SEISMIC AND STRUCTURAL ANALYSIS

FOR THE

LACROSSE BOILING WATER REACTOR

1-B DIESEL GENERATOR BUILDING

PREPARED FOR

DAIRYLAND POWER COOPERATIVE

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1. SUMMARY

This report, prepared for Dairyland Power Cooperative (DPC) presents the results of the seismic/structural analysis of the 1-B Diesel Generator Building using the NRC site-specific ground response spectra for the Safe Shutdown Earthquake (SSE) Event. It became necessary to verify the structural adequacy of the building under the Systematic Evaluation Program (SEP). The 1-B Diesel Generator Building contains safety related equipment.

A static analysis using a coeffecient of 1.5 times the peak acceleration of the NRC site-specific ground response spectra applied to the lumped masses at the column nodes was used to evaluate the building. It has been concluded from the results of the analysis that the 1-B Diesel Generator Building is capable of withstanding a SSE seismic event. Forces and stresses in all structural elements of the building were determined. All member stresses were found to be less than their allowable values. Evaluation of the effects of the SSE on the non-structural concrete block walls is not included as part of this analysis.

2. DESCRIPTION OF THE BUILDING

The 1-B Diesel Generator Building, constructed in 1975, is a single story 31'x38' addition to the southeast corner of the Turbine Building (Site Plan, Figure 2.1). It is divided into three rooms containing electrical equipment, batteries and the diesel generator (Floor Plan, Figure 2.2). The structural steel frame is founded on a 2' thick pile supported reinforced concrete slab. The Generator Building foundation is separated from the Turbine Building by a 2" wide joint for seismic independence. The walls of the building are constructed of nonreinforced 12" hollow concrete block. The roof consists of 3-1/2" thick precast lightweight concrete panels with the exception of a 8'-16" by 21'-10" poured reinforced concrete slab, 4-3/4" thick, which supports the diesel generator test load bank and exhaust muffler. The roof panel arrangement is shown in Figure 2.3. Bracing in the plane of the roof consists of double angles and wide flange sections. The exterior bays of the steel frame are braced diagonally with WT sections. The roof framing plan is shown in Figure 2.4. The 1-B diesel generator building is supported by 19 concrete filled pipe piles with a design load capacity of 50 tons per pile (Ref 5). The pile plan is shown in Figure 2.5.

3. APPLICABLE CODES, STANDARDS AND SPECIFICATIONS

The following codes of practice and regulatory guides were used in the analysis of the 1-B Diesel Generator Building.

1. Steel Construction Manual, AISC, 8th Edition, New York, NY.

2. USNRC Regulatory Guide 1.92, "Combination of Modes and Spatial Components in Seismic Reponse Analysis", Revision 1, February, 1976.

3. USNRC Standard Review Plan Section 3.7.2.

4. Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76), American Concrete Institute, Detroit, MI., 1978.

5. USNRC Standard Reivew Plan Section 3.8.4.

6. Building Code Requirements for Reinforced Concrete, (ACI 318-77), American Concrete Institute, Detroit, MI, 1977.

4. LOADS AND LOADING COMBINATIONS

The 1-B Diesel Generator Building was analyzed for each of the three components of earthquake motion (two horizontal and one vertical) individually. These results were then combined by taking the square root of the sum of the squares of the individual components to obtain the maximum effect due to simultaneous action of the three earthquake components (See USNRC Regulatory Guide 1.92).

In addition to the seismic inertia loading, the dead load and live load were included in the analysis. The building live load plan is shown in Figure 4.1 (Ref 1). The following load combination equations were used to evaluate the adequacy of the structural steel frame to withstand a seismic event using elastic working stress methods of the AISC.

1. D + L
2. D + L + E'

Seismic analysis has been performed for the SSE seismic event only since it is more limiting compared to the OBE. The magnitude of SSE acceleration is twice the OBE acceleration value for the same frequency, whereas the allowable stress values according to Standard Review Plan 3.8.4 are only 60 percent higher than that allowed for the OBE event.

The following load combination equations were used to evaluate the adequacy of the reinforced concrete roof slab and precast concrete roof panels to withstand a seismic event using the strength design method of ACI 318-77.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9E
3. D + L + E'

Both the OBE and SSE seismic events were included in the analysis of the concrete roof components. When using the strength design methods, the load factors applied to the dead and live load for the OBE load combination will cause the OBE seismic event to be more critical than the SSE.

D = Dead Load
L = Live Load (30 psf on roof)
E = OBE Seismic Loads = ½ SSE
E'= SSE Seismic Loads

5. STRUCTURAL ACCEPTANCE CRITERIA

The following allowable limits constitute the structural acceptance criteria for each of the loading combinations presented in Section 4.

Structural Steel Frame

Load Combinations	Limit
D + L	S
D + L + E'	1.6S, but not greater than Fy

Where S is the required section strength based on the design methods and the allowable stresses defined in the AISC Specifications, and Fy is the yield stress.

Reinforced Concrete Roof Slab, Precast Concrete Roof Panels

Load Combinations	Limit
1.4D + 1.7L	U
1.4D + 1.7L + 1.9E	U
D + L + E'	U

Where U is required section strength based on the strength design method defined in the ACI.

6. METHOD OF ANALYSIS

The seismic analysis of the 1-B Diesel Generator Building was performed using a static coefficient analysis. A static coefficient of 1.5 was used to take into account the effects of multifrequency excitation and multimode response (See USNRC Standard Review Plan 3.7.2). The 1.5 factor was applied to the peak acceleration from the NRC site-specific ground response spectra for 5% damping (Figure 6.1)(Ref 3). This resulted in accelerations in the two horizontal directions equal to 0.315g. The vertical acceleration was taken as 2/3 of the horizontal acceleration. This is consistent with Ref 4.

The concrete block walls were assumed not to be effective in resisting lateral loads due to the limited shear and flexural tension strength of the joint mortar. The precast concrete roof panels were not used in the analysis to transmit horizontal shear forces. They were assumed to support vertical loads only. The panels are attached to the steel roof beams at alternate corners. Therefore, all lateral loads were assumed to be resisted by the structural steel frame. A finite element model using three dimensional beam elements was made of the steel frame (See Figure 6.2 - 6.4). For the dead plus live load analysis, the roof loads were applied as distributed loads to those roof beams which support the precast concrete panels. For the seismic analysis, the roof dead load, live load, and weight of the concrete block walls was lumped at the column nodes. Each lumped mass at the column top nodes represented the contributory weight of the roof dead load, live load, and the upper one-half of the concrete block wall height. Each lumped mass at the column bottom nodes represented the contributory weight of the lower one-half of the block wall height. The steel frame weight was assumed to be negligible compared to the roof and block wall weight.

