 2
4. DESIGN OF COLUMNS B1 & B2

Figure D-2. Continued.

DISCUSSION OF MEMBER SIZES

1. The example is intended to illustrate the procedure for designing a concrete ductile moment resisting frame. The design work is complex, and several trials are required in order to achieve the optimum design.

2. The building configuration was arbitrarily made the same as that of the steel frame of example D-3.

3. Frame B will be analyzed in this example and members between grid lines 1 & 2 will be designed to illustrate the design procedure.

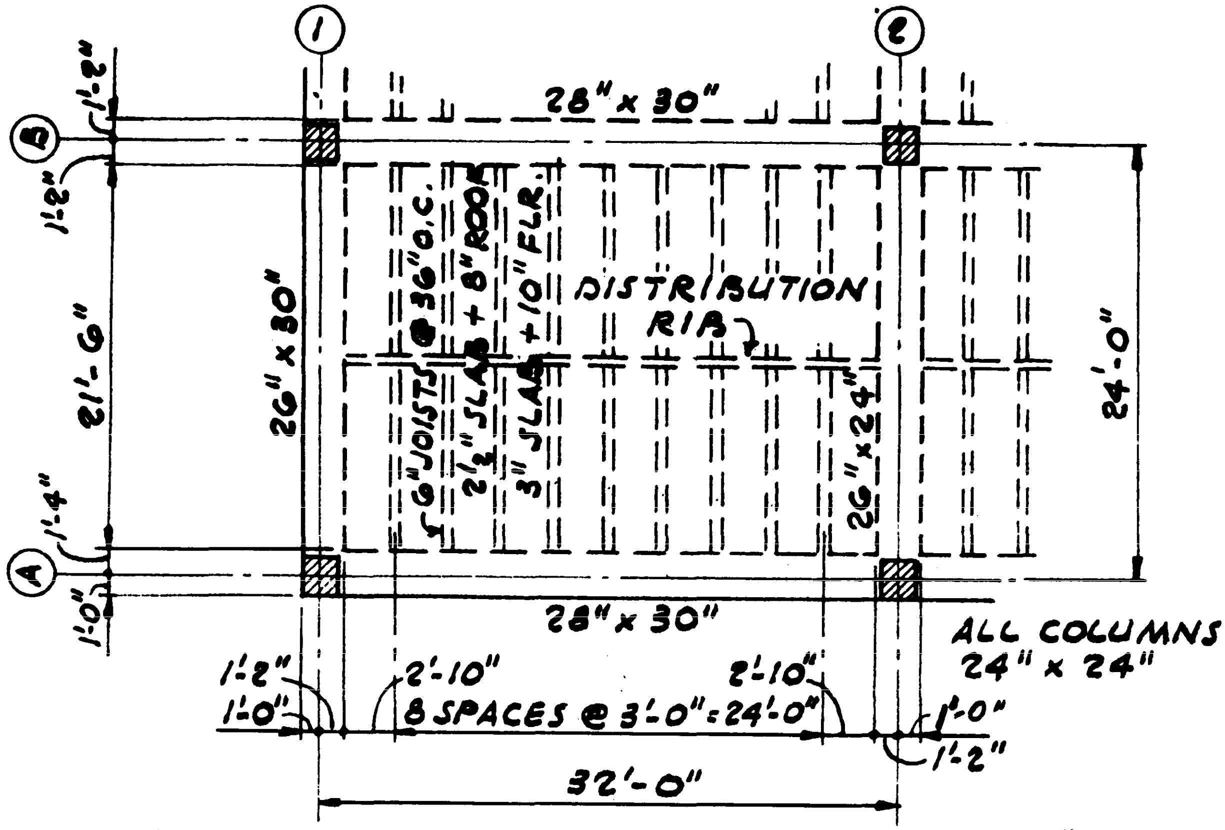
   a. The section of beam & col. sizes is a trial and error procedure. Architectural considerations, limitations on dimensions (Fig. 8-2), space for bar placement, allowable stresses of concrete and steel, etc., can affect the member sizes.

   b. The beam was assumed to be 28" x 30", and the required reinforcing and the actual ultimate moment capacity were calculated.

   c. For the min or max Pu and the required M_p (on the basis of column M_p > beam M_p), a suitable column was estimated to be 24" x 24", with 12 - #10 or 10 - #11. (Note: Biaxial loading must be considered for column forces in the transverse direction:)

4. Results of a frame analysis are given, and the example continues with representative beam, column and joint design, using sizes and design forces from this analysis. The frame analysis itself is not shown since values can be obtained by computer or by any of the various approximate methods.

Figure D-2. Continued.



STRUCTURAL PLAN SCALE: 3/32" = 1'-0"

WEIGHT OF CONCRETE IN TYPICAL 32'x50' BAY

ROOF:

LONGIT. GIRDERS	$3 \times 2.33' \times 2.5' \times 32' \times 0.150$	$= 84.0$
TRANSV. BEAM	$2 \times 2.17 \times 2.0' \times 21.5 \times 0.150$	$= 28.0$
JOISTS	$2 \times 29.67 \times 21.5 \times 0.050$	$= 63.8$
COLUMNS 24x24	$3 \times (9.5/2) \times (2.0)^2 \times 0.150$	$= 8.6$
		184.4^K

$$\omega_R = \frac{184{,}400^{\#}}{32.0' \times 50.0'} = 115 \text{ PSF}$$

FLOOR:

$$84.0 + 28.0 + \frac{61}{50}(63.8) + 2(8.6) = 207^K \text{ OR } 129 \text{ PSF}$$

Figure D–2. Continued.

BUILDING WEIGHTS

AT ROOF LEVEL

ROOF DL = $(0.131+0.010)$ KSF $\times 50' \times 194'$ = 1368^K

EXT. WALLS @ 10 PSF

PARAPET

N & S $2 \times 194' \times \left(\frac{12'}{2}+1'\right) \times 0.011$ = 30^K

E & W $2 \times 50' \times 7 \times 0.011$ = 8

$W_R = 1406^K$

AT FLOOR LEVEL

FLOOR DL = $0.164 \times 50' \times 194'$ = 1591^K

EXT. WALLS

N & S $(12'/7') \times 30$ = 51

E & W $(12'/7') \times 8$ = 14

$W_3 = W_2 = 1656^K$

TOTAL $W = \Sigma W = 1406 + 1656 + 1656 = 4718^K$

BASE SHEAR

$Z = 0.4 \qquad I = 1.0 \qquad R_W = 12$

$C_t = 0.030 \qquad h_n = 36.0'$

$$T = C_t (h_n)^{3/4} = 0.030\,(36.0)^{3/4} = 0.441$$

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.441)^{2/3}} = 3.24, \text{ USE } 2.75$$

$$V = \frac{ZIC}{R_W} W = \frac{0.4 \times 1.0 \times 2.75}{12} W = 0.0917W$$

$$= 0.0917 \times 4718 = 432^K$$

Figure D-2. Continued.

STORY FORCES & OVERTURNING

LEVEL	h_x	Δh	w_x	$w_x h_x$	$\frac{wh}{\Sigma wh}$	F_x	V_x	$V_x h$	M
R	36'		1406	50,616	.46	200^K			
		12'					200	2400	
3	24'		1656	39,744	.36	156^K			2400
		12'					356	4272	
2	12'		1656	19,872	.18	76^K			6672
		12'					432	5184	
			4718	110,232	1.00	432^K			$11,856^K$
			W			V			

RELATIVE RIGIDITIES OF FRAMES

THE FOLLOWING ASSUMPTIONS ARE MADE IN ORDER TO ESTIMATE THE FORCES TO BE APPLIED IN THE FRAME ANALYSIS.

LONGITUDINAL FRAMES

A, B & C 5 COLUMNS @ 1 = 5.0; 2 COLUMNS @ 3/4 = 1.5 } R = 6.5

$\Sigma R = 3 \times 6.5 = 19.5$

TRANSVERSE FRAMES

LINES 1 & 7 1 COLUMN @ 1 + 2 @ 3/4 = 2.5

ADJUST FOR SHORTER BEAMS = 32'/24' = 1.33; SAY 1.16* × 2.5 = 2.9

ADJUST FOR NARROW BEAMS = (26"/28") = 0.928; SAY 0.93* [0.93?] × 2.9 = 2.7

LINES 2-6 1 COLUMN @ 1 + 2 @ 3/4 = 2.5

ADJUST FOR SHORTER BEAMS 1.16* × 2.5 = 2.9

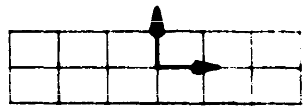
ADJUST FOR SHALLOWER BEAMS = $(24/30)^3$ = 0.51; SAY 0.75* [0.75?] × 2.9 = 2.2

$\Sigma R = (2 \times 2.7) + (5 \times 2.2) = 16.4$

* NOTE: Effects of joint rotation are not proportional to beam stiffness.

Figure D-2. Continued.

DISTRIBUTION OF FORCES TO FRAMES



UNIT FORCE, $F = 1.00^K$

FRAME	REL R	$\frac{R}{\Sigma R}$	DIRECT FORCE	d	d^2	Rd^2	$\frac{Rd^2}{\Sigma Rd^2}$	TORSION FORCE	DIRECT + TORSION
1	2.7	.165	.165	+96	9216	24,883	.312	+.031	.196
2	2.2	.134	.134	+64	4096	9,011	.113	+.017	.151
3	2.2	.134	.134	+32	1024	2,253	.028	+.008	.142
4	2.2	.134	.134	0	0	0	0		.134
5	2.2	.134	.134	-32	1024	2,253	.028	-.008	.126
6	2.2	.134	.134	-64	4096	9,011	.113	-.017	.117
7	2.7	.165	.165	-96	9216	24,883	.312	-.031	.134
	16.4	1.000	1.000 TRANSV.					SEE NOTE	
A	6.5	.333	.333	+24	576	3,744	.047	+.005	.338
B	6.6	.334	.334	0	0	0	0	0	.334
C	6.5	.333	.333	-24	576	3,744	.047	-.005	.328
	19.5	1.000	1.000 LONGIT.		ΣRd^2 =	79,782		SEE NOTE	

NOTE: SIGNS REVERSE FOR FORCE IN OPPOSITE DIRECTION. THEREFORE, DESIGN FOR ABSOLUTE VALVE OF TORSION FORCE.

TORSION:

BUILDING IS SYMMETRICAL, ∴ NO CALCULATED TORSION. ACCIDENTAL TORSION IS BASED ON ECCENTRICITY OF 5% OF MAX. DIM.

N-S EQ: $M_T = [0.05(192')] \times F = 9.6' \times 1.00^K = 9.6\ K'$

FORCE TO FRAMES 1 THROUGH 7

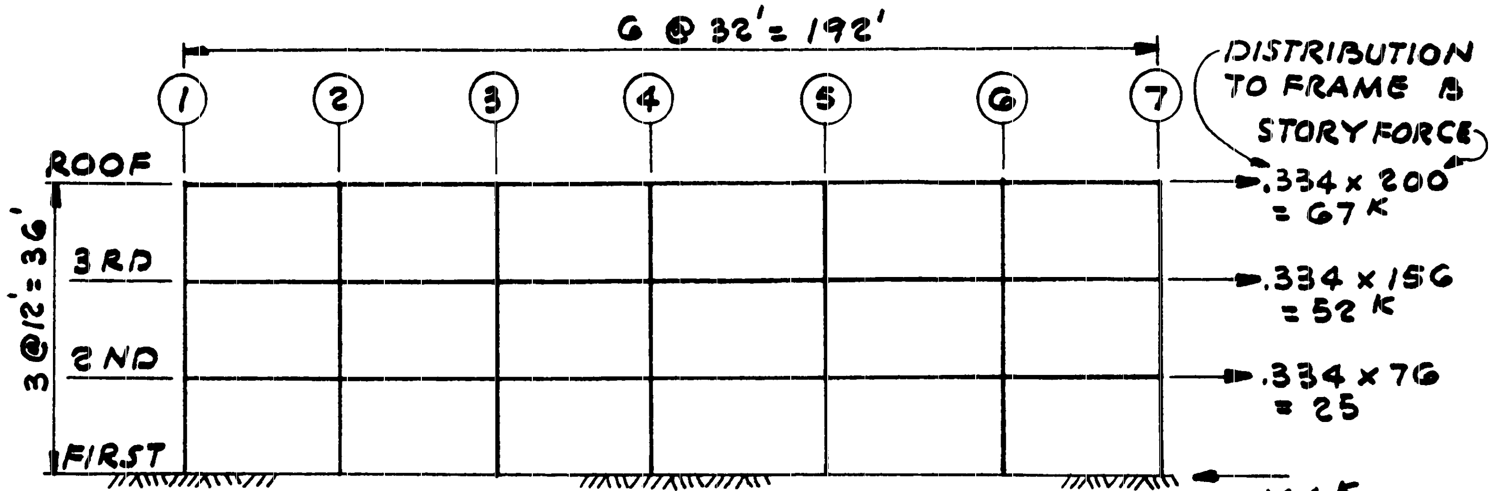
$$= \frac{(Rd^2/\Sigma Rd^2)\, M_T}{|d|} = \frac{9.6}{|d|} \cdot \frac{Rd^2}{\Sigma Rd^2}$$

E-W EQ: $M_T = [0.05(48')] \times F = 2.4' \times 1.00^K = 2.4\ K'$

FORCE TO FRAMES A, B, C

$$= \frac{(Rd^2/\Sigma Rd^2)\, M_T}{|d|} = \frac{2.4}{|d|} \cdot \frac{Rd^2}{\Sigma Rd^2}$$

Figure D-2. Continued.



PROVIDE DEEP GRADE BEAM TO STIFFEN COLUMNS. EXAMPLE ASSUMES COLUMNS FIXED AT BASE.

BEAM	b x D	I_g ft^4	W_{DL}	W_{LL}	REDUCED LL
ROOF	28"x30"	3.04	3.14 K/'	0.27 K/'	12 PSF
3RD & 2ND	28"x30"	3.04	3.94 K/'	0.72 K/'	30 PSF

ALL COLUMNS 24"x24"
$I_c = 1.33\ ft^4$

$f'_c = 4000$ PSI
(STONE AGG.)

$E = 57{,}000 \times \sqrt{4000}$
$= 3{,}605{,}000$ PSI
$= 519{,}000$ KSF

REQUIRED OUTPUT

- BEAM & COLUMN END MOMENTS AT FACE OF SUPPORT
- COLUMN SHEAR & AXIAL LOAD
- FRAME PERIOD AND HORIZONTAL DEFLECTION UNDER LATERAL LOADS

BEAM DIMENSION LIMITATIONS (FIG. 8.2)

WIDTH = 28" > 10" MIN
DEPTH = 30 < 3.33 b MAX.
WIDTH = 28 < COL. WIDTH + 1.5d MAX

FRAME ANALYSIS = INPUT - FRAME B

Figure D-2. Continued.

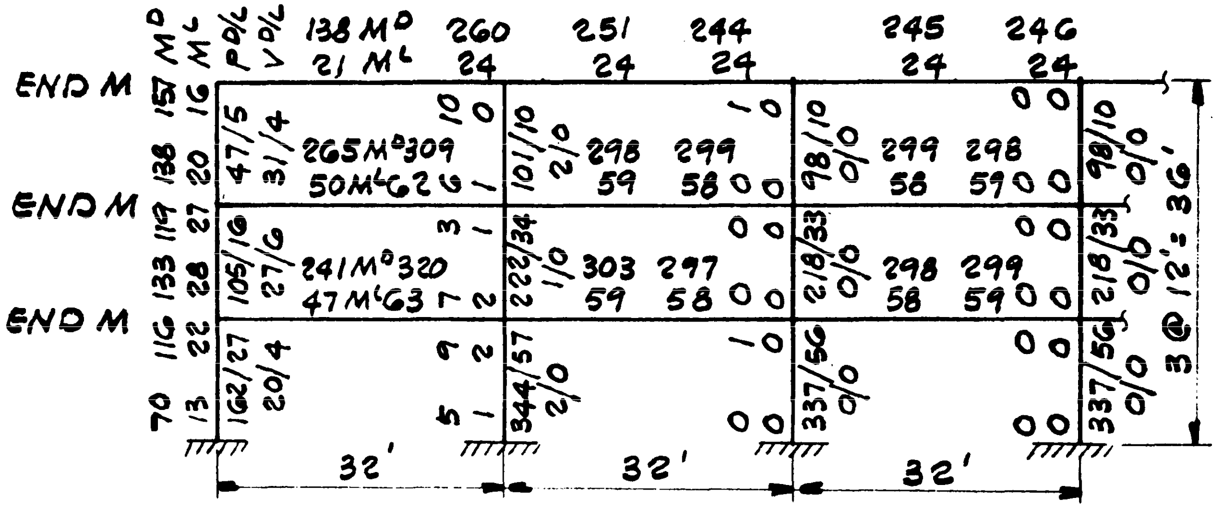
FRAME ANALYSIS RESULTS (CALCS. NOT SHOWN) FRAME B

	DL	LL	SEISMIC F
ROOF	3.14	0.27 K/'	67 K
3RD	3.94	0.72	52
2ND	3.94	0.72	25

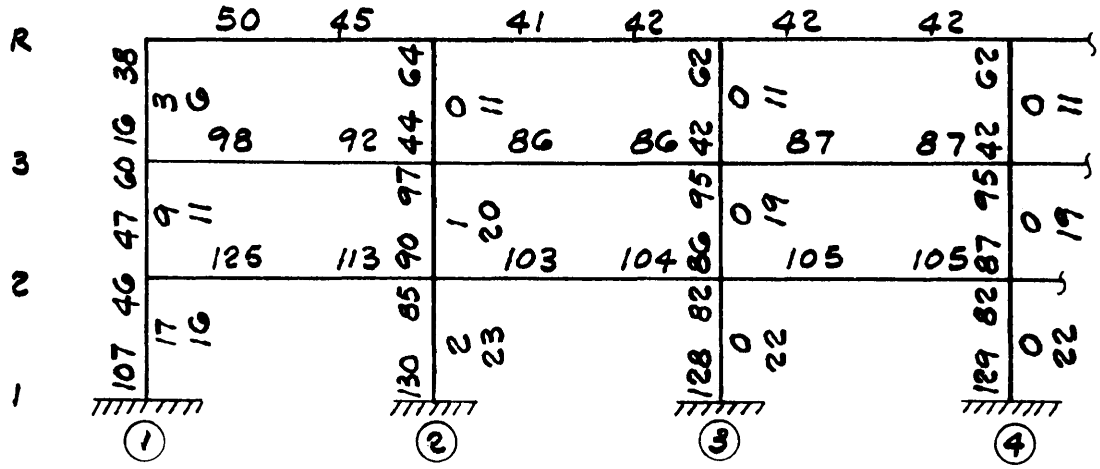
VALUES OF MOMENTS AND SHEARS ARE TAKEN AT FACE OF COLUMN OR BEAM.

ALL BEAMS 28"x 30"

ALL COLUMNS 24"x24"



VERTICAL LOAD



LATERAL LOAD

APPROX. 2ND STORY DRIFT

$$\Delta_{COL} = \frac{Ph^3}{12E_cI_c} = \frac{20(12)^3}{12(3600)1.33(12)} = .05'' \qquad \Delta_{TOTAL} = (.05''+.055'') = .105''$$

$$\Delta_{GIR} = \frac{Plh^2}{12E_bI_b} = \frac{20(30)(12)^2}{12(3600)3.04(12)} = .055'' \qquad \Delta_{ALLOW.} = \frac{.04}{R_w}h = .48''\ OK$$

Figure D–2. Continued.

DESIGN FORCES FOR BEAMS - PROCEDURE

1. Obtain end M's and V's at face of support. These are given on p. 10 for Frame B.

2. Calculate and tabulate factored M's and V's.

   a. Vertical load only

      $1.4D + 1.7L$

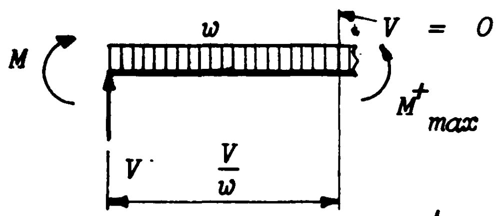
   b. Vertical plus maximum increase due to seismic

      $1.4\ (D+L+E)$ when E is in direction adding to $-M$

   c. Vertical minus reverse loading due to seismic

      $0.9D + 1.4E$ when E is in direction giving $+M$

3. Calculate and tabulate max. pos. mom. away from the end of the beam:



$+M$ produces tension on the bottom

$+V$ is upward at the left end of the beam

$$M^{+}_{max} = M + \frac{V^2}{2w} \quad (wL > V > 0)$$

4. Select maximum values for design. It is strongly recommended to sketch moment diagrams, especially when spans and loads are irregular.

5. Checkerboard loading may govern, maximum positive moments.

DESIGN FORCES FOR COLUMNS

1. Obtain P, M, V at face of support. These are given on p. 10 for Frame B

2. Calculate and tabulate factored M's and P's

   a. $1.4\ D$
   b. $1.4D + 1.7L$
   c. $1.4(D+L+E)$ for E in direction adding to vert. load
   d. $0.9D+1.4E$ for E in direction opp. to vert. load

Figure D-2. Continued.

BEAM FORCES

FRAME (B) FLOOR 2 FROM (1) TO (2)

28"×30" d = 27½ +M = TENS. ON BOTTOM	END 1		CLEAR SPAN = 30.0'			END 2	
	M	V	W	WL'	M+	M	V
(P. 10) D L E ⇄	-241 k' -47 ± 125	+56.5 +10.6 ∓ 8	3.94 k/' 0.72 -	119 k 22 -	164 31 SMALL	-820 -63 ∓ 113	+62 +12 ± 8
1.4D+1.7L M+	-417	+97			+282	-555	+106
$1.4(D+L+\overrightarrow{E})$ M+	-228	+83			+273	-694	+114
$1.4(D+L+\overleftarrow{E})$ +M	-578	+105			+273	-378	+92
$0.9D+1.4\overrightarrow{E}$	-40	+40				-446	+67
$0.9D+1.4\overleftarrow{E}$	-392	+62				-130	+44.5
MAX. NEG. MAX. POS.	-578 —*				— +282	-694 —*	
1.4 (D+L) (D+L)		94 67					103 74

* IN THIS EXAMPLE, THE SEISMIC MOMENTS ARE NOT LARGE ENOUGH TO CAUSE LOAD REVERSAL.

Figure D–2. Continued.

BEAM LONGITUDINAL REINFORCEMENT

$f_y = 60$
$f'_c = 4$
$m = \frac{f_y}{0.85 f'_c} = 17.65$

(1) FRAME B FLOOR 2 (2)

$\omega_D = 3.94^{K}/_{\prime}$
$\omega_L = 0.72^{K}/_{\prime}$

28" x 30" $d = 27.5"$ $F = 1.76$

$\ell' = 30.0'$

−M	−578		−694	−651
$K = \frac{M}{F}$	328		394	
a_u	4.24		4.19	
REQ'D $A_s = \frac{M}{a_u d}$	4.96		6.02	
TOP BARS	5-#9		6-#9	
ACTUAL A_s	5.00		6.00	MIN. ρ = .0033
ρ	.00649		.00779	MAX. ρ = .025
+M		282		
K		175		
a_u		4.37		
REQ'D A_s		2.56		
½ TOP A_s	2.48		3.01	
BOTT. BARS	3-#9	4-#9	3-#9	
ACTUAL A_s	3.00	4.00	3.00	
ρ	.00390	.00519	.00390	

ULTIMATE MOMENT CAPACITY FURNISHED M_u

K	330		391
$-M_u = KF$	581		688
K	203	269	203
$+M_u$	357	473	357

ULT. MOM. CAP'Y: $\phi = 1.0$ & STEEL AT $1.25 f_y$ ** M_p

K	406 ÷ 0.9 = 451		481 ÷ 0.9 = 534
$-M_p$	794		940
K	250 ÷ 0.9 = 278		250 ÷ 0.9 = 278
$+M_p$	489	641	489

** SOLVE BY MODIFYING ρ BY 1.25 FACTOR.

Figure D-2. Continued.

BEAM TRANSVERSE REINFORCEMENT

(1) FRAME B FLOOR 2 (2)

$w_D = 3.94$
$w_L = 0.72$

28" x 30" d = 27.5"

$\ell' = 30.0$

$f_y = 60$
$f'_c = 4$
$\phi = 0.85$

$$V_u = \frac{M_p^1 + M_p^2}{L} + 1.4\,V_{D+L}$$

$v_c = 2\sqrt{f'_c} = 126$

$v_u = 8\sqrt{f'_c} = 504$ MAX

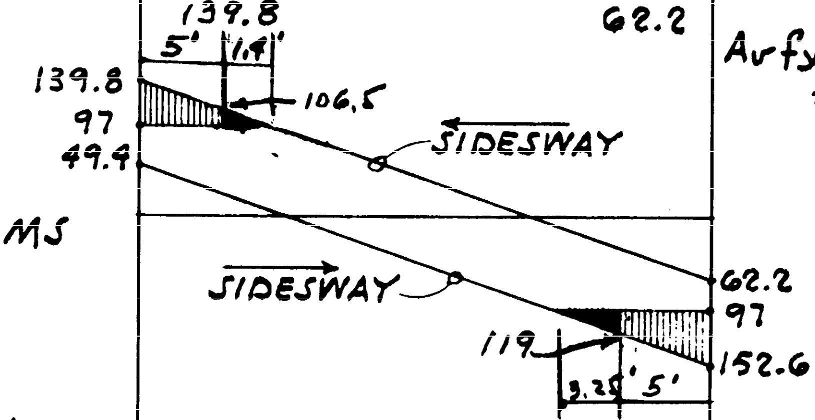
$V_c = .126 \times 28 \times 27.5 = 97^K$

$\overrightarrow{M_p}$	+489	−940
$\Sigma M/L$	−47.6	+47.6
$1.4\,V_{D+L}$	97	105
V_u	49.4	152.6
$\overleftarrow{M_p}$	−794	+489
$\Sigma M/L$	+42.8	−42.8
V_u	139.8	62.2

#3

$A_v f_y = 3 \times .11 \times 60 = 19.8^K$

SHEAR DIAGRAMS



$v_u = \dfrac{V}{\phi b d}$	$\dfrac{139{,}800}{0.85 \times 28 \times 27.5} = 214$	233
$v_u - v_c$	88	107
$s = \dfrac{A_v f_y}{(v_u - v_c)\,b}$	8.03"	6.61"
MAX. SPACING (WITHIN 2D = 5'-0" FROM COL.)	7"	6"
BEYOND 5'-0" (WITHIN SHADED AREA) $v_u - v_c$	36 (s=19")	(s=19") 56

MAX $s = \frac{d}{2} = 13.5"$
USE 13" SPACING

MIN. STIRRUPS
$\frac{d_i}{4} = 6.9"$
8 BAR DIA = 8"
24 TIE DIA. = 9"

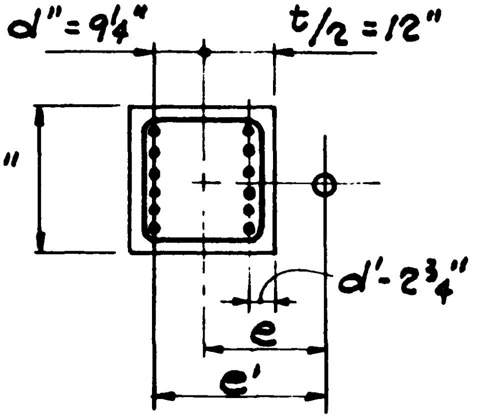
REF. FIG 8-6

Figure D-2. Continued.

COLUMN FORCES FRAME (B)

1ST STORY	COLUMN B-1			COLUMN B-2		
	AXIAL	MOMENT		AXIAL	MOMENT	
		TOP	BOTTOM		TOP	BOTTOM
D	162	116	70	344	-9	-5
L	27	22	13	57	-2	-1
E →	-17	-46	-107	-2	-85	-130
E ←	+17	+46	+107	+2	+85	+130
1.4D + 1.7L	272.7	199.8	120.1	578.8	-16.0	-8.7
$1.4(\overrightarrow{D+L+E})$	240.8	128.8	-33.6	558.6	-134.4	-190.4
$1.4(\overleftarrow{D+L+E})$	288.4	257.6	266.0	564.2	103.6	173.6
$0.9D + 1.4\overrightarrow{E}$	122.0	40.0	-86.8	306.8	-127.1	-186.5
$0.9D + 1.4\overleftarrow{E}$	169.6	168.8	212.8	312.4	110.9	177.5

COLUMN PROPERTIES (B-2)



f'_c = 4000 PSI
f_y = 60,000 PSI
6-#10 EACH FACE
$A_s = A_s'$ = 7.62 IN^2
d = 21¼"
d'/d = 0.129
$\phi = \frac{18\frac{1}{2}"}{24"} = 0.77$

DIMENSIONAL LIMITATIONS:
WIDTH = 24" > 12" OK
$\frac{MIN. DIM.}{MAX. DIM.} = \frac{24}{24} = 1 > 0.4$ OK
$h = b \geq 20 d_b$ OF BEAMS
24" > 20 × 9/8 = 22.5" OK

E_c = 519,000 KSF } P. 9
I_c = 1.33 FT^4 }
$E_c I_c$ = 690,000 K·FT^2
r = 0.3t = 0.60 FT
L_u = 9.5 FT

Figure D-2. Continued.

COLUMN SLENDERNESS — FRAME (B)

1ST STORY	COLUMN B-1		COLUMN B-2	
	TOP	BOTTOM	TOP	BOTTOM
$K = I/L$	$1.33/9.5 = 0.14$			
ΣK (COLS)	0.28	0.14	0.28	0.14
BEAM I/L	$3.04/30 = 0.10$			
ΣK (BMS.)	0.10	∞	0.20	∞
$\psi = \frac{\Sigma K\ COL}{\Sigma K\ BM}$	2.8	1 FIXED END	1.4	1 FIXED END
k	1.54		1.37	
kL/r	$1.54 \times 9.5/0.6 = 24.4$ > 22 ∴ "SLENDER" (CONT. BELOW)		$1.37 \times 9.5/0.6 = 21.7$ < 22 ∴ "SHORT" (CONT. ON P.17)	

	MAX. AXIAL	MIN. AXIAL	REMARKS
P_u	288.4	122.0	Σ AXIAL, FRAME B
ΣP_u (ALL COLS)	3440 ←		$(2 \times 272.7) + (5 \times 578.8)$
$\beta_d = \frac{M_D}{M_T}$	$\frac{116}{258} = 0.450$	$\frac{116}{40} = 2.90$	
EI	190,000	70,800	$EI = \frac{\frac{E_c I_c}{2.5}}{1 + \beta_d}$
$P_c = \frac{\pi^2 EI}{(KL_u)^2}$	8760	3260	
ΣP_c (ALL COLS)	72,900		$(2 \times 8760) + \left(5 \times \left(\frac{1.54}{1.37}\right)^2 \times 8760\right)$
$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}}$	1.049	1.056	$C_m = 1.0$ FOR UNBRACED COLUMNS
$\delta = \frac{C_m}{1 - \frac{\Sigma P_u}{\phi \Sigma P_c}}$	1.072		$\phi = 0.7$ MAX. COL. $M_u = 266$ K' (P.15)
δM	1.072×266.0 $= 285$ K'	$\approx 1.07 \times 86.8$ $= 93$ K'	REQ'D DES. M_u (NOTE: OTHER P.15 COMBIN. OF P & M WERE ALSO INVESTIGATED)

Figure D-2. Continued.

COLUMN CAPACITY FRAME (B)

1ST STORY	COLUMN B-1	COLUMN B-2
SIZE	24" x 24"	24" x 24"
BARS	8 - #9 A_{st} = 8.00	12 - #10 A_{st} = 15.24
$\frac{P_u}{A_g} = \frac{\phi P_n}{A_g}$	$\frac{288.4}{24 \times 24}$ = 0.500 KSI	$\frac{558.6}{24 \times 24}$ = 0.970

COL. MOM. CAPACITY MUST BE GREATER THAN BEAM CAPACITY SINCE $P/A_g > 0.12 f_c'$ = 0.40 KSI
(SEE CHECK BELOW)

$\rho_g = \frac{A_{st}}{A_g}$	$\frac{8.00}{24 \times 24}$ = 0.0139	$\frac{15.24}{24 \times 24}$ = 0.0264

USING ACI SP17A-(85) CHART "COL. E4 - 60.75" FIND

ϕM_n	485K' > (ΣM_u = 285)	785 K' > (M_u = 190.4)

COL. MOM CAPACITY, M_p @ ϕ = 1, STEEL @ 1.25 f_y
(which may be approximated by using 1.25 A_s)

	ρ = 1.25 x 0.0139 = 0.0174	ρ = 1.25 x 0.0264 = 0.0330
	$\frac{\phi M_n}{A_g h}$ = 0.48	$\frac{\phi M_n}{A_g h}$ = 0.79
M_p	= $\frac{0.48 \times 24^3}{0.7 \times 12}$ = 790 K'	= $\frac{0.79 (24)^3}{0.7 \times 12}$ = 1300 K'

CHECK COL. CAP'Y > BEAM CAP'Y

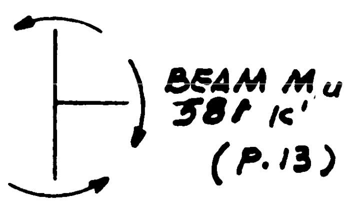
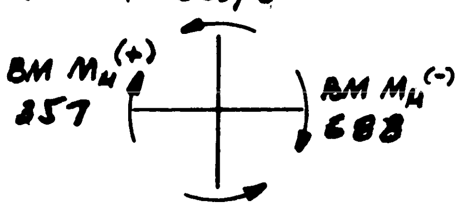
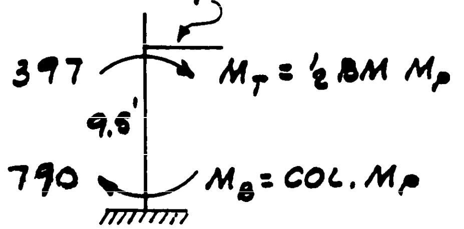
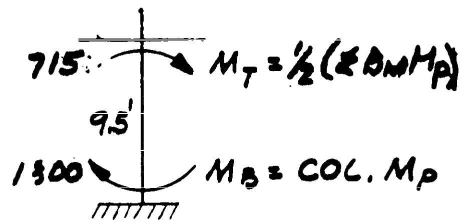
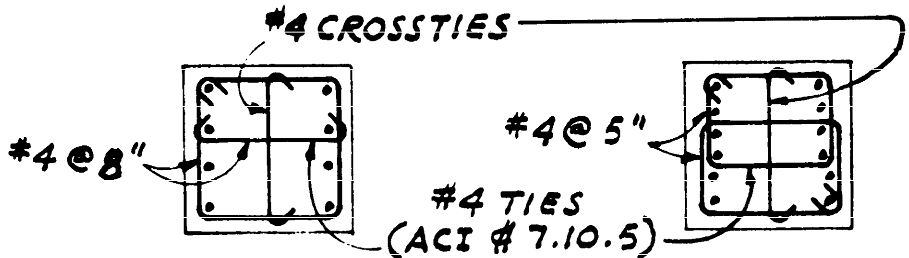
Σ COL. M $> \frac{6}{5} \Sigma$ BM. M	581/2 = 291 K'	(357 + 688)/2 = 522
		
	291 K' = ½ (BEAM M_u)	522 = ½ (Σ BM M_u)
	COL. M_u = 485 K'	COL. M_u = 782 K'
	$> \frac{6}{5} \times 291$ = 349 OK	$> \frac{6}{5} \times 522$ = 626 OK

Figure D-2. Continued.

COLUMN SHEAR — FRAME B

1ST STORY	COLUMN B-1	COLUMN B-2
	BM YIELDS BEFORE COL BM $M_p = 794$ 	
V	$\frac{397+790}{9.5'} = 125^K$	$\frac{715+1300}{9.5'} = 212^K$
$v_u = \frac{V}{\varphi A_c}$	$\frac{125}{.850 \times 420} = 0.35$ KSI P. 19	$\frac{212}{.85 \times 420} = 0.59$
$v_u - v_c$	$.350 - .126 = .224$ KSI	$.59 - .126 = .454$ KSI
TIE $S = \frac{A_v f_y}{(v_u - v_c) d_c}$	$\frac{3 \times 0.20^{\square''} \times 60^K}{.224 \text{ KSI} \times 20.5''} = 7.81''$ USE 8"	$\frac{4 \times 0.20^{\square''} \times 60^K}{.454 \text{ KSI} \times 20.5''} = 5.16$ USE 5"

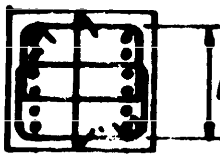


USE #4 COLUMN TIES, $A = 0.20^{\square''}$

MAX. SPACING, $S_{MAX} = \frac{\text{COL. DIM.}}{2} = 12''$

Figure D–2. Continued.

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT FOR CONFINEMENT



$h_c = 24 - 2(1\frac{3}{4}) = 20.5''$

TIE SETS @ SPACING s,
$s \leq 4''$

$A_g = 24 \times 24 = 576 \text{ IN}^2$

$A_c = 20.5 \times 20.5 = 420 \text{ IN}^2$

$f_c' = 4{,}000 \quad f_{yh}'' = 60{,}000$

A_{sh} = TOTAL AREA OF HOOPS

REF. ACI 21.4.4.1

$$(1) \quad s = \frac{A_{sh}}{0.30 h_c'' \frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_{ch}} - 1\right)} = \frac{A_{sh}}{0.30 \times 20.5 \times \frac{4}{60}\left(\frac{576}{420} - 1\right)} = 6.57 A_{sh}$$

LARGER GOVERNS

$$(2) \quad s = \frac{A_{sh}}{0.09\, h_c \frac{f_c'}{f_{yh}''}} = \frac{A_{sh}}{0.09 \times 20.5 \times \frac{4}{60}} = 8.13\, A_{sh}$$

BAR SIZE	A_s	A_{sh}	$8.13 A_{sh}$	
#3	0.11	0.44	3.58	
#4	0.20	0.80	6.50	← USE #4 GR 60 @ 6½"
#5	0.31	1.24	10.08	

EXTENT OF SPECIAL TRANSV. REINF. IS THE MAXIMUM OF:

- MAX. COL. DIMENSION = 24" ←
- 1/6 CLEAR HEIGHT = 114/6 = 19"
- 18"

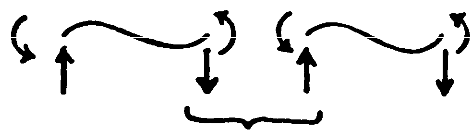
EXTEND MIN. 2'-0" ABOVE & BELOW

CONTINUED →

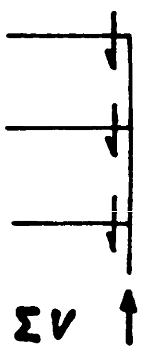
Figure D-2. Continued.

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT-CONT'D

SPECIAL TRANSV. REINF. IS ALSO REQUIRED WHERE COLUMN CAPACITY IS LESS THAN THE SUM OF THE SHEARS ABOVE. REF. APP. C PARAGRAPH C12



- ΣV IS THE COLUMN LOAD AT THIS LEVEL.
- AT INTERIOR COL'S THIS SUM IS RELATIVELY SMALL.
- END COLUMNS ARE USUALLY CRITICAL.



