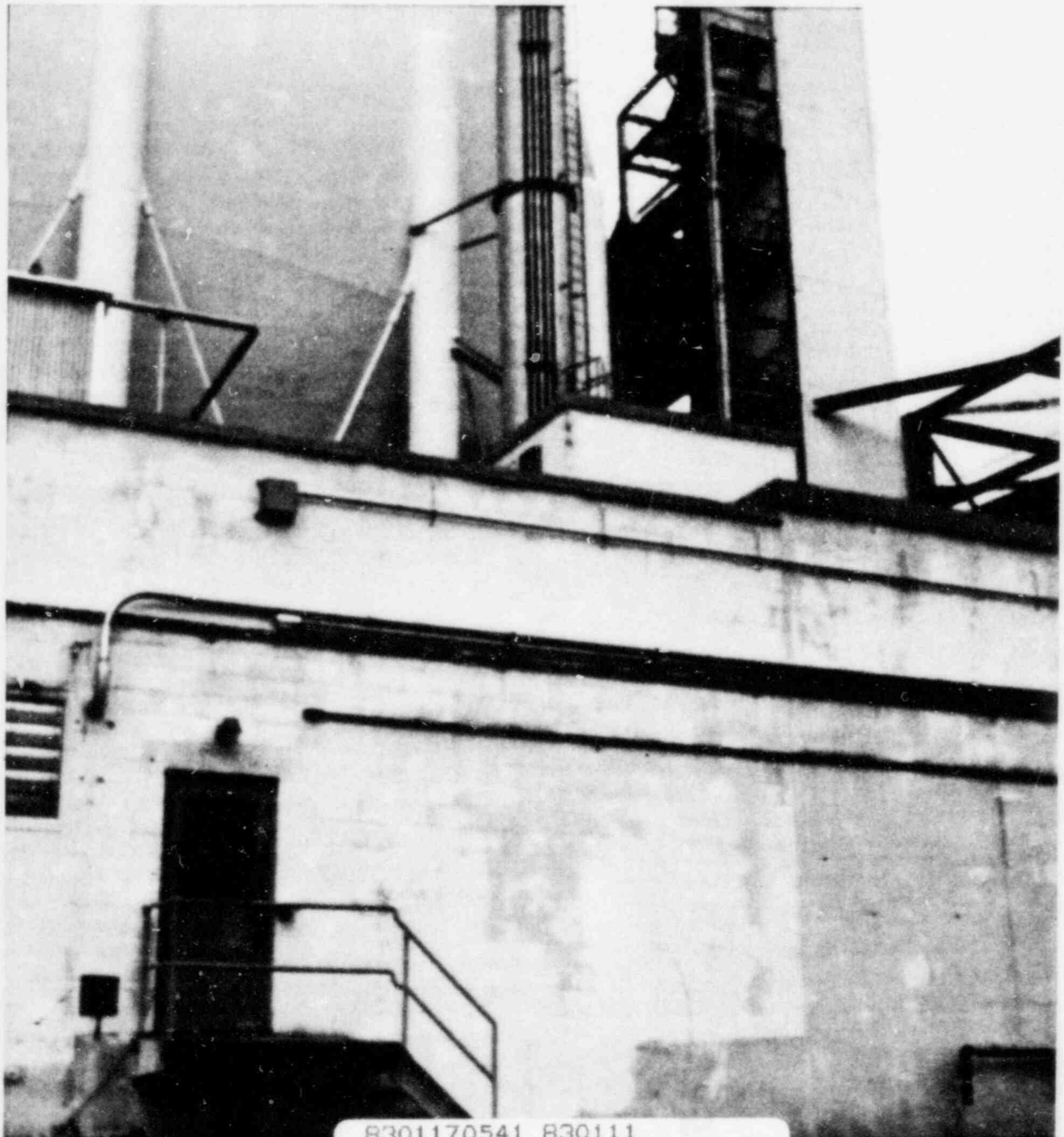
Primary Auxiliary Building and Radioactive Pipe Tunnel

Yankee Nuclear Power Station Structural Analysis Report

For: Yankee Atomic Electric Company
By: Cygna Energy Services



January 6, 1983 Rev. 2

Job No. 80023
Report No. EY-YR-80023-7
January, 1983
Rev. 2

Seismic Analysis of Primar. .uxiliary Building and Radioactive Pipe Tunnel

Yankee Nuclear Power Station

Rowe, Massachusetts

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I. EXECUTIVE SUMMARY

I.1 Purpose

The purpose of this report is to summarize the results of the seismic analyses performed by Cygna Energy Services* (Cygna) for Yankee Atomic Electric Company (YAEC). The results described in this report pertain to the Primary Auxiliary Building (PAB) and Radioactive Pipe Tunnel at the Yankee Nuclear Power Station (YNPS) at Rowe, Massachusetts.

I.2 Scope

As requested by YAEC, Cygna has performed detailed seismic analysis of the PAB. These analyses were based on the following input data for YNPS.

- Existing structural drawings (See Appendix A)
- Design Criteria (See Section III)
- Seismic ground motion (See Appendix C)

I.3 Conclusion

The PAB can functionally withstand either the YCS or the NRC spectrum loads. Minor modifications are required to upgrade the PAB to established criteria for the YCS loads. The unreinforced concrete block walls are overstressed and thus are not included in the structural analysis. The Radioactive Pipe Tunnel meets the loading criteria for the YCS and NRC spectrum.

*Cygna Energy Services is the new name for EES, Inc.
Ownership, philosophy and staffing of the firm remains the same.

II. DESCRIPTION OF STRUCTURE

The seismic evaluation of the PAB includes the PAB itself (Figs. A.1 to A.14) and the Radioactive Pipe Tunnel (Figs. A.15 and A.16). Figs. A.17 to A.22 show the interface details of the PAB with the Ion Exchange Pit. The PAB has two levels and a basement. The overall length of the building is 117'-0" and its width is 50'-0" from Column Lines 2 to 5 and 32'-0" from Column Lines 5 to 8. The weight of the building and the Radioactive Pipe Tunnel are 3798.0 kips and 998.2 kips, respectively.

As shown in Fig. A.4, the basement is bounded by Column Lines G, Fa, 2 and midway between Column Lines 4 and 5. The basement is constructed of reinforced concrete walls with thicknesses varying from 1'-0" to 2'-0". Under dead load and seismic load, the stresses developed in these walls are expected to be very small. Therefore, the basement was considered as a part of the rigid foundation and was not included in the analysis.

The first story is 16'-10" high (Fig. A.7). It is mainly constructed of reinforced concrete walls with thicknesses varying from 16" to 36" as shown in Fig. A.4. The construction along Column Line Ec between Column Lines 6 and 8 consists of composite steel columns and 12" thick concrete block walls. The first story has a reinforced concrete slab ceiling with thicknesses varying from 24" to 30" between Column Lines 2 and 6 and 6" between Column Lines 6 and 8. The tributary weight of the first story is 2501 kips.

The second story is 17'-0" high. As shown in Fig. A.5, the center part of the second story is constructed of reinforced concrete walls with thicknesses ranging from 2'-0" to 3'-0". The roof of this section is a 3'-0" thick reinforced concrete slab. The remaining parts of the second story are constructed of steel frames whose lateral and in-plane stability are provided by diagonal braces (Figs. A.12 to A.14). The tributary weight of the second story is 1297 kips.

The Radioactive Pipe Tunnel is constructed of reinforced concrete. The southerly end of the tunnel is monolithically cast with the reinforced concrete wall in the second story of the PAB. The tunnel is also supported by two 2'-10" x 5'-0" reinforced concrete columns approximately at mid-span. The northerly end of the tunnel cantilevers underneath the Steel Vapor Container (Figs. A.15 and A.16).

The Ion Exchange Pit and the Diesel Generator Building are also attached to the PAB. However, only the tributary weights and tributary stiffnesses of these two buildings were added to the proper elements of the PAB model.

The dynamic responses of the PAB were analyzed using the response spectrum method. The amplified floor response spectra at two selected locations were also generated for the analyses of the piping systems.

III. PERFORMANCE CRITERIA

III.1 Materials

a - Reinforced Concrete Walls

As shown in Figs. A.1 through A.11, the reinforced concrete walls have very small height-to-depth ratio. Therefore, their failure mode is primarily controlled by shear failure. The ultimate shear stress is calculated in accordance with Section 2611(e) of 1979 UBC [2],

$V_u = 2\sqrt{f_c'} = 2\sqrt{3000} = 110$ psi [1]

b - Steel Frames

The stresses developed in the steel members are checked against their allowable stresses as specified in the Part 1 of AISC Specification [3]. As described in Section 1.5.6 of AISC Specification, the allowable stresses may be increased by 1/3 when seismic loads are considered. For example, the allowable stress of the combined axial and bending stress shall not exceed 1.33 x 0.6 Fy = 0.8 x 33 = 26.4 ksi.

c - Diagonal Braces

The effective slenderness ratio of most of the diagonal braces exceeds 200. This significantly reduces the capacity of these braces to resist compression. For a pair of diagonal cross braces, it is assumed that only the tension brace is effective in resisting loads. More specifically, the combined tension and compression stresses in a pair of X-braces shall not exceed 1.33 x 0.6 Fy = 26.4 ksi. For a single diagonal brace, its ultimate stress shall be limited by the allowable compression stress (includes 1/3 increase) recommended by AISC [3].

d - Steel Beam-to-Concrete Wall Connections

The typical steel beam was connected to the concrete walls through two bolts embedded in the concrete as shown in Fig. A.6. In general, 3/4" Φ bolts are used except for the connection at the intersection of Column Lines 5 and Ec where 1" Φ bolts are used. The allowable shear is taken as 1.33 times the values specified in Table 36-G of UBC [2]. However, since the minimum bolt spacing of 12 diameters is not satisfied, the allowable shear is reduced to 3.3 kips for 3/4" Φ bolt and 4.0 kips for 1" Φ bolt.

e - Radioactive Pipe Tunnel and its Supporting Columns

Both the Radioactive Pipe Tunnel and its supporting columns are lightly reinforced. The forces developed in these members were checked against their cracking moment and their ultimate moment capacity as specified in Section 2610 of 1979 UBC [2].

Results of the evaluations for the tunnel are discussed in Section V.2.

f - Unreinforced Concrete Block Walls

The unreinforced concrete block walls lack the capacity to resist the seismic loads perpendicular to its plane. The allowable shear and flexural tension stress is only 10 psi and 14 psi, respectively, as specified in Seismic Reevaluation and Retrofit Criteria [1]. These block walls are expected to fail during the early stage of the severe seismic event. For this reason, the stiffness of the concrete block walls was not considered in the model so that realistic forces that can be developed in the steel frames can be obtained. The weight of the concrete block wall, however, was included in the model.

III.2 Earthquake Loadings

The PAB was analyzed using the Yankee Composite Spectrum (YCS) with 7% critical damping ratio (Fig. C.1, Appendix C). The critical damping ratio of 7% was selected as per Reference 1. The same spectrum was applied in two horizontal directions and it was scaled down to 2/3 (two-thirds) of the original magnitude when applied in the vertical direction.

The PAB was similarly analyzed using the NRC spectrum with 7% critical damping ratio (Fig. C.1, Appendix C).

IV. ANALYSIS PROCEDURES

IV.1 General

The reinforced concrete walls of the PAB were designed mainly as radiation shields. As it will be discussed in Sect. V.2., the stresses developed in these walls under seismic loads are very small. Therefore, the evaluation of the building was concentrated on studying the responses of the steel frames to the earthquake loads.

IV.2 Computer Programs

Three computer programs were adopted to perform the analyses. BATS [4] computer program was used to run the modal analyses and the response spectrum analyses. MOST [5] and INSPEC [6] computer programs were used to generate the amplified response spectra (ARS).

IV.3 Assumptions

a - Rigid Diaphragms and Subdiaphragms

The BATS computer program assumes that the floor diaphragm is rigid in its plane. As it was discussed in Chapter II, the ceiling of the first story consists of reinforced concrete slabs with thicknesses varying from 6" to 30". The concrete slabs can provide very large in-plane stiffnesses, hence, the rigid diaphragm assumption made by BATS computer program could be accurately applied. However, the same assumption could not be made for the roof of the structure. The center portion of the roof is constructed of a 36" thick reinforced concrete slab. The remainder of the roof is constructed of corrugated metal deck which has very small in-plane stiffness. In such a case, the top of the steel columns will not displace with the reinforced concrete roof in a rigid manner. To correctly model this, the subdiaphragm concept of BATS was adopted.

Under the rigid diaphragm assumption, each diaphragm has two horizontal and one torsional degree of freedom. At the second level of the building, the concrete slab was modeled as the main diaphragm. The steel roof was modeled by utilizing several subdiaphragms. These subdiaphragms were connected to the main diaphragm and to each other through flexible links which have the actual member properties of the beams or the diagonal braces which they are representing. Under this configuration, the movements of the steel frames can be accurately analyzed and the forces measured.

b - Equivalent Columns of Wall

As shown in Fig. B.1, a wall element can be replaced by a set of equivalent columns connected at the top by rigid links. The member properties of the equivalent columns were calculated such that the total area and the total moment of inertia of the columns are equal to those of the wall. In this manner, the axial and lateral stiffness of the equivalent columns are identical to those of the wall. After the analysis, all the forces developed in these columns are added to calculate the stresses developed in the wall.

IV.4Mathematical Models

a - Horizontal Model

The horizontal model of the PAB is shown in Figs. B.2 through B.4. This model was used to study the responses of the building to the horizontal earthquake loads.