The pile foundation was analyzed for two load combinations: 1) dead plus live load, and 2) dead plus live plus seismic load. The reinforced concrete mat footing was assumed to be rigid for the pile analysis. A live load of 500 psf was applied to the mat over the area occupied by the electrical equipment room and battery room. A live load of 900 psf was applied to the mat over the area occupied by the diesel generator room. The column loads from the computer program were applied statically to the pile group. The individual pile loads were found by using standard techniques for the analysis of pile footings subjected to moment (Ref 7).

7. RESULTS OF ANALYSIS AND CONCLUSIONS

The results of the seismic analysis of the 1-B Diesel Generator Building performed with the Stardyne Computer Code are contained in Reference 2. The following items were included in the analysis; structural steel frame (including typical connection and column anchorage details), pile foundation, reinforced concrete roof slab, and precast lightweight concrete roof panels. All structural elements of the building are capable of withstanding a SSE seismic event. The results of the structural evaluation are discussed below.

Structural Steel Frame

The maximum stress for each type of member in the frame for the two load combinations is shown in Tables 7.1 and 7.2. The maximum stresses in the frame are less than the allowable values. The column anchorage details are capable of resisting the uplift and lateral loads generated during a seismic event.

Pile Foundation

A summary of the maximum and minimum pile loads is shown in Table 7.3. The piles were driven to a safe bearing capacity of 100 kips per pile (Ref 5). The maximum conpressive load on a pile (109K) is greater than its rated capacity of 100K but lower than the predicted ultimate load capacity of 400K. No tensile loads exist in any of the piles.

Reinforced Concrete Roof Slab

The maximum moment in the 4 3/4" thick reinforced concrete slab is less than the ultimate moment capacity. The actual moments versus the ultimate moment capacity are summarized in Table 7.4.

Precast Lightweight Concrete Roof Panels

The 3½" thick precast lightweight concrete roof panels were designed for a minimum safe uniform load of 65 psf with a safety factor of 4 (Ref 6). The maximum uniform lc of 138.3 psf is less than the ultimate load capacity of 260 psf. The results are summarized in Table 7.5.

TABLE 7.1
DEAD LOAD + LIVE LOAD - MAXIMUM STRESSES

Beam	Element	Axial Stress f_a	Axial Stress F_a	Major Axis Bending f_{b2}	Major Axis Bending F_{b2}	Minor Axis Bending f_{b3}	Minor Axis Bending F_{b3}	Combined Stress Ratio *
W12x36	80	--	19.31	11.90	22.0	--	27.0	0.541
W12x27	70	--	17.82	13.19	22.0	--	27.0	0.600
W12x22	--	--	--	--	--	--	--	--
W10x19	--	--	--	--	--	--	--	--
W8x24	134	2.99	14.13	--	22.0	--	27.0	0.212
W8x10	--	--	--	--	--	--	--	--
WT6x18	124	0.38	9.19	--	--	--	--	0.041
Double Angles 4x3x5/16	--	--	--	--	--	--	--	--

*Axial compression and bending stresses are combined in accordance with AISC Section 1.6. Must be ≤ 1.0 for acceptability.

TABLE 7.2

DEAD LOAD + LIVE LOAD + SEISMIC - MAXIMUM STRESSES

		Axial Stress		Major Axis Bending		Minor Axis Bending		Combined Stress Ratio*
Beam	Element	fa	$Fa_{E'}$	f_{b2}	$F_{b2E'}$	f_{b3}	$F_{b3E'}$	
W12x36	81	1.08	30.90	11.90	36.0	3.82	36.0	0.472
W12x27	70	0.35	28.51	13.19	35.2	0.65	36.0	0.405
W12x22	40,41	1.80	28.38	--	36.0	1.48	36.0	0.105
W10x19	38	5.78	27.17	--	36.0	0.60	36.0	0.233
W8x24	127	4.67	22.61	--	35.2	--	36.0	0.207
W8x10	29	5.38	28.30	--	36.0	--	36.0	0.190
WT6x18	126	6.86	11.87	--	--	--	--	0.578
Double Angles 4x3x5/16	26	4.91	29.07	--	--	--	--	0.169

*Axial compression and bending stresses are combined in accordance with AISC Section 1.6. Must be ≤ 1.0 for acceptability.

Allowable stresses equal 1.6 x normal AISC allowables but not greater than Fy.

TABLE 7.3

MAXIMUM AND MINIMUM PILE LOADS

DEAD LOAD & LIVE LOAD

	Pile No.	Actual Load	Design Load
Maximum Pile Load	4	89.52 KIPS	100 KIPS
Minimum Pile Load	17	67.12 KIPS	100 KIPS

DEAD LOAD & LIVE LOAD & SEISMIC

	Pile No.	Actual Load	Predicted Ultimate Load
Maximum Pile Load	4	109.27 KIPS	400 KIPS*
Minimum Pile Load	17	51.79 KIPS	400 KIPS*

*Assume Factor of Safety of 4 Over 100K Capacity

TABLE 7.4

REINFORCED CONCRETE ROOF SLAB

LOAD COMBINATION	ACTUAL MOMENT	ULTIMATE MOMENT CAPACITY
1.4D + 1.7L	16.91 in-K/FT	73.74 in-K/FT
1.4D + 1.7L + 1.9E	19.18 in-k/FT	73.74 in-k/FT
D + L + E'	13.76 in-K/FT	73.74 in-K/FT

TABLE 7.5

PRECAST CONCRETE ROOF PANELS

LOAD COMBINATION	ACTUAL LOAD	ULTIMATE LOAD CAPACITY
1.4D + 1.7L	109.8 PSF	260 PSF
1.4D + 1.7L + 1.9E	138.3 PSF	260 PSF
D + L + E'	87 PSF	260 PSF

8. REFERENCES

1. Diesel Generator Building Drawings, Sargent & Lundy Engineers, Diesel Generator Building S-1 through S-5, A-1 through A-4.

2. 1-B Diesel Generator Building, Stardyne Structural Analysis Project 5101, Task 245, NES Computer Output Binder S-62, October, 1981.

3. Letter to all SEP owners from NRC dated June 17, 1981 (LS05-81-06-068) subject "Site Specific Ground Response Spectra for S.E.P. Plants Located in the Eastern United States."

4. NUREG/CR-0098 "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," By N.M. Newmark & W.J. Hall. May 1978. U.S. N.R.C. Washington, D.C.

5. Specification A-4109 "Foundation Piles, LaCrosse Boiling Water Reactor Project Dairyland Power Cooperative Association," Sargent & Lundy Engineers, Chicago, Ill. January 1975.

6. Specification A-4110 "General Work for the B-1 Diesel Generator Building" Sargent & Lundy Engineers, Chicago, Ill. March 1975.

7. Peck, Hanson, Thornburn; Foundation Engineering, 2nd Edition, John Wiley & Sons, Inc., New York, N.Y., 1974, pp 392-398.