ΣV

1. INCLUDE ALL BEAMS ABOVE THE COLUMN IN QUESTION.

2. $V_{u_i} = \frac{(M_p^A + M_p^B)}{L}\Big|_{p.13} + V_{D+L}\Big|_{p.12}$

3. AT THE COLUMN IN QUESTION, CALCULATE THE MAX. MOMENT TRANSFERRED TO THE COLUMN BY THE YIELDING BEAM.

4. DOES THE COLUMN HAVE THE CAPACITY TO CARRY ΣV_u WITH THIS BEAM MOMENT?
   YES: NO ADD'L REINF. REQ'D.
   NO: PROVIDE THE SPECIAL TRANSV. REINF. CALCULATED ON P. 19 FOR FULL HEIGHT.

SEE P. 21 FOR SAMPLE CALC.

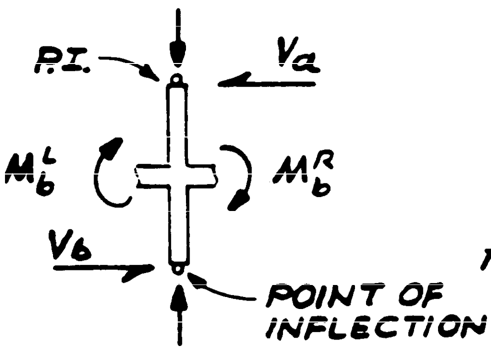
Figure D-2. Continued.

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT CONT.

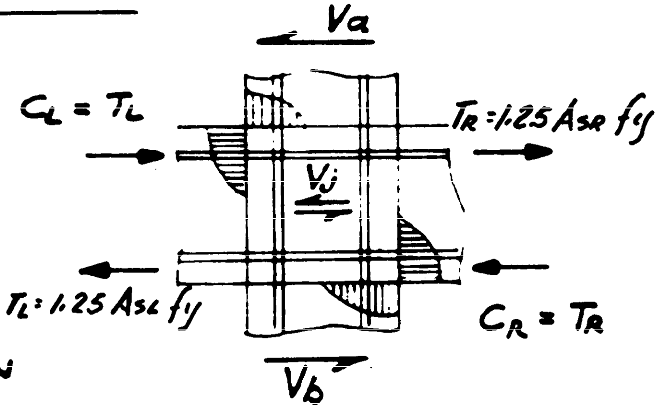
FRAME COLUMN	B 1	
ROOF BEAM $\Sigma M_p/L$ V_{D+L} V_u^R	35 55 90	CALCS. NOT SHOWN
3RD FLR. BEAM $\Sigma M_p/L$ V_{D+L} V_u^3	43 67 110	ASSUME SAME AS 2ND
2ND FLR. BEAM $\Sigma M_p/L$ V_{D+L} V_u^2	43 67 110	$= \frac{794 + 489}{30}$ (p. 13) (p. 12)
ΣV_u ABOVE	310	
M_p FROM BM	397	$= \frac{1}{2}$ BM $M_p = \frac{794}{2}$
ALLOW COL. M WITH $P = \Sigma V_u$	659	(ACI. SP 17A VOL. 2 CHART E4-60-75)
COL. M > M_p	YES	
SPEC. TRANSV. REINF.	NO	

Figure D-2. Continued.

BEAM - COLUMN JOINT



FORCES ON COLUMN



FORCES ON BEAM COLUMN JOINT

THE JOINT SHEAR, $V_j = 1.25 A_{SR} f_y + C_L - V_a$

$$= 1.25 (A_{SR} + A_{SL}) f_y - V_a$$

$$v_j = V_j / \phi A_j$$

A_j = BEAM WIDTH x COLUMN DEPTH = 28" x 24"

INTERIOR JOINTS: REF. ACI 21.6.2.3

JOINTS ARE CONFINED

$\therefore v_j \leq (20\sqrt{f_c'} = 1265 psi)$

TRANSV. HOOP REINF. MAY BE $\frac{1}{2}$ OF COLUMN HOOP REINF.

EXTERIOR JOINTS:

JOINTS ARE NOT CONFINED

$\therefore v_j \leq (15\sqrt{f_c'} = 949 psi)$

TRANSV. HOOPS SAME AS COL. HOOPS

Figure D-2. Continued.

BEAM – COLUMN JOINT – CONT. FRAME (B)

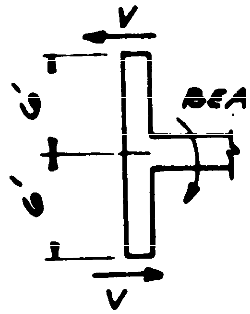
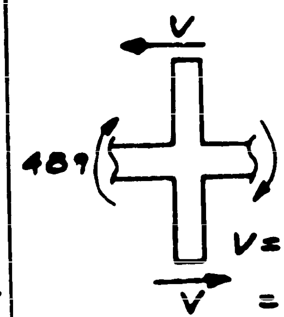
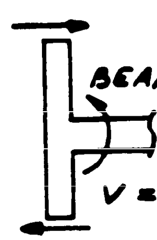
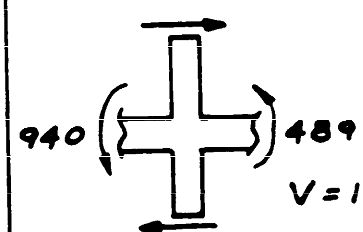
2ND STORY	COLUMN B-1	COLUMN B-2
HOOPS IN JOINT ← E	 $V = \frac{774^{k'}}{12'}$ $= 66.2^k$	 $V = \frac{940+489}{12}$ $= 119^k$
BEAM A_{SR}	$5.0\ IN^2$ (5-#9)	6.0 (6-#9)
BEAM A_{SL}	0	3.0 (3-#9)
$(A_{SR} + A_{SL})1.25 F_y$	$(5.0+0)(75) = 375^k$	$(6.0+3.0)(75) = 675^k$
V_u (see Fig 8-9)	$375 - 66 = 309^k$	$675 - 119 = 556^k$
$v_u = \frac{V_j}{\phi b d}$	$\frac{309{,}000}{0.85 \times 28 \times 24} = 541\ psi$	$\frac{556{,}000}{0.85 \times 28 \times 24} = 973\ psi$
E →	 $V = \frac{489}{12} = 40.8$	 $V = 119$
BEAM A_{SR}	$3.0\ IN^2$	$3.0\ IN^2$
BEAM A_{SL}	0	$6.0\ IN^2$
V_u (See Fig. 8-9)	$225 - 41 = 184^k$	$675 - 119 = 556^k$
v_u	332 psi	973 psi

Figure D-2. Continued.

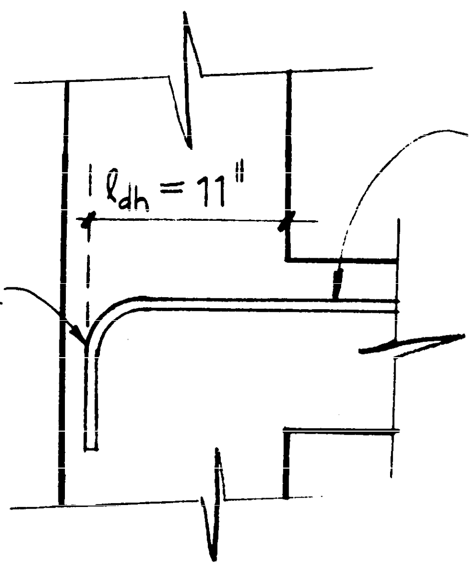
BEAM-COLUMN JOINT-CONT. FRAME (B)

2ND STORY	COLUMN B-1	COLUMN B-2
v_j	541 < 949 p.s.i. O.K.	975 < 1265 p.s.i. O.K.
#4 HOOPS	SAME AS COL. (p. 19)	½ THAT OF COL. (p. 19)
HOOP SPACING	4"	8"

ANCHORAGE OF LONGIT. BARS

A.C.I. 21.6.4

EXTERIOR JOINT OF COLUMN B-1.



#9 BARS

$d_b = 1.128''$

$$\ell_{dh} = \frac{f_y\, d_b}{65\sqrt{f_c'}}$$

$$= \frac{60{,}000 * 1.128''}{65 * \sqrt{4000}}$$

$$= 16.5''$$

NOTE: FOR INTERIOR JOINTS, COL. DEPTH > $20 d_b = 22.4''$ O.K.

Figure D-2. Continued.

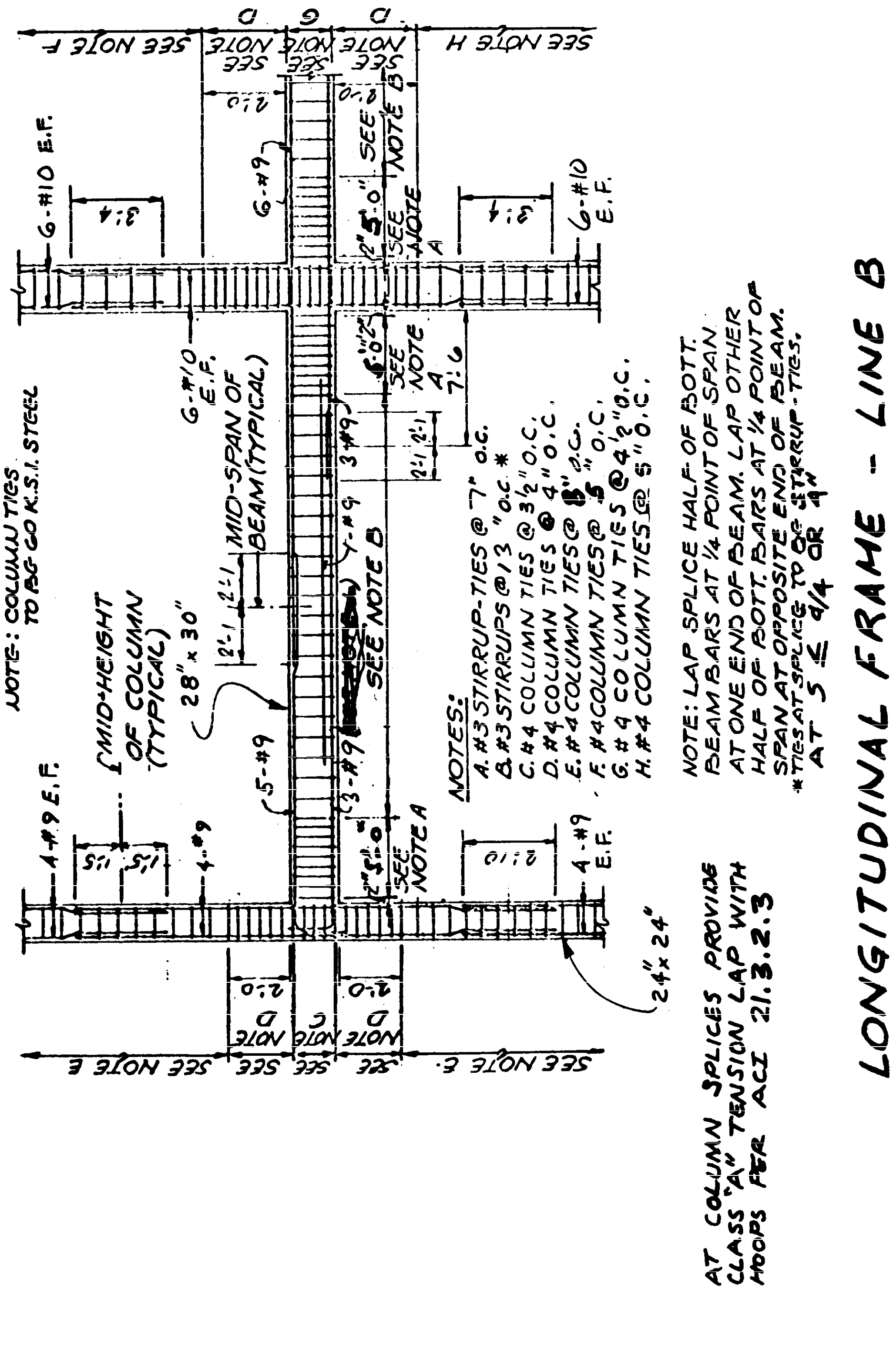


Figure D-2. Continued.

DESIGN EXAMPLE D-3

Steel Special Moment Resisting Frame and Steel Braced Frame

Description of Structure. A three-story Administration Building with transverse special moment-resisting frames and longitudinal concentric or eccentric braced frames in structural steel, using non-bearing, non-shear, exterior walls (skin) of flexible insulated metal panels. There are a series of interior vertical load-carrying column and girder bents in addition to the space frame. The structural concept is illustrated on Sheets 2 and 3.

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:
- Non-bearing, non shear, insulated metal panels.

Partitions:
- Non-structural removable drywall.

Design Concept. The transverse ductile moment-resisting frames are independent of the logitudinal braced frames. The moment frames are designed to $R_W = 12$; the concentric braced frames to $R_W = 8$; the eccentric braced frames to $R_W = 10$. The metal deck roof system forms a flexible diaphragm; therefore the roof loads are distributed to the frames by tributary area rather than by frame stiffnesses. The metal deck with concrete fill system for the floors form rigid diaphragms and the seismic loads are proportioned to the frames by the frame stiffnesses.

Discussion. Because of the importance of drift of flexible frames, the example shows several stages of design. Preliminary design to find sizes by approximate methods, using different sets of forces for stress and drift. The resulting trial sizes are then used in a computer analysis. (The frames are simple enough to be calculated by hand, but the computer makes short work of calculating design forces, frame period and drift). Final design is discussed, and examples are given for modifications to the results of the computer analysis for accommodating various stress and deflection criteria with consistent sets of member sizes, period, design force, and drift.

Figure D-3. Steel ductile moment resisting space frame and steel braced frame.

LOADS.

ROOF:

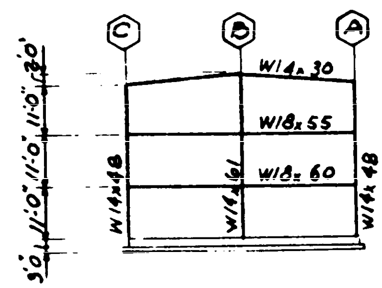
5-PLY ROOFING	=	6.0 P.S.F.
1" INSULATION	=	1.5
STEEL DECK	=	2.3
STEEL PURLINS	=	3.7
STEEL GIRDERS	=	1.2
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	25.7 P.S.F

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 & 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.

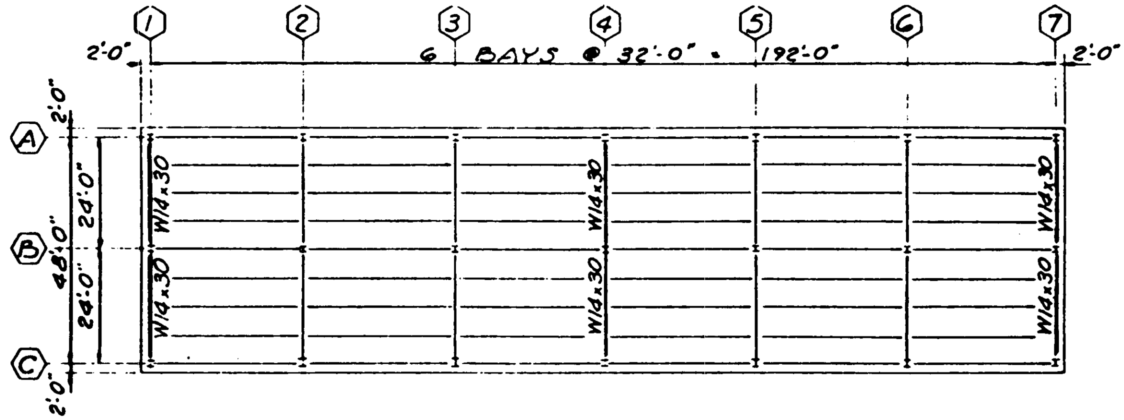


LINES 1, 4, & 7

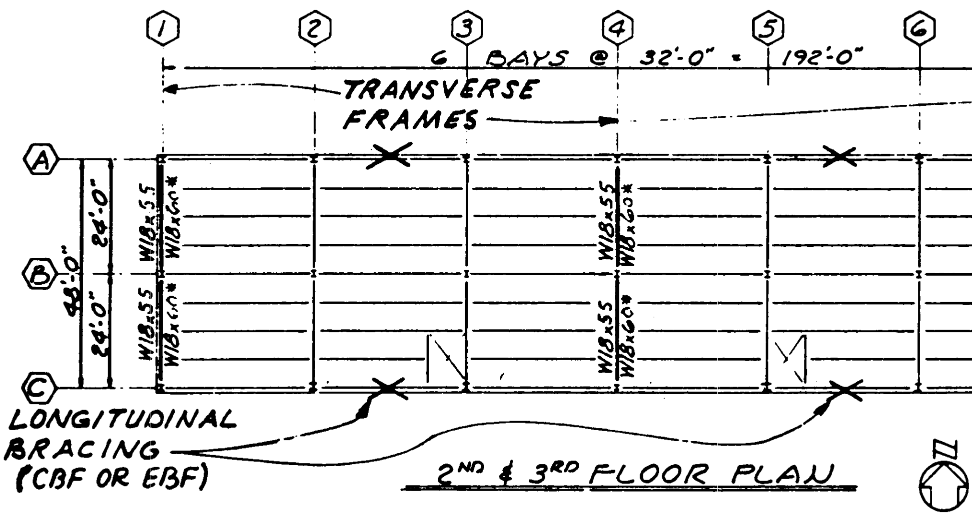
TRANSVERSE SPECIAL MOMENT RESISTING FRAMES

SEE SHT 7

Figure D-3. Continued.

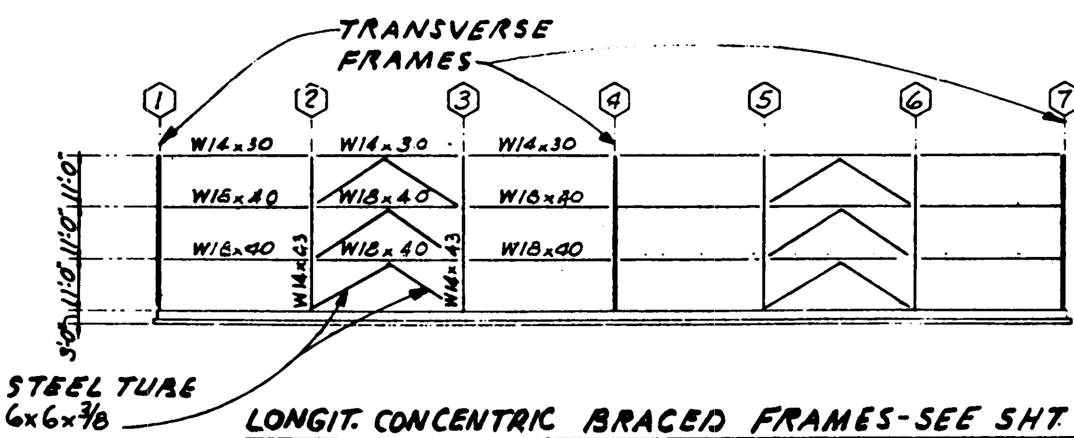


ROOF PLAN



2ND & 3RD FLOOR PLAN

.* INDICATES BEAM SIZE AT 2ND FLOOR



LONGIT. CONCENTRIC BRACED FRAMES-SEE SHT 28

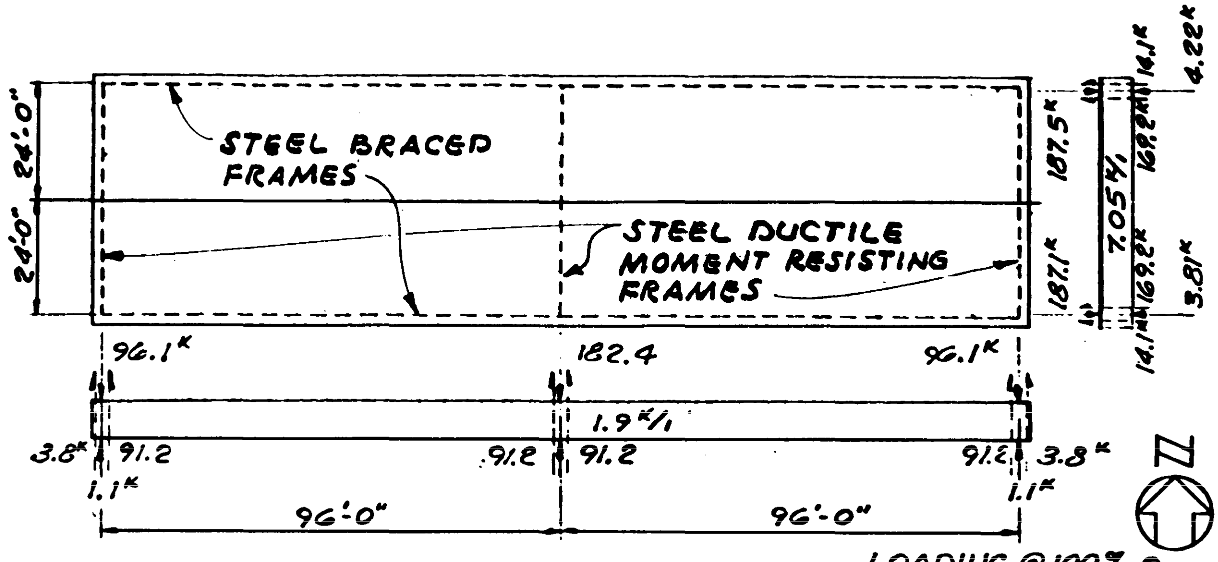
SOUTH ELEVATION (NORTH ELEV. SIM. OPP. HAND)

(FOR LONGITUDINAL ECCENTRIC BRACED FRAMES, SEE SHEET 40.)

Figure D-3. Continued.

DESIGN PROCEDURE

Figure D-3. Continued.



LOADING @ 100% g
$W_R = 374.6^K$

LOADS FOR ROOF DIAPHRAGM

EXTERIOR WALLS (NON STRUCTURAL EXTERIOR COVERING)

WALL WT. 5.3 PSF × 5.5' = 29.0 #/' — FRACTION OF SOLID WALL-WINDOWS OUT

N. WALL = 29 × .75 = 22 × 192' = 4224#

S. WALL = 29 × .68 = 19.8 × 192' = 3801#

41.8 #/'

WALL WT. 5.3 PSF × 6' = 31.8 #/'

E. WALL = 31.8 × .76 = 24 × 48' = 1.152#

W. WALL = 31.8 × .76 = 24 × 48' = 1.152#

48 #/'

N-S LOADS

ROOF = 35.7 × 52' = 1856.4

WALLS = 41.8 #/'

1898.2 #/'

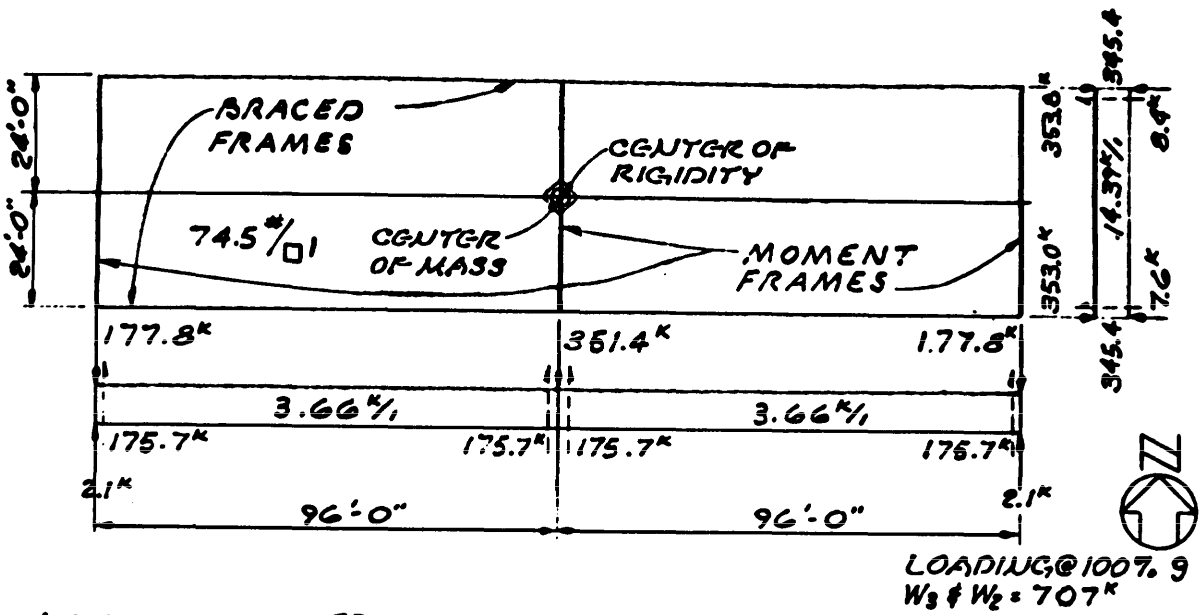
E-W LOADS

ROOF = 35.7 × 196" = 6997.2

WALLS = 48.0 #/'

7045.2 #/'

Figure D-3. Continued.



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

FLOOR WEIGHT FOR SEISMIC = 74.5 PSF

WALL WT. 5.3 PSF × 11' = 58.3 #/'

N. WALL = 58.3 × .75 = 44 × 192' = 8448#

S. WALL = 58.3 × .70 = 39.6 × 192' = 7603#

83.6 #/'

E. WALL = 58.3 × .75 = 44 × 48' = 2112#

W. WALL = 58.3 × .75 = 44 × 48' = 2112#

88 #/'

N-S LOADS

FLOOR = 74.5 × 48' = 3576.0

WALL = 84.0

3660.0 #/'

E-W LOADS

FLOOR = 74.5 × 192' = 14304.

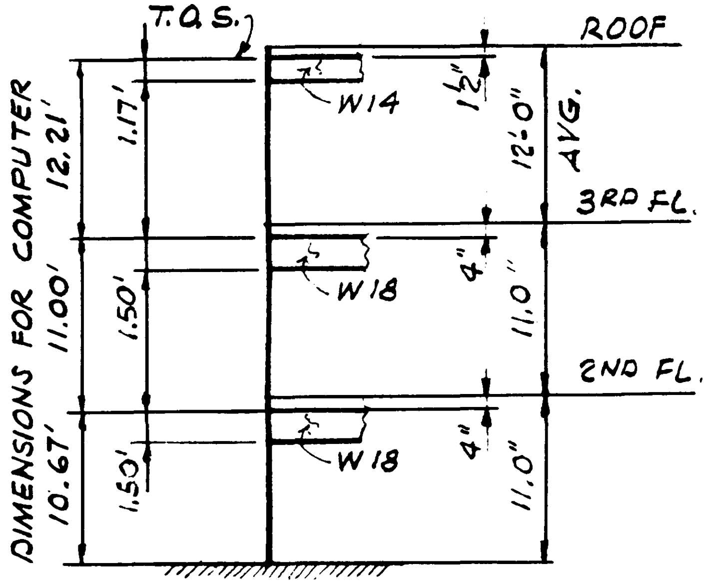
WALL 88.

14392. #/'

Figure D-3. Continued.

TRANVERSE (N-S) DIRECTION:

STEEL SPECIAL MOMENT-RESISTING FRAMES



THE COLUMN BASE IS ASSUMED FIXED.

THIS IS NOT ALWAYS FEASIBLE, ACTUAL FOUNDATION CONDITIONS SHOULD BE CAREFULLY STUDIED, AND REALISTIC ASSUMPTIONS SHOULD BE MADE FOR ANALYSIS.

FRAME CHARACTERISTICS

Gravity load: The middle frame will take twice as much gravity loads as each end frame according to tributary area.

Seismic load: All three frames will have the same proportions. Assuming the roof diaphragm is flexible, and using the tributary area approach, the middle frame will take half of the seismic load at the roof level while each end frame will take one quarter. Assuming the floor diaphragms are rigid, the third floor diaphragm will distribute some of the lateral load that originates at the roof level from the middle frame to the end frames; also, because the three frames have equal stiffness, the rigid floor diaphragms will distribute one third of the load that originates at each floor to each of the three frames. The roof diaphragm is not fully flexible: the middle frame will take something less than half and the end frames something more than one quarter each of the roof load. Also the floor diaphragms are not fully rigid: the end frames will probably not get a full third of the load. The example assumes that the middle frame keeps full half of the roof load and one third of the floor loads: what is probably an excessive load from the roof tends to offset what is probably a deficient load from the floors.

Figure D-3. Continued.

FRAME CHARACTERISTICS - cont'd

Because of accidental torsion the end frames will take some torsional forces below the third floor.

The total seismic forces being nearly the same in all frames, the design will be governed by the middle frame which takes twice as much gravity load as each end frame, and the design example will be concerned with this frame, i.e., the transverse frame on Line 4.

BUILDING PERIOD

In order to calculate lateral forces for design of the frame, the building period is needed. SEAOC provides two methods, Method A and Method B, and this example will make use of a third method, the Drift Limit Method.

Method A provides a simple formula based on the height of the building and the structural system, so it could be used as a first approximation for a preliminary design. Using C_t = 0.035 for steel frames and h_n = 34 feet, SEAOC eq 1-3 provides $T = 0.035\ (34)^{3/4} = 0.49$ seconds. Method A is intentionally conservative; it tends to be a lower bound. This is particularly noticeable with steel moment frames. It would be desirable to use a longer period in order to reduce the design forces in a more realistic representation of the building; however this must be done with care because if the period is too long the preliminary design will be undersized. The code provides a limit on T by not allowing a value of C less than 80% of the value obtained by using T from Method A.

Method B is an accurate method, but it requires frame deflections which can be calculated only after a preliminary design is established.

The Drift Limit Method provides a period based on the assumption that the frame is so limber that it is at its maximum allowable deflection under code-prescribed loads. This provides an upper-bound period in contrast to the lower-bound period of method A. The period of a frame can be approximated by the formula $T = 2\pi\sqrt{(\delta_n/a_n)}$, where δ_n is the lateral roof displacement for the peak roof acceleration a_n. For the

Figure D-3. Continued.

BUILDING PERIOD - cont'd

equivalent static force procedure, δ_n is the roof displacement due to the prescribed forces, and a_n is approximately equal to (1.7 V/W)g, or $(1.7ZIC/R_w)g$, where $C = 1.25S/T^{2/3}$ and g is the acceleration due to gravity. The formula can be written

$$T=0.66\left(\frac{\delta_n R_w}{ZIS}\right)^{3/4}$$

with δ_n in feet.

For T > 0.7 sec., the story drift limit is $0.03/R_w$. If the deflected shape is a straight line, $\delta_n = 0.030\ h_n/R_w$. But it is not likely that the deflected shape will be a straight line. Let us assume that the average story drift is 0.80 times the maximum story drift; then, $\delta_n = 0.024\ h_n/R_w$ and

$$T=0.040\left(\frac{h_n}{ZIS}\right)^{3/4}$$

for T > 0.7 sec.

For T < 0.7 sec., the story drift limit is $0.04/R_w$ and

$$T=0.050\left(\frac{h_n}{ZIS}\right)^{3/4}$$

for T < 0.7 sec.

The example will make use of this method. With $h_n = 34$, Z = 0.4, I = 1.0, and S = 1.5, T = 0.83 sec. (T > 0.7 sec.)

Period calculations, being based on framing members, are "bare-frame" periods, that is, they do not account for the participation of nonseismic frames and nonstructural elements. For <u>force</u> calculations, the calculated period will be reduced in order to account for the stiffening effects of these frames and other elements. For this example, we will divide the drift limit period of 0.83 sec. by a factor of 1.2, a number obtained by judgment.

The design lateral forces will be based on a "whole building" period of T = 0.83/1.2 = 0.69 sec.

Note that with $T_A = 0.49$ sec, $C_A = 2.75$. With T = 0.69 sec, C = 2.40 which is greater than 0.80 x 2.75 = 2.20.

Figure D-3. Continued.

LATERAL FORCES FOR PRELIMINARY DESIGN

USE T = 0.69 SEC. FOR STRESS ANALYSIS.

BUILDING : A-3

DIRECTION : TRANSVERSE

$Z = 0.4;\quad I = 1.0 \quad R_w = 12$

$C = \frac{1.25\,s}{T^{2/3}} = \frac{1.25 \times 1.5}{(0.69)^{2/3}} = 2.40$

$W = 1789$ KIPS

$T = 0.69$ SEC.

$F_T = 0.07\,TV \quad = 0^*$

$F_x = (V - F_T)\frac{wh}{\Sigma wh} = 1.0\,V\frac{wh}{\Sigma wh}$

$V = \frac{ZIC}{R_w}W = 0.080\,W$

$= 143$ KIPS

$^*F_T = 0$ WHEN $T \leq 0.7$ SEC.

LEVEL (1)	h FT (2)	Δh FT (3)	w KIPS (4)	Σw (5)	(2)×4 wh (6)	$\frac{wh}{\Sigma wh}$ (7)	F (8)	Σ(7) V KIPS (9)	(3)×(9) ΔOTM K-FT (10)	Σ(10) OTM K-FT (11)	(8)÷(4) $\frac{F}{w}$ (12)	(9)÷(5) $\frac{V}{\Sigma w}$ (13)
						$F_T = 0$						
R	34	12	375	375	12,750	0.35	50.0	50	600		0.133	0.133
3	22	11	707	1082	15,554	0.43	61.5	111.5	1226	600	0.087	0.103
2	11	11	707	1789	7,777	0.22	31.5	143.0	1573	1826	0.045	0.080
										3399		**
Σ			1789		36,081	1.00	143.0					

** ALL < 0.14, ∴ USE 0.14 FOR DIAPHRAGMS. (SEAOC 1 H2j)

Figure D-3. Continued.

DISTRIBUTION OF FORCES TO FRAMES

Since the roof diaphragm is relatively flexible, the roof forces are distributed by tributary area.

The 2nd and 3rd floor diaphragms distribute the floor forces to the frames according to their relative rigidities.

The transverse frames on lines 1, 4 and 7 are alike, and for preliminary design we may take their rigidity proportional to

see page 1?

$$K_1 = \frac{1/3(BASE\ SHEAR)}{DRIFT} = \frac{1/3(143)}{0.0025(34')} = 561\ k/ft$$

see page 9

The longitudinal frames on lines A and C have a rigidity based on preliminary trials:

see page 2?

$$K_A = \frac{1/2(BASE\ SHEAR)}{DRIFT} = \frac{1/2(250)}{0.30''/12} = 5000\ k/ft$$

prelim calcs (not shown)

Use Rel. $K_1 = 1$ and Rel. $K_A = \frac{5000}{561} = 8.91$, *say* 9

Because of symmetry there is no "calculated" torsion. The "accidental" torsion is the story force, F, times the nominal eccentricity of 5% of the max. building dimension: perpendicular to the direction of force under consideration:

For forces in the transverse direction.

$$M_t = F_{transv.} \times 0.05 \times 192' = 9.6F_{transv.}$$

$$\text{TORSIONAL SHEAR} = \frac{Kd}{\Sigma Kd^2}\ 9.6F_{transv.}$$

For forces in the longitudinal direction.

$$M_t = F_{long} \times 0.05 \times 48 = 2.4F_{long.}$$

$$\text{TORSIONAL SHEAR} = \frac{Kd}{\Sigma Kd^2}\ 2.4F_{long.}$$

$$= \frac{216}{28{,}800} \times 2.4\ F_{long} = 0.02\ F_{long}$$

Figure D-3. Continued.

DISTRIBUTION OF FORCES – CONT.

FRAME	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR	DESIGN SHEAR
1	1	96	96	9216	.33 F_T	$\pm$ .03 F_T	.36 F_T
4	1	0	0		.33 F_T	0	.33 F_T
7	1	96	96	9216	.33 F_T	$\mp$ .03 F_T	.36 F_T
			192				
A	9	24	216	5184	.50 F_L	$\pm$ 0.02 F_L	0.52 F_L*
C	9	24	216	5184	.50 F_L	$\mp$ 0.02 F_L	0.52 F_L
			432				
				Σ=28,800			

*THESE WILL BE USED FOR DESIGN OF THE LONGITUDINAL FRAMES. SEE P. 28.

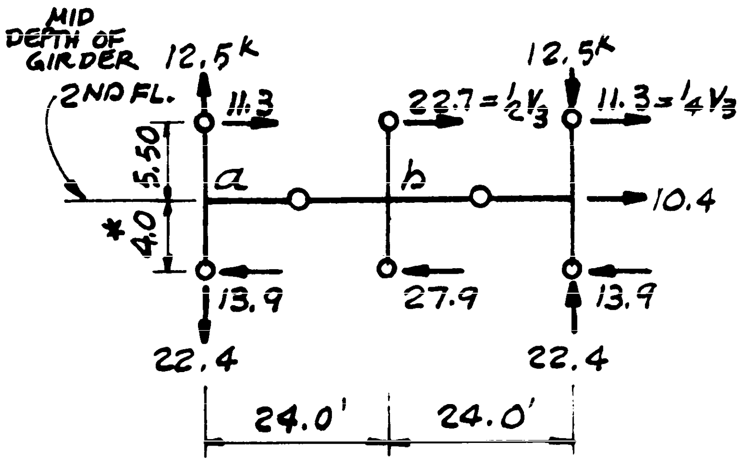
DISTRIBUTION TO TRANSVERSE FRAMES

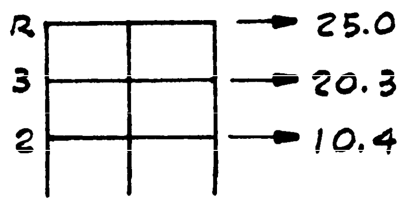
FRAME	1	4	7
ROOF (BY TIBUTARY AREA)			
50.0	×.25 = 12.5	×.50 = 25.0	×.25 = 12.5
3RD. (BY REL. RIGIDITY)			
61.5	×.36 = 22.1	×.33 = 20.3	×.36 = 22.1
2ND.			
31.5	×.36 = 11.3	×.33 = 10.4	×.36 = 11.3
143.0^K	45.9^K	55.7^k	45.9^k

Figure D-3. Continued.

PRELIMINARY DESIGN

MEMBER FORCES BY PORTAL METHOD - FRAME 4





FRAME FORCES

EXTERIOR COLUMN, (MOM. AT ℄ GIRD.)

ABOVE a, $M = 11.3^k \times 5.50' = 62.2$

BELOW a, $'' = 13.9^k \times 4.0' = 55.6$

$117.8^{k'}$

INTER. COLUMN (MOM. AT ℄ GIRD)

ABOVE b, $M = 22.7 \times 5.50 = 124.9$

BELOW b, $'' = 27.9 \times 4.0 = 111.6$

236.5^k

GIRDER (MOM. AT ℄ COL.)

$M_a = 117.8 \quad M_b = \dfrac{236.5}{2} = 118.3$

$$V = \frac{117.8 + 118.3}{24'} = 9.84^k$$

* ESTIMATED LOCATION OF INFLECTION CONSIDERING FIXITY OF BASE.

AT UPPER POINT OF INFLECTION, **

$M = (25.0^k \times 18.6'^{**}) + (20.3^k \times 6.7'^{**})$

$= 601^{k'}$

$$\text{AXIAL} = \frac{601^{k'}}{48} = 12.5^k$$

AT LOWER P. I.