In the horizontal model, the reinforced concrete walls were modeled by the equivalent columns as discussed in Section IV.3.b. The corrugated steel roof at Level 2 was grouped into four subdiaphragms as shown in Figs. B.2 through B.4. These subdiaphragms were connected to the main diaphragm, which represented the concrete slab, and to each other through eighteen flexible links as shown in Fig. B.4. The Radioactive Pipe Tunnel was also modeled as a subdiaphragm (Fig. B.4). Flexible Link No. 19 has the member properties of the tunnel itself. The supporting columns of the tunnel are not connected to the building at the first level. Under the rigid diaphragm assumption of the BATS computer program, however, the columns would be rigidly tied to the concrete slab. To correct this situation, a dummy subdiaphragm was introduced at Level 1. This dummy subdiaphragm is completely separated from the main diaphragm which represents the concrete slabs at Level 1. The supporting columns of the tunnel were then connected to the subdiaphragm properly representing the columns not connected to the concrete slabs.

Generally, the horizontal model has one main diaphragm and one dummy subdiaphragm at the first level. At the second level, it has one main diaphragm and five subdiaphragms.

b - Vertical Model

The vertical model of the PAB is shown in Fig. B.5. This model was used to study the response of the building to vertical seismic loads.

The BATS computer program cannot readily handle the motion in the vertical direction. Therefore, in order to use the program correctly, the vertical model was turned to the horizontal direction as shown in Fig. B.5. In this figure, Spring K_1, represents the axial stiffness of the reinforced concrete walls at the first story. Spring K_2 represents the twin columns which support the Radioactive Pipe Tunnel. Spring A_1

represents the reinforced concrete walls at the second story. Springs A_2 through A_5 represent the columns of the steel frames at the second story and these columns are also grouped into Subdiaphragms SS2 to SS5, respectively, as shown in Fig. B.3 of the horizontal model.

V. STRUCTURAL CHARACTERISTICS AND ANALYTICAL RESULTS

V.1 Structural Characteristics

With the rigid diaphragm idealization used in the BATS computer program, each diaphragm has three degrees of freedom. As discussed in Section IV.4.2, the horizontal building model has eight diaphragms and consequently, it has twenty-four degrees of freedom. Only the first sixteen frequencies were included in the dynamic analysis of the building. Table D.1 lists the frequencies and description of the mode shapes of these sixteen modes and Table D.2 lists the modal mass, percentage of the modal mass to the total mass and the cumulative percent mass of each mode. As shown in Table D.2, at the sixteenth mode, the cumulative modal masses in the N-S and E-W directions are almost equal to the total mass. Therefore, selection of the first sixteen modes is adequate to obtain an accurate dynamic response of the building due to horizontal earthquakes.

From Tables D.1 and D.2, it can be seen that the first two modes and several other modes are associated with local vibration of part of the steel frames at the second level. The third mode is the lowest mode associated with the E-W movement of the Radioactive Pipe Tunnel. The seventh and the eighth modes represent the vibration of the building and the Radioactive Pipe Tunnel as a unit in the N-S and E-W directions, respectively. Consequently, the third, seventh and eighth modes have larger modal masses than the others.

As shown in Fig. B.5, the vertical building model has seven degrees of freedom and consequently it has seven modes. The frequencies and the mode shapes of these seven modes are shown in Table D.3 and their modal masses are shown in Table D.4. As shown in these two tables, the first mode represents the vertical vibration of the Radioactive Pipe Tunnel. The vertical vibration of the steel frames at the second level is represented by the second through the fifth modes. The sixth mode has the largest modal mass and it represents the in-phase vibration of the concrete slabs at the first and the second levels. The last mode represents the out-of-phase vibration of the concrete slabs at those two levels. Generally speaking, the vertical earthquake has less effect on the safety of the building than the horizontal earthquake.

V.2 Analytical Results

a - Yankee Composite Spectrum (YCS)

The response of the building to the YCS applied in two horizontal directions and the vertical direction is summarized in Tables E.1 through E.6. The member forces due to the earthquake applied in two horizontal directions were combined by the square root of the sum of the squares (SRSS) method. Table E.1 shows the shear forces developed in the reinforced concrete walls. Because walls were mainly designed as radiation shields, they are capable of withstanding earthquake loads. Under YCS, the maximum shear stress developed in these walls reached only 15% of the ultimate shear stress.

Table E.2 shows the axial forces developed in the diagonal cross braces. As discussed in Section III.1.c, the strength of the brace in compression was ignored and all the forces developed in one pair of braces were assumed to be carried by its tension member only. These brace pairs can also take YCS loads.

The single diagonal brace located at Column Line 8, however, has a very small allowable compression stress (4.34 ksi). As shown in Table E.3, the compression stress developed in that member under YCS load is 1.38 times the allowable stress. Therefore, it is recommended that an additional diagonal brace which will form an inverse "V" shape with the existing brace be installed at Column Line 8 so as to take the earthquake load developed in that column line. The single diagonal brace located at Column Line G is capable of withstanding YCS loads.

Both the steel column-to-bottom slab connection and the beam-to-column connection can take very small moments (Figs. A.12 to A.14). Therefore, these connections were modeled as frictionless hinges and consequently only axial forces can be developed in the steel columns. Table E.4 shows the column axial compression stress due to dead load and YCS loads. Compared to the allowable compression stress of these columns, they can resist combination of dead load and YCS loads.

The steel beam-to-concrete wall connections are the weak part of the building. As indicated in Table E.5, the first three connections require modification. For the connection located at the intersection of Column Lines 5.5 and G, the beam is just sitting on the reinforced concrete wall without any anchor bolts. Therefore, this connection has zero shear capacity.

The Radioactive Pipe Tunnel is lightly reinforced. Therefore, the ultimate moment capacity of the tunnel is only slightly larger than the crack moment as shown in Table E.6. To ensure the safety of the tunnel and to prevent leakage of radiation, the earthquake moments developed in the tunnel were checked against its crack moment and ultimate moment. As shown in Table E.6, both the tunnel and its supporting columns will not be overstressed under YCS loads. Note that the moment capacities of the columns were evaluated under 529 kips compression force. This compression force is due to dead load minus the axial force produced by the vertical YCS loads.

b - NRC Spectrum

The response of the building to the NRC spectrum applied in two horizontal directions and the vertical direction, is summarized in Tables E.7 through E.12. Generally speaking, the reinforced concrete shear walls, diagonal cross braces and steel columns will not be overstressed under the NRC spectrum loads. The single diagonal brace located at Column Line 8 and the steel beam-to-concrete wall connections are overstressed (Table E.11).

The Radioactive Pipe Tunnel and its supporting columns can withstand NRC spectrum loads.

VI. GENERATION OF AMPLIFIED RESPONSE SPECTRA

The amplified response spectra (ARS) were generated for the analysis of the piping systems attached to the PAB. These spectra were generated at two points, both located at Elevation 1039'-6" as shown in Fig. F.1.

As discussed in Section IV.2, the MOST and the INSPEC computer programs were used to generate ARS. The inputs to the MOST program were the earthquake ground acceleration time-histories and the modal characteristics of the building model. The synthetic acceleration time-histories of the response spectra shown in Fig. C.1 were used as the ground acceleration time-histories. For each direction of the earthquake input, an independently generated synthetic acceleration time-history was used. As discussed in Section V.1, the dynamic characteristics of the sixteen horizontal modes and seven vertical modes were generated. Except the vertical mode associated with the vibration of the Radioactive Tunnel, the dynamic characteristics of the other twenty-two modes were input to the MOST program. The analytical results obtained from this program were the acceleration time-histories at the locations where ARS were needed.

These acceleration time-histories were later input to the INSPEC program to generate ARS. These ARS were then broadened by 15% in accordance with Regulatory Guide 1.122.

The amplified response spectra due to YCS have 2% critical damping ratio. The amplified response spectra generated for the three directions of Points 1 and 2 are shown in Figs. F.2 through F.4 and Figs. F.5 through F.7, respectively.

The amplified response spectra due to the NRC spectrum with 2%, 3%, and 5% critical damping ratios for the same Points 1 and 2 are shown in Figs. F.8 through F.13.

VII. SUMMARY OF THE REVIEW - PAB

1. The reinforced concrete walls of the building can resist YCS and NRC spectra loads.

2. For the diagonal cross braces, the tension member is capable of withstanding the combined forces developed in the pair of cross braces due to YCS and the NRC spectra.

3. The single diagonal brace located in Column Line 8 will be severely overstressed under YCS and NRC spectra. An additional diagonal brace is recommended to be installed at Column Line 8 to form an inverse "V" shape with the existing diagonal brace. The single diagonal brace located at Column Line G, however, will not be overstressed under YCS and NRC spectra.

4. The steel columns can resist dead load plus YCS or NRC spectra loadings.

5. Under YCS loads, the steel beam-to-concrete wall connections located at the intersection of Column Lines 5.5 and G, 6 and Fb as well as 6 and Ec need to be strengthened.

6. Both the Radioactive Pipe Tunnel and its supporting columns are capable of resisting YCS and NRC spectrum loads.

7. The unreinforced concrete block walls will be overstressed at the early stage of YCS and NRC spectrum loads and are not considered in the structural integrity calculations.

8. In general, only minor modifications are required to upgrade the PAB to resist YCS loads.

APPENDIX A

BUILDING PLANS

APPENDIX A

BUILDING PLANS

FIG. NO.	CONTRACT DRAWING NO.
FIG. A.1	S & W DWG 9699 - FA-16A
A.2 & A.3	FA-16B & 16C
A.4 & A.5	RC-40A & 40B (PARTS I & II)
A.6 to A.11	FC-40C to 40H
A.12 to A.14	FS-19A to 19C
A.15 & A.16	FC-47A & 47B
A.17 to A.20	FC-40J to 40M
A.21 & A.22	FC-40P & 40Q