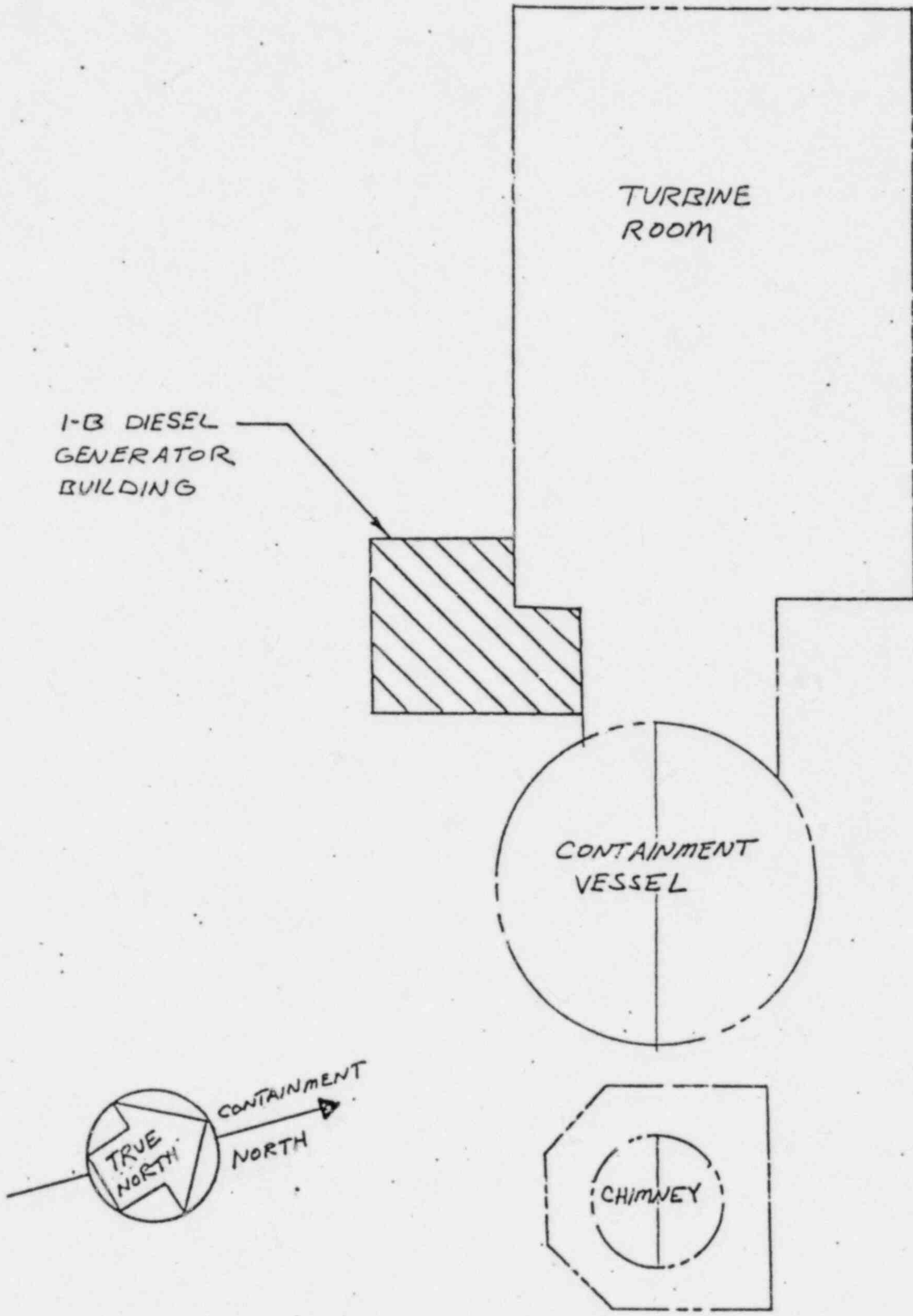


FIGURE 2.1 - SITE PLAN

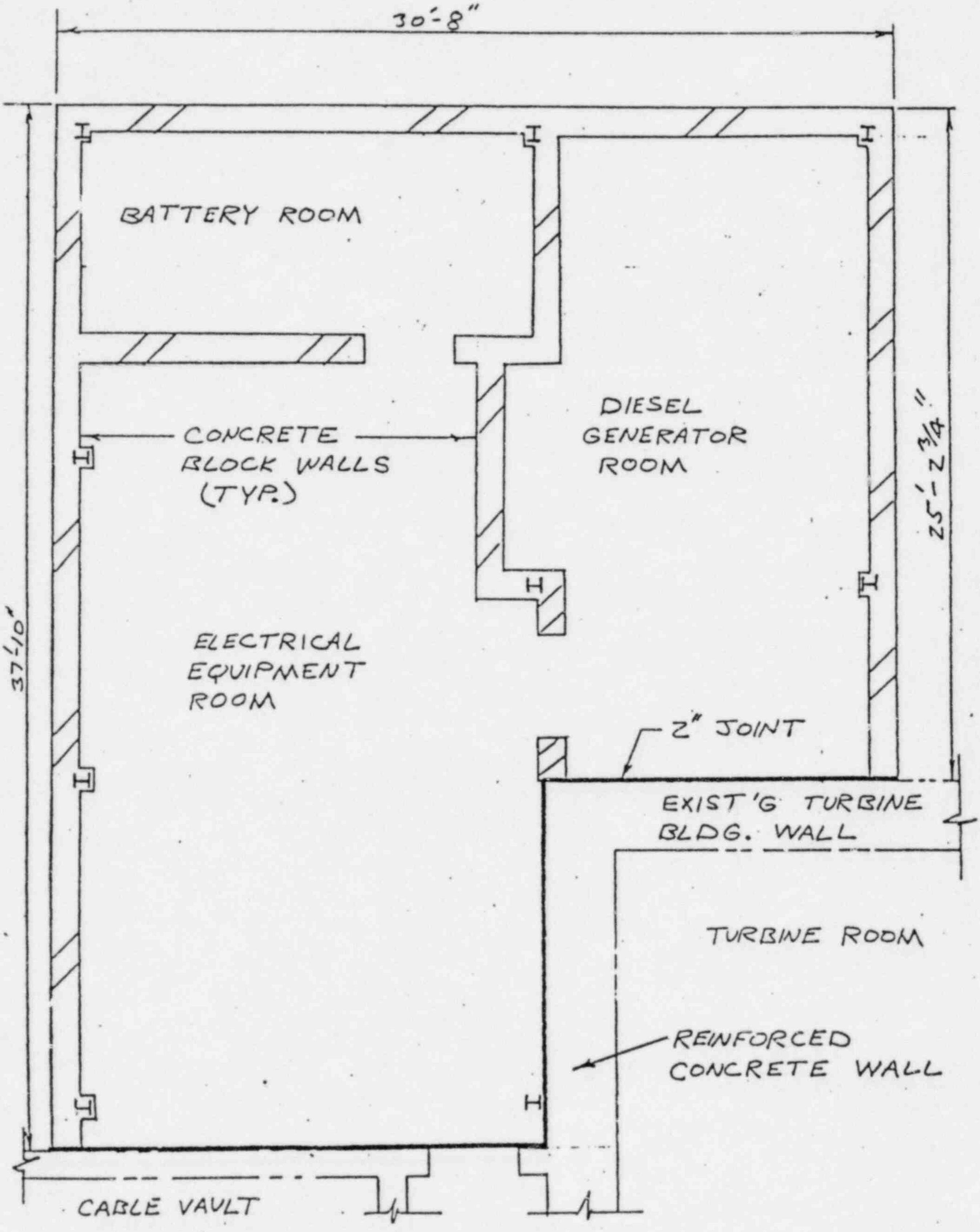


FIGURE 2.2 - FLOOR PLAN