$M = 601 + (45.3 \times 9.5) + (10.4 \times 4.0)$

$= 1073^{k'}$

$$\text{AXIAL} = \frac{1073}{48} = 22.4^k$$

**18.6' IS APPROXIMATE DISTANCE FROM ROOF TO POINT OF INFLECTION. 6.6' IS APPROXIMATE DISTANCE FROM TOP OF 3RD FLOOR SLAB TO POINT OF INFLECTION.

Figure D–3. Continued.

PRELIMINARY DESIGN - CONT. FRAME 4

INTERIOR COLUMN

a. VERTICAL LOAD ON CENTER FRAME

ROOF DL: $0.0257^{KSF} \times 24' \times 32' = 19.8$

3RD FLR. $DL+LL = (0.0745 + 0.021) \times 24 \times 32' = 73.3$

2ND FLR. RED. LL ✓ $= 73.3$

$W = 166.4^K$

BY SYMMETRY, $M = 0$

b. SEISMIC LOAD, FROM P. 13

$P = 0$

$M = 27.9^K \times (4.00 - 0.75) = 90.7^{K\text{-}ft}$ AT FACE OF GIRDER

c. VERTICAL + SEISMIC

$P = 166 + 0 = 166^K$

$M = 0 + 90.7 = 90.7^K$

USE AISC 9TH EDITION, P. 3-9

TRY W14 × 61, P. 3-24 $B_x = 0.194$

$$P_{EQUIV.} = \frac{166^K + 0.194(90.7 \times 12''/') = 284^K}{1.33}$$

FOR $h' = 9.5'$ $\frac{W14 \times 61}{I = 640}$ ALLOWS 334^K

EXTERIOR COLUMN

VERTICAL + SEISMIC

$$\left.\begin{array}{l} P \simeq 166/2 + 22.4 = 105^K \\ M \simeq 50^{K\text{-}'}(est) + 90.7/2 = 95^{K\text{-}'} \end{array}\right\} P_{EQUIV.} = 246$$

$\frac{W14 \times 48}{I = 485}$ ALLOWS 246^K

USE 14" COLUMNS FOR CONTROL OF DEFLECTIONS AND USE THE SAME SECTION FULL HEIGHT.

Figure D-3. Continued.

PRELIM. DESIGN - CONT. FRAME 4

GIRDER– 2ND FLOOR

VERTICAL LOAD AT CENTER COLUMN

$R = 0.08 \times 32 \times 24 = 61.4\%$ OR $23.1(1+74.6/50) = 57.5\%$

$RED.LL = 0.425 \times 50 = 21\ PSF \qquad (1 - 0.575 = 0.425)$

$w_{D+L} = (0.0746 + 0.021) \times 32' = 2.38 + 0.67 = 3.05^{K}/'$

$W_{D+L} = 3.05 \times 24' = 73.2^{K}$

$M \approx \frac{WL}{12} = \frac{73.2 \times 24}{12} = 146.4$

SEISMIC $M = \underline{118.3}$

VERT. + SEISMIC $M = 265$

USE AISC BEAM CHART, p. 2-166, 9TH ED., WITH $M = \frac{265}{1.33} = 199^{K'}$, AND UNBRACED LENGTH OF 6' FOR NEG. BENDING

$\frac{W18 \times 60}{I = 986}$ ALLOW $216^{K'}$

Figure D-3. Continued.

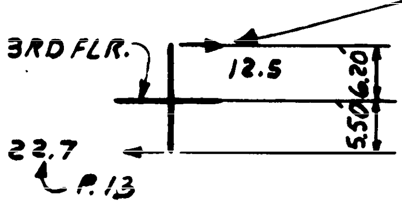
PRELIM. DESIGN - CONT. FRAME 4

GIRDER - 3RD FLOOR

VERTICAL LOAD - SAME AS 2ND, $M = 146^{K'}$

SEISMIC

$V_R = 25.0$ — 6.25^K TO EXT. COL.; 12.5^K TO INT. COL.



$\text{℄ COL. } M = (12.5^K \times 6.2') + (22.7 \times 5.5)$
$= 202^{K'}$

$\text{GIRDER } M = \dfrac{202}{2} = 101^{K'}$

VERT. + SEISMIC = $M = 146 + 101 = 247 \div 1.33 = 186^K$

W18x55 ALLOW 197

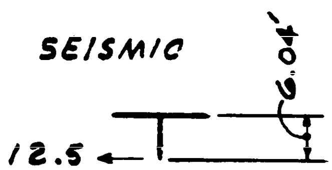
GIRDER - ROOF

VERTICAL LOAD

ROOF DL+LL = $32'(0.0257 + 0.020) = 0.82 + 0.64 = 1.46^{K/'}$

$W_D = 0.82 \times 24' = 19.7^K$ $M = \dfrac{19.7 \times 24'}{12} = 39.4^{K'}$

SEISMIC



$\text{GIRDER } M = \dfrac{12.5 \times 6.04}{2} = 37.8$

VERT. + SEISMIC = $M = 39.4 + 37.8 = 77.2 \div 1.33 = 58.0^{K'}$

W14x22 ALLOW 58 OK FOR STRESS

USE W14x30 ALLOW 83 USE WIDER FLANGE FOR BETTER DETAILS

Figure D-3. Continued.

CRITERIA FOR FINAL DESIGN - FRAME 4

1. BUILDING PERIOD

   a. For calculating frame design forces, use the whole building period estimated as T = 0.69 sec. (p. 9)

   b. For calculating drift, use the bare frame period. This will be obtained from the computer analysis or from the use of Method B. Method B will give a frame period of 0.83 sec.

2. DESIGN FORCES

As indicated item 1a above, use the building period of T = 0.69 sec. and the associated base shear of 143^k and frame shear of 55.7^k (p. 12). This is the input for the computer analysis (p. 18).

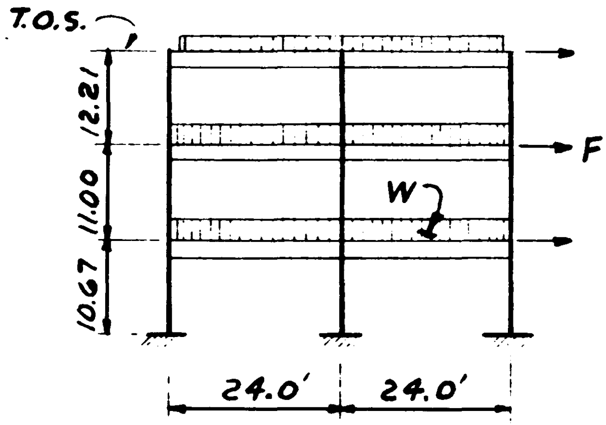
3. DEFLECTIONS

The deflections obtained from this analysis, based on whole building forces, will be modified for the calculation of drift under bare-frame forces (p.20).

Figure D-3. Continued.

FRAME ANALYSIS — FRAME 4

COMPUTER INPUT



KIPS, FEET, SECONDS

RIGID FRAME.

STEEL: E = 4,176,000 KSF

COLUMN BASES FIXED.

	EXT. COL.	INT. COL.
SIZE	W14×48	W14×61
I	0.0234	0.0309
A	0.0979	0.1243
Aw	0.0325	0.0365

LEVEL	GIRDER						TRIB W	MASS =W/g	LATERAL FORCE
	SIZE	I	A	Aw	W_{DL}	W_{LL}			
R	W14×30	0.0140	0.0613	0.0260	0.82	0.64	187^{K}	5.81	25.0
3	W18×55	0.0430	0.1125	0.0491	2.38	0.67	233	7.24	20.3
2	W18×60	0.0476	0.1229	0.0527	2.38	0.67	233	7.24	10.4
DATA FROM PAGE					14 & 15		5 & 6		11

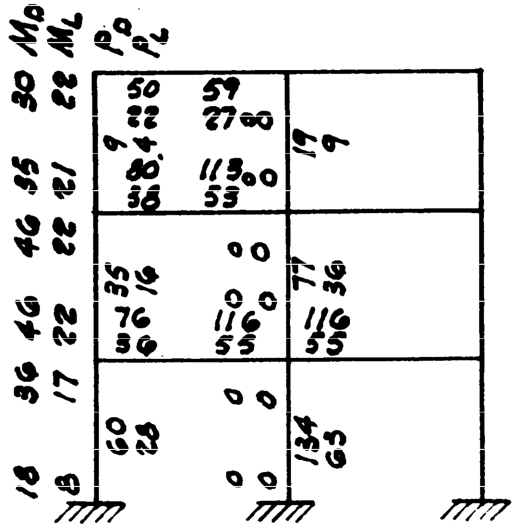
TRIB. ROOF WT. IS ½ OF TOTAL SINCE DIAPH. IS FLEXIBLE

TRIB. FLOOR WT. IS 0.33×TOTAL ACCORDING TO REL. RIG. OF FRAMES. $g = 32.2\ FT./SEC.^2$

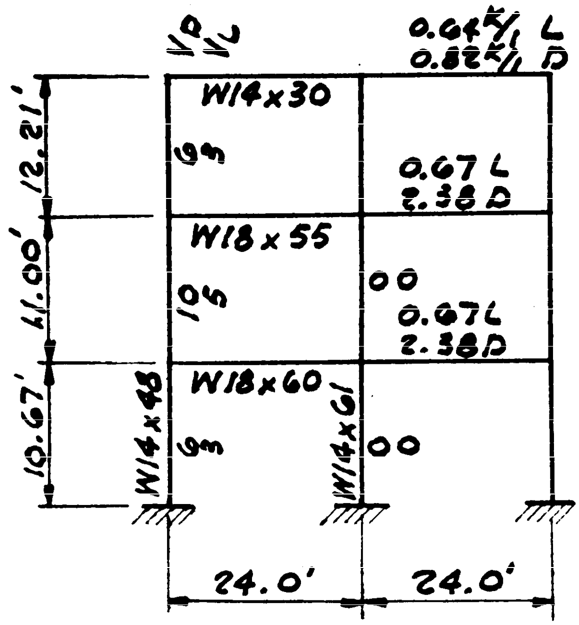
Figure D-3. Continued.

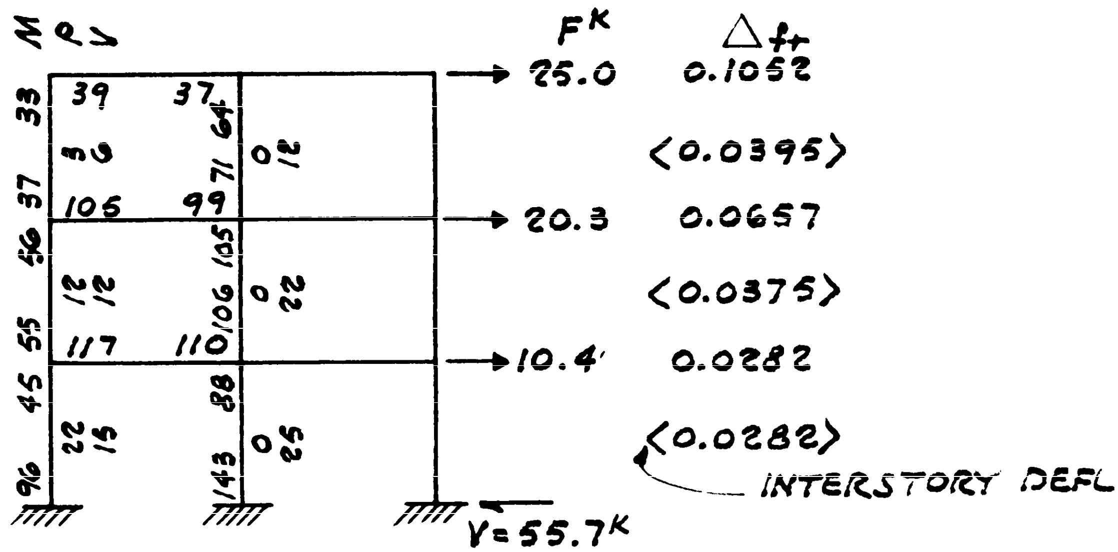
FRAME ANALYSIS - CONT. FRAME 4

COMPUTER OUTPUT



LIVE LOAD
DEAD LOAD





SEISMIC

$$\text{RIGIDITY:} \quad K = \frac{V}{\Delta_R} = \frac{55.7}{0.1052} = 529 \, ^{K}/_{ft}$$

Figure D-3. Continued.

FINAL DESIGN - CONT. FRAME 4 - DRIFT

Before proceeding with the detailed final design, we will check the drift.

From the computer analysis, the maximum drift is 0.0395 ft. (p. 19). This is based on the whole building period T = 0.69 sec. and the frame shear of 143^k (p. 10).

For the drift check, use the deflection associated with the bare frame period T = 0.93 sec. (p. 9).

$C = (1.25 \times 1.5)/(0.93)^{2/3} = 2.12$
$CS = ZIC/R_w = (0.4 \times 1.0 \times 2.12)/12 = 0.071$

Base Shear = $0.071 \times 1789 = 127^k$

Multiply deflections from frame analysis by the ratio $127/143 = 0.89$.

Maximum drift = $0.89 \times 0.0395' = 0.035'$

Allowed drift = $(0.03/12) \times 12' = 0.030'$ $(T = 0.93 > 0.7)$

The frame drift is 17% over the limit. It should be stiffened. There are three options:

(1) increase the member sizes
(2) use more than three frames
(3) make the roof diaphragm rigid

We will change the interior column to W14x68 and proceed with the detailed design. Further changes may be necessary, and we will make a final drift check after other checks are completed.

Note that the assumed condition of fixed columns bases is difficult to achieve. If the bases are not fully fixed, the frame will be more flexible than assumed, and the frames would have to be further stiffened.

Figure D-3. Continued.

FINAL DESIGN - CONT. FRAME 4
MEMBER STRESSES

(1) SAMPLE CALCULATION FOR 2ND FLR. GIRDER

	M_D	M_L	M_E	M_{D+L}	$\frac{M_{D+L+E}}{1.33}$
AT EXT. COL.	76	36	117	112	172
AT INT. COL.	116	55	110	171	211

DES. M = 211 K' UNBRACED LENGTH = 6'

W18 x 60 ALLOW M = 216 K' AISC 9TH ED., P. 2-166

PROVIDE GIRDER BRACING PER SEAOC 4F8:

MAX UNBRACED LENGTH = $96 r_y$

$= \frac{96(1.69)}{12} = 13.5 \text{ FT.} > 6 \text{ FT.}$

BRACING IS ADEQUATE

(2) SAMPLE CALCULATION FOR COLUMN,
SEE NEXT PAGE.

Figure D-3. Continued.

FINAL DESIGN - CONT. - FRAME 4

MEMBER STRESSES - CONT.

2.) SAMPLE CALCULATION FOR COLUMNS FIRST STORY AT BASE

$K_Y = 1.0$ (COLUMNS ARE BRACED BY BEAMS)

USE $K_x = 1.0$ PER SEAOC 4D3

COL. SHOULD ALSO BE CHECKED FOR ~~SEAOC 4D3~~. THAT PROVISION WILL NOT GOVERN IN THIS EXAMPLE, BUT WILL BE ILLUSTRATED IN THE BRACED FRAME EXAMPLE

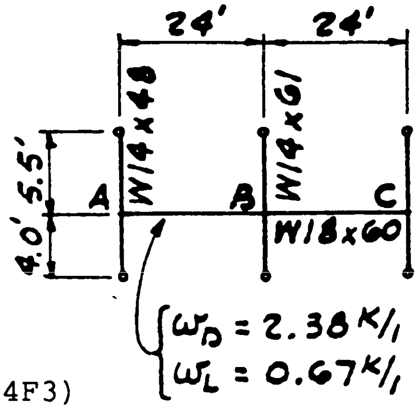
	EXT.		INT.	
	W14x48		W14x68	
	A = 14.1, S = 70.2		A = 20.0, S = 103	
	P	M	P	M
D	60	18	134	0
L	28	8	63	0
D + L	88	26	197	0
E	22	96	0	143
$\frac{D+L+E}{1.33}$	83	92	148	108
f_a	5.89 KSI		7.40 KSI	
f_b	15.7 KSI		12.6 KSI	
K_x	1.0		1.0	
ℓ	119"		119"	
r_x	5.85"		6.01	
$K\ell/r_x$	20.3		19.8	
F_a	28.3		28.3	
K_y	1.0		1.0	
ℓ	119		119	
r_y	1.91		2.46	
$K\ell/r_y$	62.3		48.6	
F_a	22.3		24.6	
F'_{ex}	362		381	
F_{bx}	33.0		33.0	
f_a/F_a	0.264		0.301	
f_b/F_b	0.476		0.382	
$\left\{ C_{mx} / \left(1 - \frac{f_a}{F'_{ex}}\right) \right\} \times (f_b/F_b)$	0.411		0.331	
	0.675		0.632	
$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} =$	0.672		0.629	

ALL SUMMATIONS < 1.0 OK

Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder data W18x60 Gr. 36



Girder strength in flexure

This is $M_s = ZF_y$ (SEAOC 4C2).
This is the same as plastic moment M_p given in the in the AISC Plastic Design Selection Table:

$M_p = 369$ k'

Girder stability W18x60

$b/2t_f = 5.44 < 52/\sqrt{36} = 8.66$ (SEAOC 4F3)
$d/t_w = 43.5 \leq 412/\sqrt{36} = 68.7$ (AISC Ch.N)

Requirements for girder-column connection

SEAOC 4F1a requires development of the lesser of the strength of the girder in flexure (M_s) and the moment associated with the panel zone shear strength. The requirement of this manual is to develop the strength of the girder in flexure. This is accomplished according to SEAOC 4F1b.

Girder flange connection to column

Provide full butt-weld connections of the flanges to the columns. (SEAOC 4F1b(1))

Girder web connection to column

Design shall be based on the gravity loads plus the seismic load associated with compliance with SEAOC 4F1a. (SEAOC 4F1b(2))

Vert. shear: V_G = (2.38+0.67)24'/2 + (171-112)/24' = 39.1
Seismic shear = $2M_s/L$ = 2x369k'/22.83' = $\underline{32.3}$
Design V = 71.4k

The method of developing the flexural capacity of the web depends on the relative size of the flanges, i.e., whether the flange strength ($bt_f(d-t_f) \times F_y$) is greater or less than 70% of the total strength ($0.7Z_xF_y$)

Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder web connection to column - cont.

$b = 7.555"$; $t_f = 0.695"$; $d = 18.24"$; $Z_x = 123 in^3$

For 70% of entire section:
$0.7\ Z_x = 0.7 \times 123 = 86 in^3$

For the flanges alone:
$bt_f(d-t_f) = (7.555")(0.695")(18.24-0.695") = 92.12\ in^3$

As 92.12 > 86, the connection can be made by welding and/or high-strength bolting according to (SEAONC 4F1b(2)(a)

If the chosen girder had had flange strength less than 70% of the total strength, the conventional web connection would have had to be supplemented with additional welding (to the web at the top and bottom of the shear tab on the column) according to SEAOC 4F1b(2) (b).

Girder web connection design

Assume 1" A325-SC bolts are selected,

Use 4 bolts: Bolt Strength, $V = 4 \times 1.7 \times 13.7 = 93.2k > 71.4$

Shear plate:

$$Z = 0.3125\ (12.5)^2/4 = 12.2\ in^3$$
$$f = 71.4k \times 2"/\ 12.2 = 11.7\ ksi < 36$$
$$v = 71.4/(0.3125 \times 12.5) = 18.3\ ksi < 0.55 \times 36 = 19.8$$

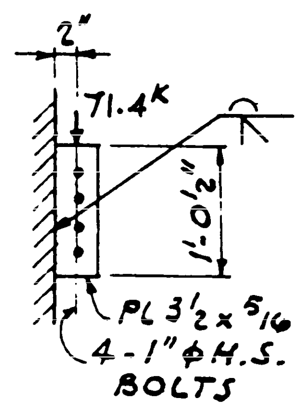


Figure D-3. Continued.

GIRDER-COLUMN CONNECTION

Girder-column panel zone -- Sample calc. for Joint B

Panel zone strength (SEAOC 4F2)

This is the moment corresponding to the development of the panel zone shear strength. This moment shall be the moment due to gravity loads plus 1.85 times seismic loads but need not exceed 0.8 times the summation of M_s at the beams.

Calculate moment arm between girder flanges:
$d-t_f$ = 18.24-0.69 =17.55"

Panel zone shear:

Gravity + 1.85 x seismic

One side of column, with D + L:
M_{D+L} = 171; 1.85 M_E = 1.85 x 110 = 204

Other side, with D:
M_D = -116; 1.85 M_E = 204

Sum of girder moments = 171 + 204 -116 + 204 = 463

0.8 M_s

Sum of girder moments = 2 x 0.8 x 369 = 590 k'

Use panel zone strength associated with 463 k'.

(See SEAOC 4F10 for Drift calculations.)

Shear

One side, top flange force = (171+204) (12)/17.55"
= 256 k
Other side, top flange force = (-116+204)(12)/17.55"
= 60 k
Column shear above joint:

sum girder moments = 463
column height = 4.0 + 5.5 = 9.5'
column shear = 463 / 9.5 = 49 k

Shear = 256 + 60 − 49 = 267 k

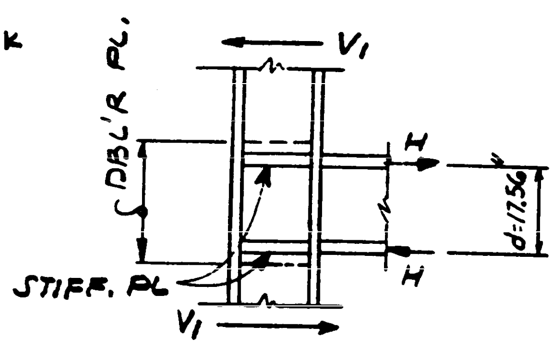


Figure D-3. Continued.

GIRDER-COLUMN CONNECTION -- Joint B -- cont.

Panel zone thickness

For thickness, t', the panel zone can develop

$$V_j = 0.55\ F_y dt' \quad \text{(modified SEAOC formula 4-1)}$$

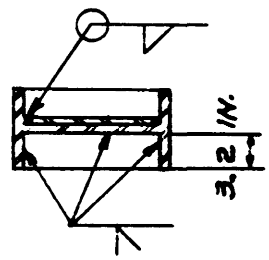
where t' is the effective thickness which consists of the combined thickness, t, of the web and doubler plate modified by the contribution of the the columns flanges (see below)

For W14 x 68 Grade 50 column

$$V_j = 0.55\ (50)(14.04)t' = 386t' \text{ kips}$$

For joint shear of 235 k,

req'd t' = **267 / 386 = 0.69"**



$t' = t\ [1 + 3b_c t_{cf}^2/d_b d_c t]$
$\quad = t\ [1 + 3(10.03)(0.72)^2/(18.24)(14.04)t]$
$\quad = t\ [1 + 0.061/t] = t + 0.061$

0.69" = t + 0.06
t = **0.63"**

For column web thickness of 7/16" req'd thickness of doubler plate = **0.63"** - 7/16 = **0.19** "

Use a 1/4" minimum Grade 50 doubler plate, or consider using a column with a thicker web.

Check SEAOC 4F2b:

$d_z = d - 2t_f = 18.24 - 2 \times 0.695 = 18.10$ (from girder)
$w_z = \quad 14.04 - 2 \times 0.720 = 12.50$ (from column)

$(d_z + w_z) / 90 = 0.34 < $ **0.63"** OK

Continuity plates

Provide continuity plates per SEAOC 4F2c

Figure D-3. Continued.

FINAL DESIGN - CONT. - FRAME 4 - STRONG COLUMN/WEAK BEAM

$$\Sigma Z_c(F_{yc}-f_a)/\Sigma Z_b f_{yb} > 1 \quad \text{(SEAOC 4F5)}$$

Interior column, second floor (Joint "b", p. 13)
W14x68: A = 20.0, Z_c = 115
P_{D+L+E} = 166
f_a = 166/20.0 = 8.30
The girder is W18x60, with Z_b = 123

2(115)(36-8.30)/2(123)(36) = 0.72 which is < 1

Change to Grade 50 steel (make both columns Gr. 50).

2(115)(50-8.3)/2(123)(36) = 1.08 which is > 1

Exterior column, second floor (Joint "a")
W14x48: A = 14.1, Z_c = 78.4
P_{D+L+E} = 105 (p.14)
f_a = 105/14.1 = 7.45 ksi
The girder is W18x60, with Z_b = 123

With one column above the joint and one below,

2(78.4)(50-7.45)/123(36) = 1.50 which is > 1

For both columns f_a < 0.4(36) = 14.41; therefore, the columns are not required to be stronger than the beams; however, strong column/weak beam design is strongly recommended.

Figure D-3. Continued.

LONGITUDINAL DIRECTION: STEEL BRACED FRAMES

LATERAL FORCES

BUILDING: A-3

DIRECTION: LONGITUDINAL

$Z = 0.40 \quad I = 1.0 \quad R_w = 8$

$C = \frac{1.25(1.2)}{(0.28)^{2/3}} = \cancel{3.50} \rightarrow$ USE 2.75

$W = 1789$ KIPS

$T = 0.02(34)^{3/4}$

$= 0.28$ SEC.

$F_t = 0.07TV = 0 \; V^*$

$F_x = (V - F_t)\frac{Wh}{\Sigma Wh} = 1.0 \; V \frac{Wh}{\Sigma Wh}$

$V = \frac{ZIC}{R_w} W = \frac{(0.40)(1)(2.75)}{8} W = 0.137W,$ SAY $0.14W = 250K$

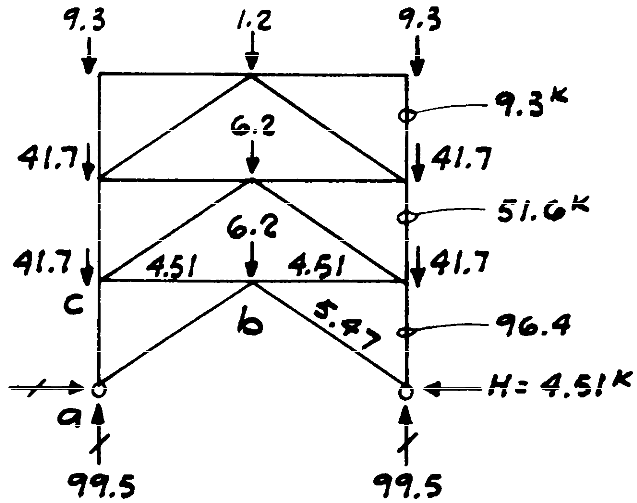
$* \; F_t = 0$ WHEN $T \leq 0.7$ SEC.

LEVEL (1)	h FT. (2)	Δh FT. (3)	W KIPS (4)	ΣW (5)	(2)×(4) Wh (6)	$\frac{Wh}{\Sigma Wh}$ (7)	F KIPS (8)	Σ(7) V KIPS (9)	(3)×(9) ΔOTM K-FT. (10)	Σ(10) OTM K-FT. (11)	(8)÷(4) $\frac{F}{W}$ (12)	(9)÷(5) $\frac{V}{\Sigma W}$ (13)
						$F_t = 0$						
R	34	12	375	375	12,750	0.35	88	88	1056		.235	.235
3	22	11	707	1082	15,554	0.43	107	195	2145	1056	.151	.180
2	11	11	707	1789	7,777	0.22	55	250	2750	3201	.078	.140
										5951		
		Σ	1789		36,081	1.00	250					

Figure D-3. Continued.

BRACED FRAME - CONT.

VERTICAL FORCES ALL EXCEPT ROOF LL



TRIB. AREA AT COLUMN

TRANSV. GIRDER : $9' \times 32' = 288$

EDGE BEAM $\left(\frac{32}{2} + \frac{16}{2}\right) \times 3' = 72$ } 360 SF

TRIB. AREA AT BRACE $16' \times 3' =$ 48 SF

LOADS AT COLUMNS (REDUCED LL) AT BRACES

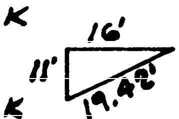
ROOF $(25.7\ PSF + 0) \times 360\ SF = 9.3^K$ $\times 48\ SF = 1.2^K$

FLOOR $(74.5 + 36) \times 360\ SF = 39.8$ } 41.7^K $\times 48\ SF = 5.3^K$ } 6.2^K

WALL $(5.3\ PSF \times 11') \times 32' = 1.9$ $\times 16' = 0.9$

BRACE FORCE = $V = \frac{6.2}{2} = 3.1$ $H = \frac{16'}{11'}(3.1) = 4.51^K$

AXIAL FORCE = $\frac{19.4}{11}(3.1) = 5.47^K$



FULL LOADS =

ROOF $P_D = 9.3^K$ $P_L = 7.2^K$

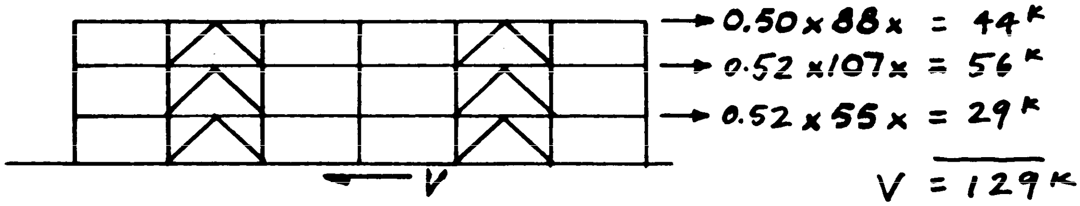
FLOOR $P_D = 28.7^K$ $P_L = 18.0^K$

Figure D-3. Continued.

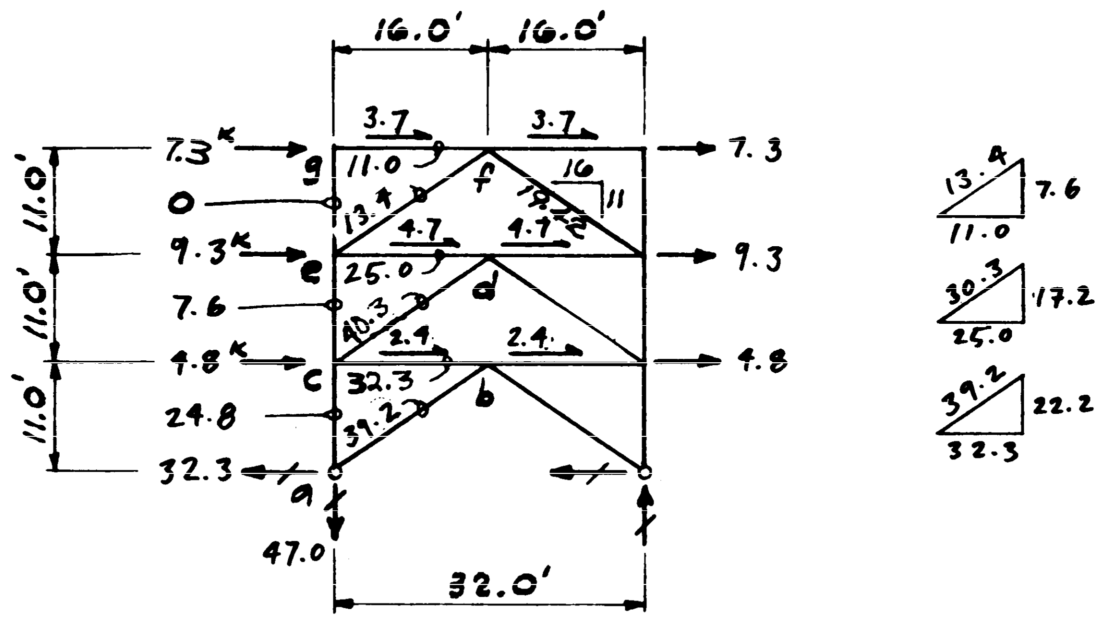
BRACED FRAME - CONT.

SEISMIC FORCES

FRAMES SHARE ROOF LOAD ACCORDING TO TRIBUTARY AREA. AT FLOORS THE FRAMES TAKE 0.52 x STORY FORCE (P. 12).



APPLY 1/2 FORCE TO EACH BRACED BAY



TYPICAL BRACED BAY
SEISMIC FORCES

DESIGN CHEVRON BRACE MEMBERS FOR 1.5 x SEISMIC FORCES.

Figure D-3. Continued.

BRACED FRAMES CONT

MEMBER SIZES

BRACE (FIRST STORY)

CHEVRON BRACE
SEAOC 4G

$P_E = \pm 39.2$ DESIGN F = $1.5 \times 39.2 = 58.8$

$P_D = -5.5$ (p. 29)

BRACE FORCE, $P_B = -5.5 \pm 58.8 \begin{cases} +53.3 \\ -64.3 \end{cases}$ ←

$$\frac{L}{r} \le \frac{720}{\sqrt{F_y}} = \frac{720}{\sqrt{46}} = 106 \quad \text{(SEAOC 4G1a)}$$

TS 6×6×3/8 $r = 2.27$

$$\frac{L}{r} = \frac{228}{2.27} = 100 < 106 \quad \text{OK}$$

$P_a = 115^k$ AISC ASD p. 3-42

$$\beta = 1 \Big/ \left[1 + \frac{Kl/r}{2C_c}\right] = 1 \Big/ \left[1 + \frac{100}{2(112)}\right] = 0.69$$

(SEAOC 4G1b)

$$(P_a)_{Brace} = 0.69 \times 115 = 79.5 > 64.3$$

CHECK $\frac{b}{t}$ OF TUBE WALL

$$\text{ASD CHAPT. N: } \frac{190}{\sqrt{F_y}} = 28$$

$$b = 6 - 2\left(\frac{3}{8}\right) = 5\,1/4$$

$$b/t = \frac{5\,1/4}{3/8} = 14 < 28$$

Figure D-3. Continued.

BRACED FRAMES CONT.

COLUMN (SEAOC 4D1)

TRY W14 x 43 FOR COMPATIBILITY WITH OTHER COLUMNS

$$P_c = 1.0\,P_{DL} + 0.7\,P_{LL} + 3\left(\frac{R_w}{8}\right)P_E$$

$$= 1.0\,(66.7) + 0.7\,(43.2) + 3\left(\frac{8}{8}\right)(39.2) = 215^k$$

$L = 10;\ F_y = 36$

ASD p. 3-24

$$P_{allow} = 1.7\,(215) = 366^k > 215 \text{ OK}$$

NOTE:

THIS IS A CHECK FOR COLUMN STABILITY. NO BENDING MOMENTS ARE INCLUDED IN THIS CHECK. FOR SITUATIONS WITH LARGE COLUMN BENDING, A STANDARD COLUMN CHECK INCLUDING BENDING AND CODE LEVEL AXIAL FORCE MAY GOVERN OVER THIS.

COLUMN SPLICES

DESIGN FOR THE FOLLOWING TENSION:

$$P_t = 0.85\,P_{DL+LL} - 3\left(\frac{R_w}{8}\right)P_E$$

$$= 0.85(66.7) - 3\left(\frac{8}{8}\right)(39.2) = -60.9K$$

Figure D-3. Continued.

BRACED FRAME · CONT.

MEMBER DESIGN - CONT.

EDGE BEAMS - FULL LATERAL BRACING BY STEEL DECK. ASSUME WALL LATERAL LOADS TRANSMITTED DIRECTLY TO THE STEEL DECK: NO TORSION OR HORIZONTAL LOAD.

BEAM WITHOUT DIAGONAL BRACE $L = 32'$

BEAM MUST STILL CARRY VERTICAL LOAD EVEN IF THE BRACES SHOULD FAIL IN A LARGE EARTHQUAKE. SEAOC 4G 3a (3)

FLOOR

$$\left.\begin{array}{l} W = (0.745 + .050 \text{ KSF}) \times 3' \times 32' = 12.0 \\ \text{WINDOW } 0.0053 \text{ KSF} \times 11' \times 32' = 1.9 \end{array}\right\} 13.9^K$$

W18x40

BEAM WITH BRACE $L = 16'$

$$\left.\begin{array}{l} \text{ROOF USE W14x30} \\ \text{FLOOR USE W18x40} \end{array}\right\} \text{FOR CONSISTENT DETAILS WITH UNBRACED BAYS}$$

$$\text{FLOOR} = 3' \times (74.5 + 50 \text{ PSF}) = 373.5 \ \#/'$$

$$\text{WALL} = 5.3 \text{ PSF} \times 11' = 58.3 \ \#/'$$

$$w = .432 \ K/'$$

$$\text{VERT. } W = .432^{K/'} \times 16' = 6.90^K$$

$$M = \frac{6.90 \times 16'}{8} = 13.8^{K'}$$

$$f = \frac{13.8 \times 12}{68.4} = 2.42 \text{ KSI LOW}$$

$$\text{AXIAL } P = +4.51 \pm 32.3 = \begin{cases} +36.8 \\ -27.7 \end{cases}$$

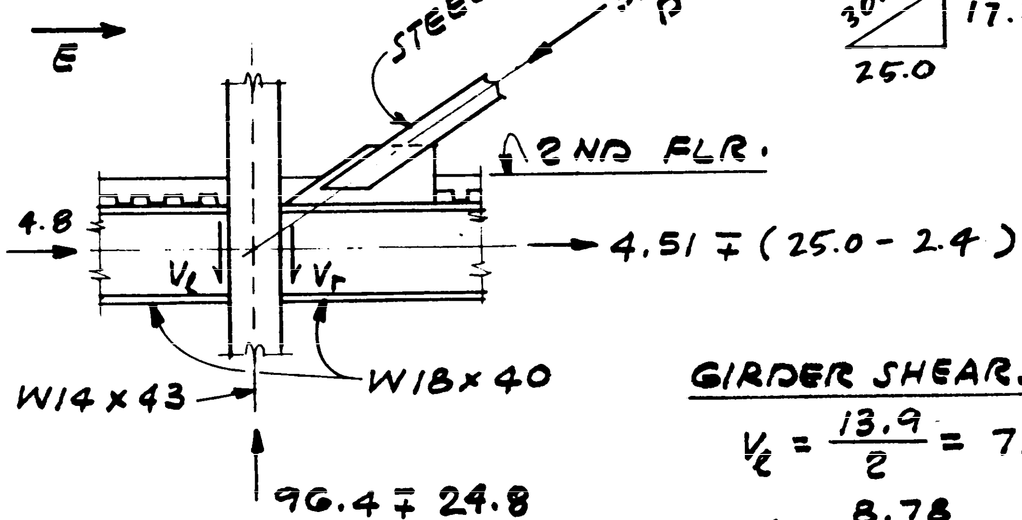
$$f_a = \frac{36.8}{11.8} = 3.12 \text{ KSI LOW}$$

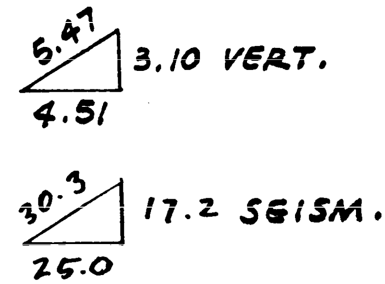
Figure D-3. Continued.

BRACED FRAME - CONT.