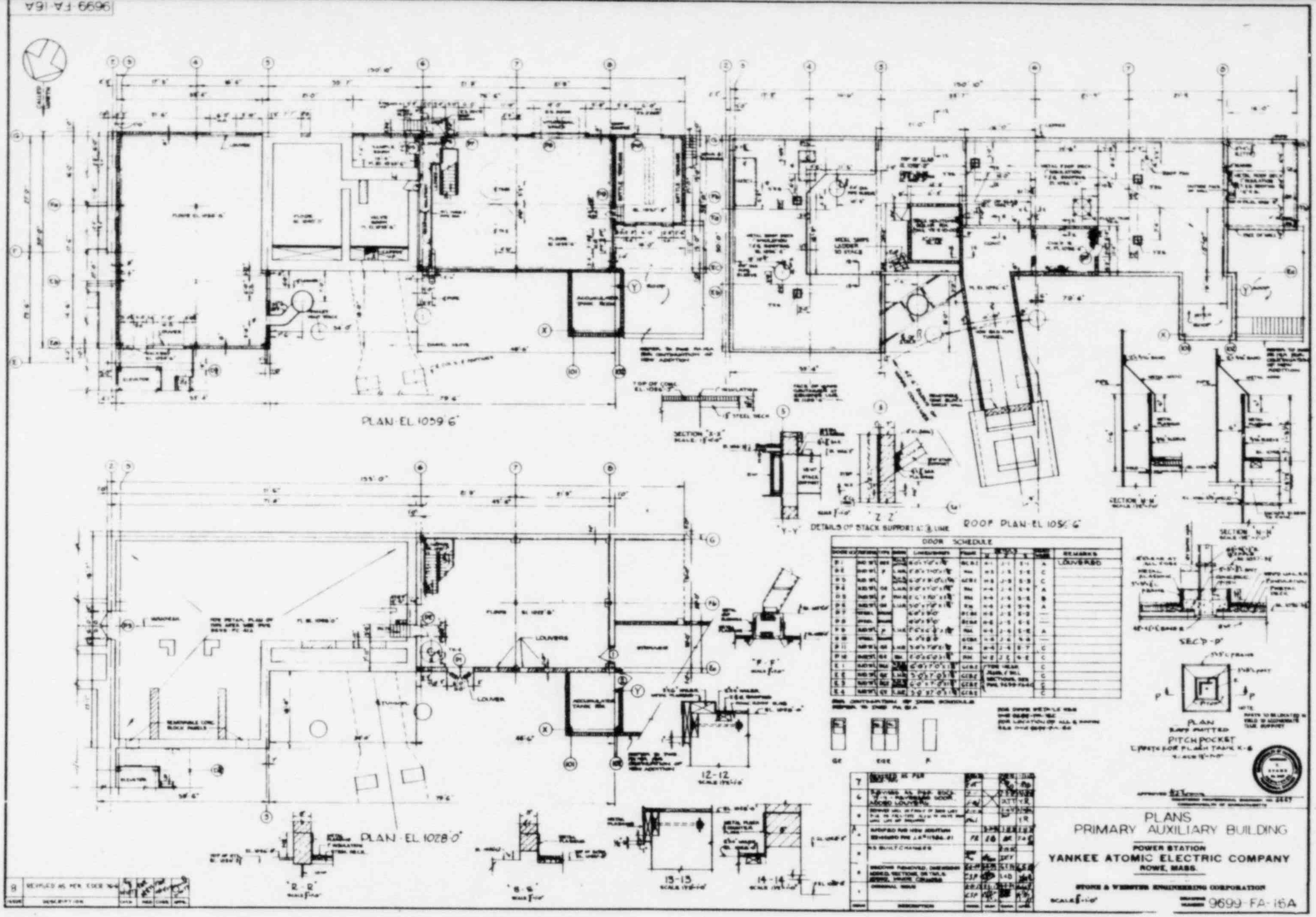

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FIG. A.1

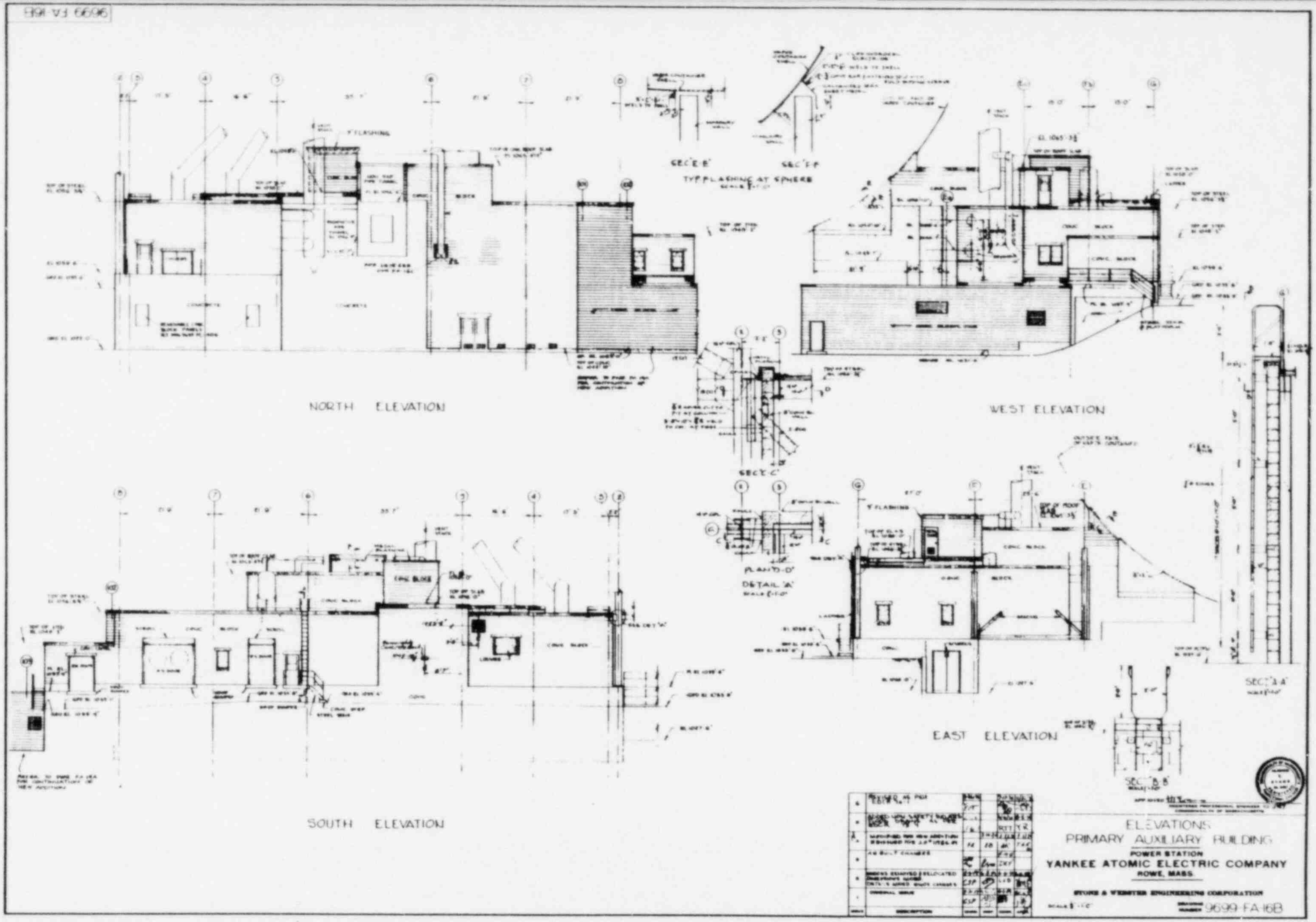

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FIG. A.2

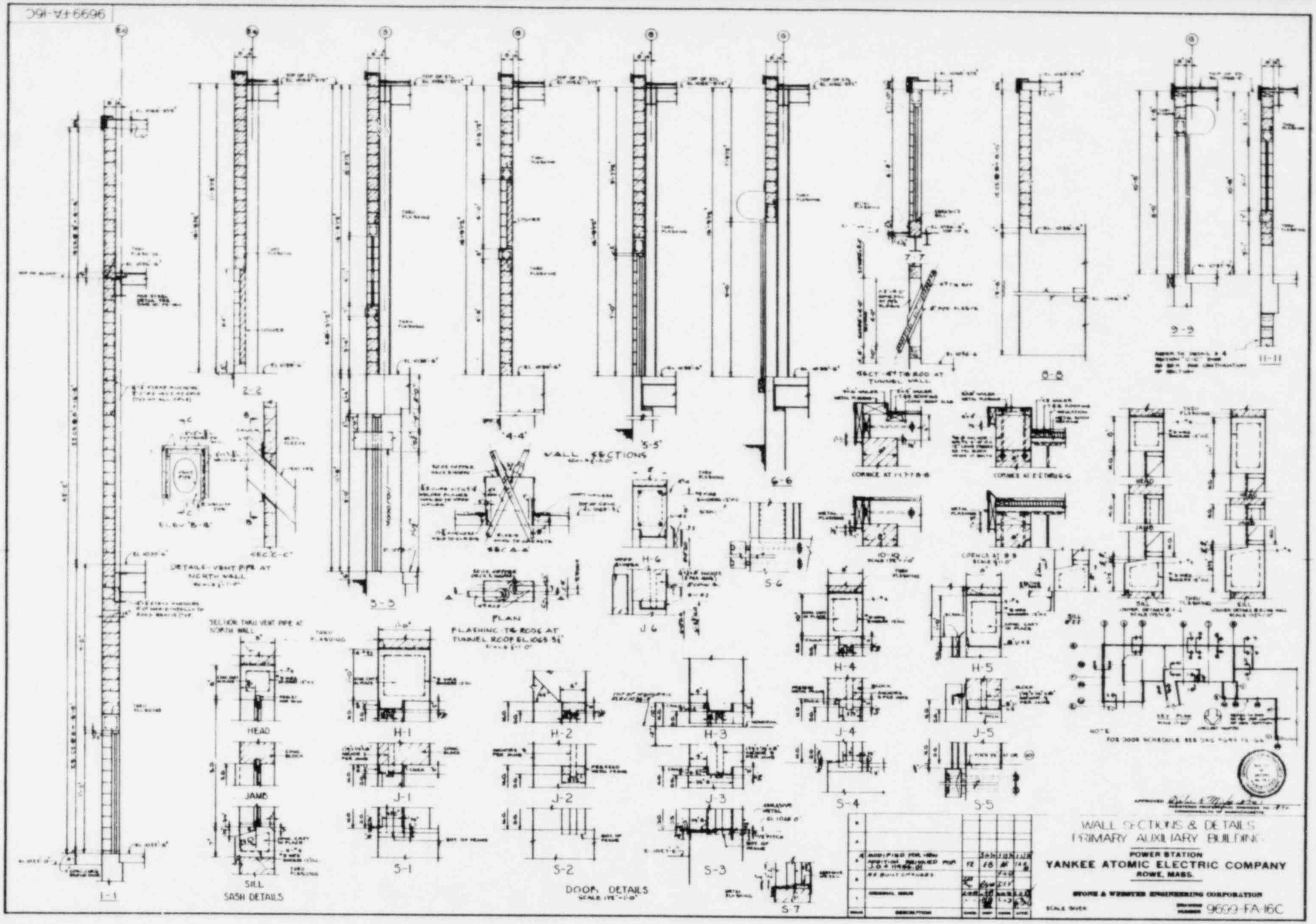

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FIG. A.3

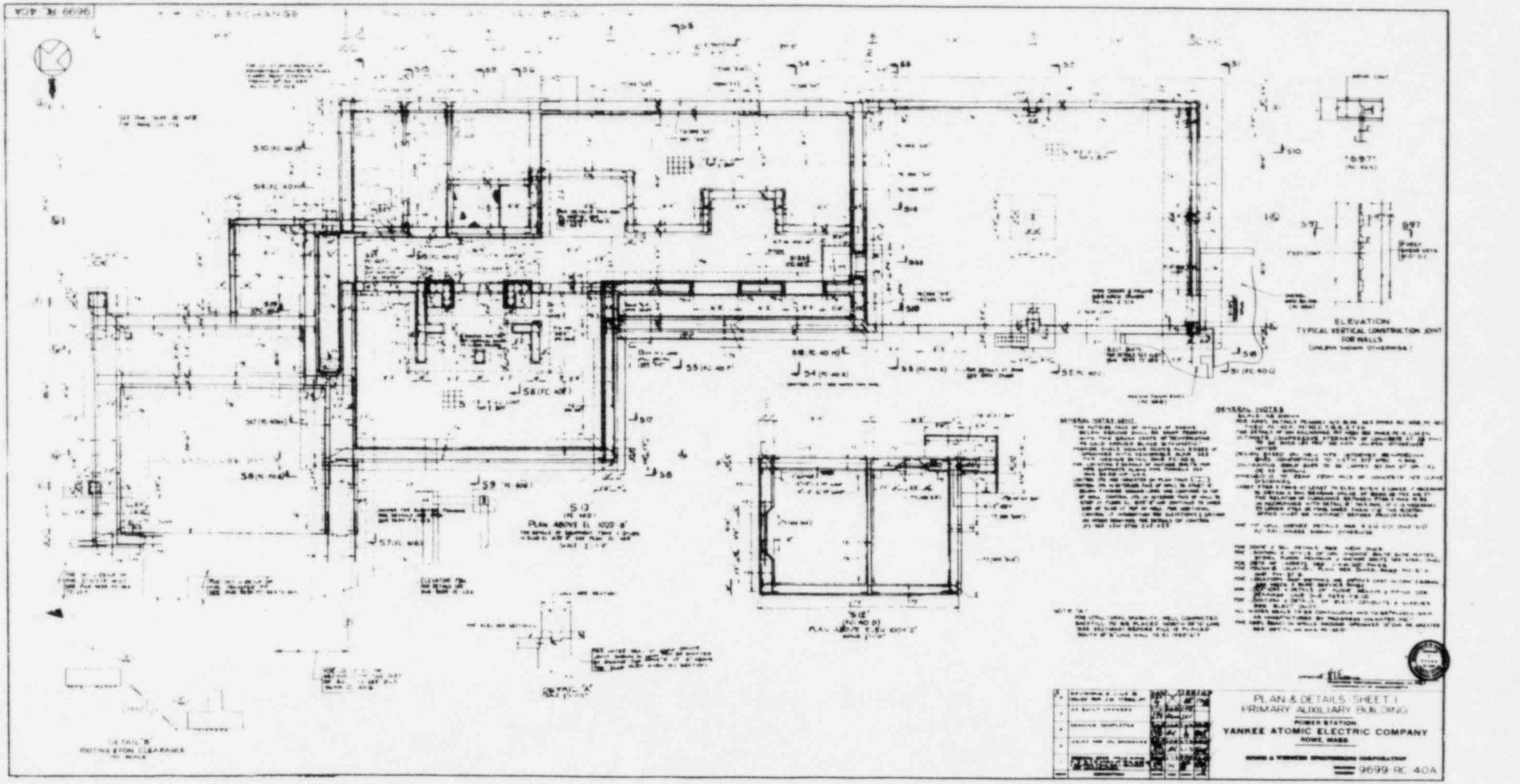

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FIG. A.4

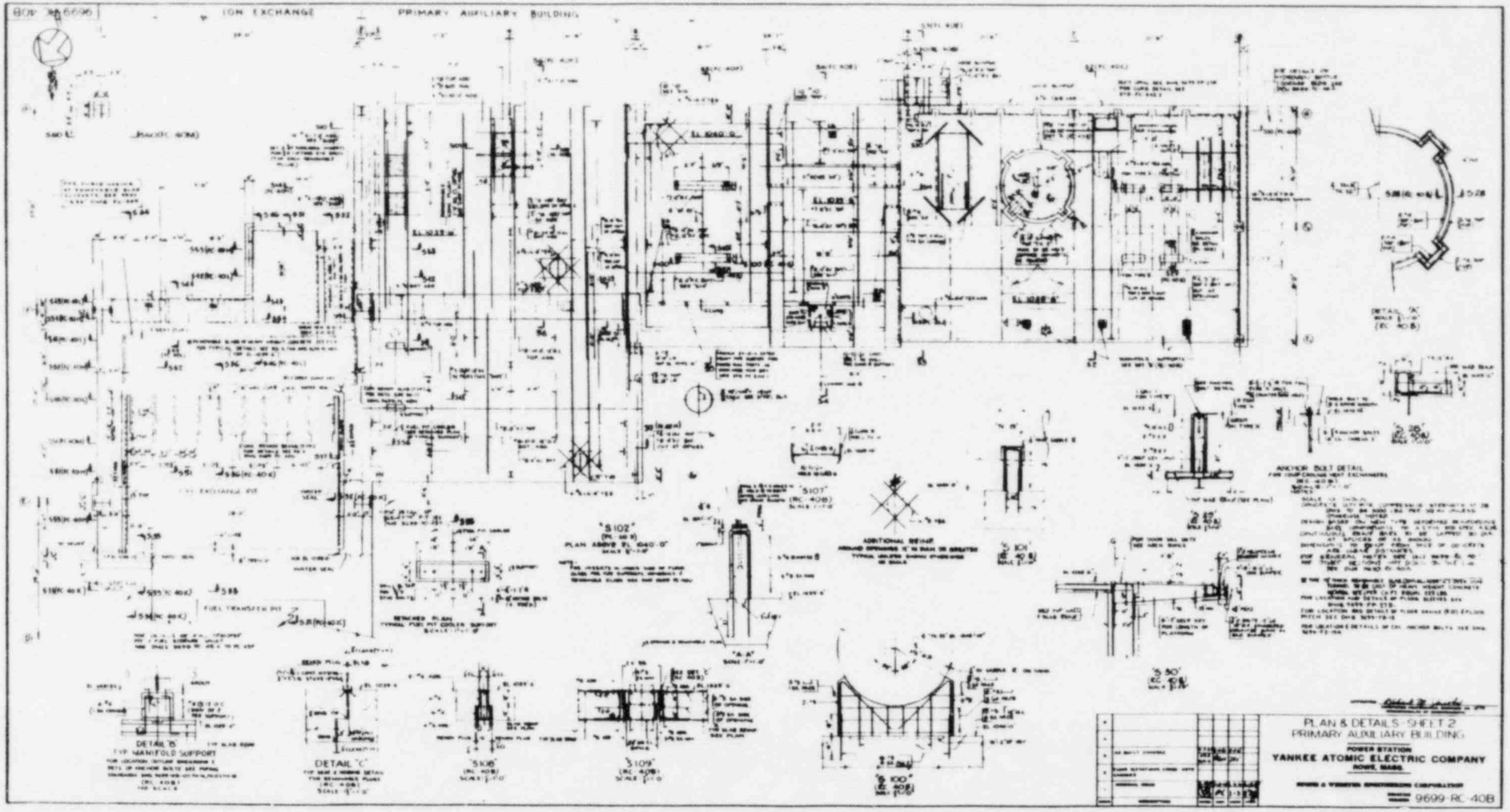