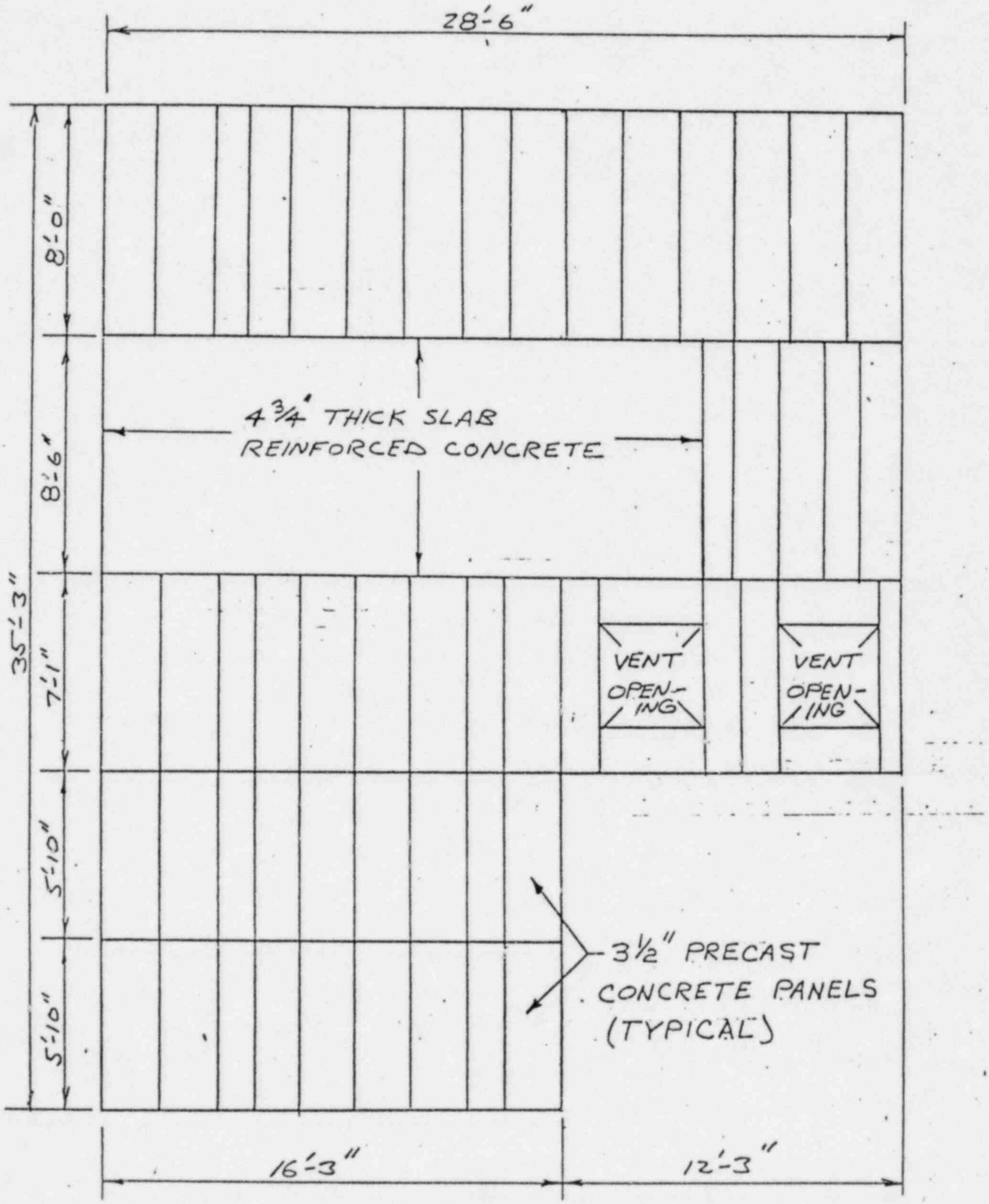


FIGURE 2.3 - PRECAST CONCRETE ROOF PANEL ARRANGEMENT

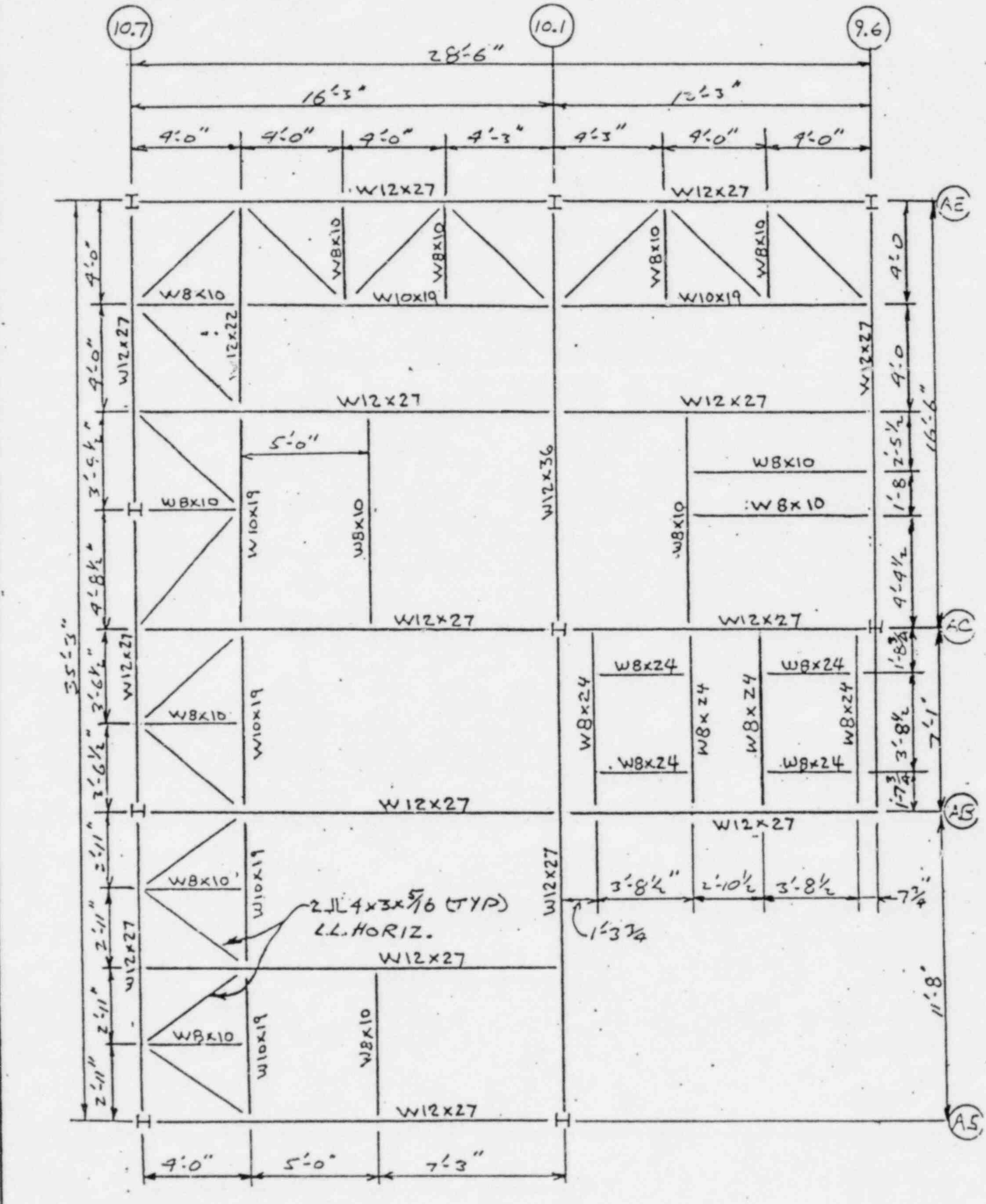


FIGURE 2.4 - ROOF FRAMING PLAN