CONNECTIONS

JOINT "C"



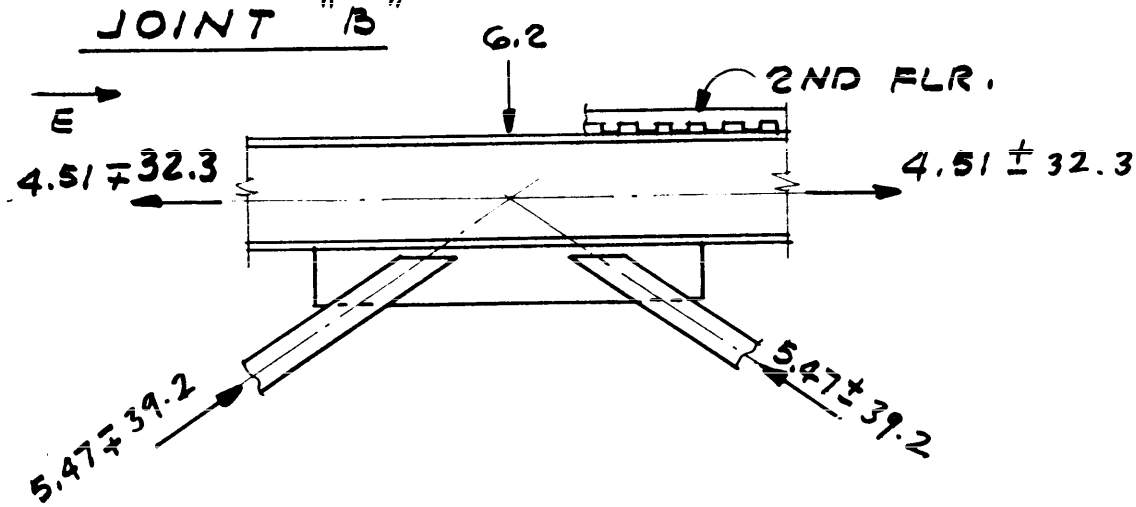


GIRDER SHEARS (VERT. LOAD)

$$V_l = \frac{13.9}{2} = 7.0^k$$

$$V_r = \frac{8.78}{2} = 4.4^k$$

JOINT "B"



NOTE: CONNECTIONS SHALL BE DESIGNED FOR THE ABOVE FORCES WITHOUT THE USUAL ONE-THIRD INCREASE, OR SHALL BE DESIGNED TO DEVELOP THE FULL CAPACITY OF THE MEMBERS

Figure D-3. Continued.

BRACED FRAME - CONT.

CONNECTIONS - CONT.

connections shall be designed for the least of:

(1) TENSILE STRENGTH OF THE BRACING

(2) $3\left(\frac{R_W}{8}\right)$ TIMES THE FORCE IN THE BRACE DUE TO PRESCRIBED SEISMIC FORCES.

(3) THE MAX. FORCE WHICH CAN BE TRANSFERRED TO THE BRACE BY THE SYSTEM.

BOTTOM LEVEL

TENSILE STRENGTH OF BRACE

$P_{ST} = F_y A = 46\,(8.08) = 372^k$

$3/8\ R_W \times$ SEISMIC $\longrightarrow \frac{3}{8}(8)(39.2) = \underline{117.6}^k \longleftarrow$ GOVERNS

$117.6 + 5.47 = \underline{\underline{123.1}}^k$

(p. 29)

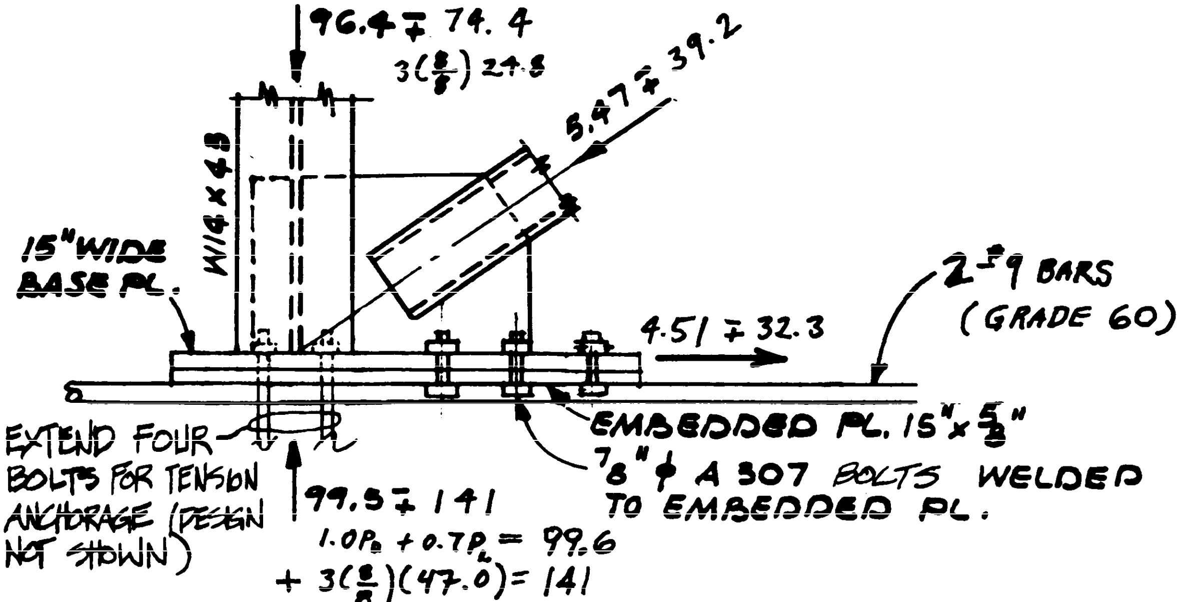
ANCHORAGE TO FOUNDATION

THIS IS A BRACED FRAME CONNECTION. AS SUCH, IT MUST MEET SEAOC 4G2, OR, IF CONSIDERED AS A COLUMN, SEAOC 4D2. THE RESULT IS THE SAME. SEE NEXT PAGE.

Figure D-3. Continued.

BRACED FRAME - CONT.

JOINT "a" → E



VERT. BEARING ON CONC. $P = 99.6 + 141 = 241^k$

FOR FOUNDATION MUCH WIDER THAN THE LOADED AREA, ULTIMATE BEARING STRESS =

$\sqrt{A_2/A_1}\ \phi\,(0.85 f_c')A_1 = 2(0.70)(0.85)(3)(A_1) = 3.57A_1$

REQ'D $A = \dfrac{241}{3.57} = 67.5\ IN^2$ $\left[\left(\sqrt{A_2/A_1}\right)_{max} = 2\right]$

MIN. LENGTH $= \dfrac{67.5}{15} = 4.5"$

SHEAR BOLTS $V = 4.61 + 3(\frac{1}{8})32.3 = 101^k$

USE $10 - \frac{7}{8}"\phi$ A 307 STUDS IN S.S.

STRENGTH: $10 \times 1.7 \times 6.01 = 102^k$

REINFORCING BARS

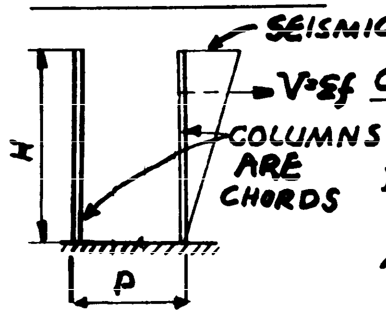
REQ'D $A = 101^k / 0.9(60) = 1.87\ IN^2$ 2-#9

LOCATE BASE PLATES IN A POCKET SO THAT THE #9'S ARE CONFINED WITHIN THE GRADE BEAM BAR CAGE. EXTEND BARS 4'-0 EA. SIDE OF POCKET. WELD TO DEVELOP THE BARS. COMPLY WITH ACI 12.14.3..

Figure D-3. Continued.

BRACED FRAME - CONT.

DEFLECTION

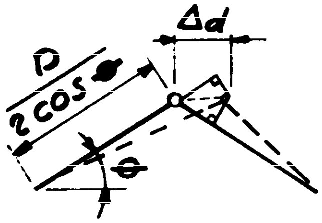


CHORDS : $\Delta_c = \dfrac{11\,VH^3}{60\,EI}$ (BEAM WITH INVERSE TRIANGULAR LOADING)

$$I = 2 \cdot A \cdot \left(\frac{D}{2}\right)^2 = \frac{AD^2}{2}$$

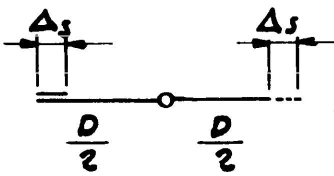
A = AREA OF ONE COLUMN

$$\Delta_c = \frac{11\,VH^3}{60E\,\frac{AD^2}{2}} = \frac{11\,VH^3}{30\,EAD^2}$$



DIAGONALS : $\Delta_d = \dfrac{P\,\frac{D}{2\cos\theta}}{EA\cos\theta} = \dfrac{f_d\,D}{2E\cos^2\theta}$

WHERE f_d IS THE STRESS IN THE DIAG.



STRUTS : $\Delta_s = \dfrac{P \times \frac{D}{2}}{EA} = \dfrac{f_s\,D}{2E}$

WHERE f_s IS THE STRESS IN THE STRUT

$$\Delta = \Delta_c + \Delta_d + \Delta_s$$

$$= \frac{11\,VH^3}{30\,EAD^2} + \sum \frac{f_d\,D}{2E\cos^2\theta} + \sum \frac{f_s\,D}{2E}$$

IN COMPUTATION OF DEFLECTIONS USE STRESSES BASED ON SEISMIC FORCES, PAGE 28.

V FROM SHEET 30 ($2 \times 32.3 = 64.6^k$) $D = 32'$

MEMBER FORCES P FROM SHEET 30 $E = 29{,}000$ KSI

A = AREA OF MEMBER $\cos\theta = \dfrac{16}{[11^2+16^2]^{1/2}} = .824$

Figure D-3. Continued.

BRACED FRAME - CONT.

DEFLECTION - CONT.

CHORDS W14 x 43 A = 12.6 IN² Δ_c

$$\Delta_c\ (ROOF) = \frac{11}{30}\frac{VH}{EA}\left(\frac{H}{D}\right)^2 = \frac{11}{30}\ \frac{64.6 \times (33 \times 12)}{29000 \times 12.6}\left(\frac{33}{32}\right)^2 = 0.027''$$

$\Delta_c\ (3^{RD}) = .55\ \Delta_{ROOF}$ (AT 2/3 H UNIFORM BEAM) $\doteq$ 0.015"

$\Delta_c\ (2^{ND}) = .17\ \Delta_{ROOF}$ (AT 1/3 H UNIFORM BEAM) $\doteq$ 0.005"

DIAGONALS 6x6x3/8 TUBE A = 8.08 IN²

	MEMBER P	$f_d = P/A$	$\frac{f_d D}{2E \cos^2\theta}$	Δ_d
ROOF				0.100"
3RD	13.4	1.66	0.0162	0.084"
2ND	30.3	3.75	0.0366	0.047"
1ST	39.2	4.85	0.0473	0

STRUTS

		P	A	f_s	$\frac{f_s D}{2E}$	Δ_s
ROOF	W14x30	11.0ᵏ	8.85 in²	1.24 ksi	0.0082	0.040"
3RD	W18x40	25.0ᵏ	11.8 in²	2.14 ksi	0.014	0.032"
2ND	W18x40	32.3ᵏ	11.8 in²	2.74 ksi	0.0181	0.018"

TOTAL DEFLECTION

	Δ_c	Δ_d	Δ_s	Δ
ROOF	0.027	0.100	0.040	0.17"
3RD	0.015	0.084	0.032	0.13"
2ND	0.005	0.047	0.018	0.07"

EST. STIFFNESS OF WALL A & C

$$K_A = \frac{2\ \text{BRACED FRAMES/WALL}}{\Delta_R} = \frac{2 \times 64.6}{0.17''/12''/ft} = 9120\ ^{k}/FT$$

Figure D-3. Continued.

FINAL PROPERTIES

TRANSVERSE PERIOD - SEAOC FORMULA 1-5
FOR STIFFNESS USE THE F, Δ VALUES FROM THE ANALYSIS FOR FRAME A (P. 19). FOR MASS USE THE STORY WEIGHTS (P. 10)

LEVEL	W	F	Δ	$W\Delta^2$	$F\Delta$
R	375^K	25.0^K	0.1052 FT.	4.15	2.63
3	707	20.3	0.0657	3.05	1.33
2	707	10.4	0.0282	0.56	0.29
				7.76	4.25

FOR THE WHOLE BUILDING (W'S ABOVE) THERE ARE THREE FRAMES; SO USE $3 \times F\Delta = 12.75$

$$T = 2\pi\sqrt{\frac{W\Delta^2}{gF\Delta}} = \sqrt{\frac{7.76 \text{ K-FT.}^2}{32.2 \text{ FT/SEC}^2 \times 12.75 \text{ K-FT}}}$$

$= 0.86$ SEC. (BARE FRAME)

STIFFNESS (SEE P. 11 & 12)

IN FINAL DESIGN, $K_1 = 529^K/\text{FT}$ (P. 19), REL. $K_1 = 1$
$K_A = 9120^K/\text{FT}$ (P. 38), REL. $K_A = \frac{9120}{529} = 17.2$

FOR FRAME A OR C,
$Kd = 17.2 \times 24 = 414$, $Kd^2 = 9930$ $\Sigma Kd^2 = 39,720$

TORSIONAL SHEAR $= \frac{414}{39,720} \times 2.4 F_L = 0.02 F_L$

DESIGN BASED ON $0.02 F_L$ IS STILL O.K. (p. 12)

Figure D-3. Continued.

LONGITUDINAL DIRECTION: ECCENTRIC BRACED FRAMES

This section of the example illustrates the use of eccentric braced frames as an alternate to the concentric braced frames used in the previous section of the example. It should be noted that EBF's would not be an economical choice for this building because the seismic demands are relatively low. The EBF scheme is not to be compared with the CBF scheme for costs for this building: the EBF scheme is presented here only to illustrate the design procedure.

AISC formulas are from Chapter N of the ASD Manual, 9th edition.

DESIGN PROCEDURE

Figure D-3. Continued.

A. LATERAL FORCES

1. Base shear

$V = (ZIC/R_w)W$

$Z = 0.4$, $I = 1.0$, $R_w = 10$, $S = 1.5$, $C_t = 0.03$, $h_n = 34$ ft.

$T = C_t(h_n)^{3/4} = 0.03 \times (34)^{3/4} = 0.422$ sec.

$T < 0.70$ sec., therefore $F_t = 0$.

$C = (1.25\ S) / T^{2/3} = (1.25 \times 1.5) / (0.422)^{2/3} = 3.33$

$C > 2.75$, therefore use $C = 2.75$

$C/R_w = 2.75/10 = 0.275 > 0.075$

$W = 1789$ kips

$V = (0.4 \times 1.0 \times 2.75 / 10)\ W = 0.11\ W = 0.11 \times 1789$
$= 197$ kips.

2. Story forces and shears

Level	w_i	h_i	$w_i h_i$	$wh/\sum wh$	F_i	V
R	375	34	12,750	.353	70	
						70
3	707	22	15,554	.431	85	
						155
2	707	11	7,777	.216	42	
						197
	1789		36,081			

3. Horizontal distribution of lateral forces

At the flexible roof diaphragm, the four braced frames each take 0.25 times the story force because of equal tributary areas; at the rigid floor diaphragms, the frames each take 0.30 times the story force because of equal stiffness and consideration of accidental torsion.

Figure D-3. Continued.

B. TYPICAL BEAM SIZING

1. Typical Roof Beam

W12x 19 $d = 12.2"$ $S = 21.3\ in^3$
$t_w = 0.235"$ $I = 130\ in^4$
$Z = 24.7\ in^3$

$l = 32$ ft.

$w = 6' \times (26\ DL + 20\ LL\ psf)$, or 0.276 k/ft.

$M = wl^2/8 = 35.3$ k-ft.

$f_b = M/S = 19.9$ ksi

$\Delta_{LL} = (20/46)5wl^4/384EI = 0.75$ in. $= 1/510$ OK

$V_p \leq 0.55F_y d t_w = 56.8$ kips ASD formula N5-1

2. Typical Floor Beam

W16x36 $d = 15.9"$ $S = 56.5\ in^3$
$t_w = .295"$ $I = 488\ in^4$
$Z = 64\ in^3$

$l = 32$ ft.

$w = 6' \times (75 + 50)$, or 0.750 k/ft.

$M = wl^2/8 = 96$ k-ft.

$f_b = M/S = 20.4$ ksi

$\Delta_{LL} = (50/125)5wl^4/384EI = .54$ in. $= 1/711$ OK

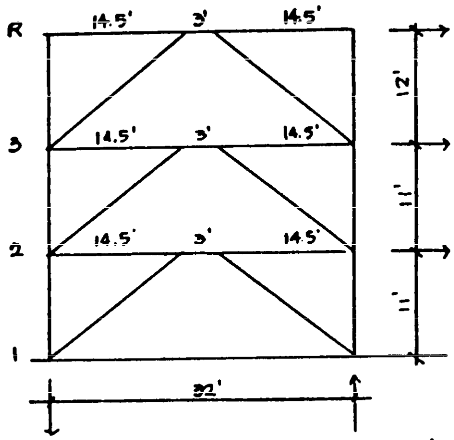
$V_p \leq 0.55F_y d t_w = 92.9$ kips

Figure D-3. Continued.

C. PRELIMINARY FRAME MEMBER SIZING

1. Frame loads

Load factor is 1.3 (ASD Sect. N1)
Distribution factors are 0.25 (roof) and 0.30 (floors)



1.3 (0.25 x 70) = 22.8 kips

1.3 (0.30 x 85) = 33.2 kips

1.3 (0.30 x 42) = 16.4 kips

Link beam shears associated with this factored frame loading:

$(V_p)_R$ = (22.8k x 12') / 32' = 8.55k
$(V_p)_3$ = (56.0k x 11') / 32' = 19.3k
$(V_p)_2$ = (72.4k x 11') / 32' = 24.9k

2. Beam sizes

Size is governed by link-beam strength with a link-beam rotation limit. This example chooses the criterion of 0.06 radians for a clear length of 1.6 M_s/V_s or less, with $M_s = ZF_y$ and $V_s = 0.55F_y dt_w$ (SEAOC 4H2a and 4H3a).

W12x26 at Roof:

M_s = 112 k' (obtained as M_p in the ASD Plastic Design Selection Table)

V_s = 0.55 x 36 x 12.22 x 0.230 = 55.6 kips

Clear length must be ≤ 1.6 x (M_s/V_s) = 3.2' > 3.0

W16x36 at 3rd: (W16x36 also at 2nd)

Ms = 192 k'
Vs = 92.6 k

Clear length must be ≤ 1.6 x (M_s/V_s) = 3.3'

Figure D-3. Continued.

PRELIMINARY FRAME MEMBER SIZING - CONT.

Design braces and columns for forces associated with yield of the following link beams:

R: W12x26 V_s = 55.6 kips)
3: W16x31 V_s = 92.6 kips)
2: W16x31 V_s = 92.6 kips)

{Note: These beam sizes {are larger than required {for the low seismic forces {at this example. Beam {sizes were selected to {match typical framing {members.

Following are the lateral forces associated with yielding of the link beams:

$12' \times F_R / 32' = 55.6$ kips

$F_R = 148$ kips

$11' \times (F_R + F_3) / 32' = 92.6$ kips

$F_3 = 121$ kips

$F_2 = 0$

Assume forces enter braced bay equally from both sides.

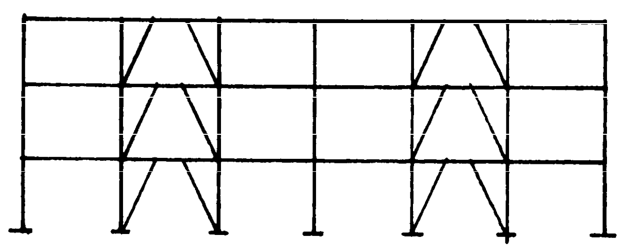


Figure D-3. Continued.

PRELIMINARY FRAME MEMBER SIZING - CONT.

3. Preliminary brace sizes

3rd story:

* brace length = 18.8'
* 1.5 is the load multiplier per SEAOC 4H12
* 148/2 is the horiz. component of the brace force
* 18.8'/14.5' is the geometric multiplier for the axial force

Brace design force = 1.5 x (148k/2 x 18.8/14.5) = 144 k
For use of the table in the AISC manual, the equivalent allowable axial force is

144k/1.7 = 85k

Use TS 7 x 7 x 1/4 P_{all} = 115 kips

2nd and 1st stories: brace length = 18.2'

F_{BR} = 1/1.7 x 1.5 x (270/2 kips x 18.2'/14.5') = 150 kips

Use TS 8 x 8 x 5/16 P_{all} = 186 kips

4. Preliminary column sizing

Size for first story; use that size above

l = 11'; Trib. A = 32' x 12' = 384 sq. ft.

P_{D+L} = 384 (26 + 0 psf)
+ 2 x 384 x (75 + 40 psf)
+ 32' x 34' x 5.3 psf (for the wall)

= 104 kips

With a load multiplier of 1.25 per SEAOC 4H13,

P_E = 1.25 x [(148kips x 23' + 122 x 11)/32']

= 185 kips

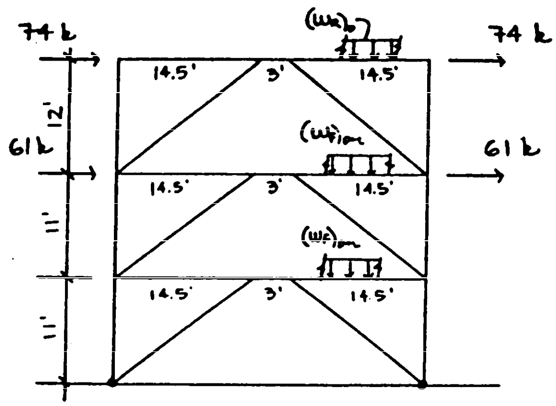
Σ = 289 kips; Equiv. allow. load = 289/1.7 = 170

Use W14x61 P_{all} = 322 kips

Figure D-3. Continued.

D. BRACED FRAME ANALYSIS

Computer input



Load Case I: DL + LL

$(w_R)_D$ = 3'(0.026ksf)
= 0.078 k/ft

$(w_F)_{D+L}$ = 3'(0.075 +0.050)
+ 11'(0.0053) wall
= 0.433 k/ft

Load Case II: Seismic

(at link yield)

$E = 29 \times 10^3$ ksi

$G = 11.6 \times 10^3$ ksi

Column bases are pinned; all other joints are rigidly connected.

Member Properties:

Member	Section	A in^2	A_v in^2	I in^4
Roof beams	W12x26	7.65	2.81	204
Floor beams	W16x36	10.6	4.69	448
Roof braces	TS 7x7x1/4	6.59	---	49.4
Floor braces	TS 8x8x5/16	9.36	---	90.9
Columns	W14x61	17.9	---	640

Figure D-3. Continued.

E. FINAL MEMBER SIZING

1. Roof Beam

W12X26

$A = 7.65$ $\quad t_f = 0.38$
$b_f = 6.5$ $\quad d = 12.2$
$r_x = 5.17$ $\quad t_w = 0.23$
$r_y = 1.51$

a. Link beam stability:

$b_f/2t_f = 8.55 \le 52/\sqrt{F_y} = 8.67$ (SEAOC 4H1)
$d/t_w = 53.0 \le 640/\sqrt{F_y} = 107$ (ASD B5.1)

b. Link beam strength:

Assumed design forces at the link:

$V_E = V_S = 56$ k $\quad V_D = 0$
$M_E = 1.5' \times V_S = 84$ k' $\quad M_D = 0$
$P_E = 0$ $\quad P_D = 0$

$V_s = 55.6$ k
$M_S = 112$ k'

Using beam flanges only, since link beam strength is governed by shear,

$M_S^* = t_f b_f F_y (d-t_f)/12 = 87.6$ k' $\quad > 84$ OK

c. Beam strength (check as columns per ASD N4).

Assumed design forces outside of link:

V_E: OK by inspection with link
$M_E = 69.4$ k'*
$P_E = 74$ k (bm.axial @ link yield - see comp.out.)
$(M_D)_u = 1.3 \times 0.9^* = 1.2$ k' $\quad M_u = 69.4 + 1.2 = 70.6$
$(P_D)_u = 1.3 \times 0.4^* = 0.5$ k $\quad P_u = 74 + 0.5 = 74.5$

Unbraced lengths:

$l_x = 14.5'$ $\quad (kl/r)_x = 33.7$
$l_y = 7.3'$ (brace @ midspan) $\quad (kl/r)_y = 58$

$F_a = 17.6$ ksi

$(F_e')_x = 129$ ksi

* from computer analysis

Figure D-3. Continued.

FINAL MEMBER SIZING - cont. Roof Beam - cont.

c. Beam strength - cont.

$P_{CR} = 1.7AF_a$ = 229 k (ASD formula N4-1)
$P_e = 23/12\ A(F_e')_x$ = 1891 k (see N4-1)
$P_y = AF_y$ = 275 k

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 108\ k' \leq M_s \quad \text{(ASD N4-5)}$$

Formula N4-2: $P_u/P_{CR} + C_m M_u/(1-P_u/P_e)M_m \leq 1.0$

74.5/229 + (0.85x70.6)/[(1-(74/1891)) x 108]

= 0.33 + 0.58 = 0.91 OK

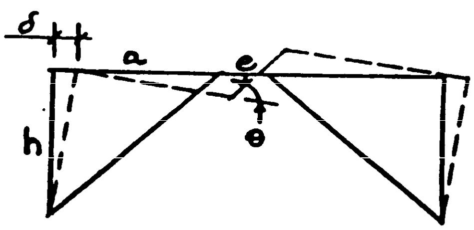
Formula N4-3: $P_u/P_y + M_u/(1.18\ M_p) \leq 1.0;\ M_u \leq M_s$

74.5/275 + 70.6/(1.18x112)

= 0.27 + 0.53 = 0.80 OK

and 70.6 < 112 OK

d. Link beam rotation:



θ at $3(R_w/8) \times \delta_{code} \leq$ 0.06 rad.

δ_{code} = [(0.25 x 70)/148] x 0.383"*
= 0.0453"

$3(R_w/8) \times \delta_{code} = 3.75\ \delta_{code}$ = 0.170"

θ = (1 + 2a/e)(0.170)/h
= 0.013 radians < 0.060 rad.

e. Link beam web:

$V_{code} \leq 0.8\ V_s$ (SEAOC 4H4)

$0.8\ V_s$ = 44.5 k

V_{code} = (0.25x70) x 12 / 32
= 6.56 k OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

2. Floor Beam

W16X36

$A = 10.6$ $t_f = 0.43$
$b_f = 7$ $d = 15.9$
$r_x = 6.51$ $t_w = 0.295$
$r_y = 1.51$

a. Link beam stability:

$b_f/2t_f = 8.14 \leq 52/\sqrt{F_y} = 8.67$
$d/t_w = 53.9 \leq 640/\sqrt{F_y} = 107$

b. Link beam strength:

Assumed design forces at the link:

$V_E = V_S = 92.9$ k $V_{D+L} = 0$
$M_E = 1.5' x V_S = 139$ k' $M_{D+L} = 0$
$P_E = 0$ $P_{D+L} = 0$

$V_s = 92.9$ k
$M_s = 192$ k'
$M_s^* = t_f b_f F_y (d-t_f)/12 = 140 > 139$ OK

c. Beam strength

Assumed design forces outside of link:

V_E: OK by inspection with link
$M_E = 117$ k'*
$P_E = 135$ k
$(M_{D+L})_u = 1.3 \times 6^* = 7.8$ k' $M_u = 117 + 7.8 = 125$
$(P_{D+L})_u = 1.3 \times 5^* = 6.5$ k $P_u = 135 + 6.5 = 141$

Unbraced lengths:

$l_x = 14.5'$ $(kl/r)_x = 26.7$
$l_y = 7.3'$ (brace @ midspan) $(kl/r)_y = 57.6$

$F_a = 17.6$ ksi

$(F_e')_x = 205$ ksi

* from computer analysis

Figure D-3. Continued.

FINAL MEMBER SIZING - cont. Floor Beam - cont.

c. Beam strength - cont.

$P_{CR} = 1.7AF_a$ = 317 k
$P_e = 23/12\ A(F_e')_x$ = 4165 k
$P_y = AF_y$ = 382 k

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_S = 184\ k' \lesssim M_S$$

$P_u/P_{CR} + C_m M_u/(1-P_u/P_e)M_m \le 1.0$

= 0.45 + 0.59 = 1.04 Say OK

$P_u/P_y + M_u/(1.18\ M_S) \le 1.0;\ \ M_u \le M_S$

= 0.37 + 0.55 = 0.92 OK

and 70.6 < 112 OK

d. Link beam rotation at 3rd Floor:

θ at $3(R_w/8) \times \delta_{code} \le 0.06$ rad.

$\delta_{code} \approx [(0.25x70)+(0.30x85)]/(148+122) \times 0.375"$ *

= 0.0597"

$3(R_w/8) \times \delta_{code} = 3.75\ \delta_{code} = 0.224"$

$\theta = (1 + 2a/e)(0.224)/h$
= 0.018 radians < 0.060 rad.

e. Link beam web:

$V_{code} \le 0.8\ V_S$

$0.8\ V_S$ = 74.3 k

V_{code} = [(0.25 x70 + 0.30x85 + 0.30x42) x 11] / 32
= 19.1 k OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

3. Brace size - 3-R

TS 8x8x1/4

$A = 7.59\ in^2$
$F_y = 46$ ksi
$b = d = 8$ in.
$t = 0.25$ in
$r_x = r_y = 3.15$ in.

Assumed design forces:

$P_E = 1.5 \times 99^* = 149$ k
$M_E = 1.5 \times 15^* = 22.5$ k'

$(P_D)_u = 1.3 \times 1\ k^* = 1.3$ k
$(M_D)_u = 0$

$P_u = 149 + 1.3 = 150$
$M_u = 22.5 + 0 = 22.5$

Unbraced lengths:

$l_x = l_y = 18.8'$ $(kl/r)_x = (kl/r)_y = 72$

$F_a = 20.7$ ksi

$(F_e')_x = 28.8$ ksi

* from computer analysis

$P_{CR} = 1.7AF_a = 267$ k
$P_e = 23/12\ AF_e' = 419$ k'
$P_y = AF_y = 349$ k
$Z = bd^2/4 - b_1d_1^2/4 = 22.5\ in^3$
$M_s = ZF_y = 86.4$ k'

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 79.1\ k' \leq M_s$$

$P_u/P_{CR} + C_mM_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.56 + 0.27 = 0.83$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_s) \leq 1.0;\quad M \leq M_s$

$= 0.43 + 0.225 = 0.65$ OK

Width\thickness $= b_1/t = 30 \approx 190/\sqrt{F_y} = 28$ Say OK
Depth\thickness $= d/t = 32 \leq 257/\sqrt{F_y} = 38$ OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

4. Brace size - 2-3 and 1-2

TS 8x8x3/8

$A = 11.1\ in^2$
$F_y = 46$ ksi
$b = d = 8$ in.
$t = 0.375$ in
$r_x = r_y = 3.09$ in.

Assumed design forces:

$P_E = 1.5 \times 171 = 257$ k
$M_E = 1.5 \times 22 = 33$ k'

$(P_{D+L})_u = 1.3 \times 6.0\ k^* = 8$ k
$(M_{D+L})_u = 1.3 \times 0.6\ k^* = 1$ k'

$P_u = 257 + 8 = 265$
$M_u = 33 + 1 = 34$

Unbraced lengths:

$l_x = l_y = 18.2'$ $(kl/r)_x = (kl/r)_y = 71$

$F_a = 20.9$ ksi

$(F_e')_x = 29.6$ ksi

* from computer analysis

$P_{CR} = 1.7AF_a = 394$ k
$P_e = 23/12\ AF_e' = 630$ k'
$P_y = AF_y = 511$ k
$Z = bd^2/4 - b_1d_1^2/4 = 32.7\ in^3$
$M_S = ZF_y = 125$ k'

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_S = 115\ k' \leq M_S$$

$P_u/P_{CR} + C_mM_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.67 + 0.31 = 0.98$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_S) \leq 1.0;\ M \leq M_p$

$= 0.52 + 0.23 = 0.75$ OK

Width\thickness = $b_1/t = 19 \approx 190/\sqrt{F_y} = 28$ Say OK
Depth\thickness = $d/t = 21 \leq 257/\sqrt{F_y} = 38$ OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

5. Column size - 1-2

W14x53

$A = 15.6\ in^2$
$b_f = 8$ in. $d = 13.9$ in.
$t_w = 0.37$ in. $t_c = 0.66$ in.
$Z = 87.1\ in^3$
$r_x = 5.89$ in. $r_y = 1.92$ in.

Assumed design forces:

$P_E = 1.25 \times 139 = 174$ k
$M_E = 1.25 \times 15 = 18.8$ k'

$(P_{D+L})_u = 1.3 \times 104\ k^* = 135K$ $P_u = 174 + 135 = 265$
$(M_{D+L})_u = 1.3 \times 3.7\ k^* = 4.8$ k' $M_u = 18.8 + 4.8 = 24$

Unbraced lengths:

$l_x = l_y = 11'$ $(kl/r)_x = 22$
$(kl/r)_y = 69$

$F_a = 16.5$ ksi

$(F_e')_x = 309$ ksi

* from computer analysis

$P_{CR} = 1.7AF_a = 438$ k
$P_e = 23/12\ AF_e' = 9240$ k'
$P_y = AF_y = 562$ k
$M_S = ZF_y = 261$ k'

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_S = 245\text{ k}' \leq M_S$$

$P_u/P_{CR} + C_mM_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.71 + 0.06 = 0.77$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_S) \leq 1.0;\ M \leq M_p$

$= 0.55 + 0.08 = 0.63$ OK

Figure D-3. Continued.

DESIGN EXAMPLE D-4

Dual Bracing System With Concrete Frame

Description of Structure. A two-story Office Building in Zone 4 with a complete reinforced concrete vertical load-carrying space frame. The lateral forces are resisted by a dual system consisting of concrete special moment resisting frames and concrete shear walls. The structural concept is illustrated on Sheet 2. The East-West direction is considered.

Construction Outline.

Roof:
Built-up, 5-ply.
Concrete joists and girders.
Suspended ceiling.

2nd & 3rd Floors:
Concrete joists and girders.
Asphalt tile.
Suspended ceiling.

1st Floor:
Concrete slab-on-grade.

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

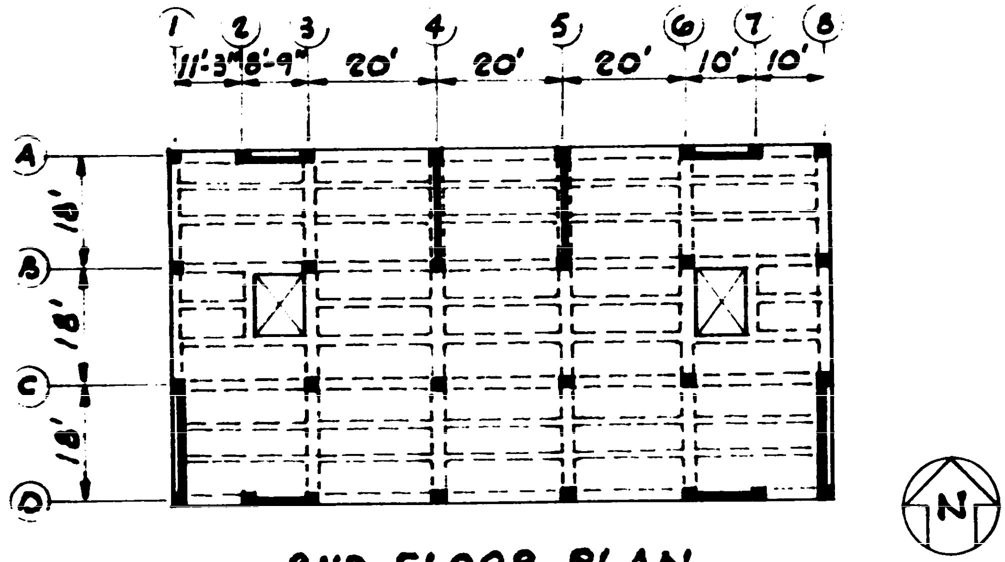
Partitions:
Non-structural removable drywall, except concrete as structurally required.

Design Concept. The structure is a dual system meeting SEAOC 1D6d, and it qualifies for an R_w-factor of 12 as follows: (1) the SMRF is designed to resist not less than 25 percent of the prescribed lateral forces; (2) the shear wall and the special frame systems are designed to resist the total prescribed lateral forces in proportion to their relative rigidities. In this example it is assumed that the rigidity of the frame system is negligible when compared to the wall system. (Compare stiffness as proportional to the third power of the width of one element: 14^3 for a column is much less that 120^3 for a wall.) Therefore, the wall system is designed for 100 percent of the base shear and the frame system for 25%. The roofs and floors form rigid diaphragms, and the seismic loads to the frames are proportioned according to their stiffnesses and loads to the walls according to theirs. The building is assumed to be symmetrical about both axes so that only accidental torsion is involved.

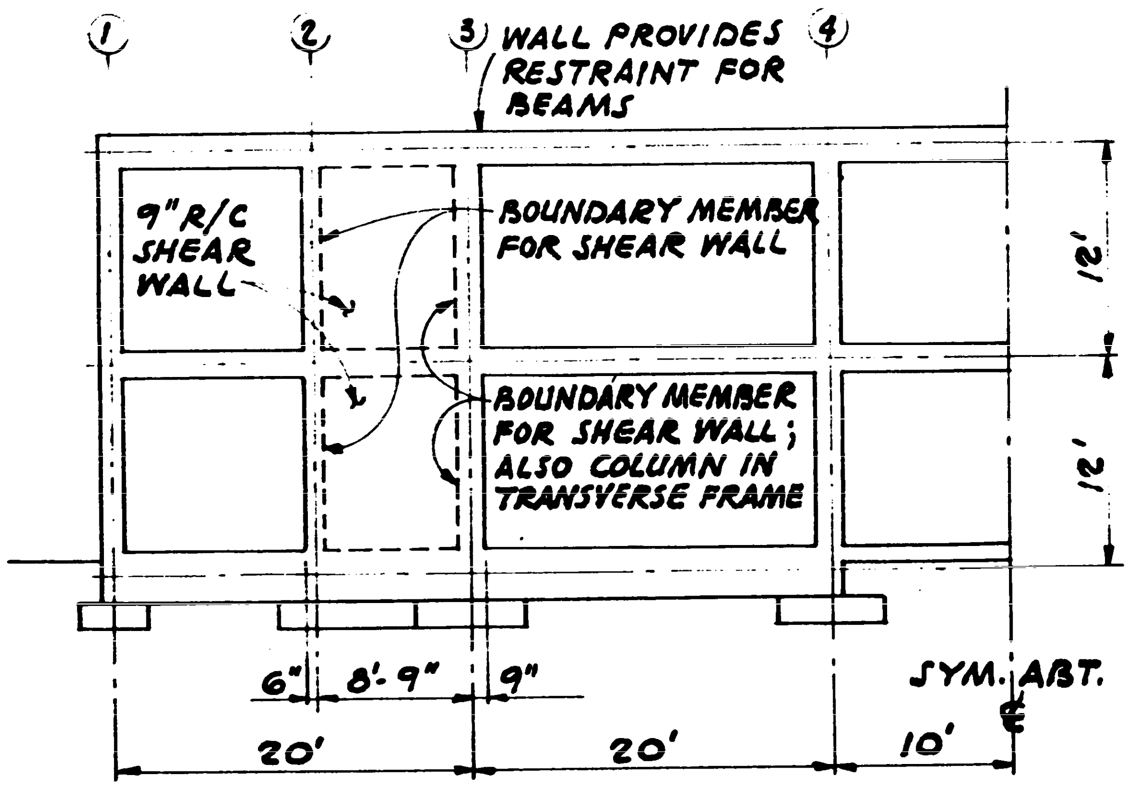
Discussion. Vertical and lateral forces are pre-computed. (See Example A-5 for a typical computation.) The shear walls in the south wall (Line D) are designed for the given lateral forces. The seismic frames would be designed for 25% of these forces, using the methods of Example A-2. Deformation compatibility is investigated for the nonseismic frames.