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FIG. A.5

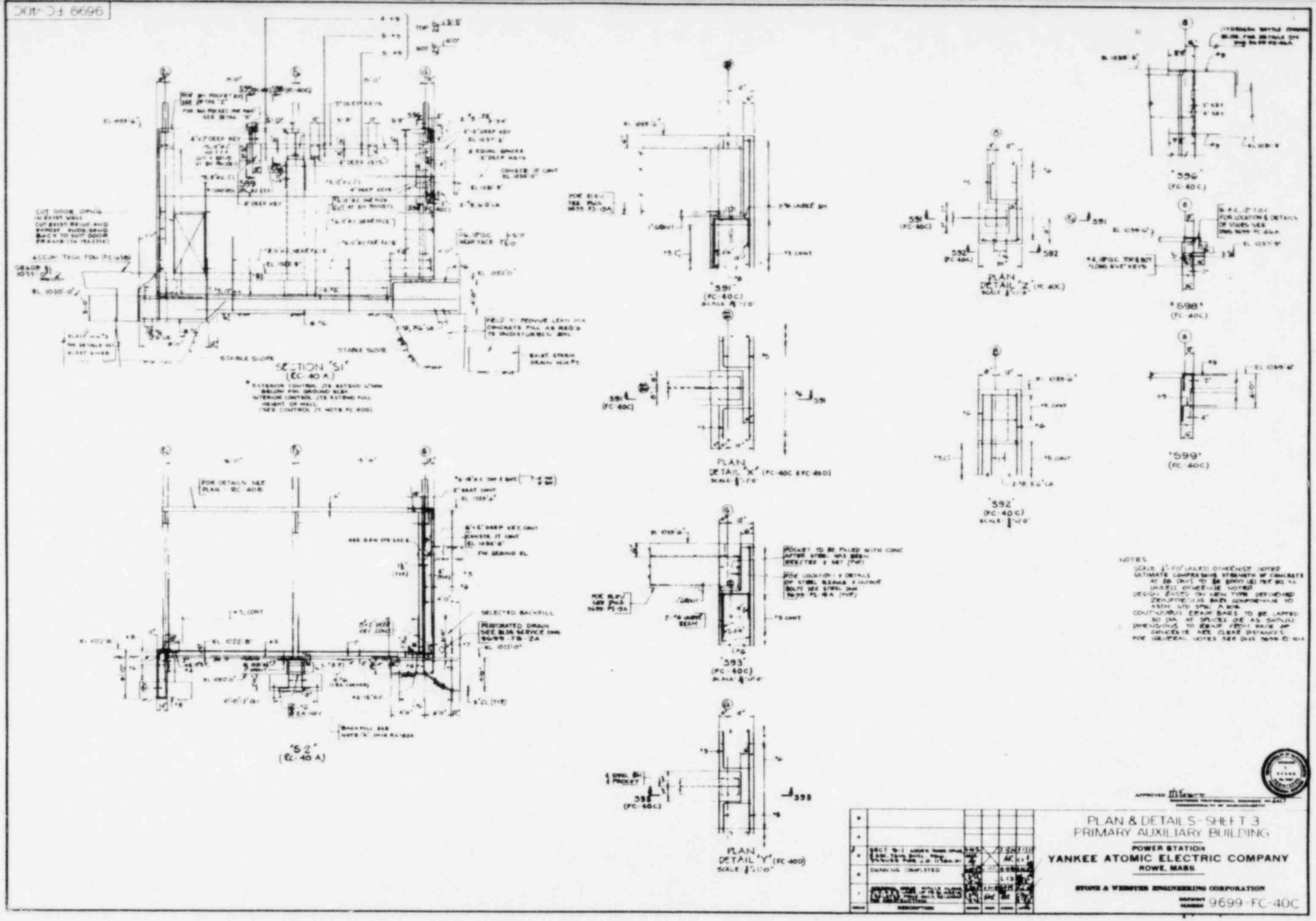

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FIG. A.6

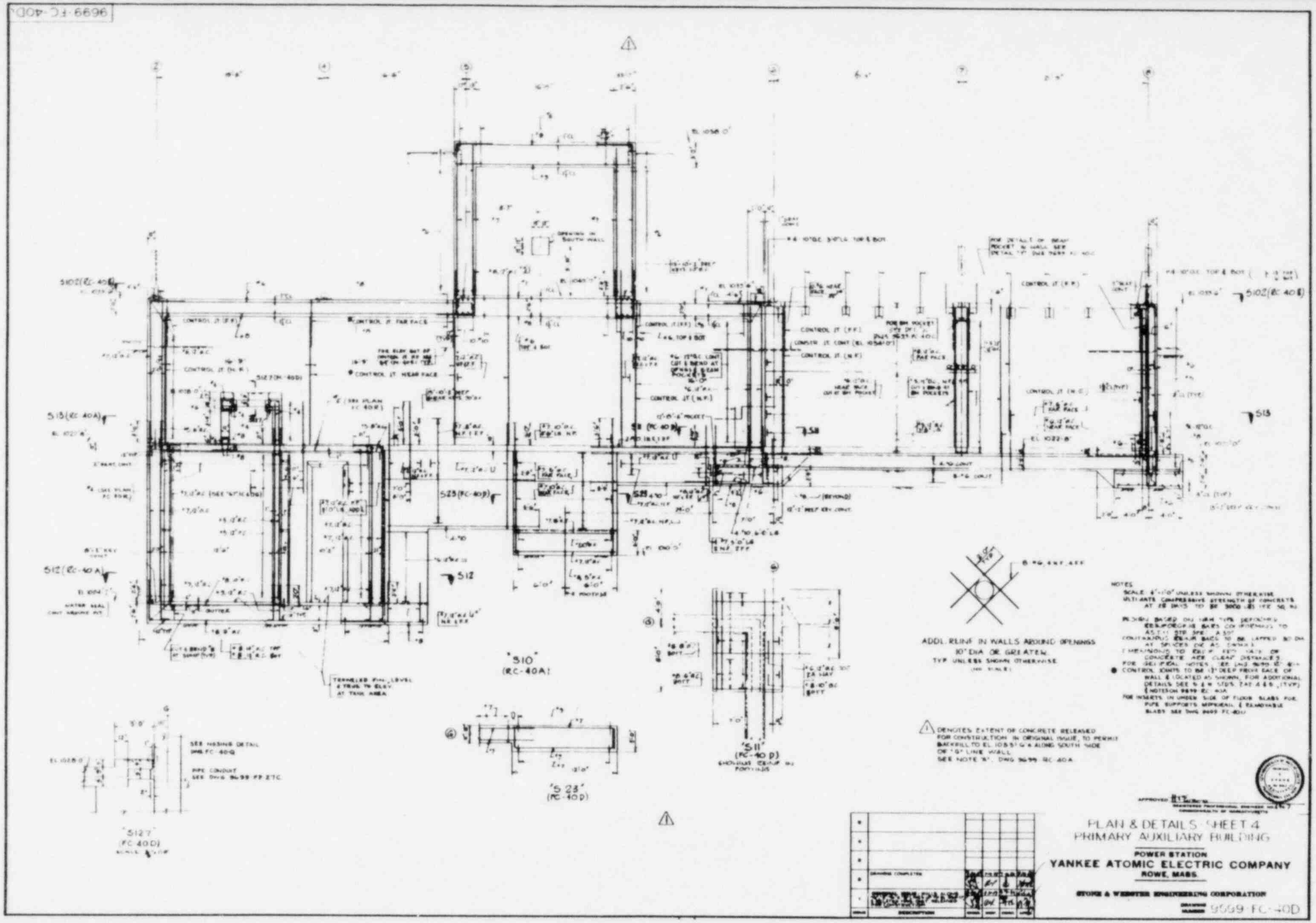

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FIG. A.7

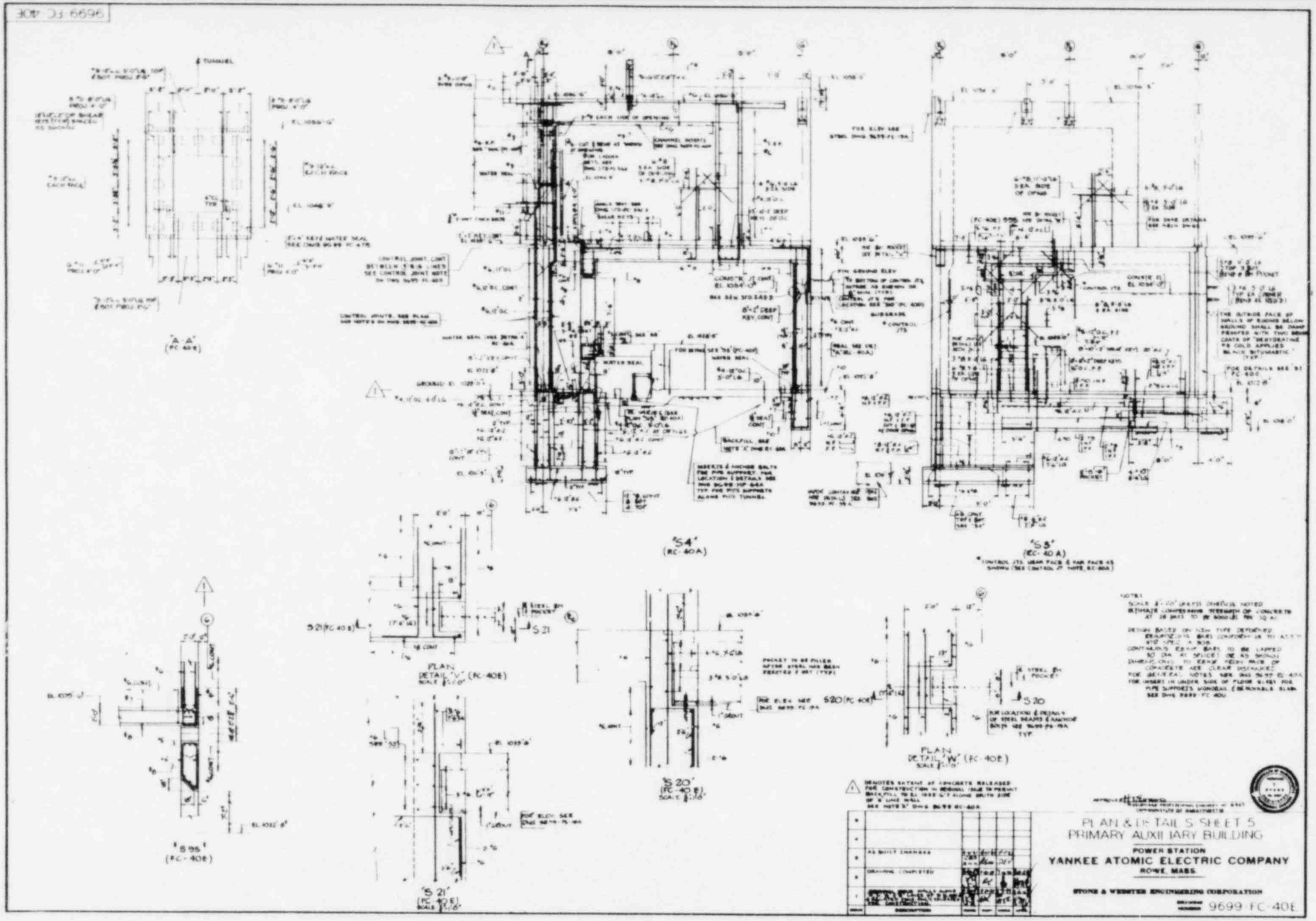

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FIG. A.8

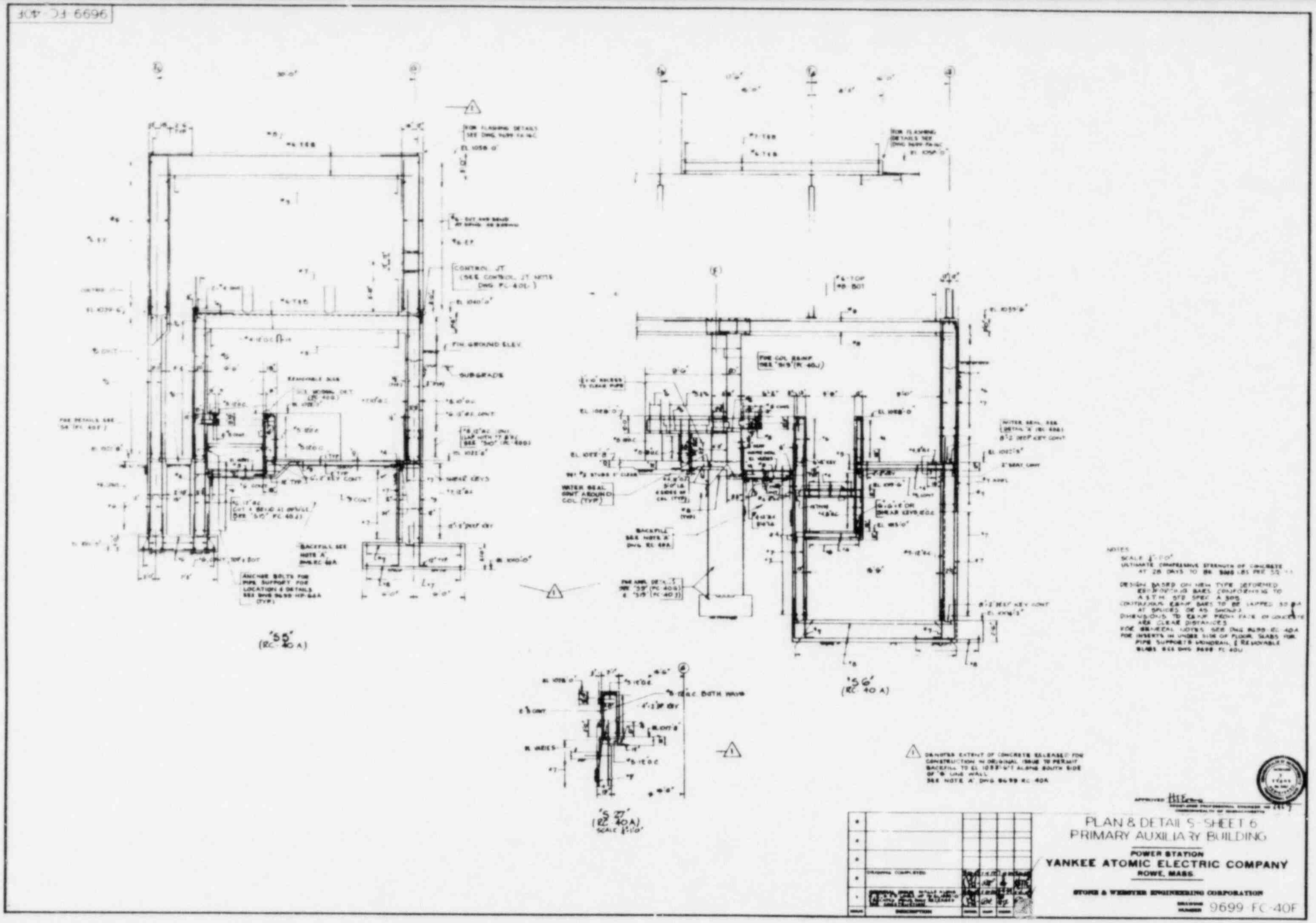