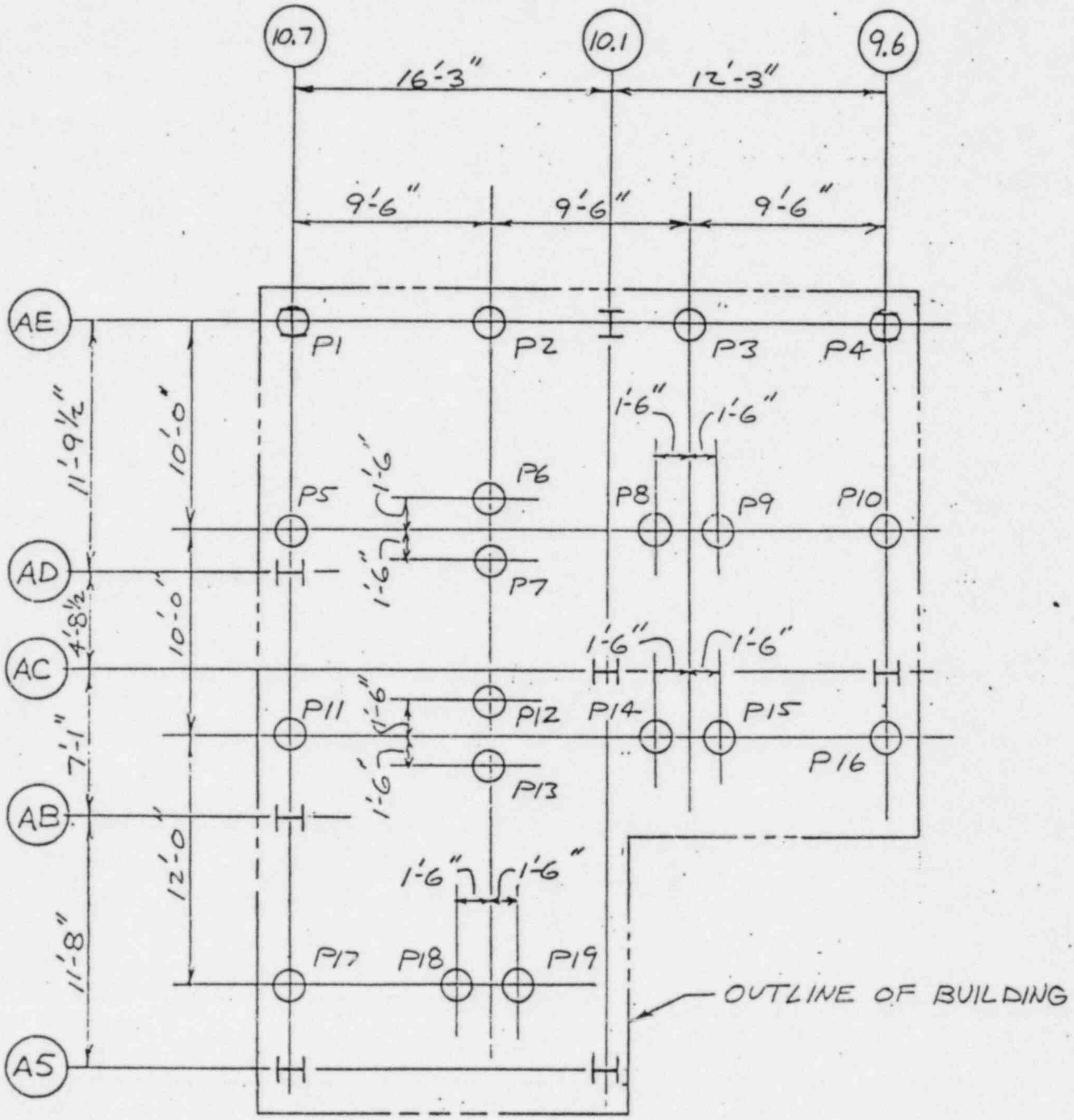


FIGURE 2.5 - PILE PLAN

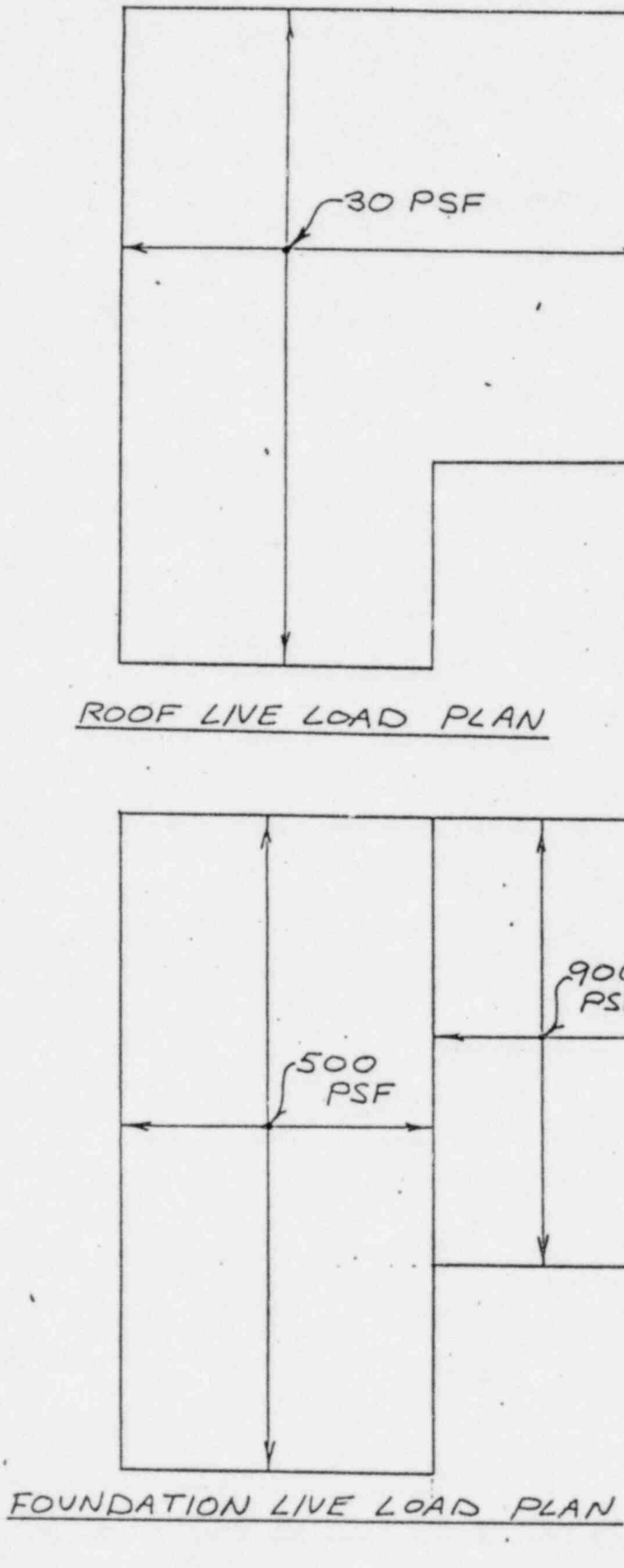


FIGURE 4.1 - LIVE LOAD PLAN
(Reference 1)