Materials. Conc. f'_c = 3,000 psi; reinf. steel f_y = 60,000 psi.

Figure D-4. Dual bracing system.

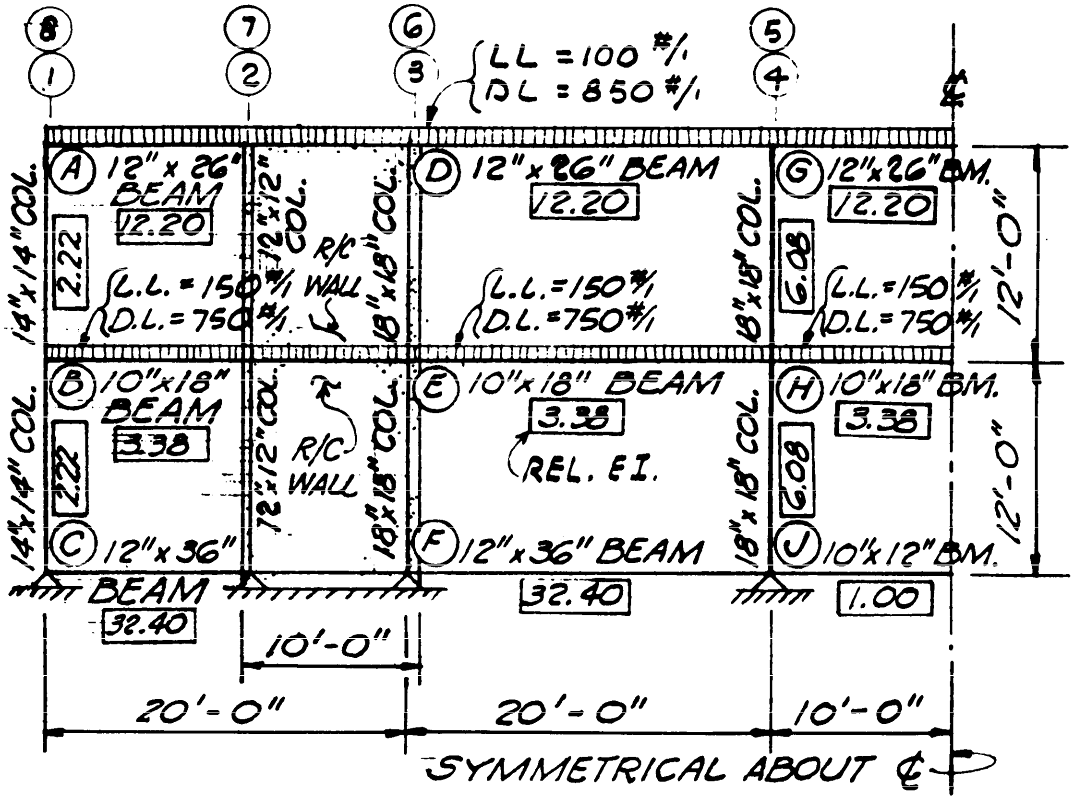


2ND FLOOR PLAN



HALF ELEVATION OF SOUTH WALL-LINE D

Figure D–4. Continued.



MEMBERS AND LOADS - SOUTH WALL - LINE D

FRAME PROVISIONS

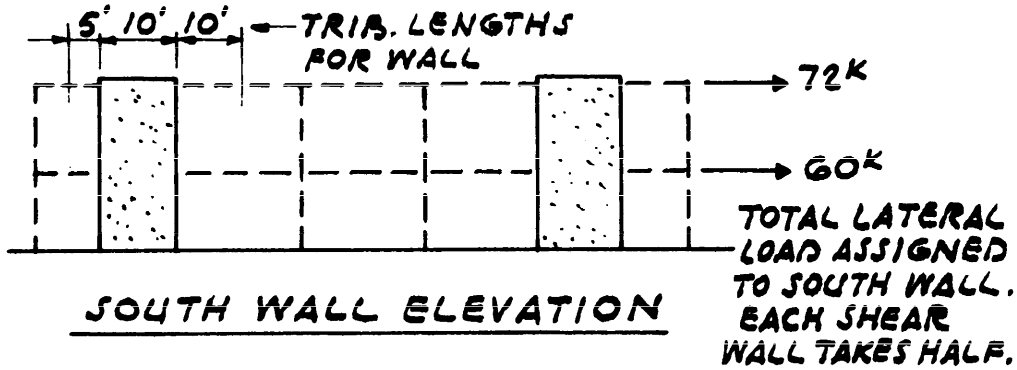
The two interior frames (Lines B and C) will be designed as special moment resisting frames to carry 25% of the total required lateral force. See Example A-2. This example will deal only with Lines A and D which have shear walls that carry 100% of the lateral force. Deformation compatibility (SEAOC 1H2d) must be investigated for the vertical load-carrying frames on Lines A and D (see p. 11).

As an alternate, the interior frames could be designed for vertical load only (with an investigation for deformation compatibility), and the lateral forces would be carried by ductile moment resisting space frames on Lines A and D. In these frames there is a choice concerning the columns on Lines 2 and 7: the columns may be treated as columns with adjacent girders of 10' span, or they may be treated as boundary members for the shear walls. In the latter case, the girders must still be designed, together with the columns, for the actual 10' spans, but they must also be designed to span 20' from 1 to 3 and from 6 to 8 in case the shear walls and boundary members fail.

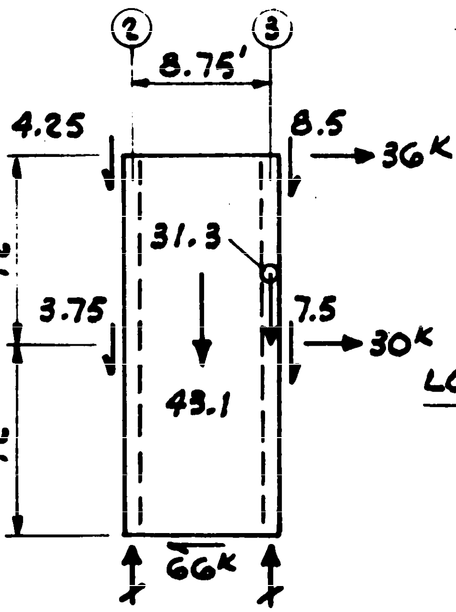
Figure D-4. Continued.

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 ELEVATION



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	0.85x10	=	8.5
FLOOR	0.75x10	=	7.5
WALL	10x24x0.113	=	27.1
	TOTAL DL		43.1K
FLOOR LL	0.15x10	=	1.5K

LINE 3 GIRDER REACTION

ROOF + FLOOR DL = 31.8K
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

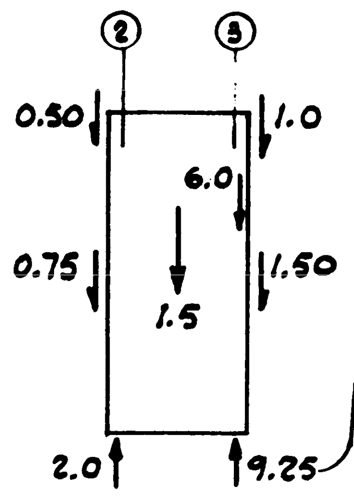
$P = 0.9D - 1.4E$

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

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)



VERT. LL

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

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 $V_u = 1.4E$, $\varphi = 0.60$

$V_u = 1.4\left(\frac{72+60}{2}\right) = 92.4$ $A_c = 9'' \times 10' \times 12''/1 = 1080 \text{ IN}^2$

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

$V_c = 0.110 \times 1080 = 119^K$ $< 8\sqrt{f'_c}$ OK

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

$A_v = \frac{V_u' s}{f_y d}$ $s = \frac{A_v f_y d}{V_u'}$

Try #4 @ 18" CC EACH WAY EACH FACE

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

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

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: $\quad$ 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:

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

USE $\gamma = 0.60$

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

REQ'D $\rho_g = 0.010$

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

TENSION CONTROLS:

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

VERTICAL COLUMN CORE REINFORCEMENT

Figure D–4. Continued.

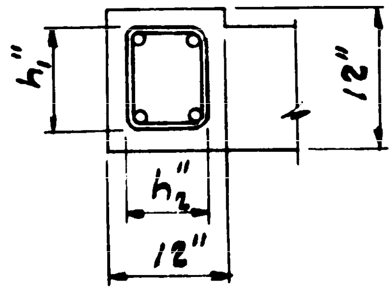
SHEAR WALL DESIGN - CONT'D

VERTICAL BOUNDARY MEMBERS - CONT'D

SPECIAL TRANSVERSE REINFORCEMENT:

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

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



FROM ACI 21.4.4.1

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

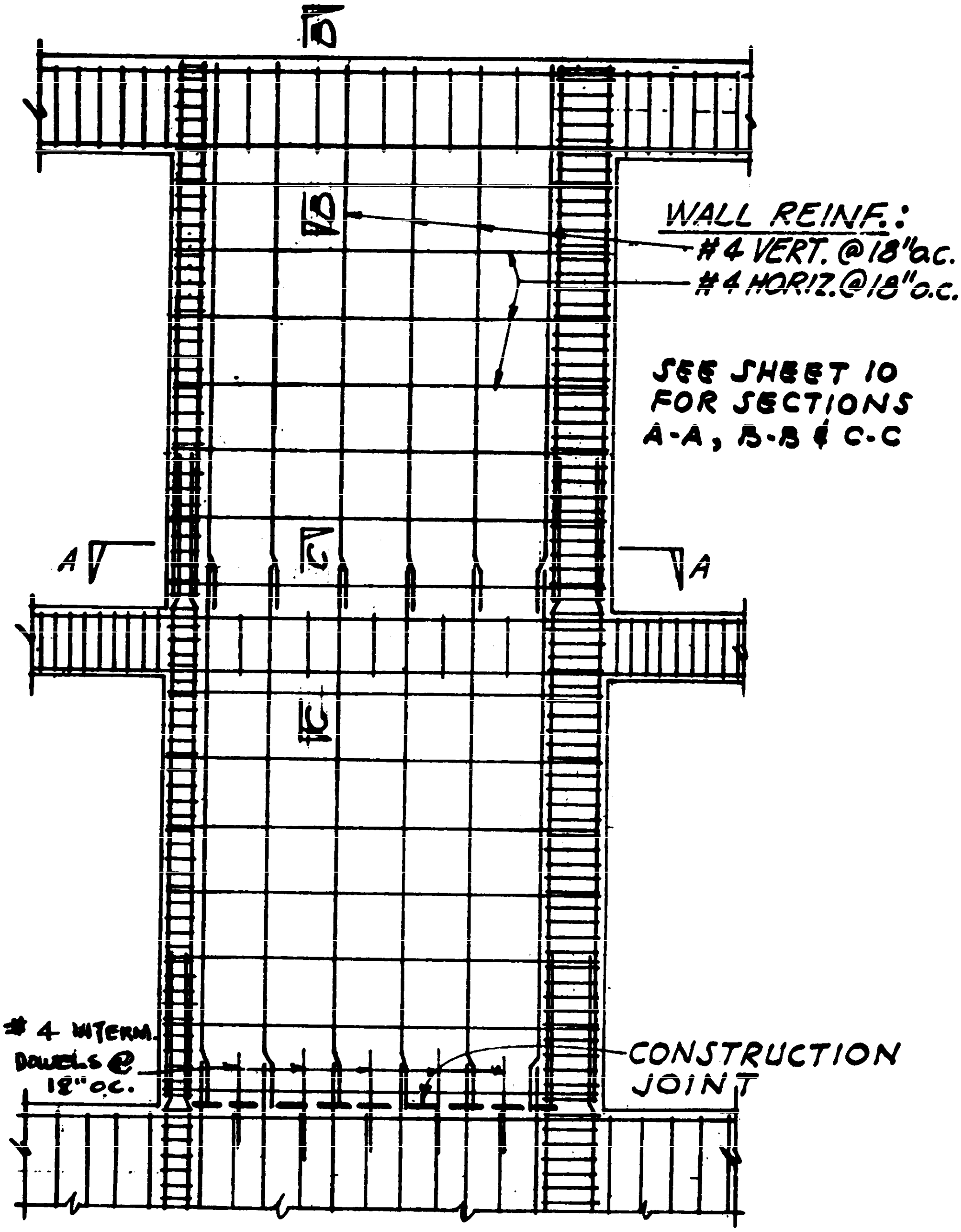
$$= \frac{.40}{0.30(9.0)\frac{3}{60}\left[\frac{12 \times 12}{9 \times 9} - 1\right]} = 3.81''$$

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

$S_{MAX} = 3.81''$ ∴ USE #4 HOOPS @ $3\frac{1}{2}''$ O.C. THRU-OUT LENGTH OF COL. CORE

Figure D-4. Continued.

SHEAR WALL DESIGN - CONT'D



WALL ELEVATIONS

Figure D-4. Continued.

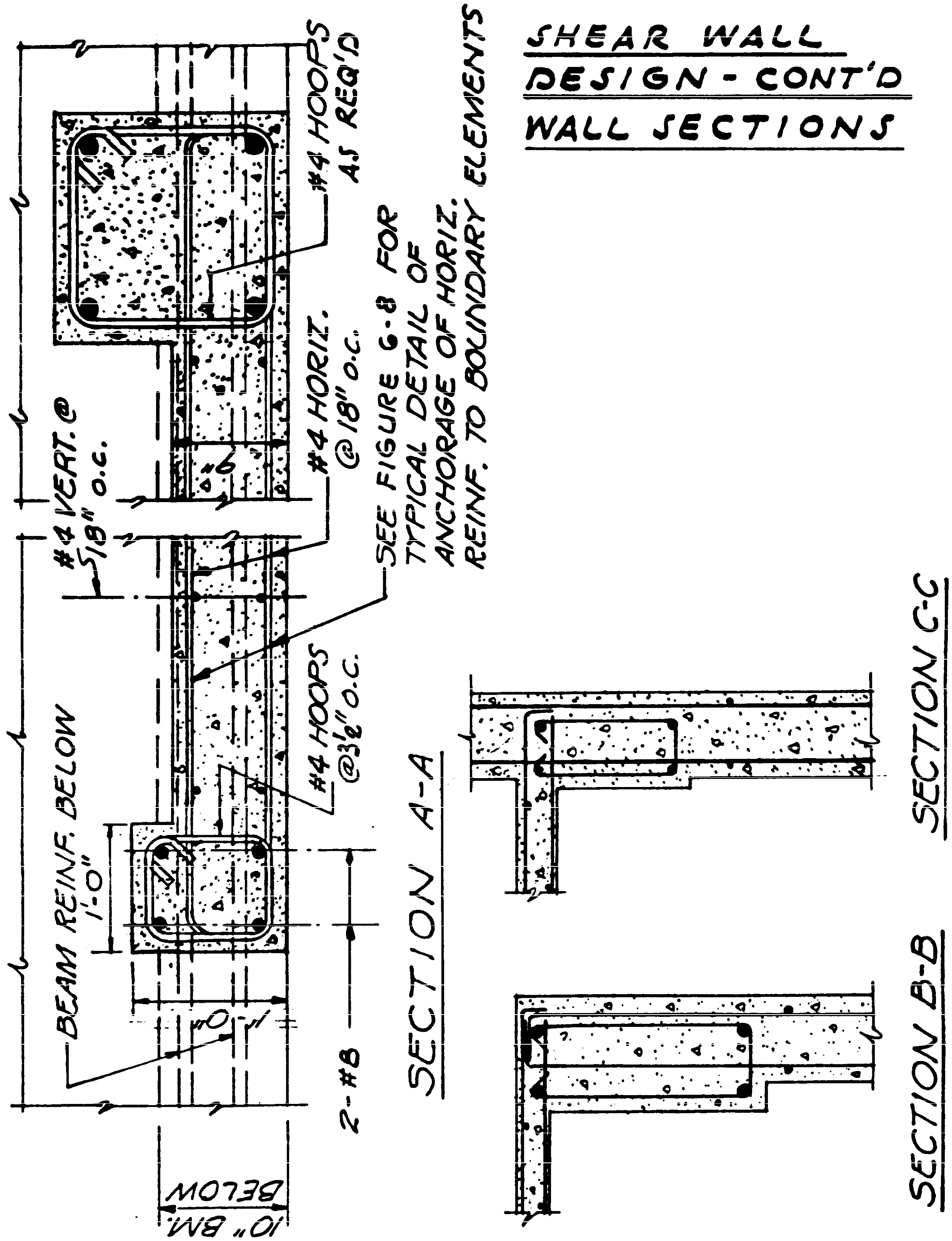
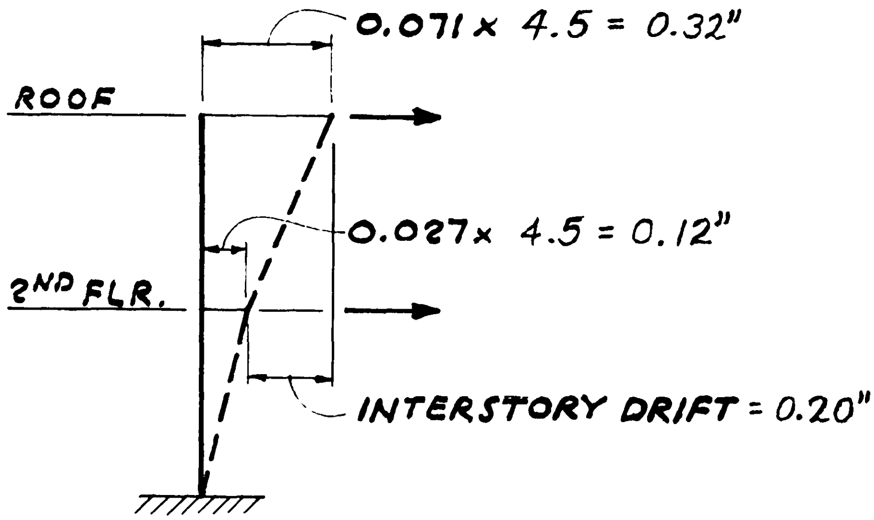


Figure D-4. Continued.

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.

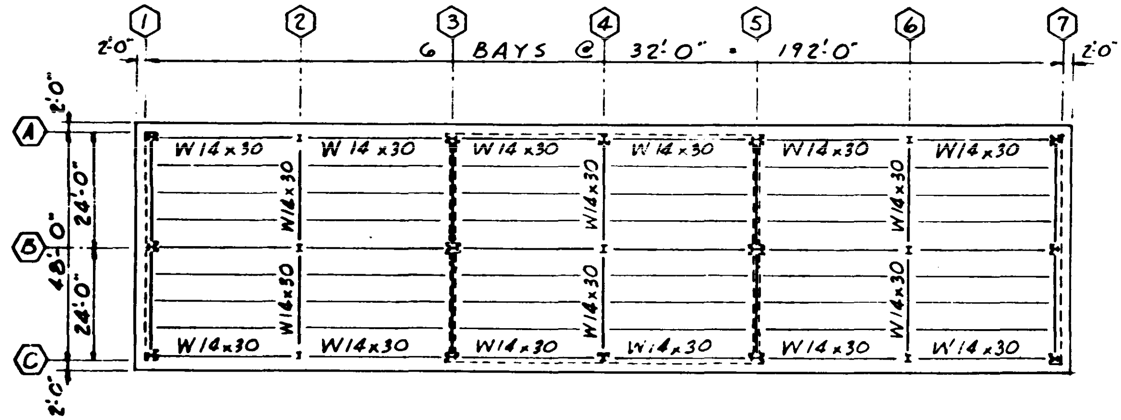
Exterior Walls:
- Bearing Walls in concrete and non-bearing, non-shear insulated metal panels.

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

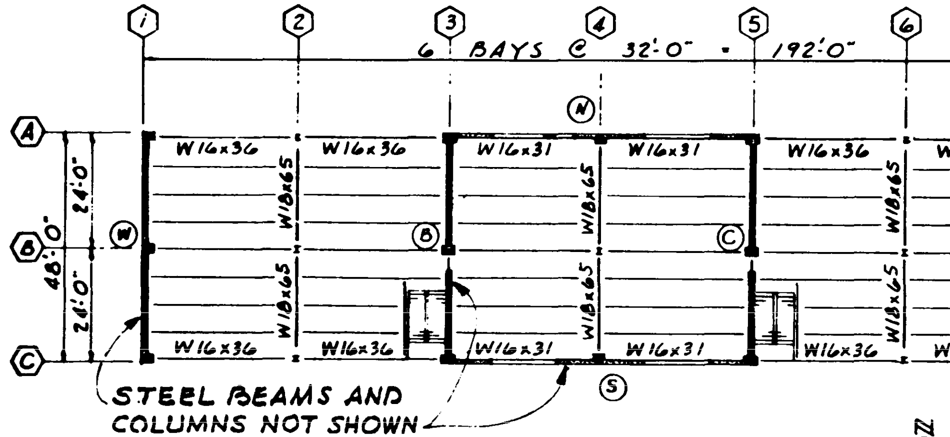
Design Concept. The building has a complete steel space frame capable of carrying all gravity loads. Lateral loads are resisted by a dual system. Concrete shear walls are designed to carry 100% of the prescribed lateral loads. (The transverse walls are on lines B, C, E and W; longitudinal walls are on 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.

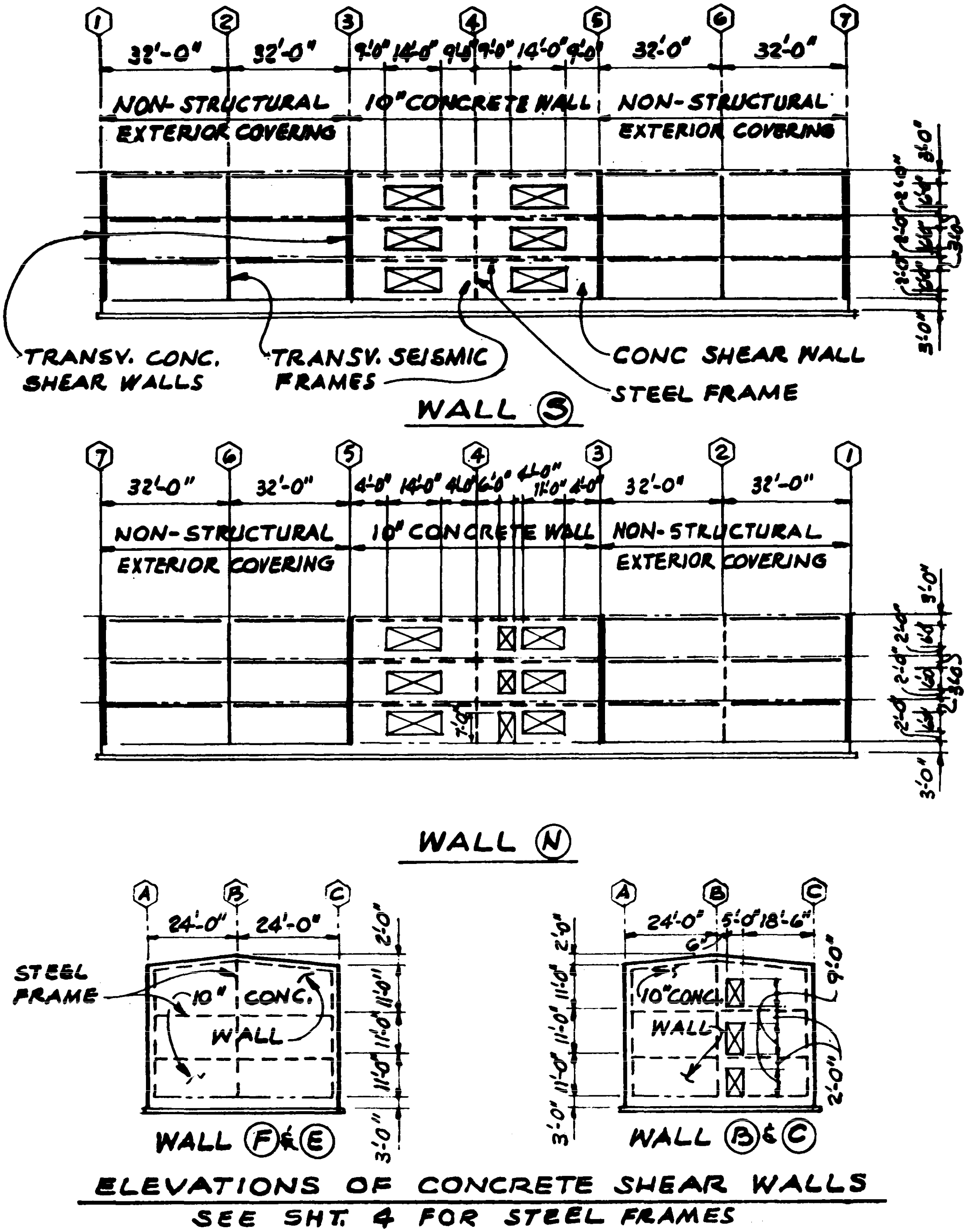
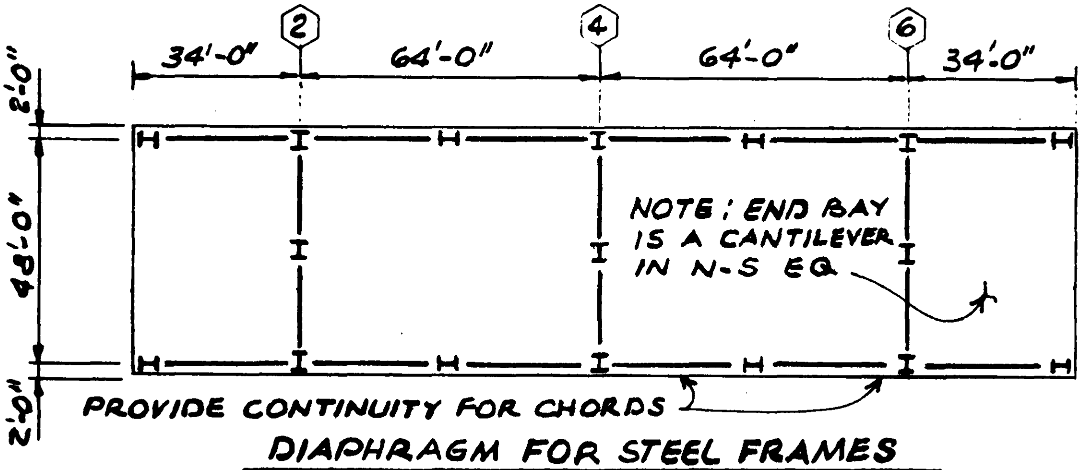
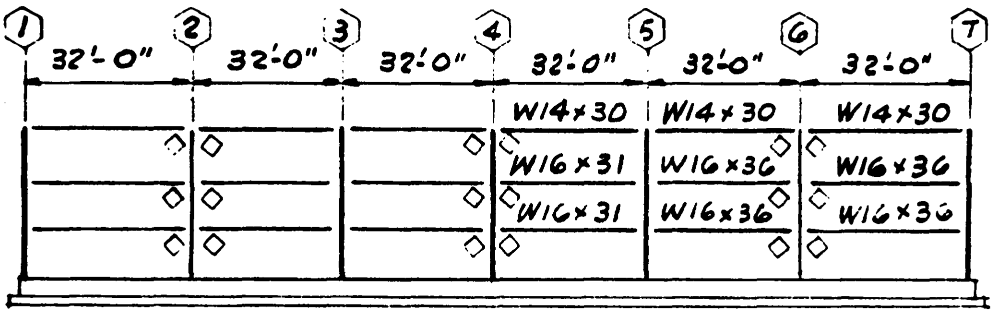


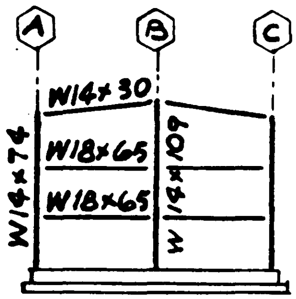
Figure D-5. Continued.



DIAPHRAGM FOR STEEL FRAMES

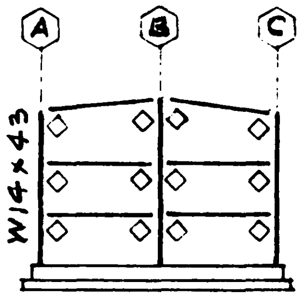


STEEL FRAMES AT (N) & (S) WALLS



VERTICAL LOAD + SEISMIC

LINES 2, 4, 6.



VERTICAL LOAD ONLY

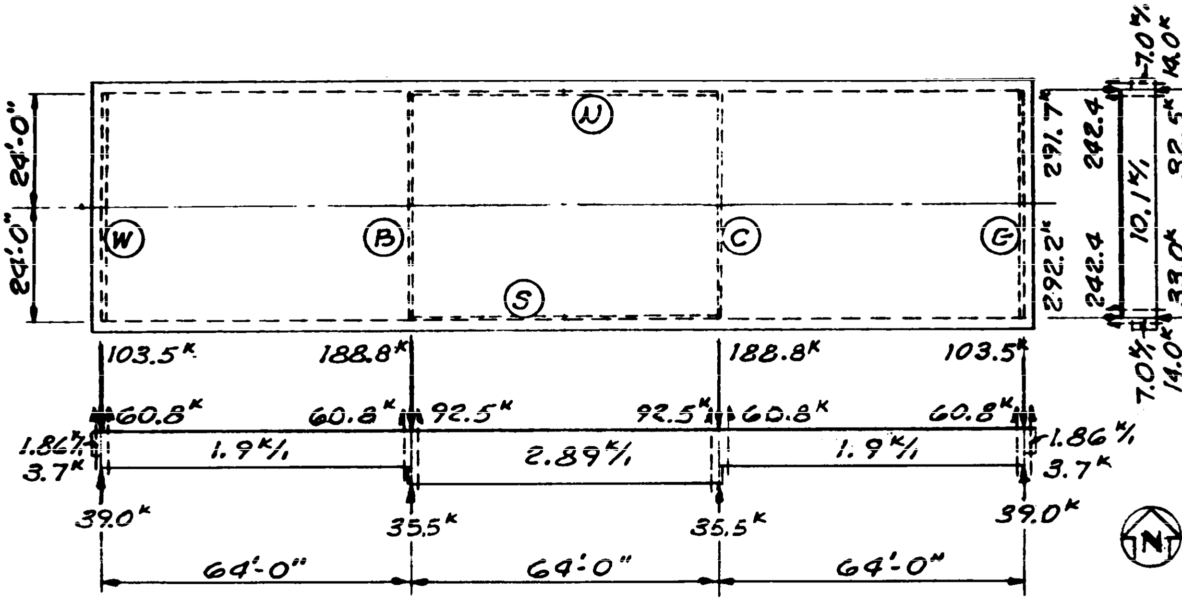
LINES 1, 3, 5, 7.

TRANSVERSE STEEL FRAMES

◇ 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.



TOTAL W_R 584K
LOADING @ 100% g

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 = 813 × 48 = 39,024#
1626#/'

N = 687 × .75 = 522 × 63.2 = 32,548#

S = 687 × .76 = 522 × 63.2 = 32,900#
1037#/'

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

E-W LOADS

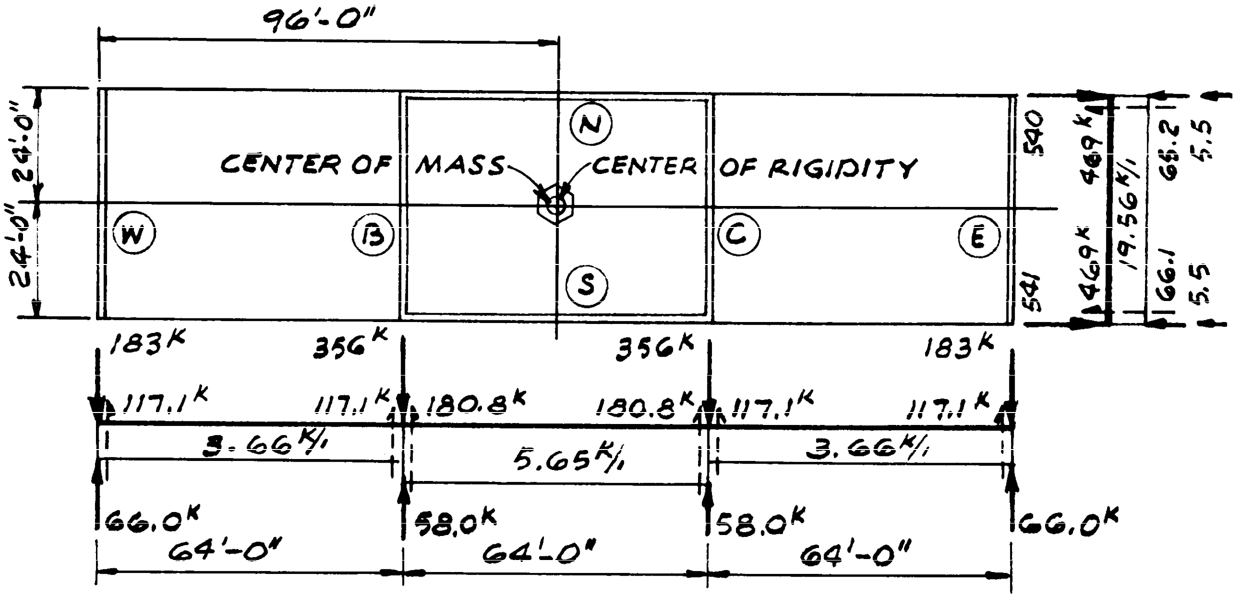
ROOF = 35.7 × 196 = 6997#/'
WALLS E & W = 1626
WALLS B & C 2×740 = 1480
10,103#/'

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% g

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' = 2.76^K × 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			

$V = 0.092 \times 2740 = 252^k$ $F_T = 0$ 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.

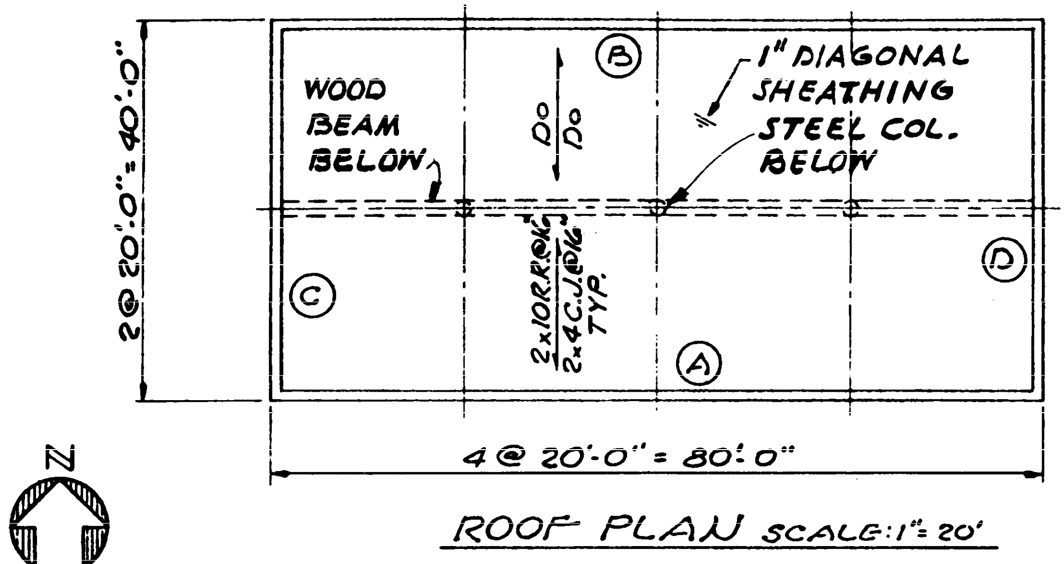
Exterior Walls:
- Wood stud bearing walls with exterior and interior plaster.

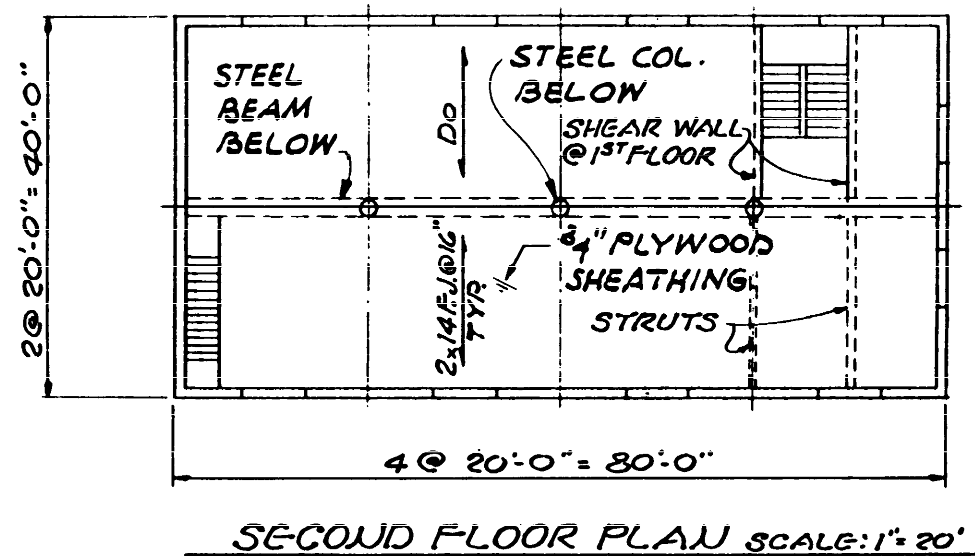
Partitions:
- The stair enclosure walls are wood stud with plywood sheathing on one. Other interior walls are removable drywall.

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.





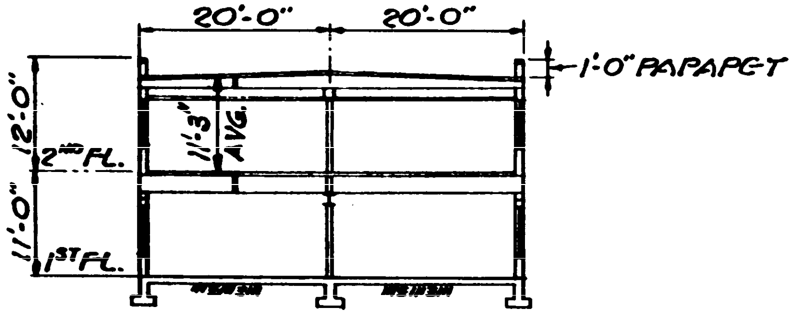


Figure D-6. Continued.