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FIG. A.9

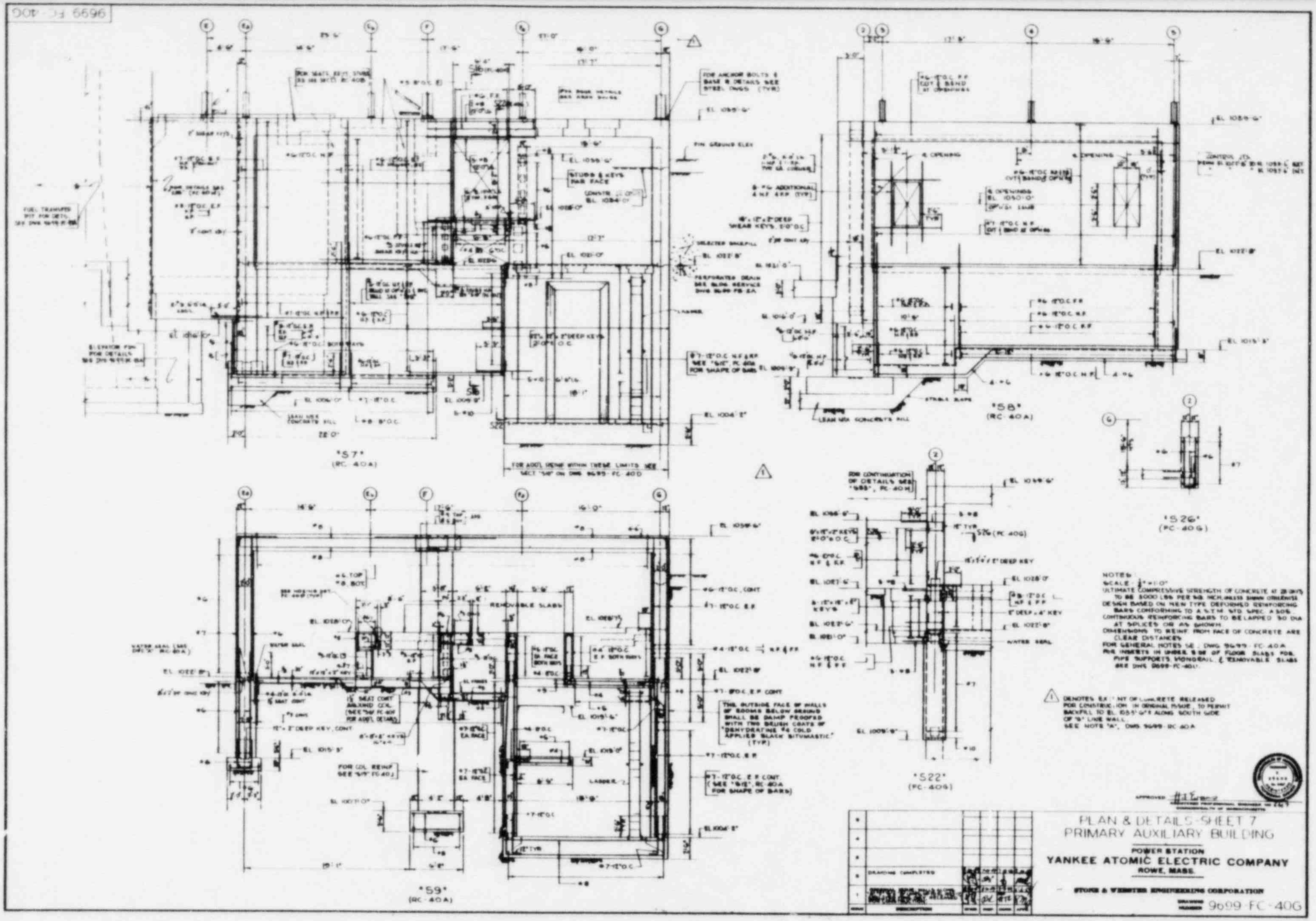

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FIG. A.10

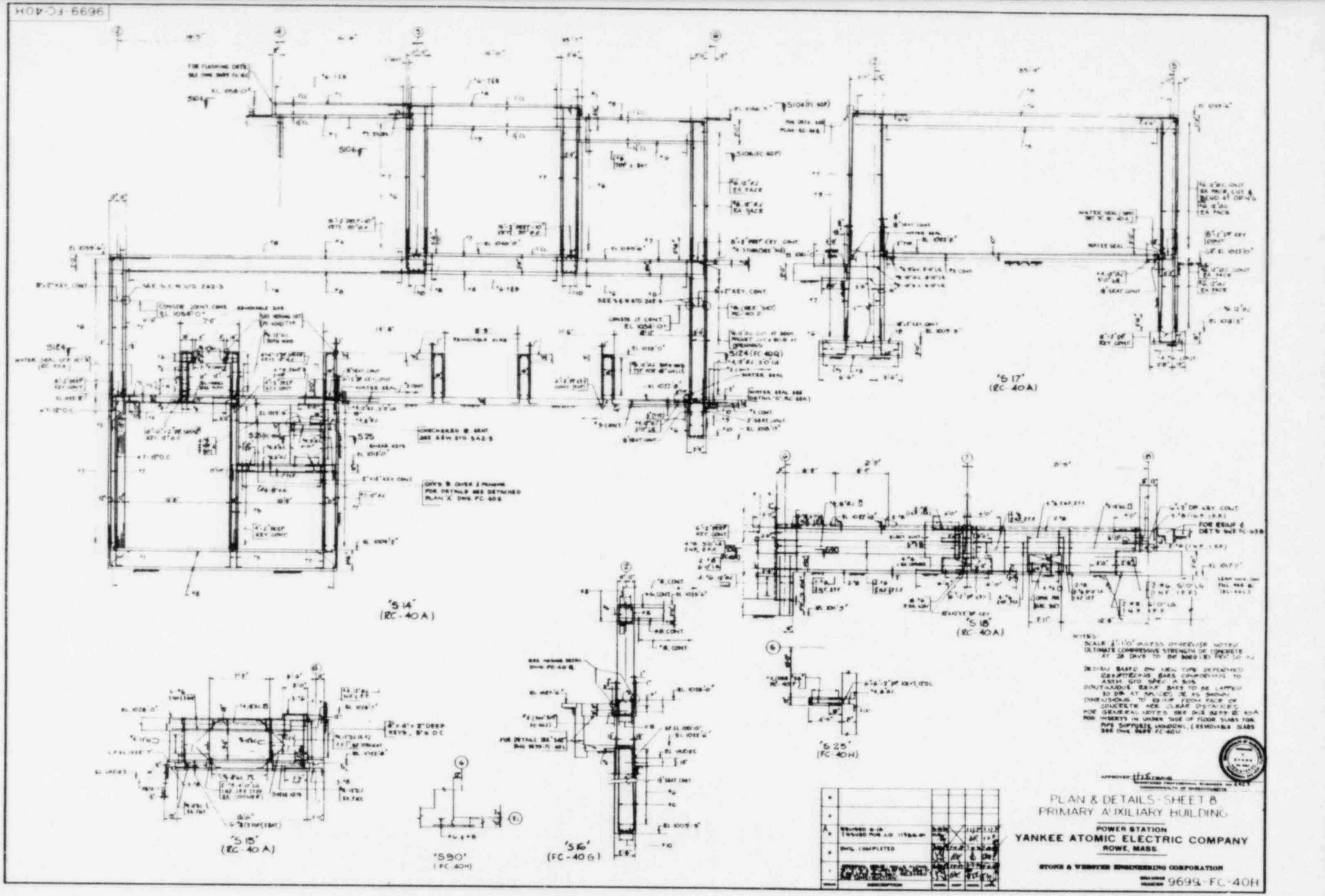

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FIG. A.11

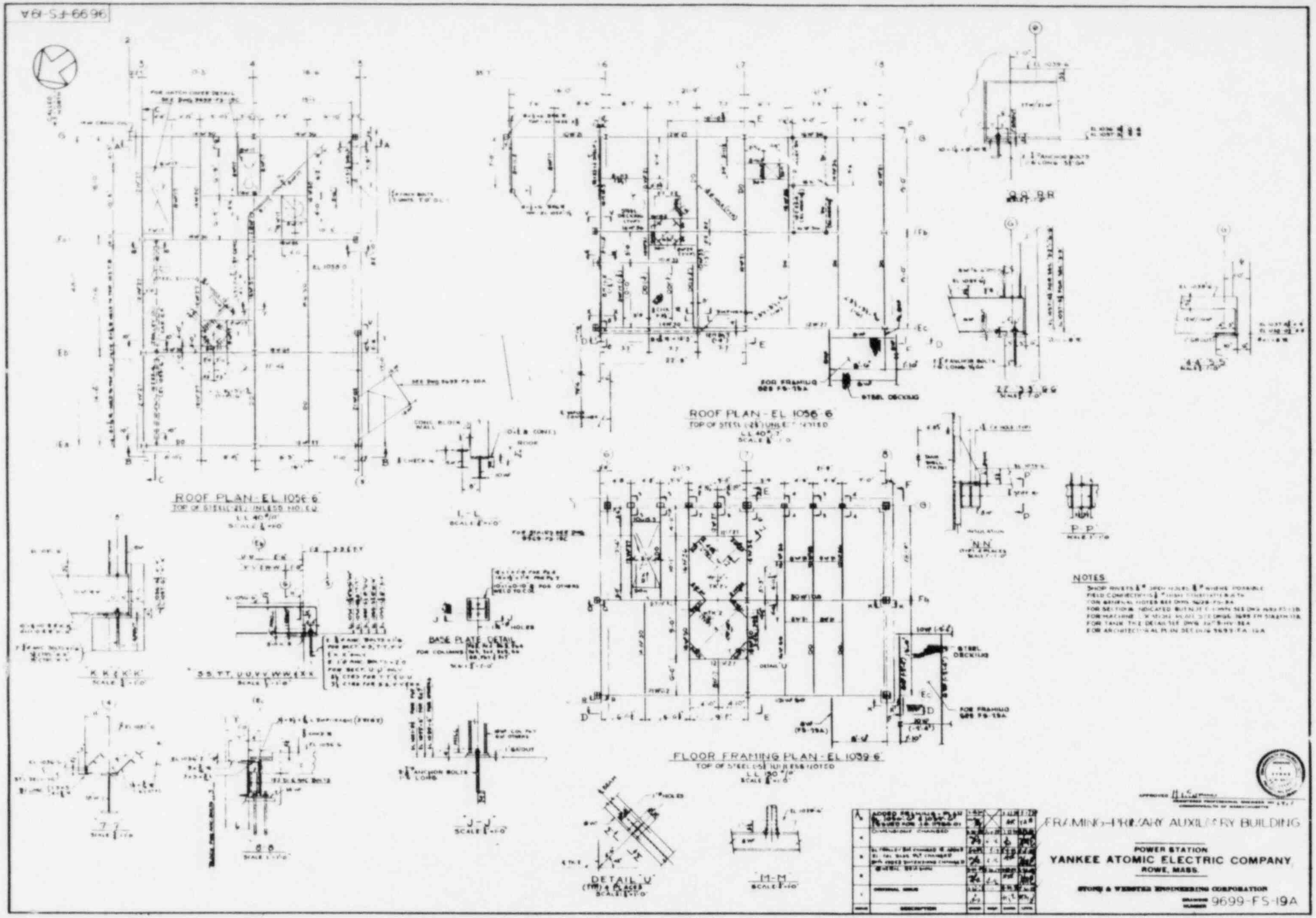

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FIG. A.12

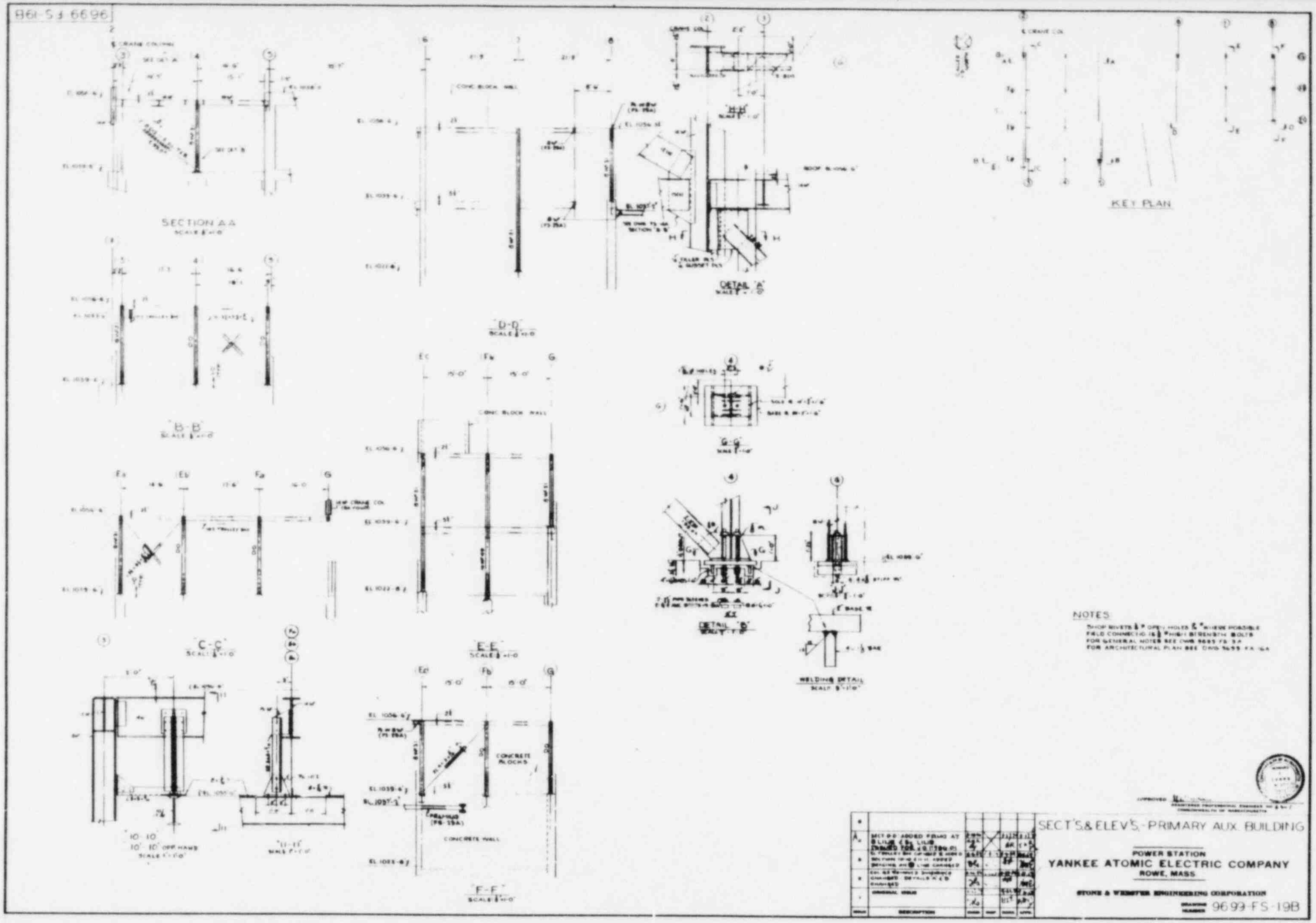

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FIG. A.13