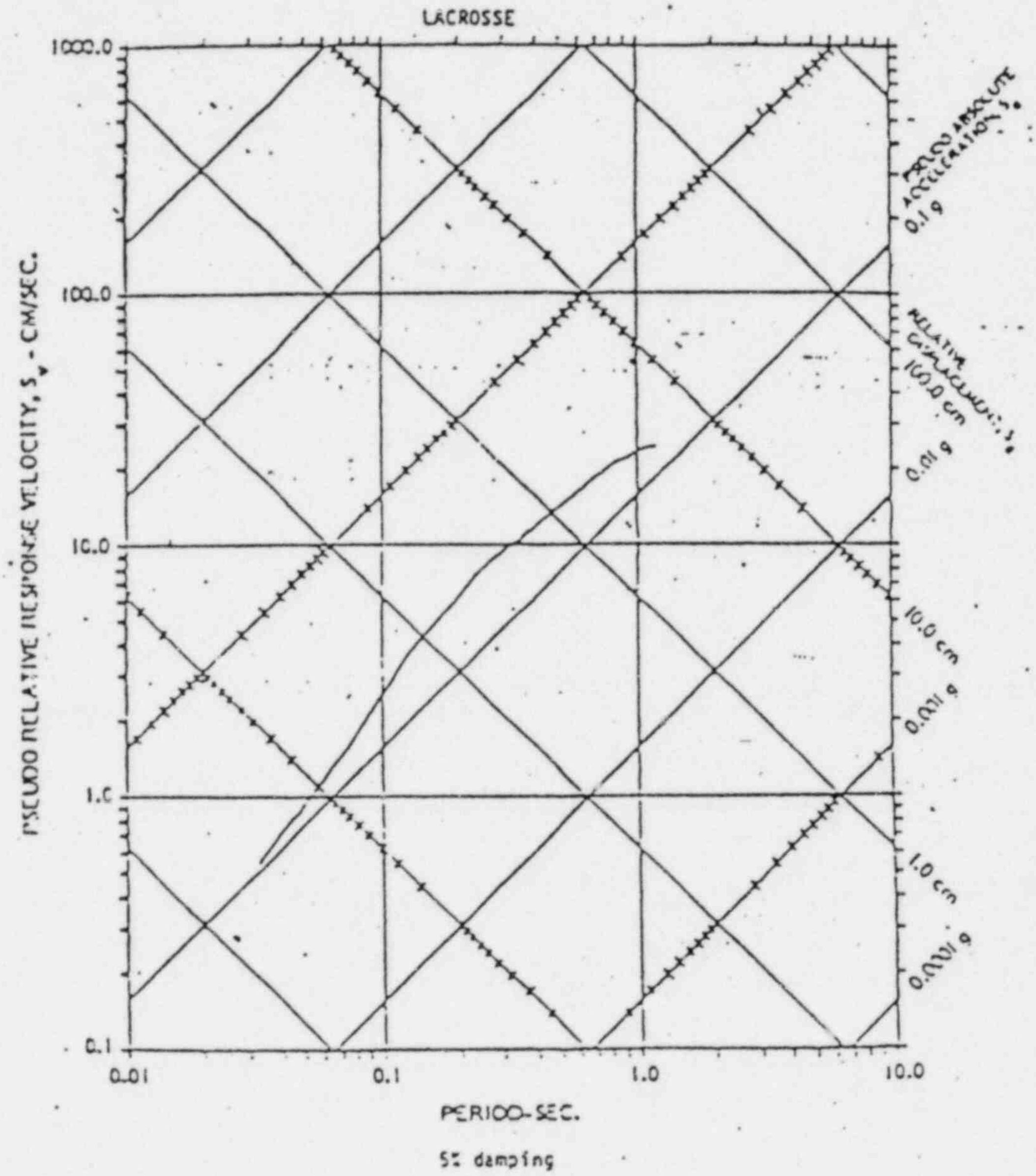


FIGURE 6.1 - LACBWR SITE-SPECIFIC GROUND RESPONSE SPECTRA