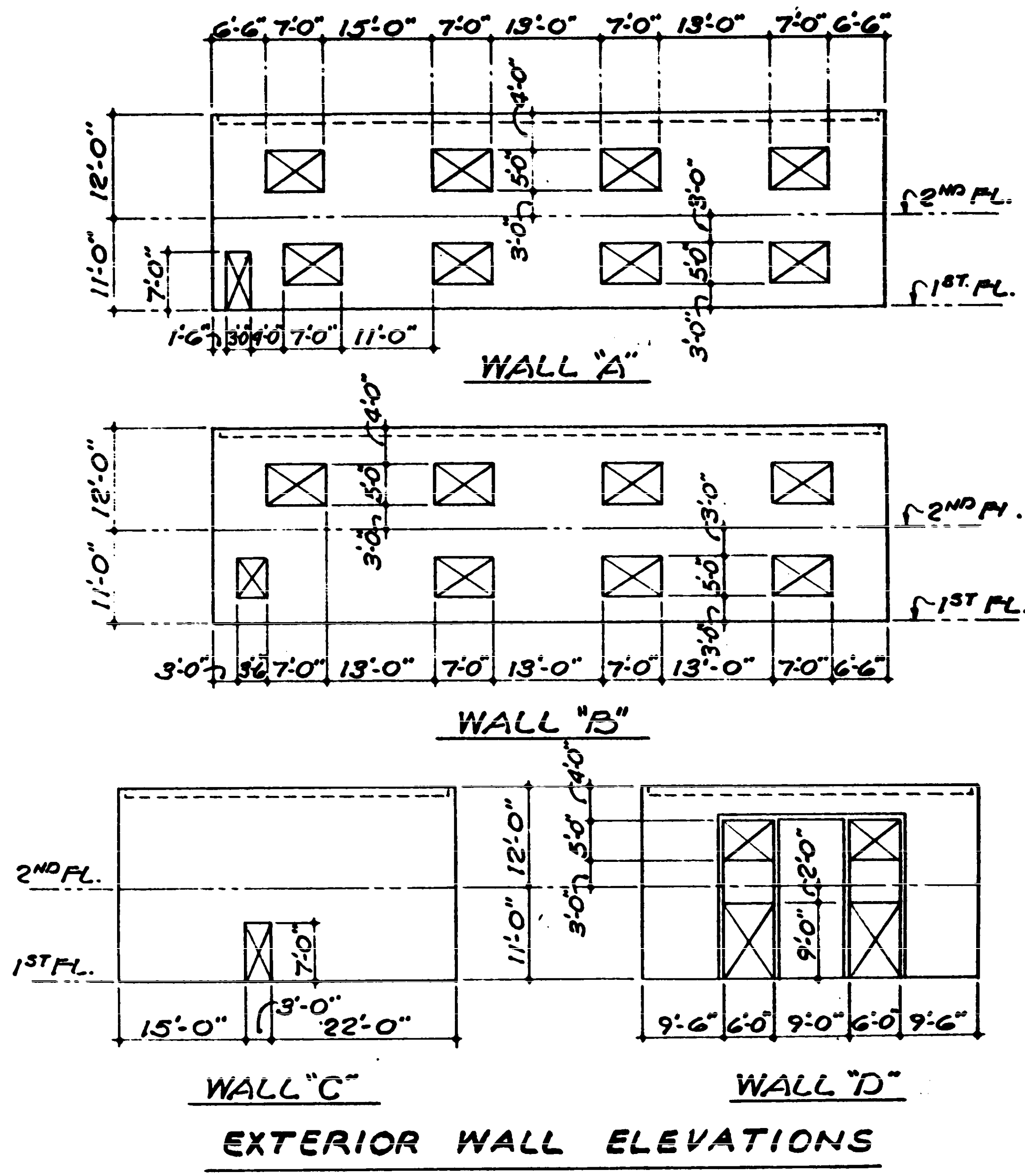


Figure D-6. Continued.

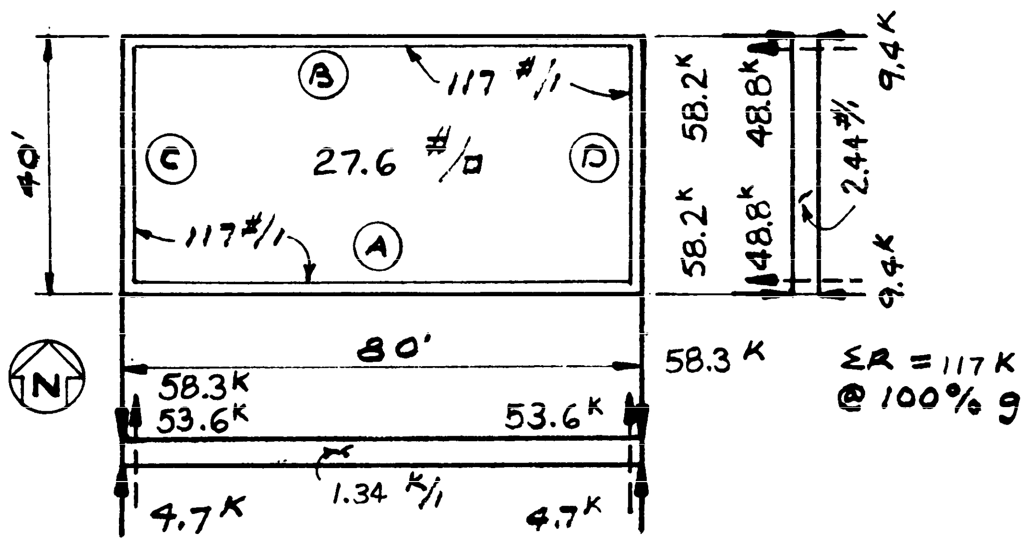
LOADS FOR ROOF DIAPHRAGM

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 #/1

LOADING DIAGRAM - ROOF DIAPHRAGM



(N-S) LOADS

27.6 #/□' × 40' = 1104 #/1
117 #/1 × 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

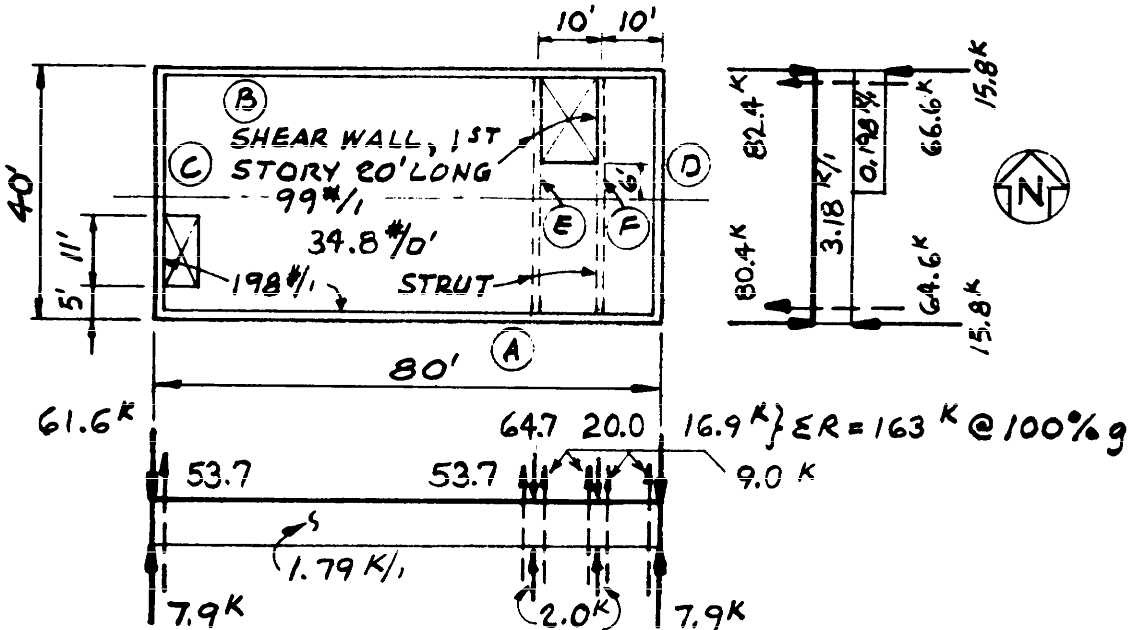
ASPHALT TILE	=	1.0 #/□'
3/4" PLYWOOD SHEATHING	=	2.3
FLOOR JOISTS	=	4.6
STEEL BEAMS & COLUMNS	=	1.0
CEILING & MECHANICAL	=	5.9
PARTITIONS	=	20.0
FLOOR DEAD LOAD	=	34.8 #/□'

WALLS

EXTERIOR: 18 #/□' × 11' = 198 #/'

INTERIOR: 18 × 5.5 = 99

LOADING DIAGRAM - 2ND FLOOR DIAPHRAGM



(N-S) LOADS

34.8 × 40 = 1392 WALL C OR D = 198 × 40 = 7920#

198 × 2 = 396 WALL E OR F = 99 × 20 = 1980#

1788 #/'

(E-W) LOADS

34.8 × 80 = 2784 WALL E & F = 99 × 2 = 198 #/'

198 × 2 = 396 WALL A OR B = 198 × 80 = 15,840#

3180

Figure D–6. Continued.

LATERAL FORCES

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

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

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

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

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

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

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

LEVEL	W_x	h	wh	$\frac{wh}{\Sigma wh}$	F_x
ROOF	117^K	$22.25'$	2603	0.59	17.0
2^{ND} 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. $F_{px} = 0.35 ZI W_{px} = 0.35 \times .3 \times 1 \times W_{px} = 0.105\ W_{px}$

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

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 $v_{MAX.} = 190^{\#}/'$

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 = $3 \times 1350^{\#} \times 1.33 = 5.40^K$.

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 = $7 \times 107^{\#} \times 1.33 = 996^{\#}$

DIAPHRAGM CONNECTION WALL A OR B $v = 90^{\#}/'$

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

284

DIAPHRAGM CONNECTION TO WALL C OR D $v = 190^{\#}/'$

RAFTER TO BLOCKING & BLOCKING TO TOP PLATE (Sect. C, FIGURE 5-23) USE 16d @ 8" O.C. CAPACITY = $(107^{\#} \times 1.33 \div 0.67 = 212^{\#}/FT$

284

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-$\frac{5}{8}$"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

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

$$= 3.45^{K}$$

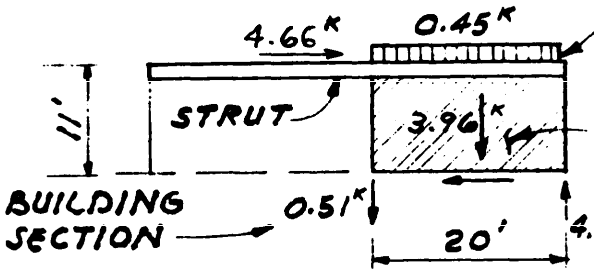
USE 2-$\frac{3}{4}$"ϕ BOLTS, DOUBLE JOIST TO $\angle 4 \times 4 \times \frac{1}{4}$ AND 2-$\frac{3}{4}$"ϕ BOLTS, ANGLE TO DOUBLE TOP PLATE OF WALL E.

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 x FLOORWEIGHT



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

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

TOTAL DL = 4410#

SEISMIC LOADS (0.072 x FLOOR WT.)
DIAPH. LOADING DIAGRAM

FROM THE WEST	0.072 x 53.7^K	= 3.87^K
FROM THE EAST	x 9.0	= 0.66
FROM THE WALL	x 2.0	= 0.14
	WALL V	= 4.66^K

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

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# x 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 2½" NET, WITH METAL SIDE PLATES: ALLOW 2x(1.25x1020#) x1.33 = 3392# > 824# ANCHOR BOLT: 5/8" ϕ ALLOW 1.33x3000 = 4000# ∠4x4x5/8 x0'-3½": S = (3½ − 7/8")(5/8")²/6 = .173 IN³ M = 511# x (2½ − 1) = 767#" F = 767/.173 = 4333 PSI

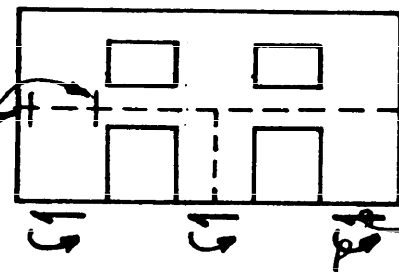
Figure D-6. Continued.

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.



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

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

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

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

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

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 2½" WITH 2-⅝" 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) 3-¾"φ BOLTS TO 4x4 POST, SINGLE SHEAR IN 2½" NET WITH METAL SIDE PLATE. ALLOW $3 \times (1.25 \times \frac{2870^{\#}}{2}) \times 1.33 = 7157^{\#}$

⅞"φ ANCHOR BOLTS WITH 3"x3" WASHER

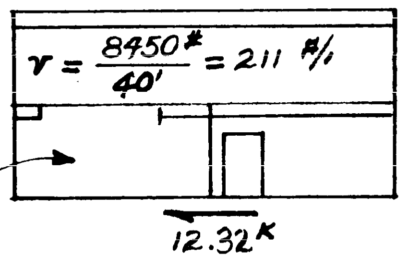
NET AREA OF WASHER = 8 IN.^{2}
ALLOW 600 PSI IN B'R'G: $600 \text{ PSI} \times 8 \times 1.33 = 6400^{\#}$

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)



$\rightarrow .145 \times 58.3^K = 8.45^K \times 22.3 = 189^K$

$\rightarrow .072 \times 53.7^K = 3.87^K \times 11.0 = 43$

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

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

WT. OF WALL $= 18\,psf \times 23$

$= 414^{\#}/'$

RESISTING MOMENT

$M_R = 0.414^{K}/' \times 40' \times 20'$

$= 331 > 232^{K'}$

$\therefore$ 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 $0.145 \times 58.2 =$	8.44	8.44	52'	$162^{\#}/'$
FLOOR $0.072 \times 80.4 =$	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

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

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

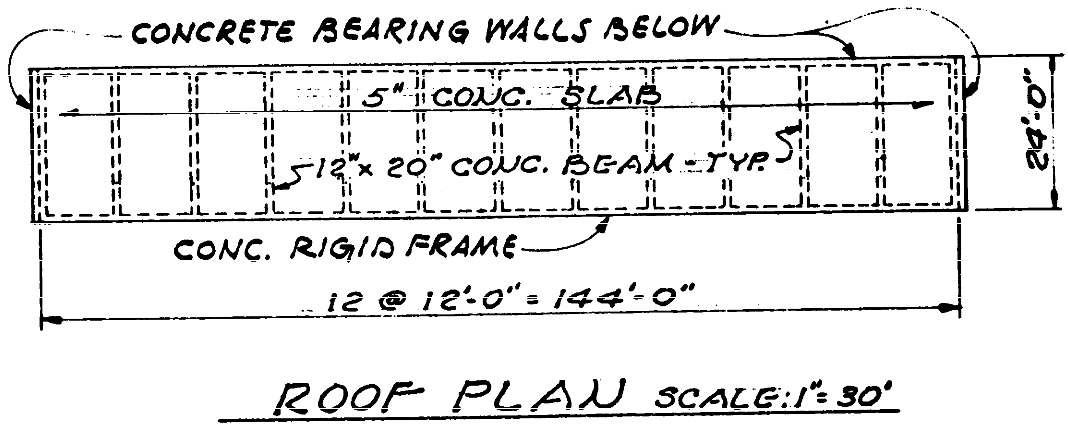
$$T = 0.020\,(h_n)^{3/4} \qquad h_n = 16.0'$$

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

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

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

Figure D-7. Special configuration.



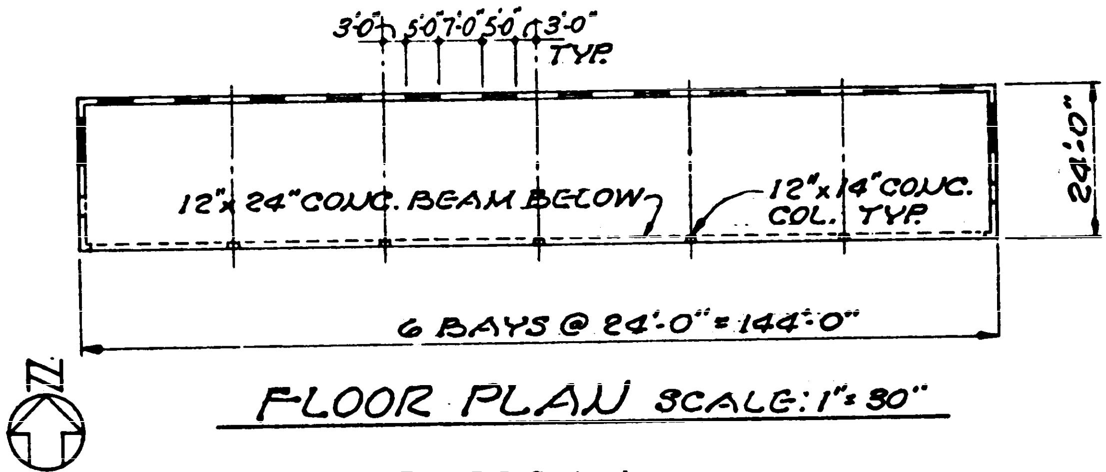


Figure D-7. Continued.

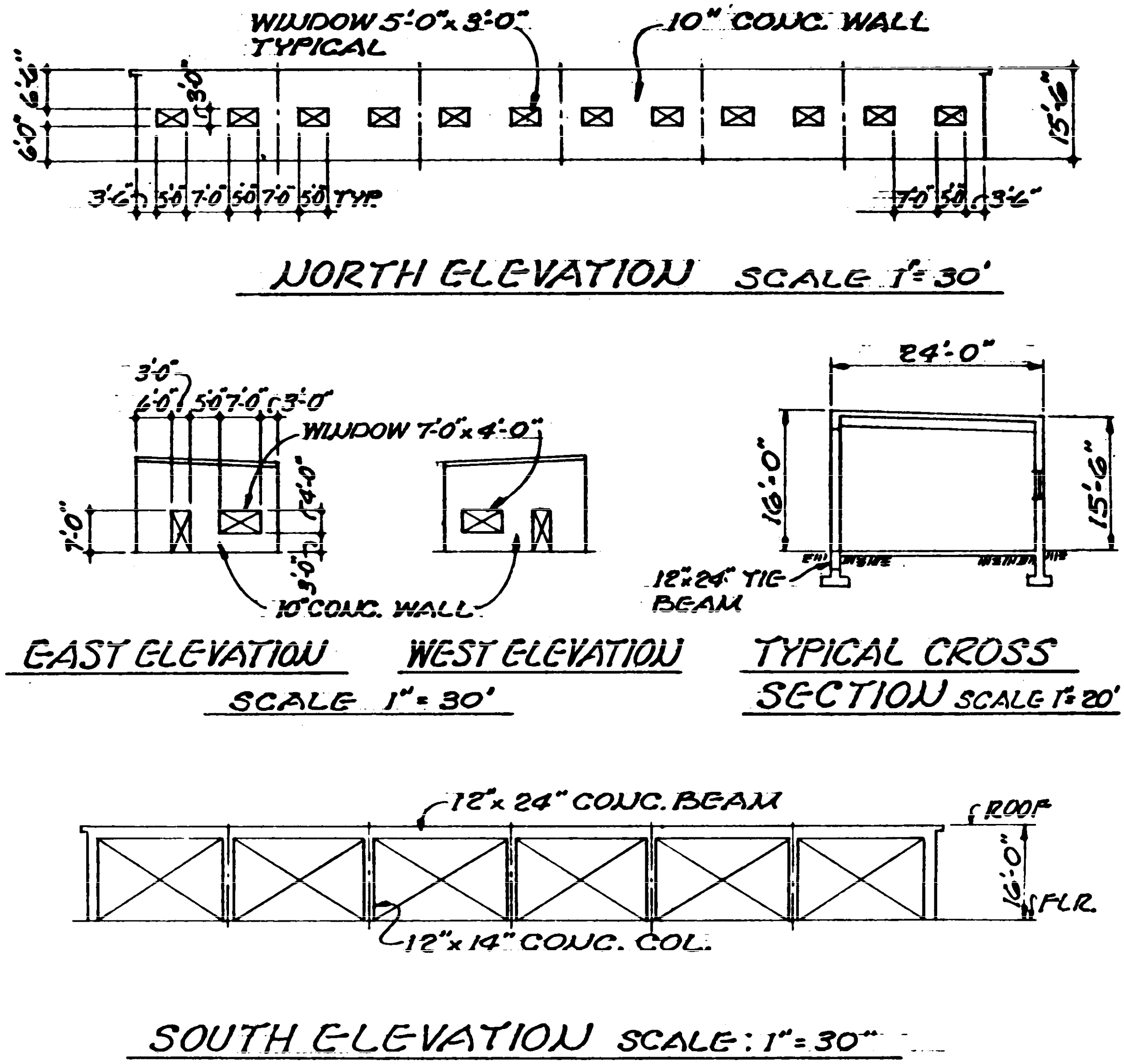


Figure D-7. Continued.

ROOF D.L.

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

EXTERIOR WALLS
N. WALL 10" CONC. 125x7.55 = 945 #/'
E. & W. END WALLS 125x7.88 = 985 #/'

12" x 24" CONC. BEAM = 150 x 1.58 = 237 #/'
COLUMNS = 1 x 1.17 x 150 x 14/2 = 1228 #

SEISMIC N-S

ROOF 86 x 24 = 2060
N. WALL = 945
BEAM = 237
COLS. = 5x1228/144 = 43
DOOR OR COVER 10x7 = 70
.138 x 3355 = 463 #/'

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

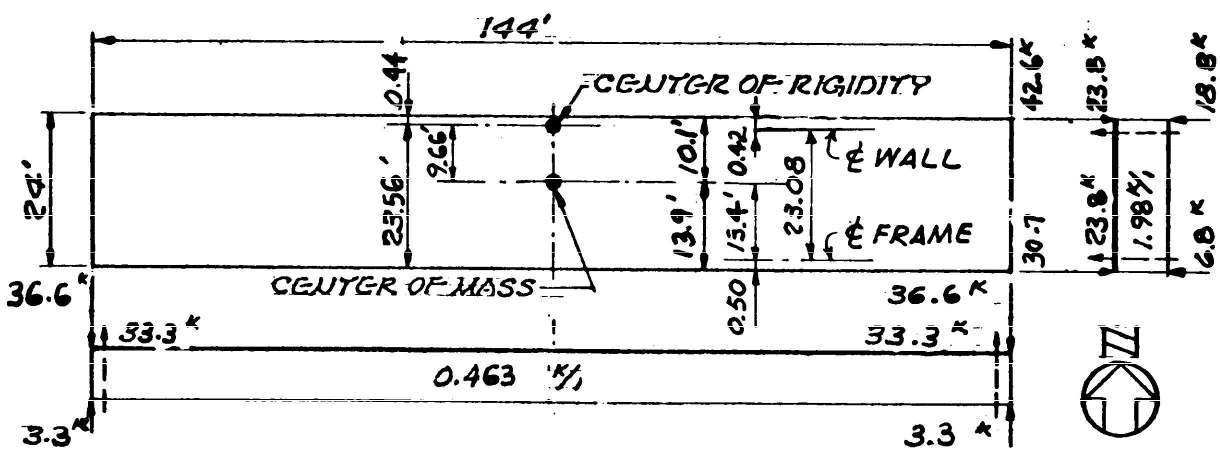
SUMMARY

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 = 34,200
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	

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

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

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

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 (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	

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

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

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

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

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

$$= 0.0038''$$

CENTER OF RIGIDITY

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $K = 82.6 \times 23.08 = 1906$

SOUTH WALL $K = \underline{0.0445} \times 0 = \underline{0}$

$82.64 \qquad 1906$

$$\bar{X} = \frac{1906}{82.64} = 23.06$$

AT N. WALL

CENTER OF MASS

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $R = 42.6 \times 23.08 = 983.2$

SOUTH WALL $R = \underline{30.7} \times 0 = \underline{0}$

$73.3^k \qquad 983.2$

$$\bar{X} = \frac{983.2}{73.3} = 13.4'$$

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 =

$M_A = V(0.05L) = 73.3(0.05\times144) = 528_{K'}$

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 $M_T = 73.3\times9.66 = 708_{K'}$

$M_A = 73.3\ (0.05\times24) = 88_{K'}$

Figure D-7. Continued.

E-W EARTHQUAKE

NORTH WALL

FACTORED DESIGN LOAD = $1.4 \times 73.3 = 103^K$

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

SOUTH WALL

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

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

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 =
$1.4 \times 40.4 = 56.6K$; $M_{OT} = 56.6 \times 15.8' = 894 K'$

FACTORED DESIGN FORCE FOR SHEAR =
$1.4 \times 40.4 = 56.6 K$

Figure D-7. Continued.

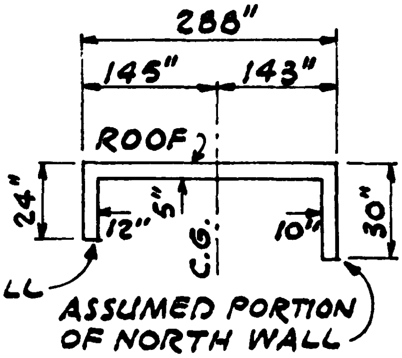
TRANSVERSE FRAMES

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



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

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

$$\Delta_f = \frac{5 \times 463 \times 143.17^4 \times 1728}{384 \times 3 \times 10^6 \times 19.12 \times 10^6} = 0.076$$

SHEARING DEFLECTION OF WEB

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

$$F = \frac{10^4}{8.5 \times 5 \times 150^{1.5}\sqrt{3000}} = 0.234$$

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

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

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

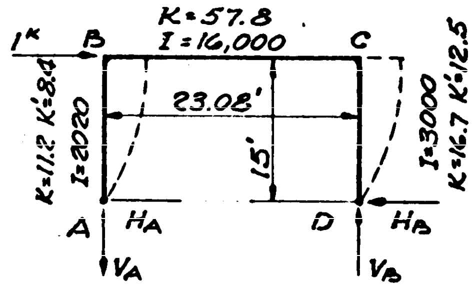
$K = \dfrac{2020}{15 \times 12} = 11.2$ FOR T-BEAM

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

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

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

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



DEFLECTION OF FRAME FOR 1K LOAD. FIXED JOINTS @ B & C
FIXED END MOMENTS OF COLUMNS $= -\dfrac{3EK\Delta}{L}$

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
	-.175EΔ			-.236EΔ

FIXED-END MOMENTS:

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

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

FINAL MOMENTS FOR $P = 1^K$:

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

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

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

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

FRAME DEFORMATION COMPATIBILITY

FOR REQ'D FRAME DEFLECTION OF 0.207";

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

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

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.

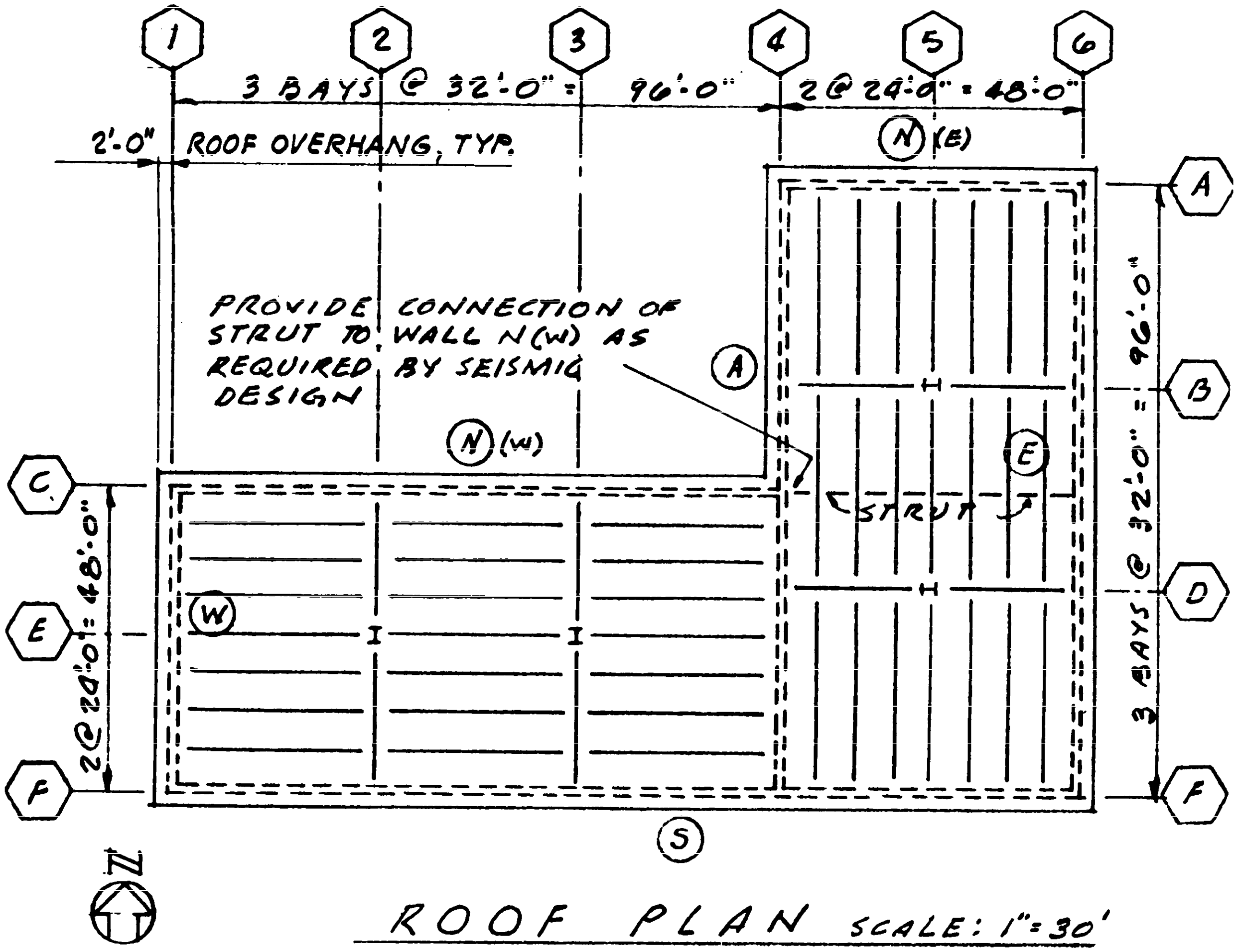
Roof:

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.

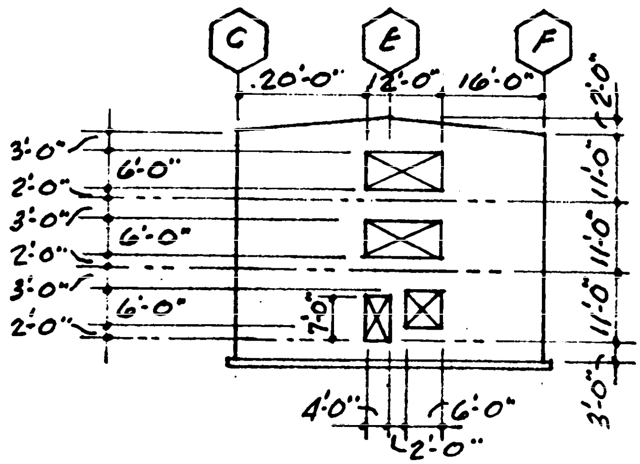
2nd & 3rd Floors:

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'

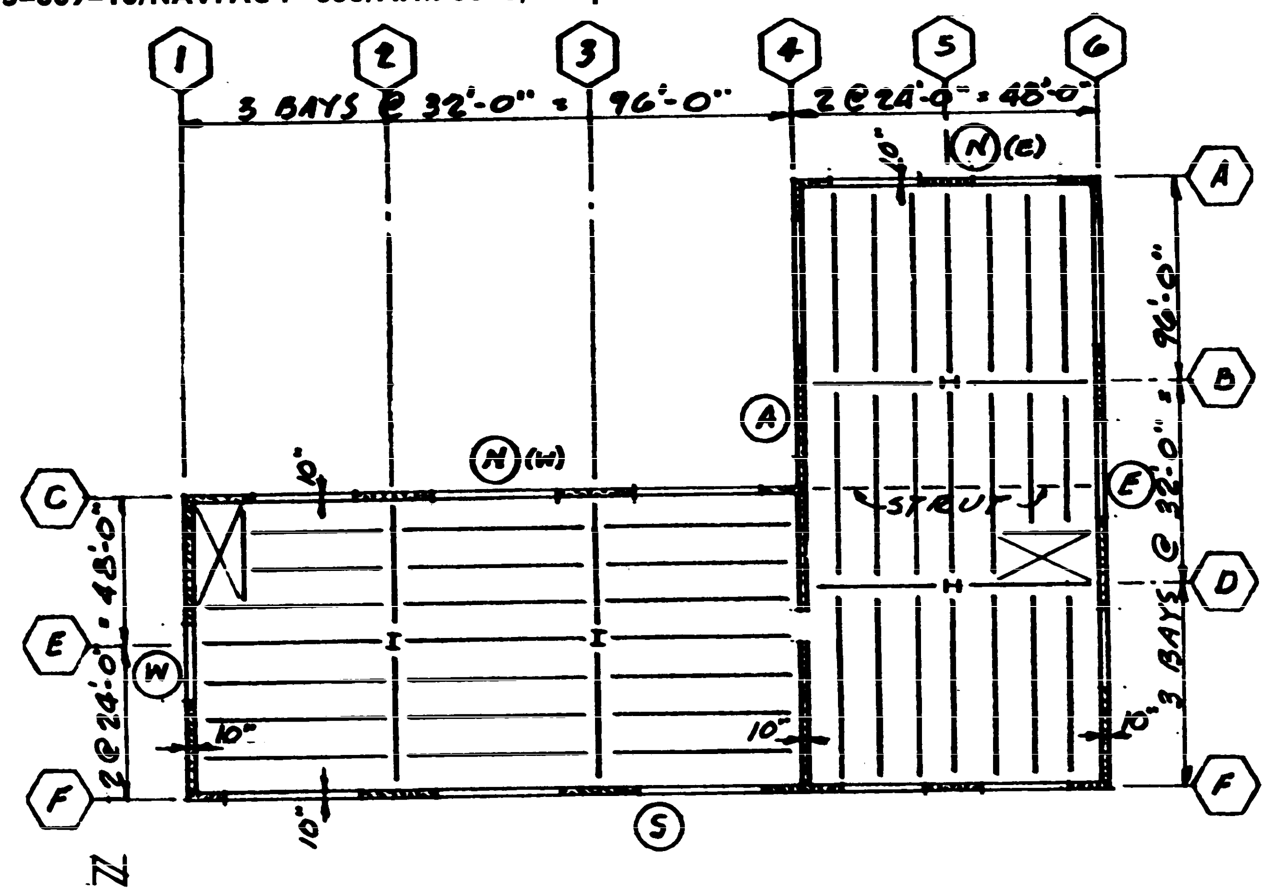


WALL (W)

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.

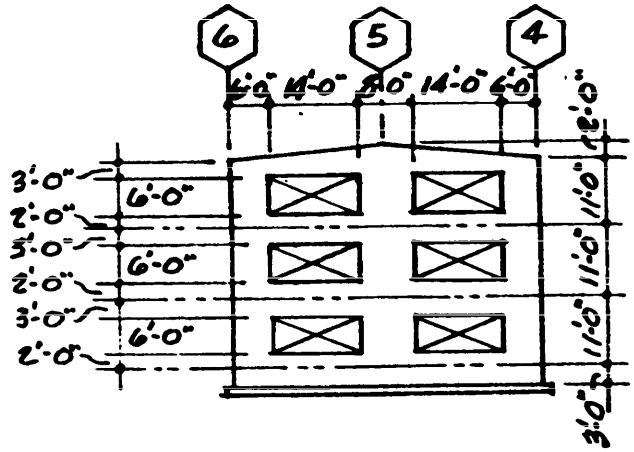
Figure D-8. Continued.



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.