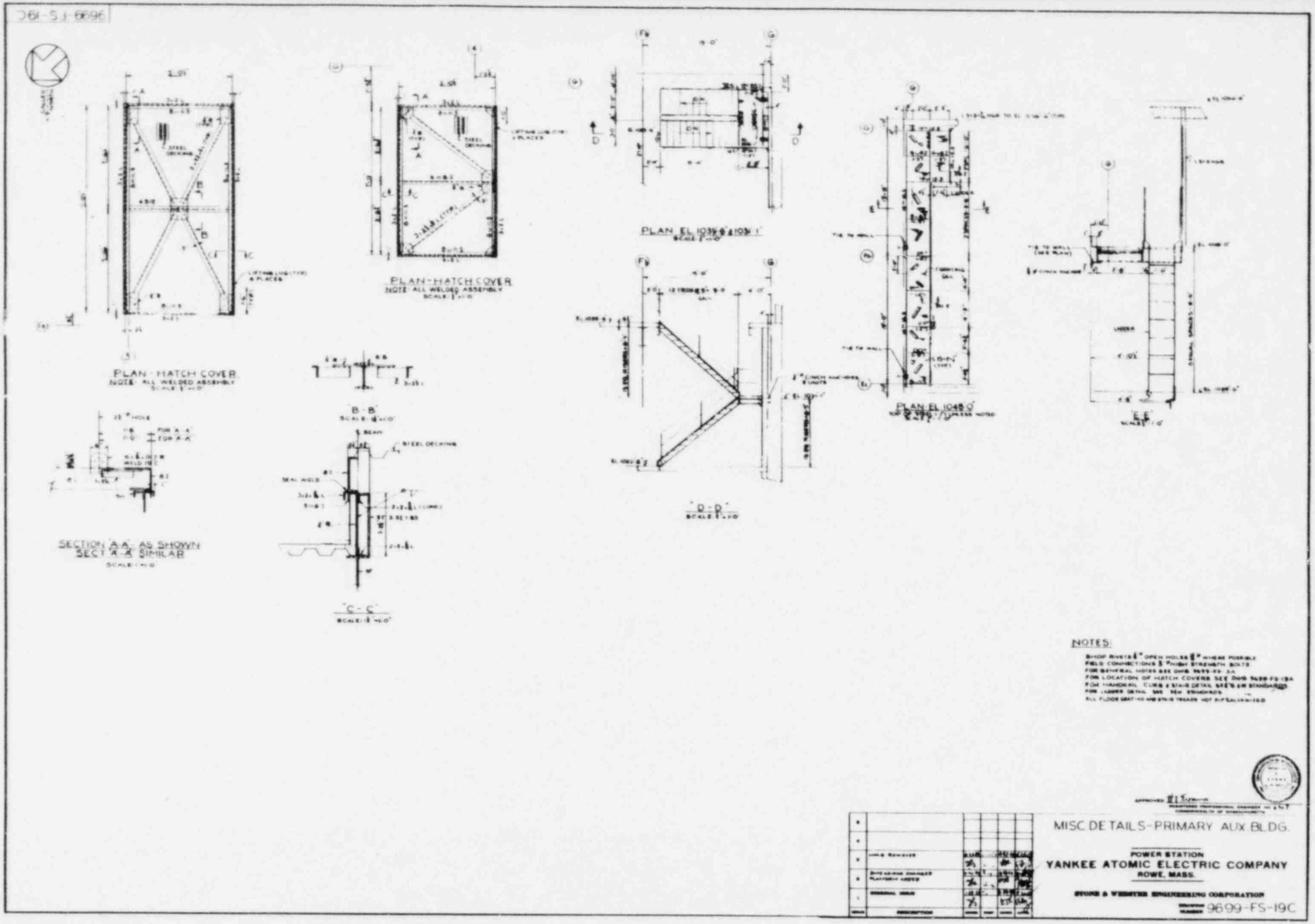

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FIG. A.14

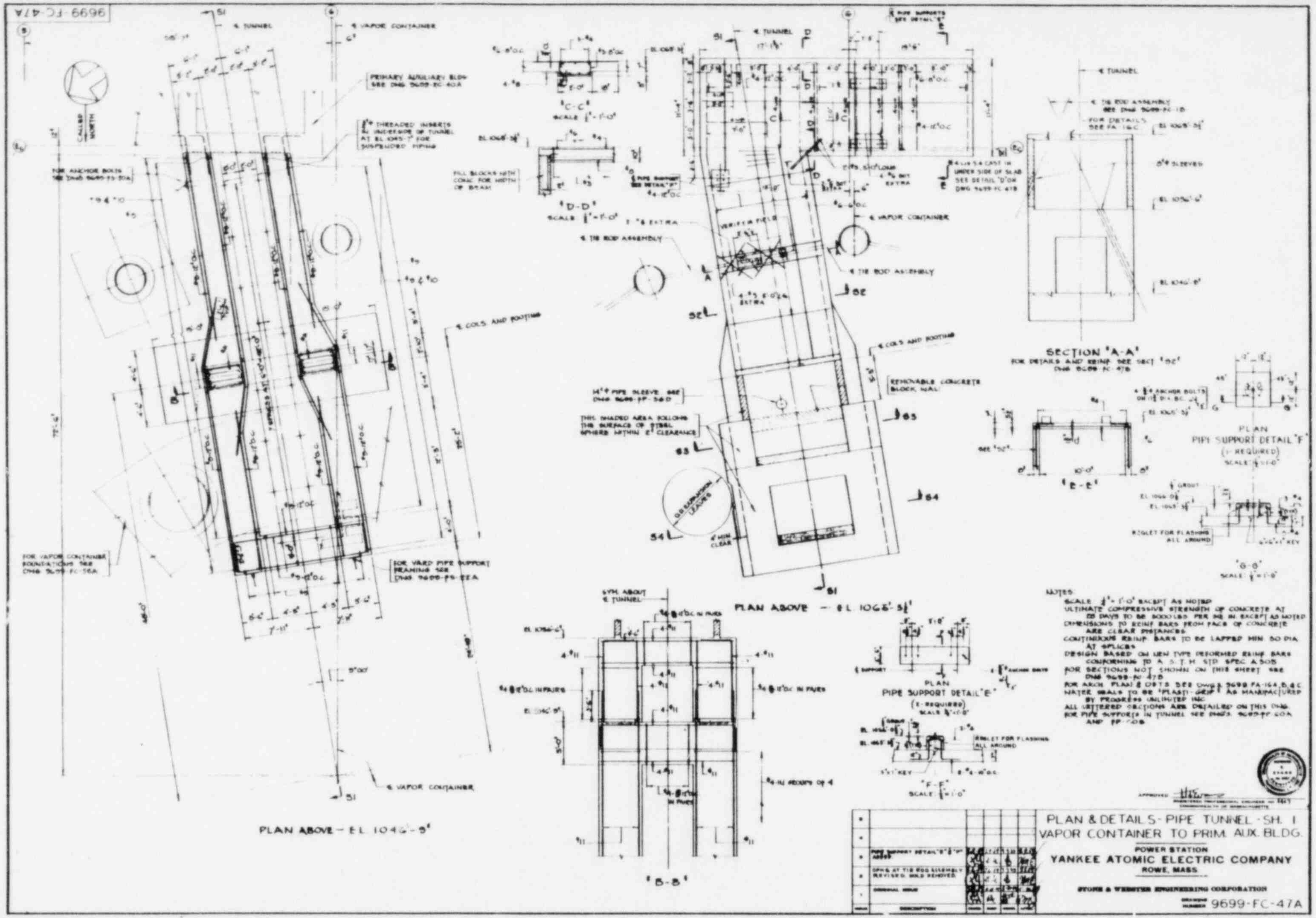

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FIG. A.15

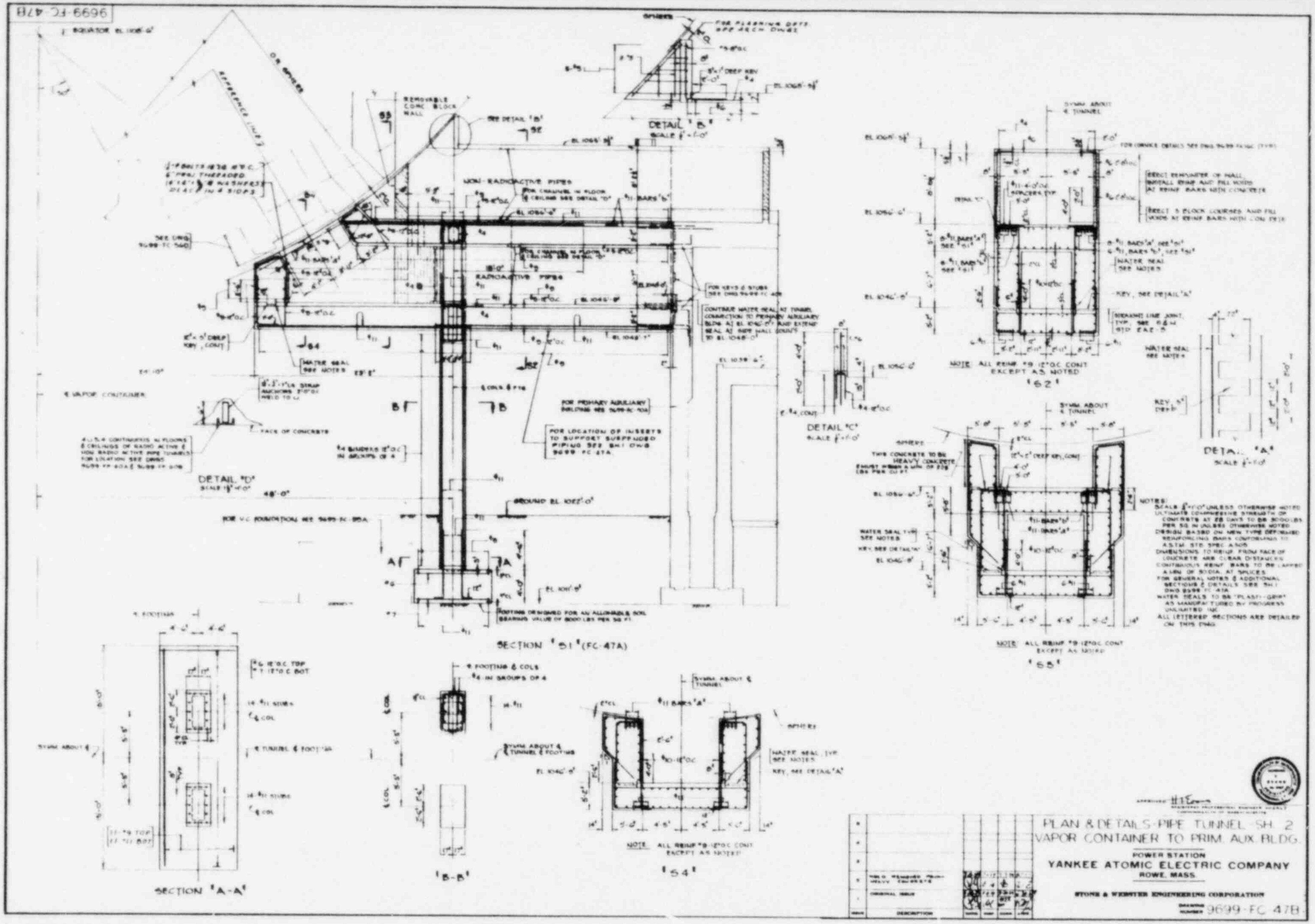

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FIG. A.16

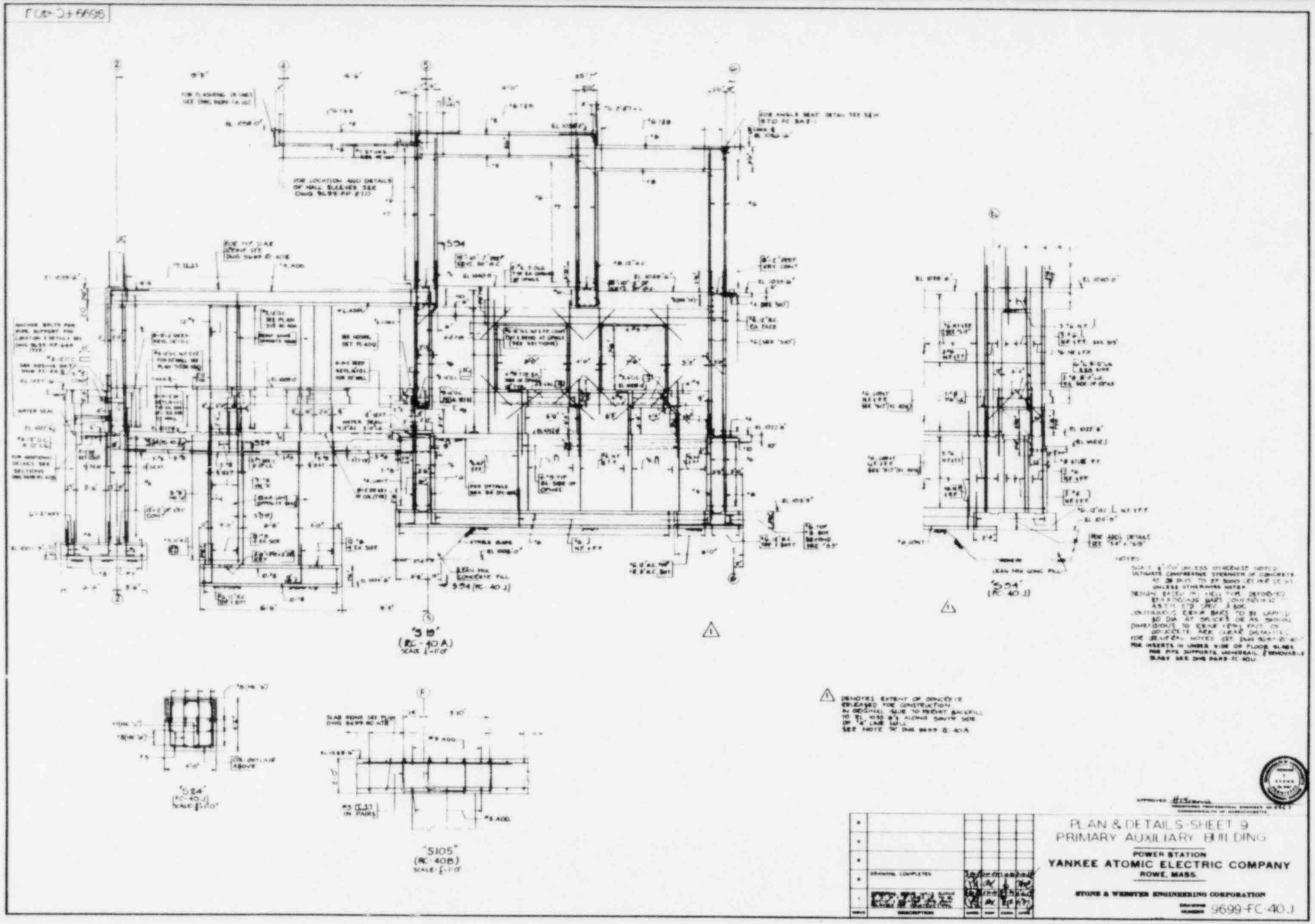

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FIG. A.17