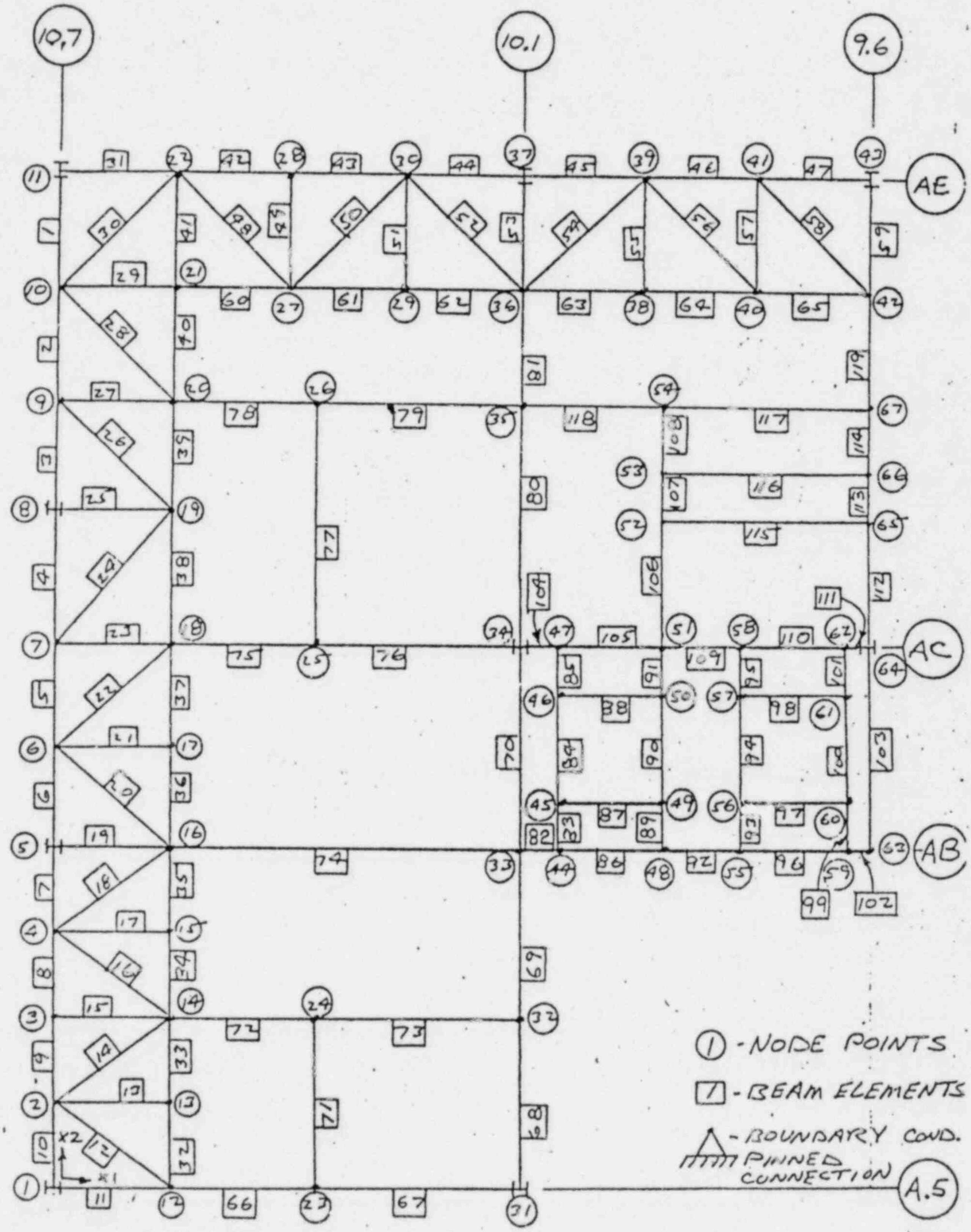
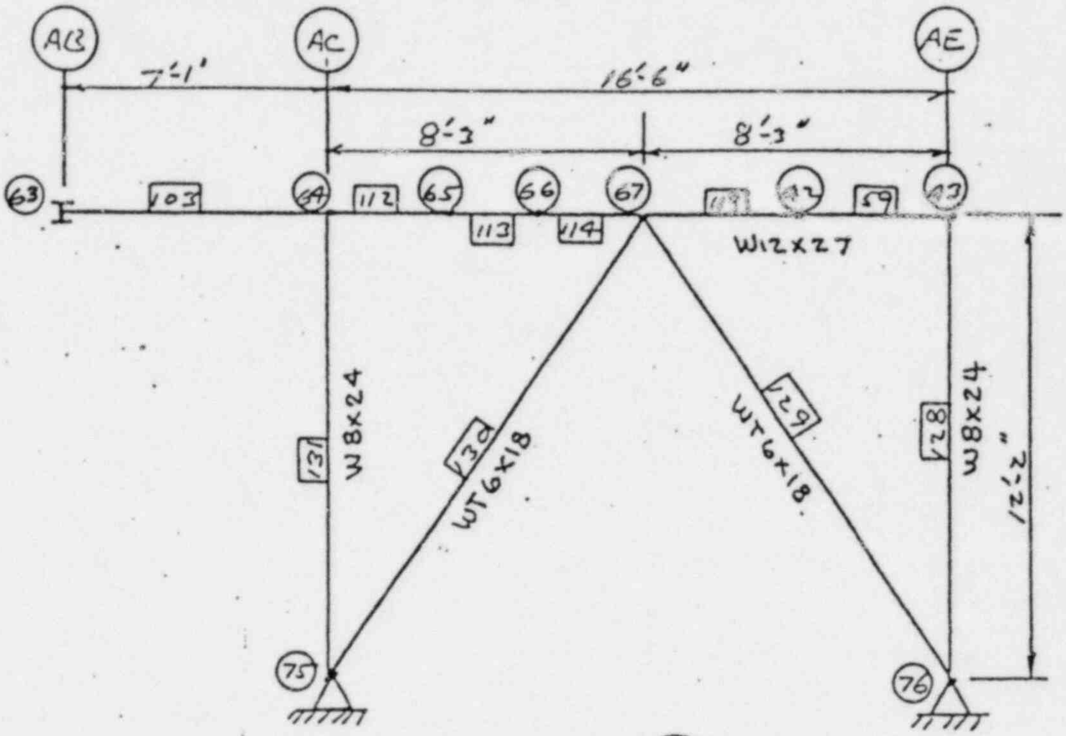