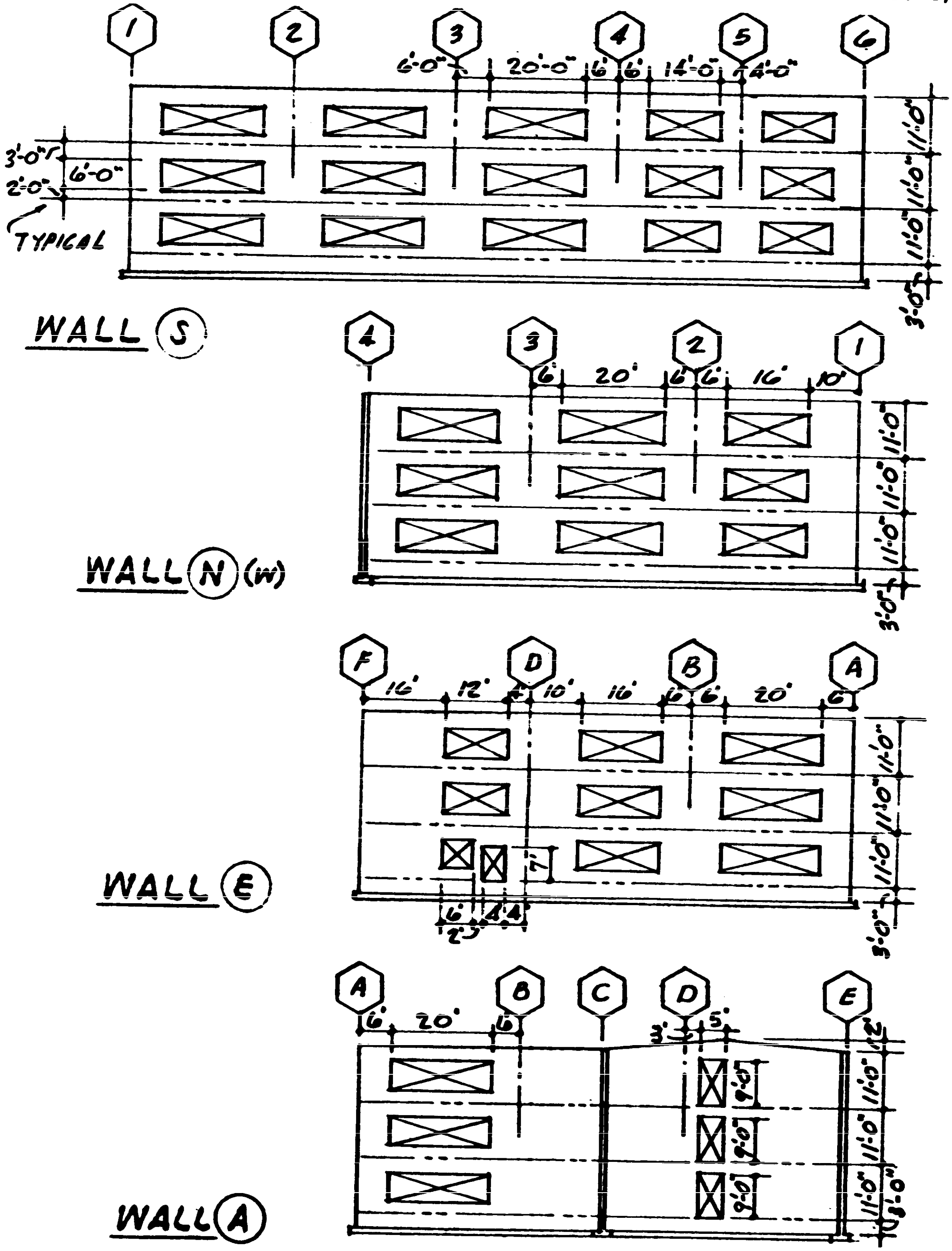


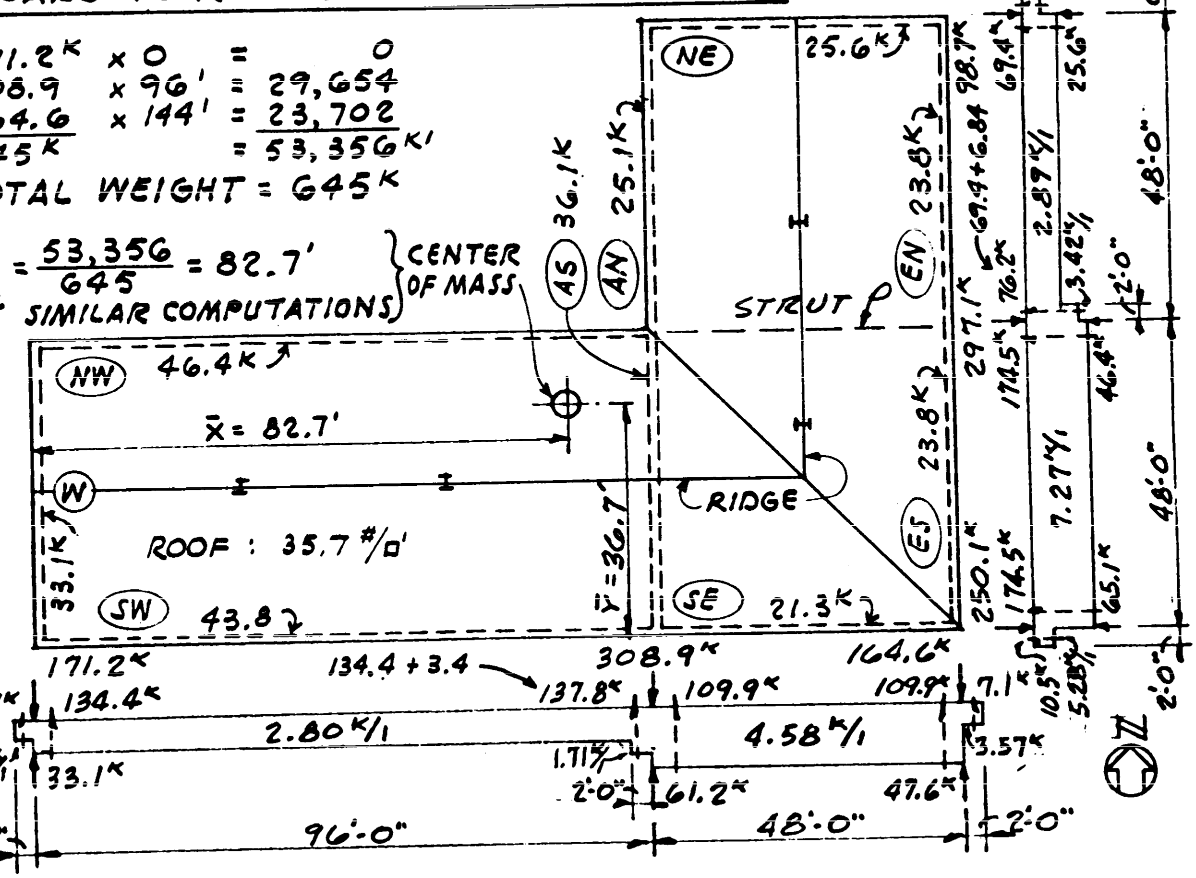
Figure D–8. Continued.

LOADS TO ROOF DIAPHRAGM 100% g

$171.2^K \times 0 = 0$
$308.9 \times 96' = 29{,}654$
$164.6 \times 144' = 23{,}702$
$645^K \quad = 53{,}356^{K'}$

TOTAL WEIGHT = 645^K

$\bar{X} = \frac{53{,}356}{645} = 82.7'$ } CENTER OF MASS
$\bar{Y}$ SIMILAR COMPUTATIONS



WALL	SOLID WT. #/1	% TOTAL AREA	NET WT. #/1	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	813	.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 #/1		948	1013	1981	1031
				ROOF WIDTH		52'	100'	148'	52'
				ROOF WEIGHT #/1		1856	3570	5284	1856
				TOTAL #/1		2804	4583	7265	2887
				K/1		2.80	4.58	7.27	2.89

10" WALL WT.
125 #/# × 6.5' = 813 #/1 *
125 × 5.5' = 687 #/1
*GREATER HT. TO RIDGE

Figure D–8. Continued.

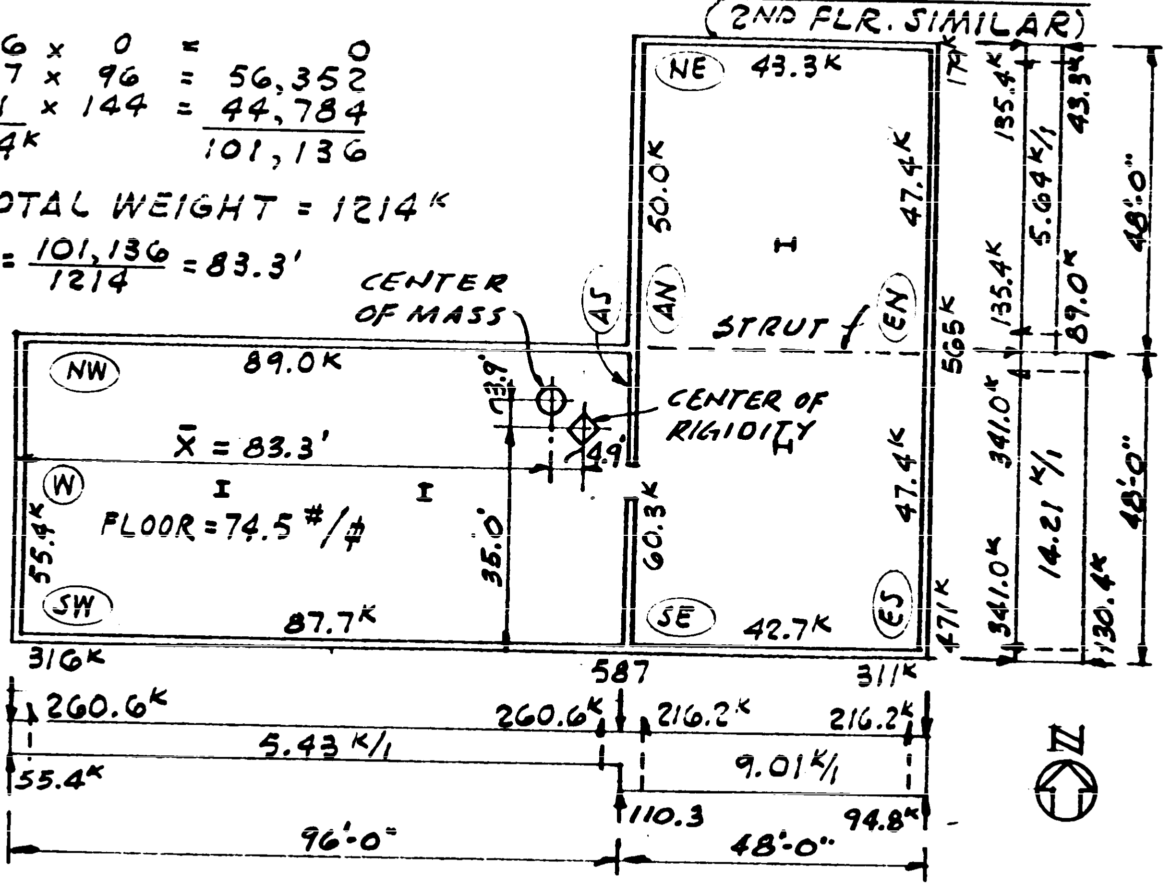
LOAD TO FLOOR DIAPHRAGM 100% 3RD FLR.

(2ND FLR. SIMILAR)

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

TOTAL WEIGHT = 1214^K

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



WALL	SOLID WT. #/l	%TOTAL AREA	NET WT. #/l	L'	W^K	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.				WALLS #/l		1856	1856	3479	2063
125#/□ × 11' = 1375#/l				FLOOR WIDTH		48'	96'	144'	48'
				FLOOR WEIGHT		3578	7152	10728	3576
				TOTAL #/l		5434	9008	14,207	5639
				K/l		5.43	9.01	14.21	5.64

Figure D-8. Continued.

LATERAL LOADS

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

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

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

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

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

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

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

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

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

	h_x	Δh	w_x	$w_x h_x$	$\frac{w_x h_x}{\Sigma w_x h_x}$	F_x	V_r	$V_x h = \Delta M_x$	M_v
ROOF	33	11	645	21285	.347	149	149	1639	
3RD FLR.	22	11	1214	26708	.435	187	336	3696	1639
2ND FLR.	11	11	1214	13354	.218	94	430	4730	5335
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 $\quad \frac{F}{W} = \frac{149.5}{645} = 0.232$

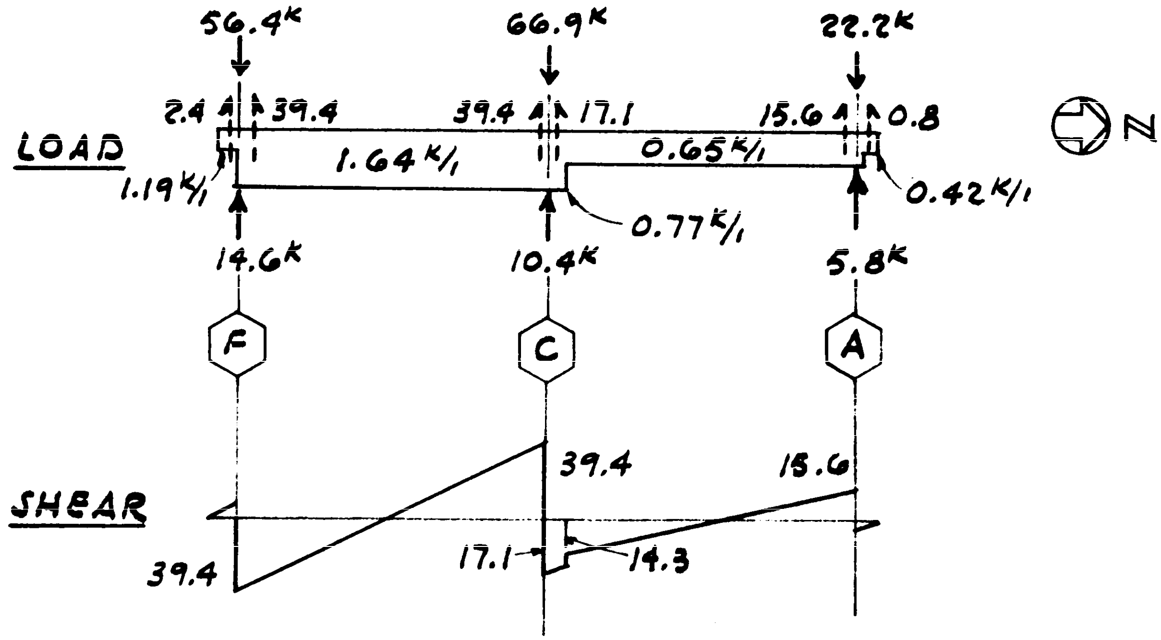
DIAPH. FORCE PER SEAOC 1-11

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

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

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: $V = \frac{48'}{144'} \times 39.4 = 13.1$

$\Big\} 28.7^K$

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.

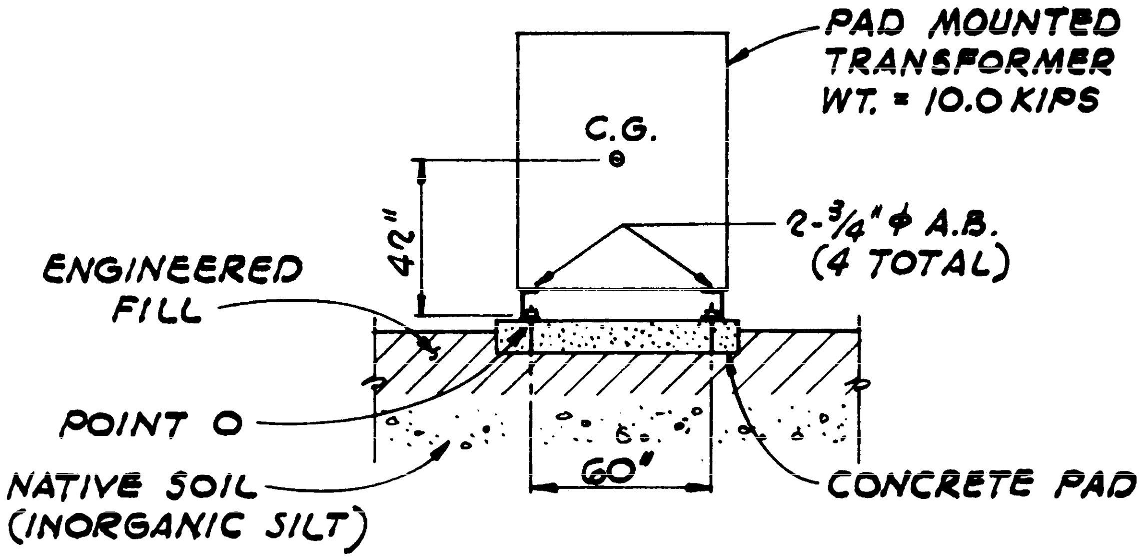
APPENDIX E

DESIGN EXAMPLES—MECHANICAL AND ELECTRICAL EQUIPMENT

E-1. Introduction. The design examples in this appendix are to illustrate principles, factors, and concepts involved in seismic design. These are not mandatory; and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

E-2. Design Examples:

Fig. No.	*Description of Design Examples*
E-1	*Pad-Mounted Transformer.* Illustrates the seismic design of a typical, rigidly mounted item of equipment on the ground.
E-2	*Cooling Tower in Building.* Presents analysis for a rigidly mounted cooling tower in a multi-story building.
E-3	*Unit Heater—Flexible Brace.* Analysis of a unit heater not rigidly braced.
E-4	*Unit Heater—Rigid Support.* Demonstrates the reduction of the lateral seismic load by rigidly bracing the unit heater of figure E-3.
E-5	*Water Heater.* Indicates how a water heater in a barracks is investigated for seismic loads.
E-6	*Tank on a Building.* Demonstrates the seismic analysis of a storage tank on a building. Emphasis is placed on the period determination.
E-7	*Water Riser.* Illustrates an approximate scheme used to determine the seismic loading on pipe connections. A riser in a multi-story building is treated.



GIVEN: W = 10.0 KIPS
RIGID EQUIPMENT ON THE GROUND
ZONE 3 SEISMIC AREA AND I = 1.0

REQUIRED: CHECK ANCHOR BOLT REACTIONS DUE TO SEISMIC LOADS.

SOLUTION:

$$F_p = ZI\,(2/3\ C_p)\,W_p \qquad \text{(EQ 12-3)}$$

$Z = 0.30,\ I = 1.0,\ C_p = 0.75,\ W_p = 10.0$ KIPS

$F_p = 0.30\,(1.0)(2/3)(0.75)(10) = 1.5$ KIPS

APPLIED AT CG

SHEAR/BOLT = 1.5/4 = 0.38 KIPS/BOLT

ALLOW. SHEAR = 1.50 KIPS/BOLT

∴ 4-3/4" φ A.B. O.K.

CHECK OVERTURNING -

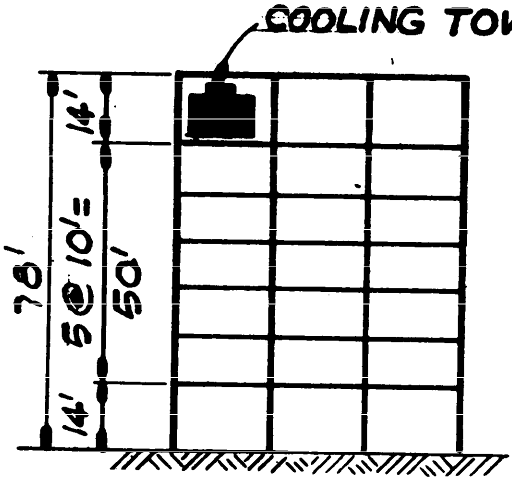
$\Sigma M_O = 0$

$42'' \times 1.5^K << \frac{60''}{2} \times 10.0^K$ ∴ OVERTURNING O.K.

Reference: Chapter 12, paragraph 12-5a

Also Check SEAOC 1I per SEAOC 1G 2d
Rigid Non-Building Structure
V = 0.5 ZIW (SEAOC EQ 1-12)
= 0.5 x 0.3 x 1 x 10 = 1.5 KIPS   Same as Above

Figure E-1. Pad-mounted transformer.



GIVEN:

WT. COOLING TOWER = 20.0 KIPS
ZONE 3 SEISMIC AREA
CONSIDER TOWER RIGIDLY MOUNTED
WT. TYP. FLOOR = 400 KIPS
100 % MOMENT RESISTING FRAME. I = 1.0.

REQUIRED:

FIND THE SEISMIC DESIGN FORCE TO BE APPLIED AT C.G. OF COOLING TOWER.

SOLUTION:

CHECK MASS RATIOS (PARA. 12-2d)
W_p/w_x FLOOR 20/400 < 0.20 O.K.
W_p/W STRUCT. 20/2800 < 0.10 O.K.

QUALIFIES AS RIGID EQUIPMENT, RIGIDLY MOUNTED IN A BUILDING (PARA. 12-3).

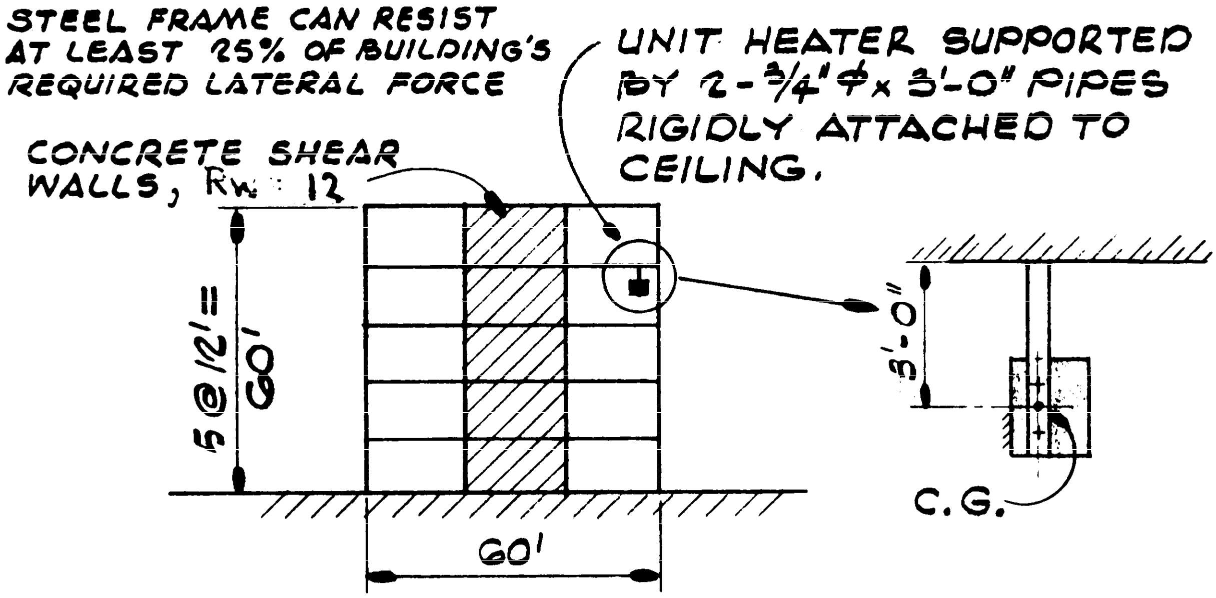
$F_p = Z I_p C_p W_p$ (SEAOC EQ 1-10)

$Z = 0.30$ (ZONE 3), $I = 1.0$

$C_p = 0.75$ (SEAOC TABLE 1-H)

$F_p = 0.30 \times 1.0 \times 0.75 \times W_p = 0.225\,W_p$
$= 0.225 \times 20 = \underline{\underline{4.5 \text{ KIPS}}}$

Figure E-2. Cooling tower in building.



GIVEN: NEGLECT EFFECTS OF ROTATION OF UNIT HEATER.

W_p = WT. UNIT HEATER = 450 LBS
w_x = WT. TYPICAL FLOOR = 500 KIPS
W = WT. STRUCTURE = 2300 KIPS
I (OCCUPANCY) = 1.0
ZONE 3 SEISMIC AREA
I_o (3/4"Φ PIPE) = 0.037 IN^4
E (PIPE) = 30×10^3 $KIPS/IN^2$

REQUIRED: FIND DESIGN SEISMIC FORCE TO BE APPLIED AT C.G. OF UNIT HEATER.

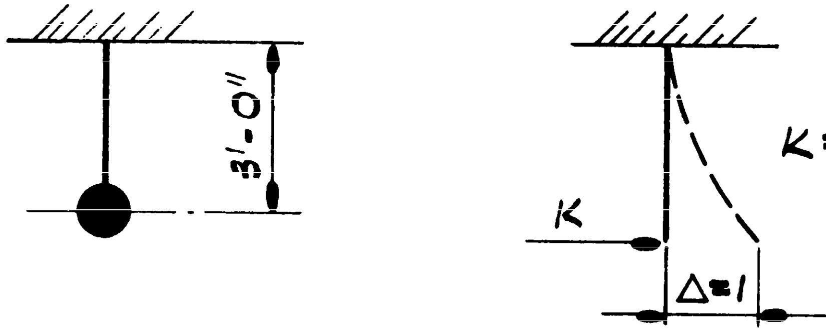
SOLUTION: CHECK MASS RATIOS: (PARA. 12-2d)
W_p/w_x FLOOR $= 0.45/500 \ll 0.20$ OK
W_p/W STRUCT. $= 0.45/2300 \ll 0.10$ OK

INVESTIGATE AS FLEXIBLY MOUNTED EQUIPMENT IN BUILDINGS PARA. 12-4

$$F_p = Z I_p A_p C_p W_p \qquad \text{(EQ 12-3)}$$

Figure E-3. Unit heater—flexible brace.

$Z = .30$ (ZONE 3), $I = 1.0$, $C_p = 0.75$
A_p, WHICH RANGES FROM 1.0 TO 5.0 IS DEPENDENT ON PERIODS T_a (EQUIP.) AND T (BLDG.)
REFER TO PARA. 12-4a.



$$K = \frac{3EI}{L^3}$$

$$k = 2\left\{\frac{3(30\times10^3)(0.037)}{36^3}\right\} = 0.142 \text{ KIPS/INCH.}$$

$$T_a = 0.32\sqrt{\frac{W_p}{k}} = 0.32\sqrt{\frac{.35}{.142}} = 0.50 \text{ SEC.} \qquad (10\text{-}1)$$

T = 0.6 SEC. (FROM ANALYSIS OF BUILDING, SEAOC EQ 1-5 AND PARA 12-4c (1))

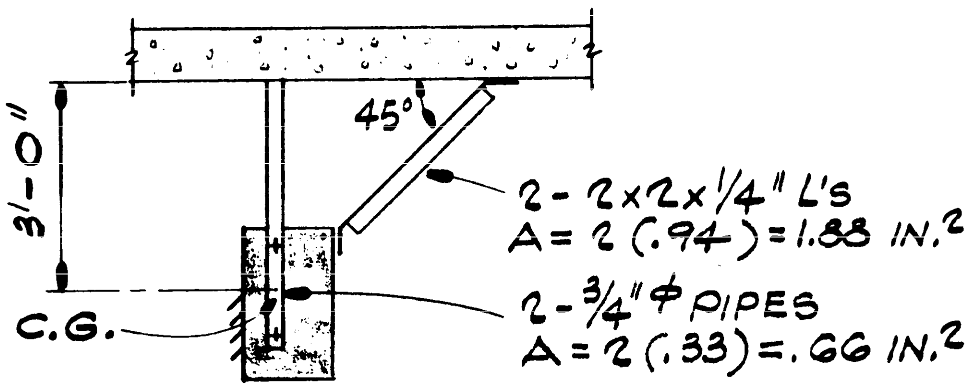
$$\frac{T_a}{T} = \frac{0.50}{0.60} = 0.83$$

USE FIGURE 12-3b: $A_p = 4.90$ (TABLE 12-1)

$$F_p = 0.30 \times 1.0 \times 4.9 \times 0.75\, W_p = 1.10\, W_p$$
$$= 1.10 \times 350 = \underline{\underline{386 \text{ LBS.}}}$$

NOTE: A LATERAL FORCE OF 386 LBS. WILL OVERSTRESS THE ¾"φ PIPE BRACES; THEREFORE ADD DIAGONAL SUPPORTS AS SHOWN IN FIGURE E-4.

Figure E–3. Continued.



DETAIL OF UNIT HEATER

GIVEN : USE DATA GIVEN IN FIGURE E-3

REQUIRED : FIND DESIGN SEISMIC FORCE

SOLUTION : $F_p = Z I_p C_p W_p$ (SEAOC EQ 1-10
IF RIGIDLY MOUNTED, PARA. 12-3)

CALCULATION OF T_a FOR RIGIDITY CHECK:

APPROXIMATE ANGLE CONNECTIONS BY PINS. ASSUME ALL LATERAL FORCE IS RESISTED BY BRACING ANGLES. USE ENERGY METHOD TO CALC. K_2 .

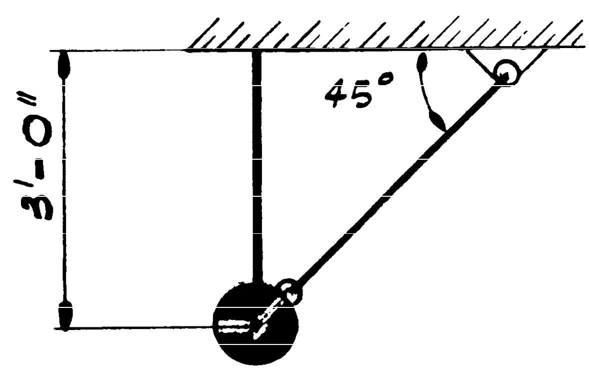
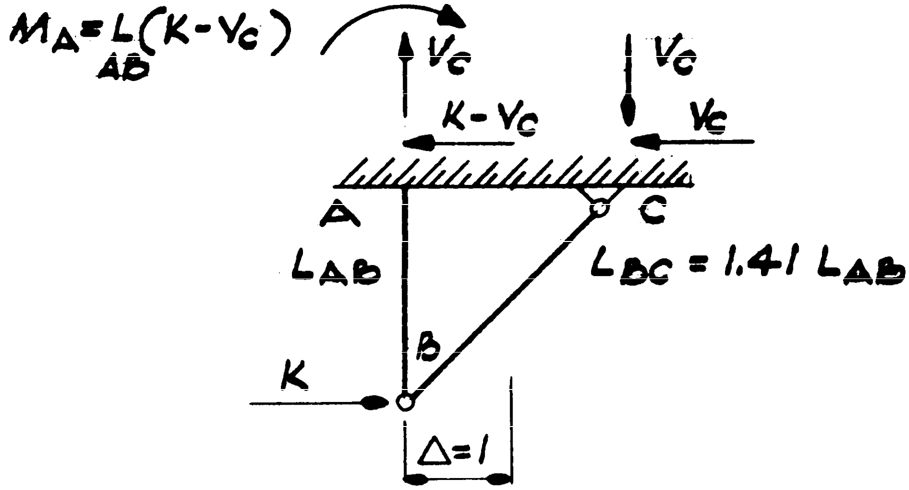


Figure E-4. Unit heater—rigid support.



ASSUME $K - V_C \doteq 0$: THIS ASSUMES ALL OF THE HORIZONTAL FORCE K IS RESISTED BY THE DIAGONAL.

$$\Sigma W_{\text{EXTERNAL}} = \Sigma W_{\text{INTERNAL}}$$

$$K\left(\frac{\Delta}{2}\right) = \frac{K^2 L_{AB}}{2 A_{AB} E} + \frac{(1.41K)^2 L_{BC}}{2 A_{BC} E}$$

$$1 = K\left(\frac{L_{AB}}{A_{AB} E} + \frac{1.41^3 L_{AB}}{A_{BC} E}\right)$$

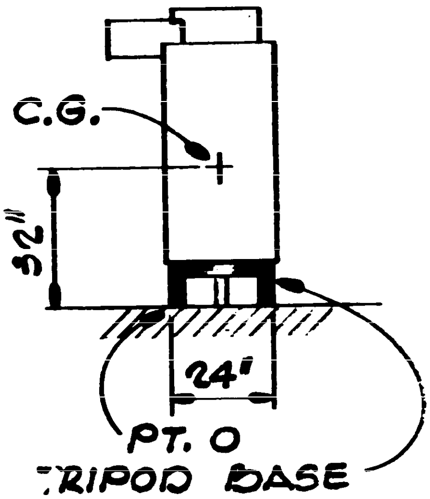
$$K = \frac{30 \times 10^6}{\left(\frac{36}{0.66} + \frac{1.41^3(36)}{1.88}\right)} = 2.78 \times 10^5 \text{ LBS/INCH}$$

$$T_a = 0.32\sqrt{\frac{350}{2.78 \times 10^5}} = 0.011 \text{ SEC.} \qquad \text{(EQ 12-1)}$$

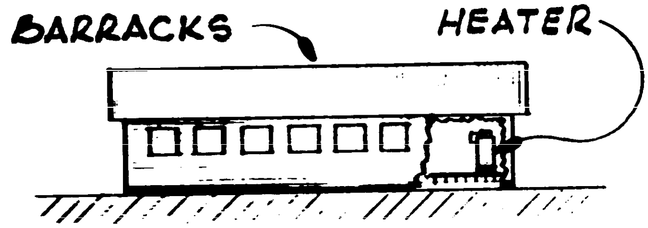
$T_a < 0.06$ SEC., THEREFORE SUPPORT IS RIGID (PARA. 12-3)

$$F_p = Z I_p C_p W_p = 0.30 \times 1.0 \times 0.75\ W_p = 0.225\ W_p$$
$$= 0.225 \times 350 = \underline{79 \text{ LBS.}}$$

Figure E–4. Continued.



GIVEN: 1445 LB. WATER HEATER IN BARRACKS, SEISMIC ZONE 4.



REQUIRED: INVESTIGATE THE WATER HEATER FOR SEISMIC LOADS.

SOLUTION: WATER HEATER WILL BE CLASSIFIED AS BEING EQUIPMENT ON THE GROUND AND WILL BE CONSIDERED TO BE A RIGID BODY. SINCE FRICTION CANNOT BE USED TO RESIST LATERAL SEISMIC FORCES, THE WATER HEATER MUST BE RIGIDLY ATTACHED TO ITS FOUNDATION. BOLT WATER HEATER LEGS TO FLOOR.

$F_p = ZI(2/3\ C_p)\ W_p$

$Z = 0.4,\ I = 1.0,\ C_p = 0.75$ (SEAOC TABLE I-H)

$F_p = 0.4 \times 1.0 \times 2/3 \times 0.75 = 0.20 W_p$

$F_p = 0.20\,W_p = 0.20 \times 1.445 = 0.29$ KIPS

$F_p = 0.29K$ APPLIED AT C.G.

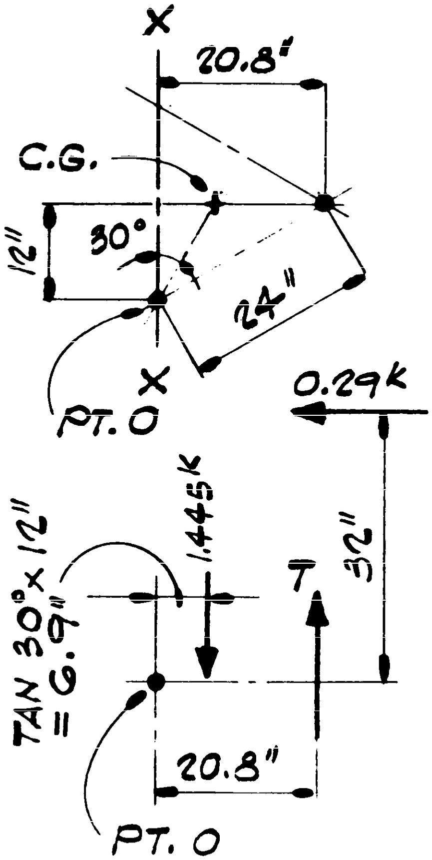
CHECK FOR OVERTURNING ABOUT POINT O.

$\Sigma M_{x-x} = 0$

$0.29^{K} \times 32'' < 1.445^{K} \times \text{TAN. } 30^\circ \times 12''$

$9.28''^{K} < 10.0''^{K}$

OVERTURNING O.K.



CHECK FOR LOAD T IN LEG OF TRIPOD.

$\Sigma M_{x-x} = 0 = T \times 20.8 + 0.29 \times 32 - 1.445^{K} \times \text{TAN } 30^\circ \times 12''$

$T = \dfrac{-9.28''^{K} + 10.0''^{K}}{20.8''} = 0.035$ KIPS COMPRESSION

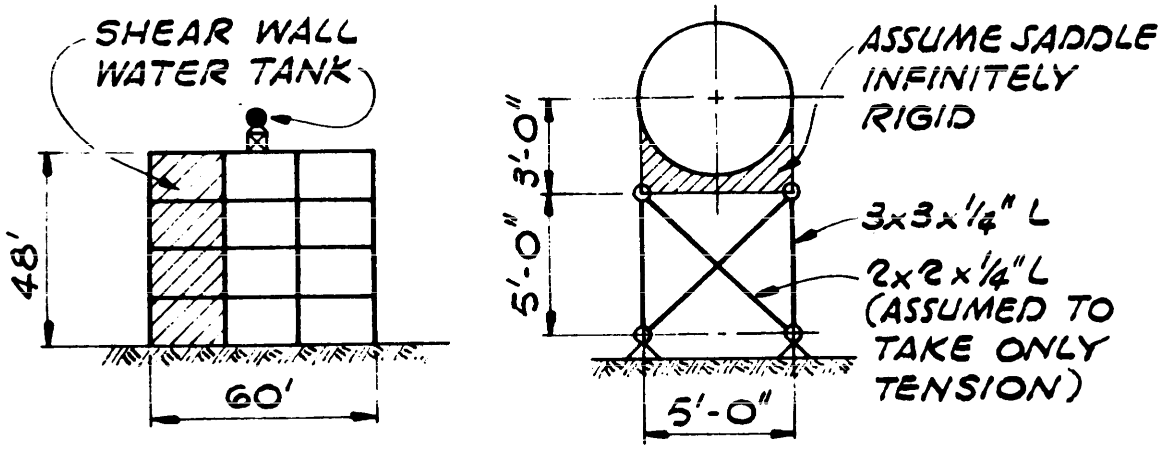
HENCE, USE NOMINAL ANCHOR BOLTS. USE 3-5/8" ϕ A.B.

ALLOW BASE SHEAR = $3(1.0^{K}) = 3K$

SHEAR O.K. $3.0K \gg 0.29K$

Note: SAME RESULTS IF CONSIDERED
NON-BUILDING STRUCTURE
V = 0.5 ZIW (SEAOC EQ 1-12)

Figure E-5. Continued.



DETAIL OF TANK SUPPORT

GIVEN: WT. OF TANK + WATER = 10.0 K/ TRUSS

ZONE 2A SEISMIC AREA AND I = 1.0 OCCUPANCY
ASSUME ALL JOINTS ARE PIN CONNECTIONS.
ASSUME CROSS MEMBERS TAKE TENSION ONLY.
NEGLECT WT. OF SUPPORT MEMBERS.

REQUIRED: FIND THE DESIGN SEISMIC FORCE.

SOLUTION: HYDRO-DYNAMIC EFFECTS ARE NEGLECTED EVEN WHEN TANK IS PARTIALLY FULL. CALCULATION OF STIFFNESS OF TANK STRUCTURE: USE ENERGY METHOD TO FIND K.

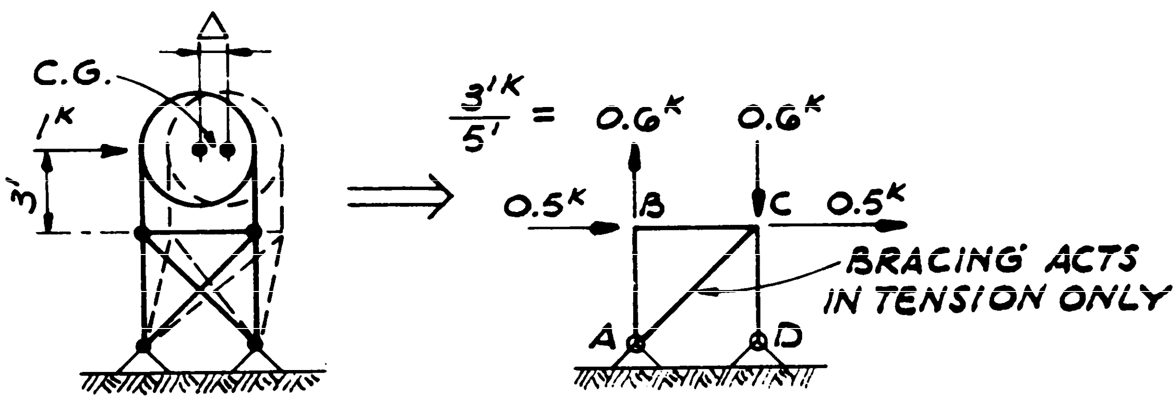


Figure E-6. Tank on a building.