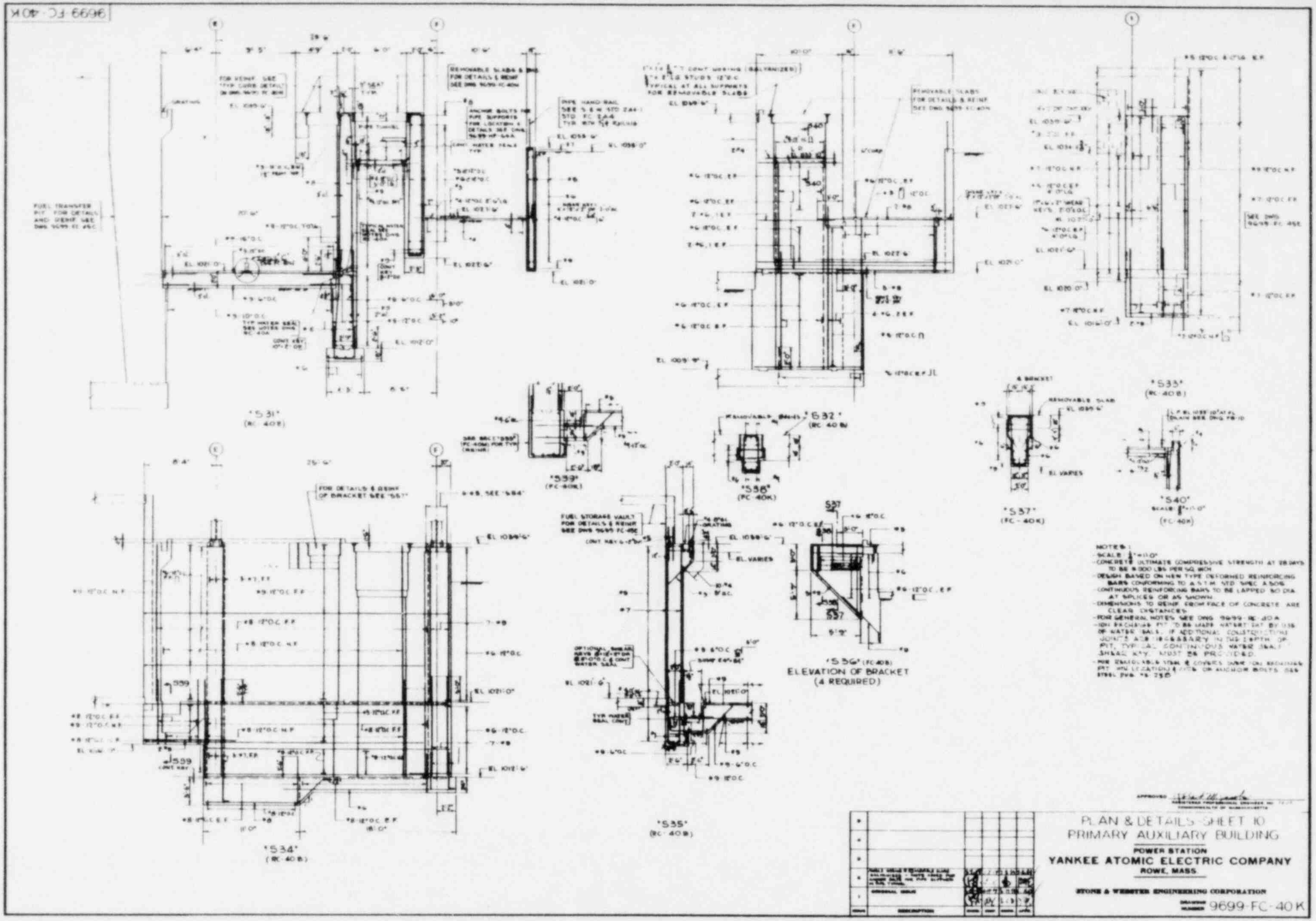

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FIG. A.18

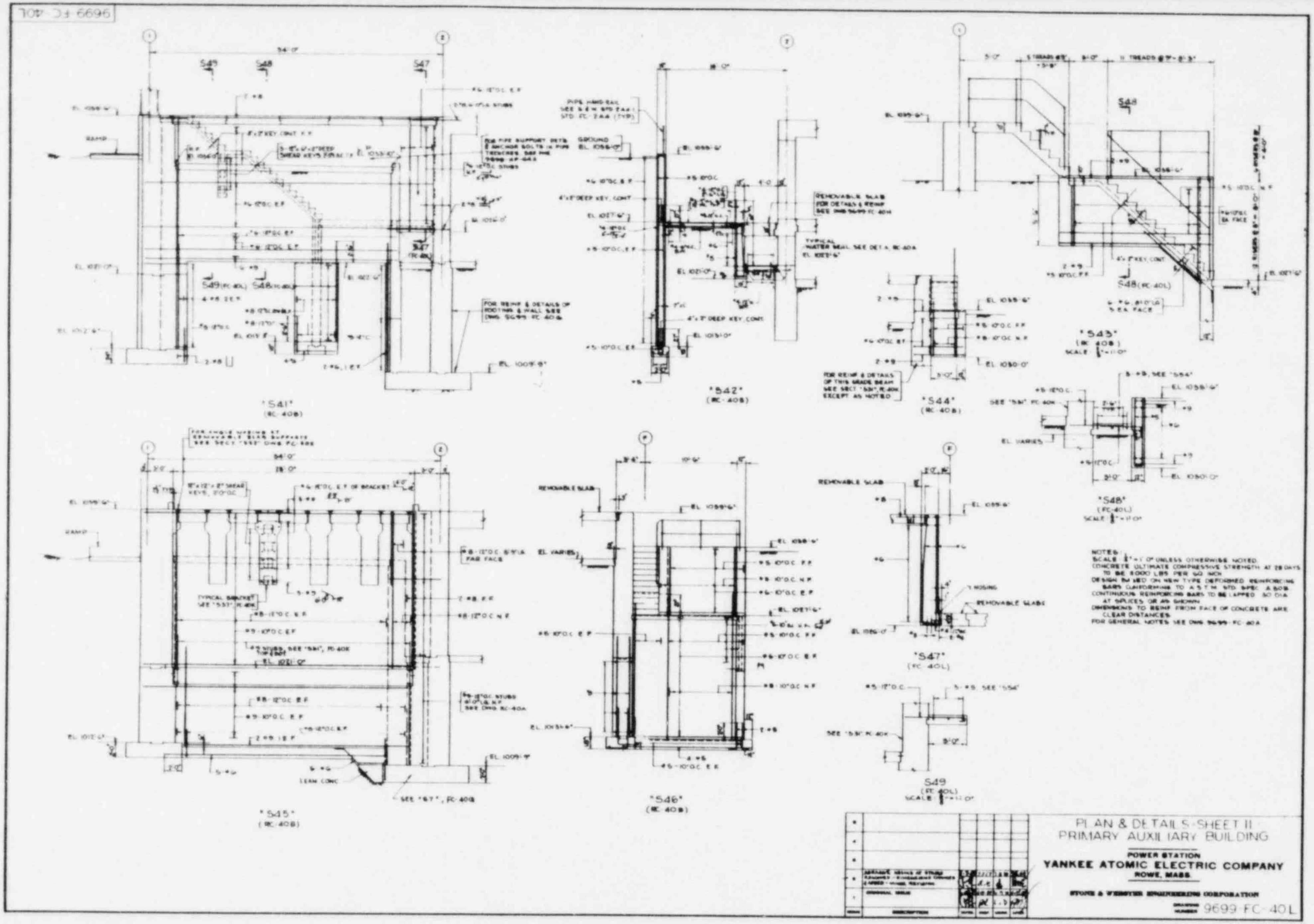

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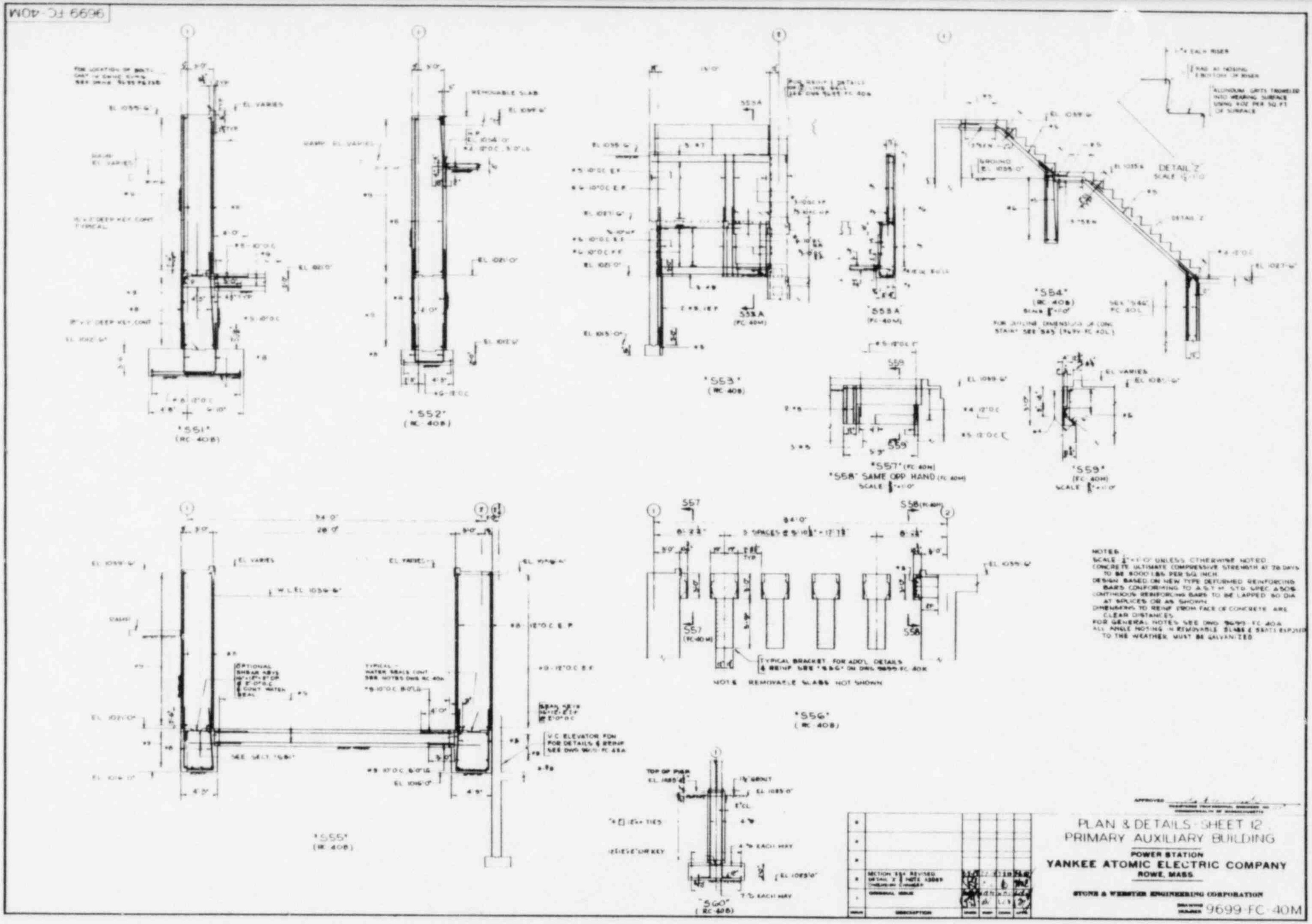
FIG. A.19



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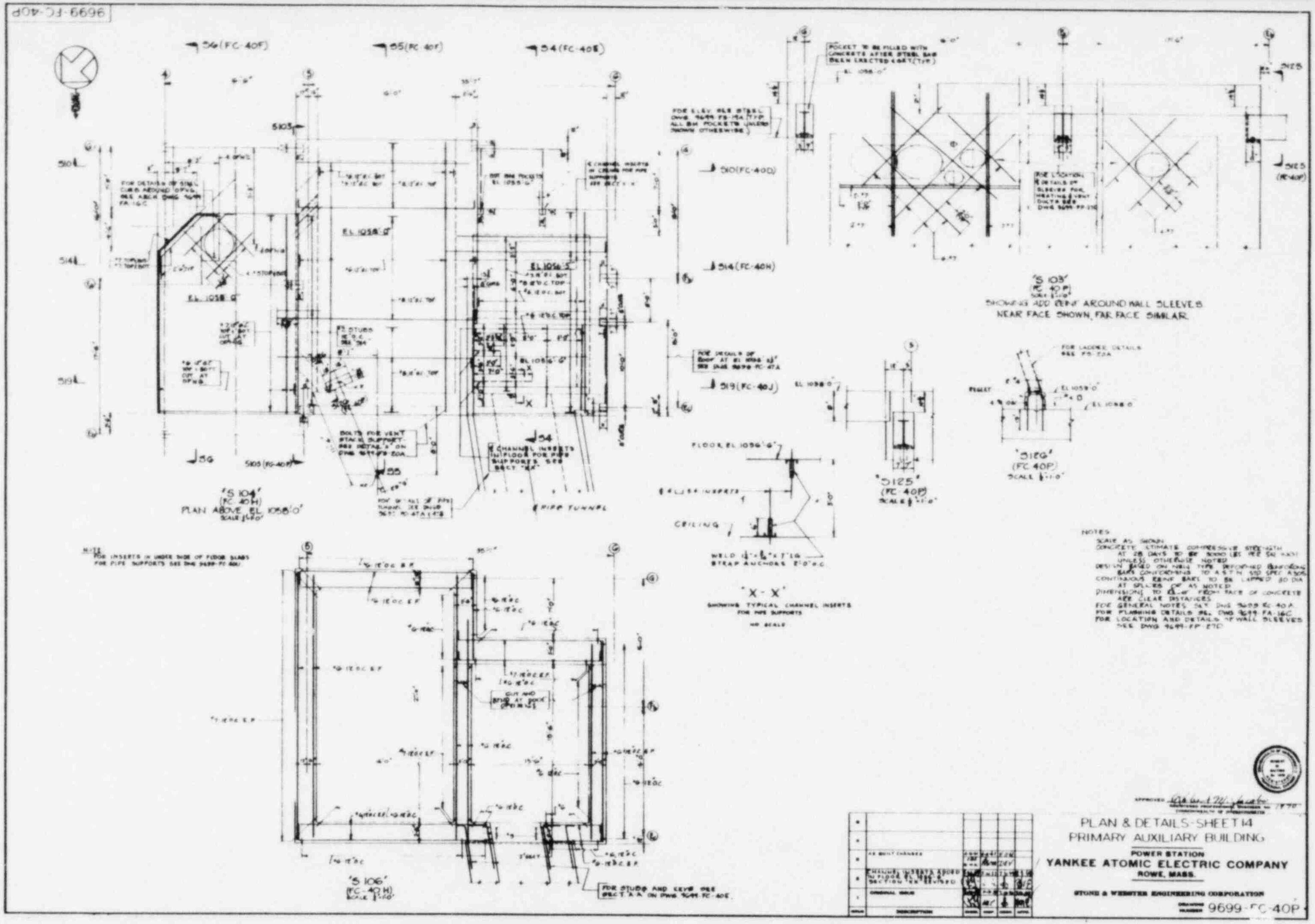
80023; EY-YR-80023-7; Rev. 2

FIG. A.20



Yankee Atomic Electric Company
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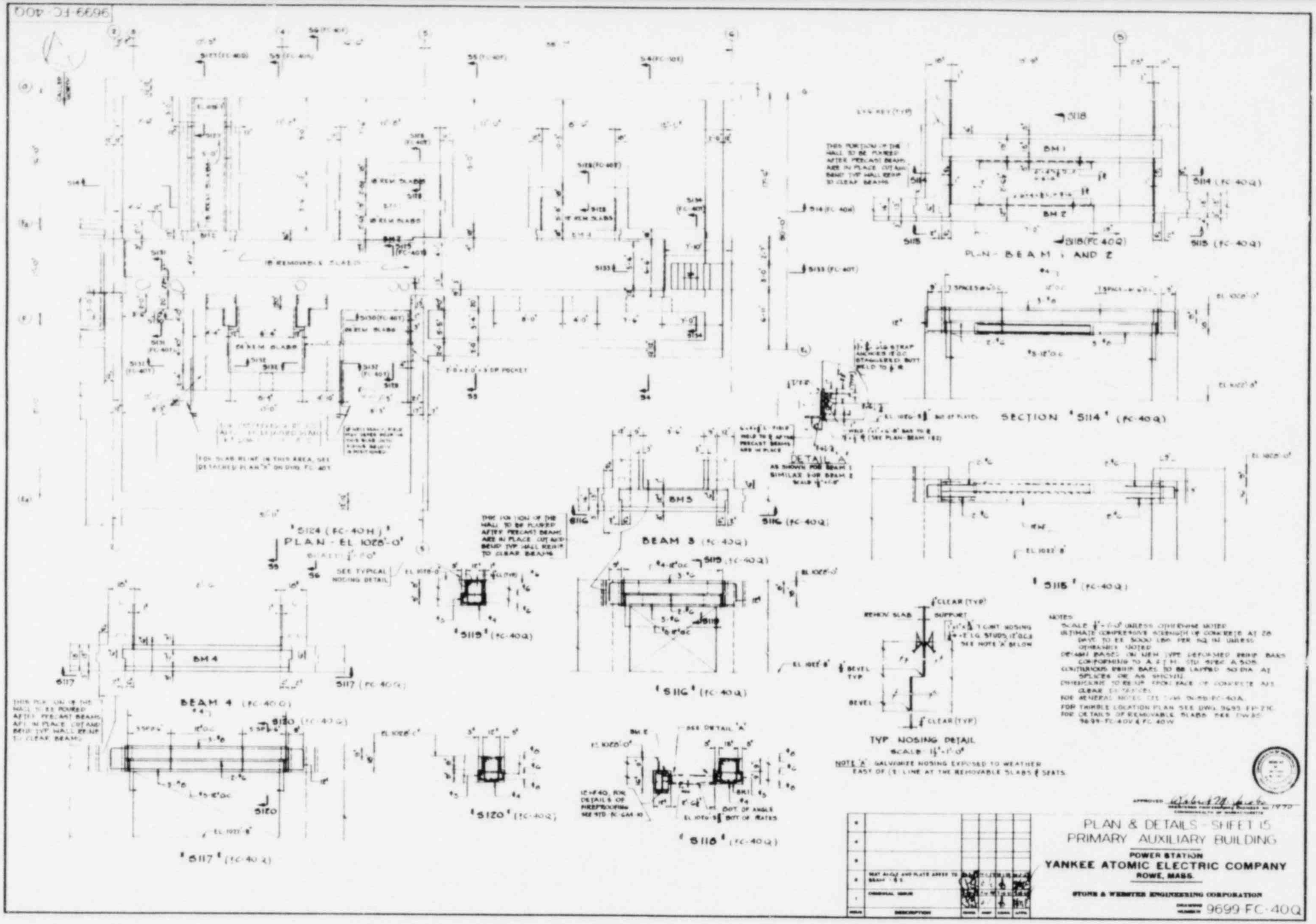