FIGURE 6.2 - FINITE ELEMENT MODEL



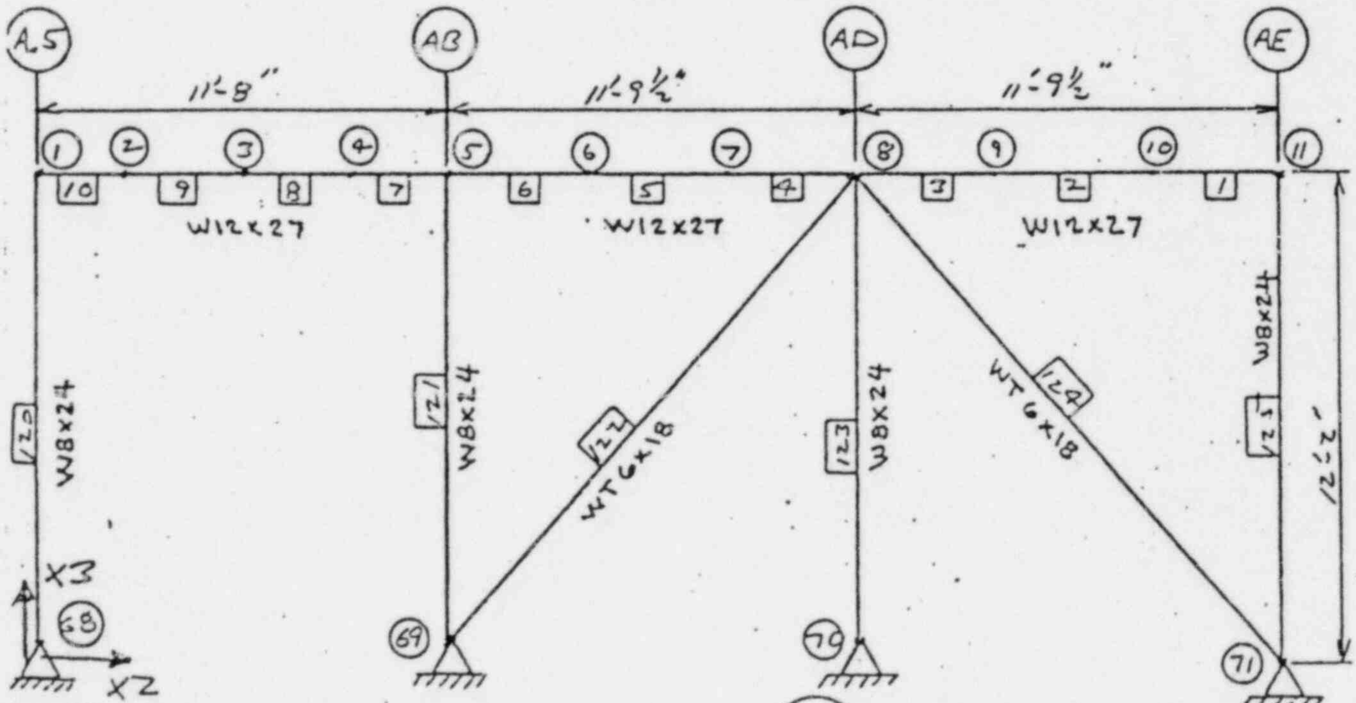


FIGURE 6.3 - FINITE ELEMENT MODEL

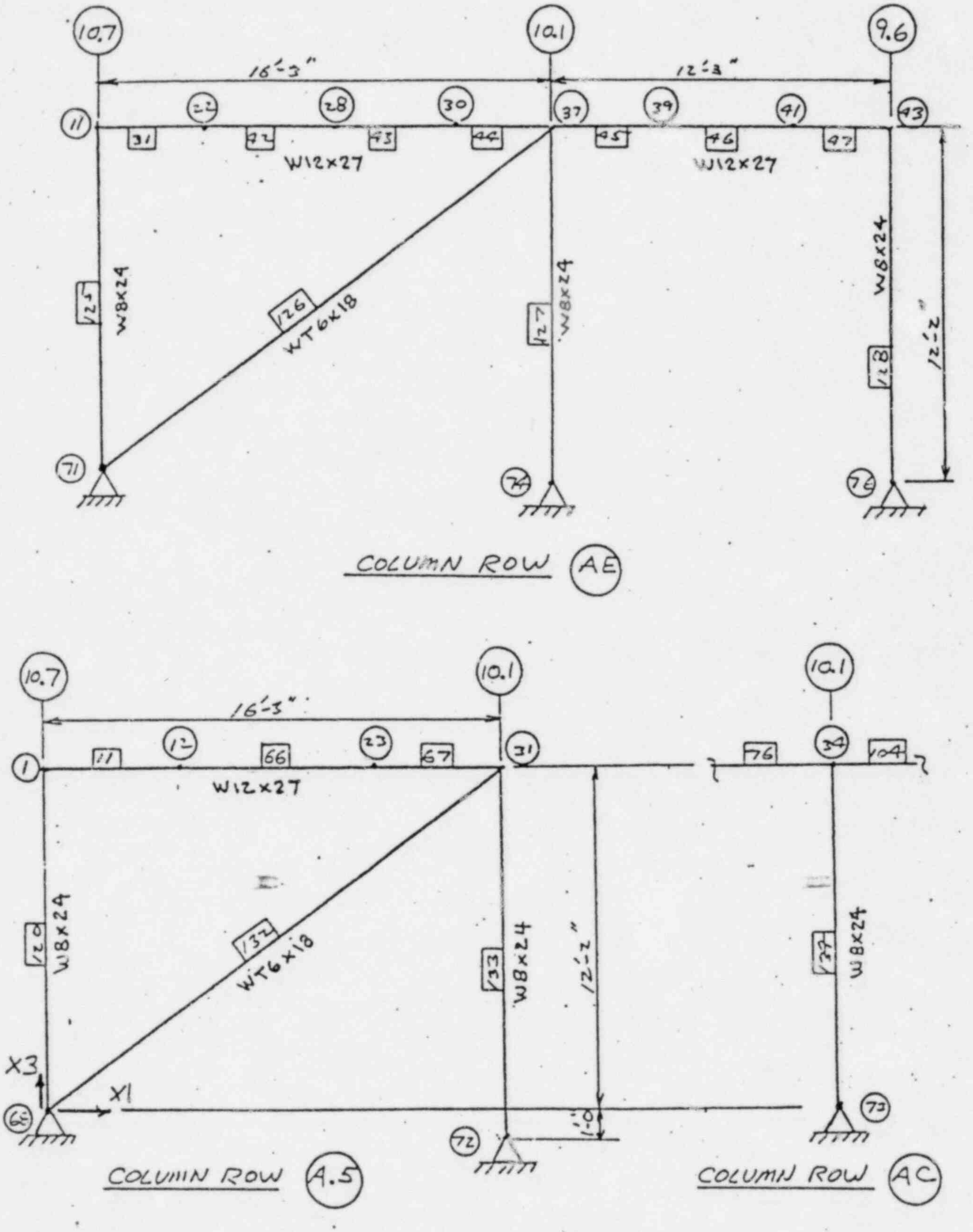


FIGURE 6.4 - FINITE ELEMENT MODEL