COMPUTATION OF Δ : $1^K \cdot \frac{\Delta}{2} = \Sigma \frac{F^2 L}{2AE}$

MEMBER	LENGTH	AREA	F	F^2L/A
AB	5.00 FT.	1.44 IN.^{2}	+ .6 K	1.25
CD	5.00	1.44	− 1.6	8.89
CA	7.07	0.94	+ 1.414	15.03
				25.17

$$\therefore 1K \times \left(\frac{\Delta}{2}\right) = \frac{25.17 \frac{K^2-FT.}{IN^2} \times 12\ IN/FT}{2(30 \times 10^3\ K/IN^2)} = 0.5025 \times 10^{-2}\ IN.-K$$

$$\Delta = 1.005 \times 10^{-2}\ IN./K$$

$$K = \frac{1}{\Delta} = \frac{1}{1.005 \times 10^{-2}} = 99.5\ K/IN.$$

$$\therefore T_a = .32\sqrt{\frac{W}{K}} = .32\sqrt{\frac{10}{99.5}} = .102\ SEC.$$

(EQ 12-1)

T_a (EQUIPMENT PERIOD) = 0.102 > 0.06 SEC.
SUPPORT IS NOT RIGID (PARA. 12-3)
DESIGN AS FLEXIBLY MOUNTED (PARA. 12-4)

T (BLDG. PERIOD) IS CALCULATED TO BE 0.31 SEC.
REFER TO PARA. 12-4c(1)
T_a/T = 0.102/0.31 = 0.33 AND T < 0.5 SEC.
FIND A_p FROM FIGURE 12-3a

$$A_p = 1 + \left(\frac{0.33 - 0.10}{0.80 - 0.10}\right)(5.0 - 1.0) = 2.31$$

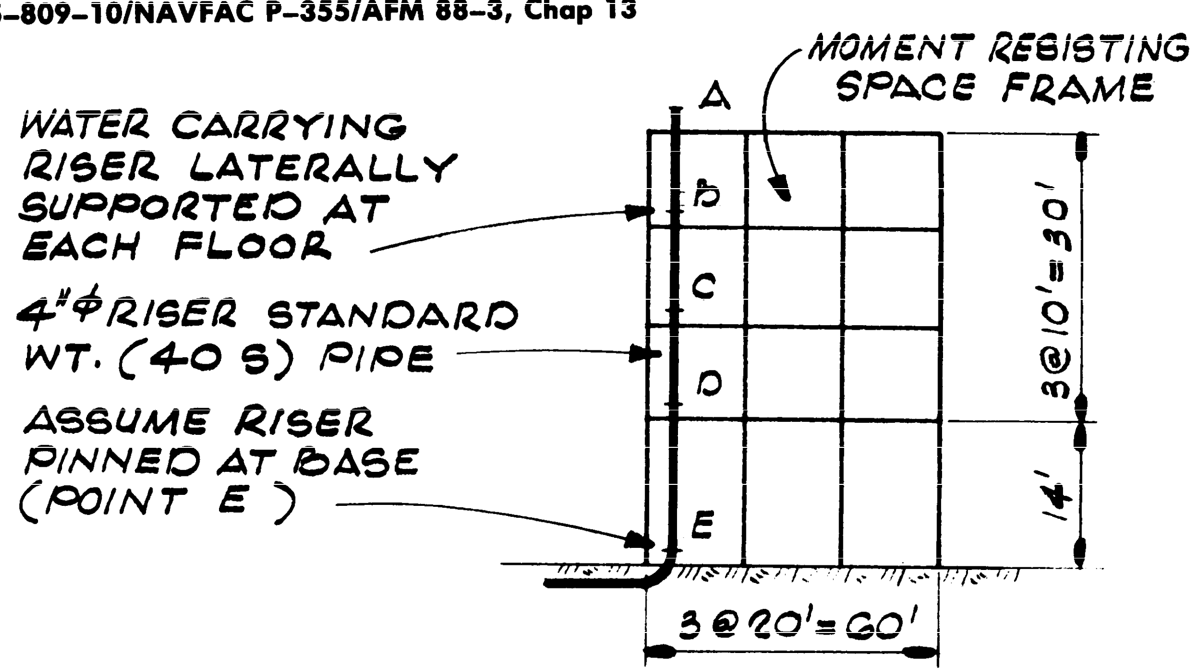
$F_p = ZIA_pC_pW_p$, (PARA. 12-4e, FORMULA 12-2)

WHERE $Z = 0.15$ (ZONE 2A)
$I = 1.0$
$C_p = 0.75$

$F_p = 0.15 \times 1.0 \times 2.31 \times 0.75\ W_p = 0.26\ W_p$

$F_p = 0.26 \times 10$ = 2.6 KIPS/TRUSS

Figure E-6. Continued.



GIVEN: RISER AS SHOWN IN MULTI-STORY BUILDING. SEISMIC ZONE 4
ESSENTIAL FACILITY BUT THE RISER IS NOT RELATED TO FIRE PROTECTION

REQUIRED: FIND SEISMIC FORCE AT EACH LATERAL RISER SUPPORT.

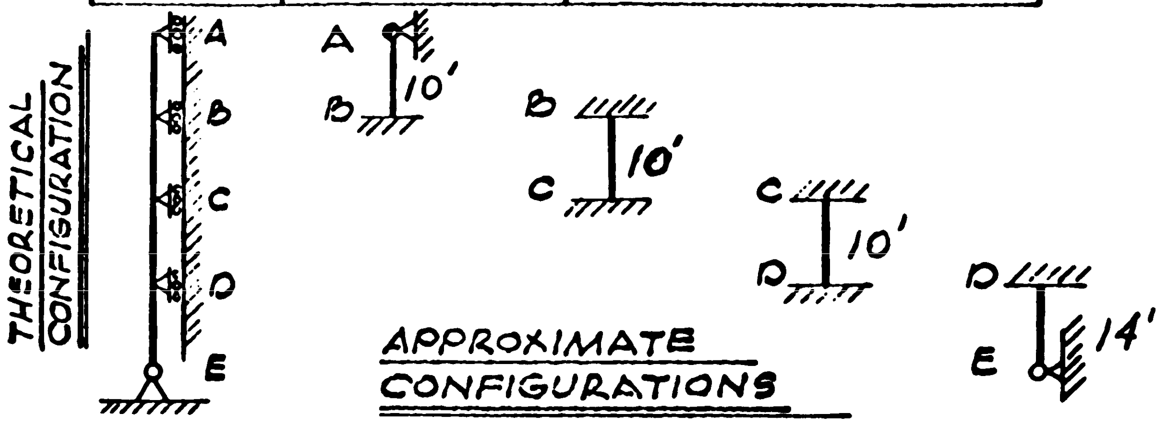
SOLUTION: AN APPROXIMATE SOLUTION WILL BE MADE.
FIRST INVESTIGATE THE ALLOWABLE SPAN FOR 4"φ (40 S) PIPE, THEN APPLY SEISMIC LOADING TO RISER.

1. IF PIPING SYSTEM IS RIGID
$F_p = Z I_p C_p W_p$ [PARA. 12-7d (2)]

2. IF PIPING SYSTEM IS NOT RIGID
$F_p = Z I_p A_p C_p W_p$ [PARA. 12-7d (3) & (4)]

Figure E-7. Water riser.

PIPE	APPROXIMATE END COND.	MAXIMUM RIGID SPANS (FIG. 12-4, 12-5 & 12-6)
A B	FIXED - PINNED	14'-6"
B C	FIXED - FIXED	17'-8"
C D	FIXED - FIXED	17'-8"
D E	FIXED - PINNED	14'-6'



PIPE SPANS ARE SHORTER THAN MAXIMUM RIGID SPAN LIMIT; $\therefore$ $F_p = Z\,I\,C_p\,W_p$ APPLIES.

$Z = 0.4$ (ZONE 4) $I_p = I = 1.25$; $C_p = 0.75$

W_p = (WT. OF PIPE + CONTENTS) = (10.8 + 5.5) LB/FT × LENGTH

$F_p = 0.40 \times 1.25 \times 0.75\,W_p = 0.38\,W_p$ = 6.1 LB/FT.

POINT	APPROXIMATE TRIBUTARY LENGTH (FT.)	APPROXIMATE CONNECTION LOAD (LBS)
A	5.0	31
B	10.0	61
C	10.0	61
D	12.0	73
E	7.0	43

Figure E-7. Continued.

APPENDIX F

DESIGN EXAMPLES—NONBUILDING STRUCTURES

F-1. Introduction. The design examples in this appendix are to illustrate principles, factors, and concepts involved in seismic design. These are not mandatory; and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

F-2. Design Examples—

Fig. No.	*Description of Design Examples*
F-1	*Elevated Tank (Braced Frame).* Four-legged, diagonal braced tower.
F-2	*Vertical Tank (On Ground).* Vertical water tank supported directly by the ground.
F-3	*Horizontal Tank (On Ground).* Typical horizontal tank supported on saddles.
F-4	*Pole-Mounted Transformer.* Equipment supported by a non-building pole structure.
F-5	*Tower-Mounted Equipment.* Tower-supported equipment is investigated for lateral seismic loads. The tower period is computed.

DESIGN EXAMPLE: F-1

ELEVATED TANK (BRACED FRAME):

Description of Structure. A 90,000 gallon steel water tank on top of a 114.5 foot high steel braced frame.

Lateral Loads.

$V = (ZIC/R_w)\ W$ (SEAOC EQ 1-1)

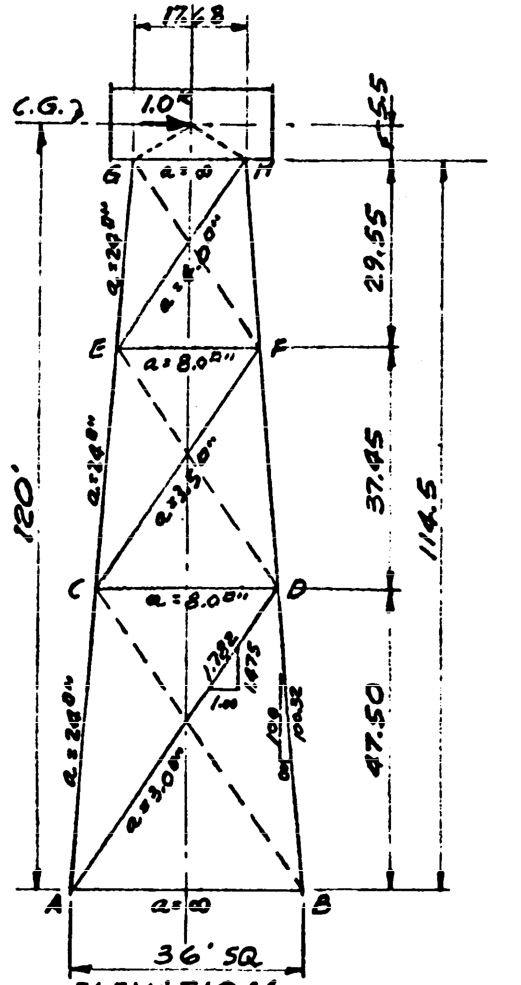
where $Z = 0.3$ (Zone 3)
$I = 1.0$
$R_w = 3$ (SEAOC TABLE 1-I)
$S = 1.5$ (Soil type S_3, SEAOC TABLE 1-B)

$T = 1.37$ (See Sheet 2)
$C = 1.25\ S/T^{2/3} = 1.52$

$V = (0.3 \times 1.0 \times 1.52/3)\ W = 0.15\ W$

MINIMUM $C/R_w = 0.50$, $V = 0.15W$ (SEAOC 1I 5a)

Figure F-1. Elevated tank (braced frame).



ELEVATION

SLOPE OF DIAGONALS IN PLANE OF SIDE

DEFLECTION Δ*

MEMBER	LENGTH	AREA	u	u^2L/A
A B	36.0	∞	+.134	0
A D	57.5'	3.0□"	+.474	4.30
A C	47.8'	24.0	+1.283	3.27
B D	47.8'	24.0	-1.678	5.60
C D	28.4'	8.0	-.296	.31
C F	45.3'	3.5	+.598	4.62
C E	37.7'	24.0	+.787	.97
D F	37.7'	24.0	-1.283	2.58
E F	22.41	8.0	-.374	.39
E H	35.8'	4.0	+.758	5.14
E G	29.7'	24.0	+.156	.03
F H	29.7'	24.0	-.787	.77
G H	17.68'	∞	-.487	0
				27.98

90,000 GALLON WATER TANK

WEIGHT OF WATER 750^k

STEEL TANK (EST) 72

WATER + TANK $822^k = W$

NEGLECT WT. OF TOWER

ASSUME BRACES CARRY TENSION ONLY.

COMPUTE THE PERIOD OF THE STRUCTURE TO DETERMINE COEFFICIENTS C AND S

$$T = 0.32\sqrt{\frac{W}{K}} \qquad \text{(EQ 12-1)}$$

$W = 822^k$

K = SPRING CONSTANT (KIPS/INCH)

IF A 1.0^k LATERAL LOAD IS APPLIED AT THE TANK C.G.,

$$K = \frac{1.0}{\Delta}$$

WHERE Δ = LATERAL DEFLECTION OF TANK DUE TO 1^k LOAD.

$$T = 0.32\sqrt{W \times \Delta}$$

$$\Delta = \frac{2^{*} \times 27.98 \times 12}{30{,}000} = 0.0224 \text{ IN}$$

FOR 1^k LOAD ON TOWER (0.5^k EACH SIDE)

$$T = 0.32\sqrt{822 \times 0.0224}$$

$= 1.37$ SEC

*0.5^k APPLIED TO EACH SIDE OF TOWER. FOR 1.0^k ON THE WHOLE TOWER:

$$\Sigma \frac{u^2L}{A} = 2 \times 27.98$$

$$\Delta = \Sigma \frac{u^2L}{AE} \text{ IN/KIP}$$

Figure F-1. Continued.

V = 0.15 W (SHEET 1 OF 3)

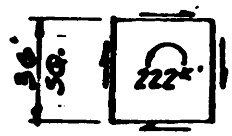
= 0.15 x 822 = 123.3 KIPS.

STRESS IN MEMBERS FOR LOAD APPLIED PARALLEL TO MAJOR AXIS. $V = 123.3^k$

MEMBER		DIRECT LOAD STRESS	ECCEN. LOAD STRESS	TOTAL STRESS	UNIT STRESS
AB	+.134 x 123.3^k	+ 16.5^k	+ 0.8^k	+ 17.3^k	—
AD	+.474	+ 58.5	+ 2.9	+ 61.4	20.5 K/□"
AC	+1.283	+158.2	0.	+158.2	6.6
BD	-1.678	-207	0.	-207	8.63
CD	-.296	- 34.5	- 1.8	- 38.3	4.79
CF	+.598	+ 73.7	+ 3.7	+ 77.4	22.1
CE	+.787	+ 97.1	0.	+ 97.1	4.05
DF	-1.283	-158.2	0.	-158.2	6.6
EF	-.374	- 46.2	- 2.3	- 48.5	6.06
EH	+.758	+ 93.4	+ 4.7	+ 98.1	24.5
EG	+.156	+ 19.2	0.	+ 19.2	0.80
FH	-.787	- 97.1	0.	- 97.1	4.05
GH	-.487	- 60.1	- 3.0	- 63.1	—

STRESSES DUE TO 5% ECCENTRICITY

$M_T = .05 \times 36 \times 123.3 = 222^{k'}$

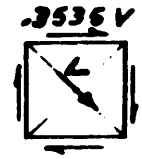


SHEAR ON EA. OF 4 SIDES = $\frac{222}{4 \times 18} = 3.08^k$

STRESS IN WEB MEMBERS = $\frac{3.08}{(123.3/2)}$ x (DIRECT LOAD STRESS)

STRESS IN COLUMNS = 0

CHECK COLUMN FORCES AND UPLIFT FOR LOAD APPLIED AT 45° TO MAJOR AXIS OF TOWER



$$P = \frac{123.3 \times 120}{1.414 \times 36} \times 1.007 = \pm 293 \text{ KIPS}$$

(NOTE: FORCE IN BD $\times \sqrt{2}$ = 207 x 1.414 = 293)

GRAVITY FORCE ON COLUMNS = $822^k \div 4$ = -206 KIPS

COLUMN DESIGN: -293 - 206 = -499 KIPS (COMPR.)

UPLIFT: +293 - 0.85(206)* = 118 KIPS (UPLIFT)

DESIGN ANCHOR BOLTS AND FOUNDATION FOR 118 KIPS UPLIFT FORCE

*REFER TO SEAOC 1H1b

Figure F-1. Continued.

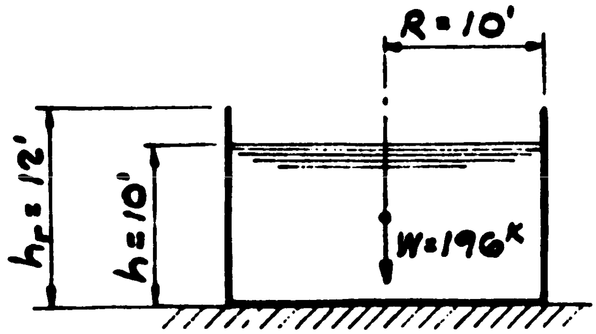
DESIGN EXAMPLE: F-2

VERTICAL TANK (ON GROUND)

Description of Structure. A cylindrical water tank on grade with a radius of 10 feet (R = 10), a height of 12 feet (h_r = 12), and a water depth of 10 feet (h = 10). The tank is located in Seismic Zone 4 and I = 1.0. The weight of the tank is 20 kips.

Required. The period of the sloshing water, the maximum vertical displacement of the water (d_{max}), and the design seismic forces.
Refer to Chapter 13, paragraph 13-4.

Figure F-2. Vertical tank (on ground).



REFER TO FIGURES 13-1 AND 13-2 FOR SEISMIC FORCE DISTRIBUTION

GENERAL

Z = 0.4, SEISMIC ZONE 4
I = 1.0
R_w = 4 (SEAOC TABLE 1-I)
C = $1.25\, S/T^{2/3} \leq 2.75$ SEAOC EQ 1-2
S = 1.5 (NOT KNOWN, SEAOC TABLE 1-B)
α = h/R = 10.0/10.0 = 1.0
W (WATER) = $\pi(10)^2(10)(0.0624)$ = 196^K
W_r (ROOF) = 0 (NO ROOF)
W_w (TANK WALLS) = 20^K

Figure F-2. Continued.

RIGID BODY FORCES [PARA. 13-4a (1)]

$$V_{RB} = ZIC/R_w \, (W_r + W_W + W_I) \qquad (13\text{-}1)$$

$$C = 2.75$$

$$ZIC/R_w = 0.4 \times 1.0 \times 2.75/4 = 0.28$$

$$W_I = 0.54\,W \quad (\text{FOR } \alpha = 1.0) \qquad (\text{TABLE } 13\text{-}1)$$
$$= 0.54 \times 196 = 106^K$$

$$V_{RB} = 0.28\,(0 + 20 + 106) = \underline{35.3^K}$$

$$h_I = 0.38h \qquad (\text{TABLE } 13\text{-}2)$$
$$= 0.38 \times 10 = 3.8 \text{ FT.}$$

$$h_I' = 0.80h \qquad (\text{TABLE } 13\text{-}2)$$
$$= 0.80 \times 10 = 8.0 \text{ FT}$$

$$M_{RB}(\text{TANK SHELL}) = ZIC/R_w \left[W_r h_r + W_W \bar{h}_W + W_I h_I\right] \qquad (13\text{-}2)$$
$$= 0.28\left[0 + 20\left(\tfrac{12}{2}\right) + 106(3.8)\right]$$
$$= \underline{146^{K\text{-}FT}}$$

$$M_{RB}(\text{BELOW BASE}) = 0.28\left[0 + 20\left(\tfrac{12}{2}\right) + 106(8.0)\right]$$
$$= \underline{271^{K\text{-}FT}}$$

Figure F-2. Continued.

SLOSHING WATER FORCE [PARA. 13-4a (2)]

PERIOD, $T = k_T\sqrt{h}$ (13-4)

$k_T = 0.84$ (TABLE 13-3)

$T = 0.84\sqrt{10} = \underline{\underline{2.66 \text{ SEC.}}}$

$V_{SL} = (ZIC/R_w)W_C$ (13-3)

$C = 1.25\,S/T^{2/3} = 0.97$

$S = 1.5$ (MAXIMUM VALUE)

$ZIC/R_w = 0.4 \times 1 \times 0.97 \times 1/4 = 0.097$

$W_C = 0.43\,W$ (TABLE 13-1)

$= 0.43 \times 196 = 84.3^K$

$V_{SL} = 0.097 \times 84.3 = \underline{8.2^K}$

$h_c = 0.60h = 0.60 \times 10 = 6.0$ FT. (TABLE 13-2)

$h'_c = 0.79h = 0.79 \times 10 = 7.9$ FT.

M_{SL} (TANK SHELL) $= (ZIC/R_w)W_C h_c$ (13-5)

$= 0.097 \times 84.3 \times 6.0$

$= \underline{49.1^{K\text{-}FT}}$

M_{SL} (BELOW BASE) $= 0.097 \times 84.3 \times 7.9$

$= \underline{64.6^{K\text{-}FT}}$

Figure F-2. Continued.

HEIGHT OF SLOSHING WATER

$$d_{MAX} = \left[\frac{0.75\,(ZIC/R_w)}{1-k_d(ZIC/R_w)}\right]R \qquad (13\text{-}6)$$

$$= \left[\frac{0.75\,(0.097)}{1-(1.75)(0.097)}\right]10.0 \qquad (k_d \text{ FROM TABLE 13-4})$$

$$= \underline{\underline{0.88 \text{ FT.}}} \quad (\text{LESS THAN } h_r - h = 2 \text{ FT, OK})$$

TOTAL DESIGN FORCES [PARA. 13-4a(3)]

$$V_{TOTAL} = \sqrt{V_{RB}^2 + V_{SL}^2} \qquad (13\text{-}8)$$

$$= \sqrt{(35.3)^2 + (8.2)^2} = \underline{\underline{36.2^K}}$$

$$M_{TOTAL} = \sqrt{M_{RB}^2 + M_{SL}^2} \qquad (13\text{-}9)$$

$$\text{FOR TANK SHELL} = \sqrt{146^2 + 49.1} = \underline{\underline{154^{K\cdot FT}}}$$

$$\text{FOR BELOW BASE} = \sqrt{271^2 + 64.6^2} = \underline{\underline{279^{K\cdot FT}}}$$

Figure F-2. Continued.

DESIGN EXAMPLE: F-3

HORIZONTAL TANK (ON GROUND):

Description of Structure. A 20,000 gallon steel tank in concrete saddles on a concrete slab on grade. Seismic Zone 2A, I = 1.0, S = 1.5 For this rigid structure T $\leq$ 0.3 sec.

Lateral Loads:

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

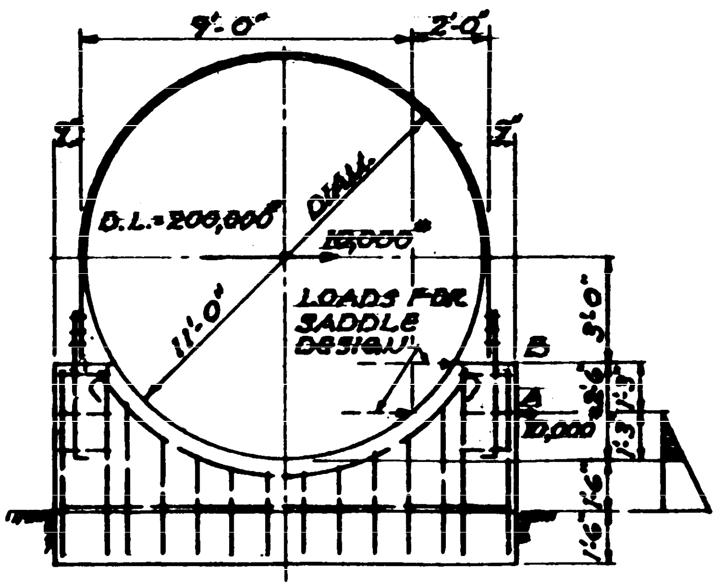
where $Z = 0.15$, $I = 1.0$, $R_w = 4$, $S = 1.5$
$C = 2.75$ (Table 4-1)
W = Weight of Tank plus contents.

$$V = \frac{0.15(1.0)(2.75)}{4} W$$

$$= 0.10\ W > 0.075\ W \quad \text{(OK)}$$

[MINIMUM $C/R_w = 0.5$; $V = 0.15 \times 0.5W = 0.075W$ (SEAOC ¶I5a)]

Figure F-3. Horizontal tank (on ground).



20,000 GALLON TANK
11'-0" DIAM. x 28'-0" LONG
WEIGHT TANK PLUS
CONTENTS 200,000 LBS.
SEISMIC LATERAL FORCE
V = 0.10 W
= 0.10 x 200,000
= 20,000 LB.
OR 10,000 LB. EA. SADDLE

STRAP DESIGN:

FOR THE PURPOSE OF THIS EXAMPLE ASSUME THE REACTION IS AT LEVEL "A" AND NEGLECT WEIGHT OF TANK AND CONTENTS.

M = 10000 x 4.25 = 42,500 '#

STRESS = 42,500/9.0 = 4,720 # IN STRAP.

SADDLE DESIGN

FOR REINFORCEMENT ASSUME THE LOAD ON THE PIER TO BE APPLIED AT LEVEL "B"

MOMENT WITH LOAD APPLIED AT LEVEL B

M = 10,000 x 2.5 = 25,000'# DESIGN REINF. TO RESIST THIS BENDING MOMENT IN ACCORDANCE WITH STANDARD PROCEDURE.

BASE DESIGN

TOTAL O.T.M. = 20,000 x 8.5 = 170,000'#

BASE 12'-6" x 24'-0" A = 12.5 x 24 = 300□'

SECTION MODULUS $S = \frac{24 \times (12.5)^2}{6} = 625$

EA. SADDLE = 5880# x 2 =	11,760#
BASE WEIGHT = 225 x 300 =	67,500#
	+200,000
	279,260# TOTAL WEIGHT

$\frac{P}{A} = \frac{279,260}{300} = 930.87 \quad \frac{M}{S} = \frac{170,000}{625} = 272$

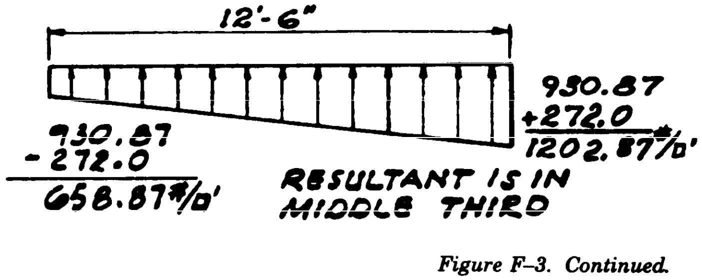
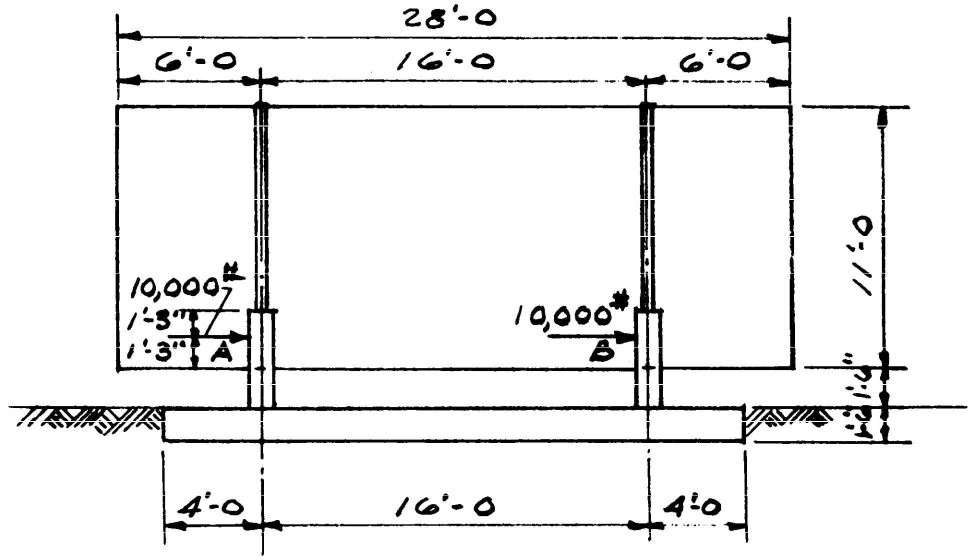


Figure F-3. Continued.



OVERTURNING ON SUPPORT IS NEGLIGIBLE AND IS NOT INCLUDED IN THIS CALCULATION

SADDLE DESIGN

$M_A = M_B$ ABOUT BASE OF TANK $= 10{,}000 \times 1.25 = 12{,}500$ '#

ABOUT FOOTING $= 10{,}000 \times 2.75 = 27{,}500$ '#

DESIGN REINF. TO RESIST THESE BENDING MOMENTS IN ACCORDANCE WITH STANDARD PROCEDURE

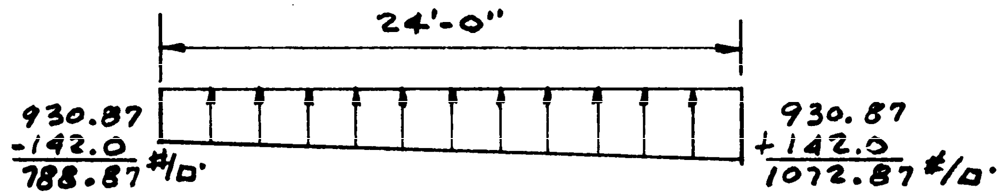
BASE DESIGN

DESIGN REINF. IN FOOTING IN ACCORDANCE WITH STANDARD PROCEDURE TO RESIST SADDLE $M = 27{,}500$ '#

TOTAL O.T.M. $= 20{,}000 \times 8.5 = 170{,}000$ '#

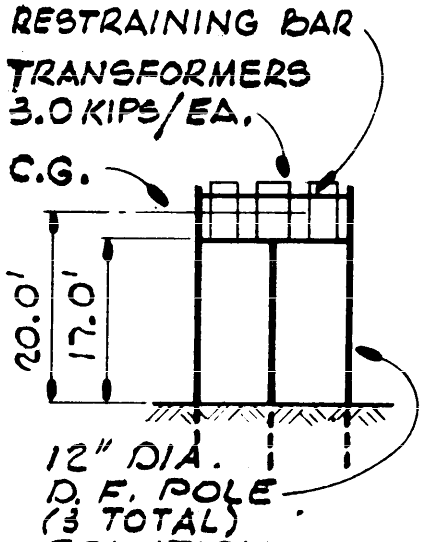
SECTION MODULUS $S = \dfrac{12.5 \times (24)^2}{6} = 1200$

$\dfrac{P}{A} = 930.87$ (FROM SHEET 2 OF 3) $\dfrac{M}{S} = \dfrac{170{,}000}{1200} = 142$



RESULTANT IS IN MIDDLE THIRD
DESIGN FOOTING FOR SOIL
PRESSURES SHOWN IN ACCORD-
ANCE WITH STANDARD
PROCEDURE.

Figure F-3. Continued.



GIVEN:

WT. TRANSFORMERS = 3.0 KIPS/EA.
WT. POLES = 35 LB/FT./POLE
E (POLES) = 1.6×10^6 LB/IN.^{2}
SOIL PROPERTIES ARE UNKNOWN
ASSUME EACH POLE ACTS AS A 20' LONG CANTILEVER
SEISMIC ZONE 3 OCCUPANCY CATEGORY 1 (ESSENTIAL FACILITY).

REQUIRED:

FIND THE SEISMIC FORCE COEFFICIENT FOR THE WEAK AXIS OF THE POLE FRAME. (I.E., NORMAL TO THE PAPER.)

SOLUTION:

CLASSIFY AS A NON-BUILDING STRUCTURE.

$$T = 0.32 \sqrt{\frac{W}{k}} \qquad \text{(EQ 12-1)}$$

$$W = 3000 + \frac{35 \times 20}{2} = 3{,}350 \text{ LB/POLE}$$

CALCULATION OF k:

$$I_o \text{ (ONE POLE)} = .785R^4 = .785(6)^4 = 1017 \text{ IN.}^4$$

$$\Delta = \frac{PL^3}{3EI_o} \quad \text{OR} \quad k = \frac{3EI_o}{L^3} = \frac{3(1.6 \times 10^6)(1017)}{(20 \times 12)^3}$$

$$= 353 \text{ LBS/IN.}$$

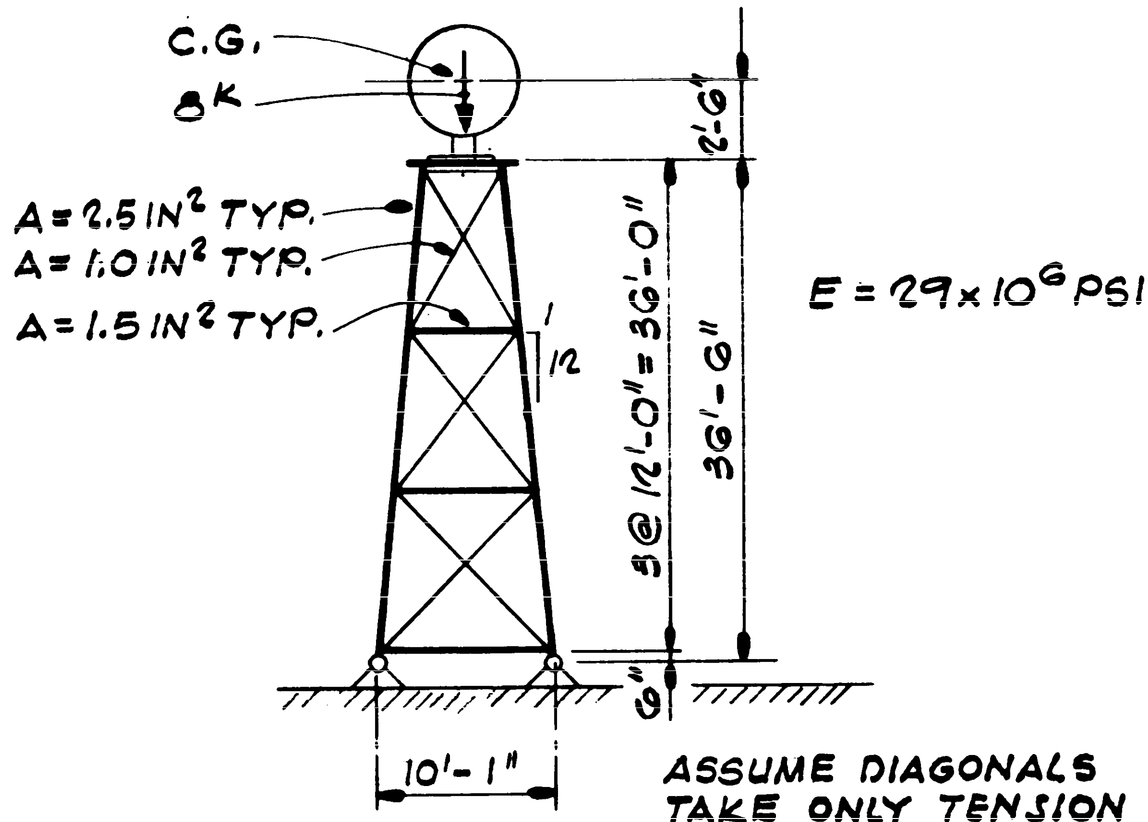
$$\therefore T = 0.32 \sqrt{\frac{3350}{353}} = 0.99 \text{ SEC.}$$

$V = (ZIC/R_w)W$ (SEAOC EQ 1-1)
$Z = 0.30$ (ZONE 3)
$I = 1.25$ (ESSENTIAL FACILITY)
$R = 3$ (INVERTED PENDULUM) SEAOC TABLE 1-I
$C = 1.88$ (TABLE 4-1 FOR $T = 1.0$ SEC)
$C/R_w = 1.88/3 = 0.63 > 0.50$ (SEAOC 1I5a)
$V = (0.30 \times 1.25 \times 1.88/3)\, W = \underline{\underline{0.236W}}$

Figure F-4. Pole-mounted transformer.

GIVEN:

MISSILE TRACKING DEVICE SITUATED ON TRUSS TOWER: SEISMIC ZONE 2B
ESSENTIAL FACILITY
SITE TYPE S_3



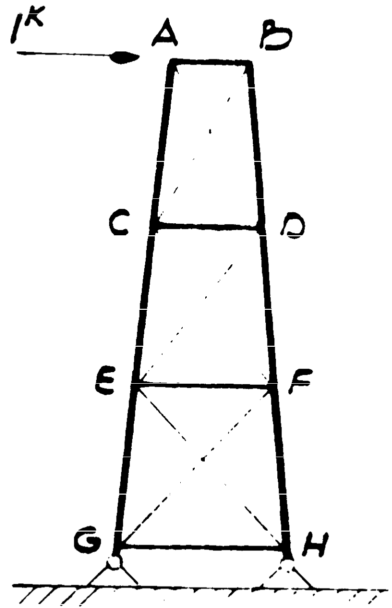
REQUIRED:

FIND THE LATERAL SEISMIC FORCE TO BE APPLIED AT THE CENTER OF GRAVITY OF THE TRACKING DEVICE. CLASSIFY AS RIGID EQUIPMENT ON A STRUCTURE OTHER THAN A BUILDING.

Figure F-5. Tower-mounted equipment.

SOLUTION:

MEMBER	P FORCE (KIPS)	L (IN.)	A (IN.^{2})	$\frac{P^2L}{A}$
AB	1.00	48	∞	0
AC	0	145	2.5	0
AD	0	156	1.0	0
BC	+2.17	156	1.0	734.6
BD	-2.02	145	2.5	236.6
CD	-0.67	72	1.5	21.5
CE	+2.02	145	2.5	236.6
CF	0	167	1.0	0
DE	+1.16	167	1.0	224.7
DF	-3.02	145	2.5	529.0
EF	-0.50	96	1.5	16.0
EG	+3.02	145	2.5	529.0
EH	0	180	1.0	0
FG	+0.75	180	1.0	101.8
FH	-3.63	145	2.5	764.3
GH	+0.30	120	1.5	7.2



NOTE: PT. H IS ASSUMED TO TAKE NO BASE SHEAR AS MEMBER EH CARRIES NO LOAD

$$1^K \cdot \frac{\Delta}{2} = \sum \frac{P^2L}{2AE} \; ; \; \sum \frac{P^2L}{A} = 3401.3\ K^2/IN.$$

$$\sum \frac{P^2L}{AE} = 1.17 \times 10^{-1} = 0.117 \text{ INCHES/KIP}$$

$$\left(\frac{1}{\Delta}\right) = k \qquad k = 8.55 \text{ KIPS/IN. PER SIDE}$$

$$T = 0.32\sqrt{\frac{W}{k}} = 0.32\sqrt{\frac{8.0}{2(8.55)}} = 0.22 \text{ SEC (EQ 12-1)}$$

$Z = 0.20$ (ZONE 2B), $I = 1.25$ (ESSENTIAL FACILITY)

$R_w = 3.0$ (INVERTED PENDULUM), $C = 2.75$ (TABLE 4-1)

$V = (ZIC/R_w)W = (0.20 \times 1.25 \times 2.75/3)W = 0.23 \times 8 = 1.84$ KIPS

NOTE: WEIGHT OF TOWER WAS NEGLECTED.

Figure F-5. Continued.

The proponent agency of this publication is the Office of the Chief of Engineers, United States Army. Users are invited to send comments and suggested improvements on DA Form 2028 (Recommended Changes to Publications and Blank Forms) direct to HQUSACE (CEMP-ET), WASH DC 20314-1000.

By order of the Secretaries of the Army, the Navy and the Air Force:

GORDON R. SULLIVAN
General, United States Army
Chief of Staff

Official:

Milton H. Hamilton

MILTON H. HAMILTON
Administrative Assistant to the
Secretary of the Army

D. E. BOTTORFF
Rear Admiral, United States Navy
Commander, Naval Facilities
Engineering Command

MERRILL A. McPEAK
General United States Air Force
Chief of Staff

Official:

WILLIAM O. NATIONS, *Colonel, USAF*
Director of Information Management

Distribution:

Army: To be distributed in accordance with DA Form 12-34-E, Block 0736, requirements for TM 5-809-10.

Air Force: F

*U.S. G.P.O.:1993-330-060