80023; EY-YR-80023-7; Rev. 2

FIG. A.21



CYGNA

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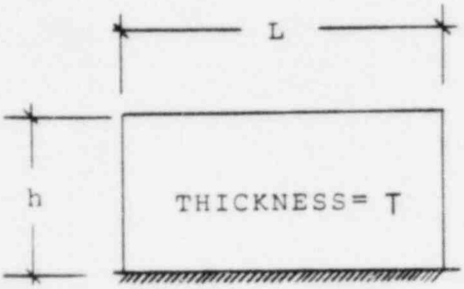
Primary Auxiliary Building
80023; EY-YR-80023-7; Rev. 2

FIG. A.22

APPENDIX B

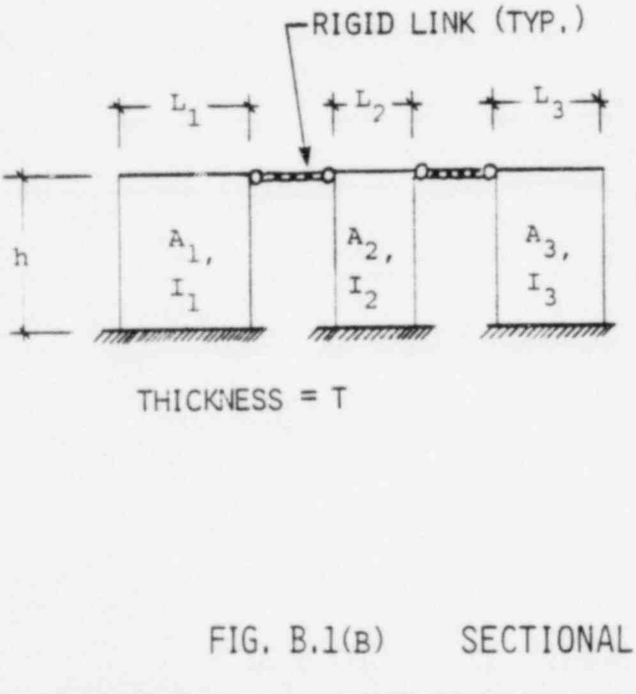
FINITE ELEMENT MODEL



CROSS-SECTIONAL AREA, $A = LT$

MOMENT OF INERTIA, $I = \frac{1}{12} TL^3$

FIG. B.1(A) SECTIONAL PROPERTIES OF WALL



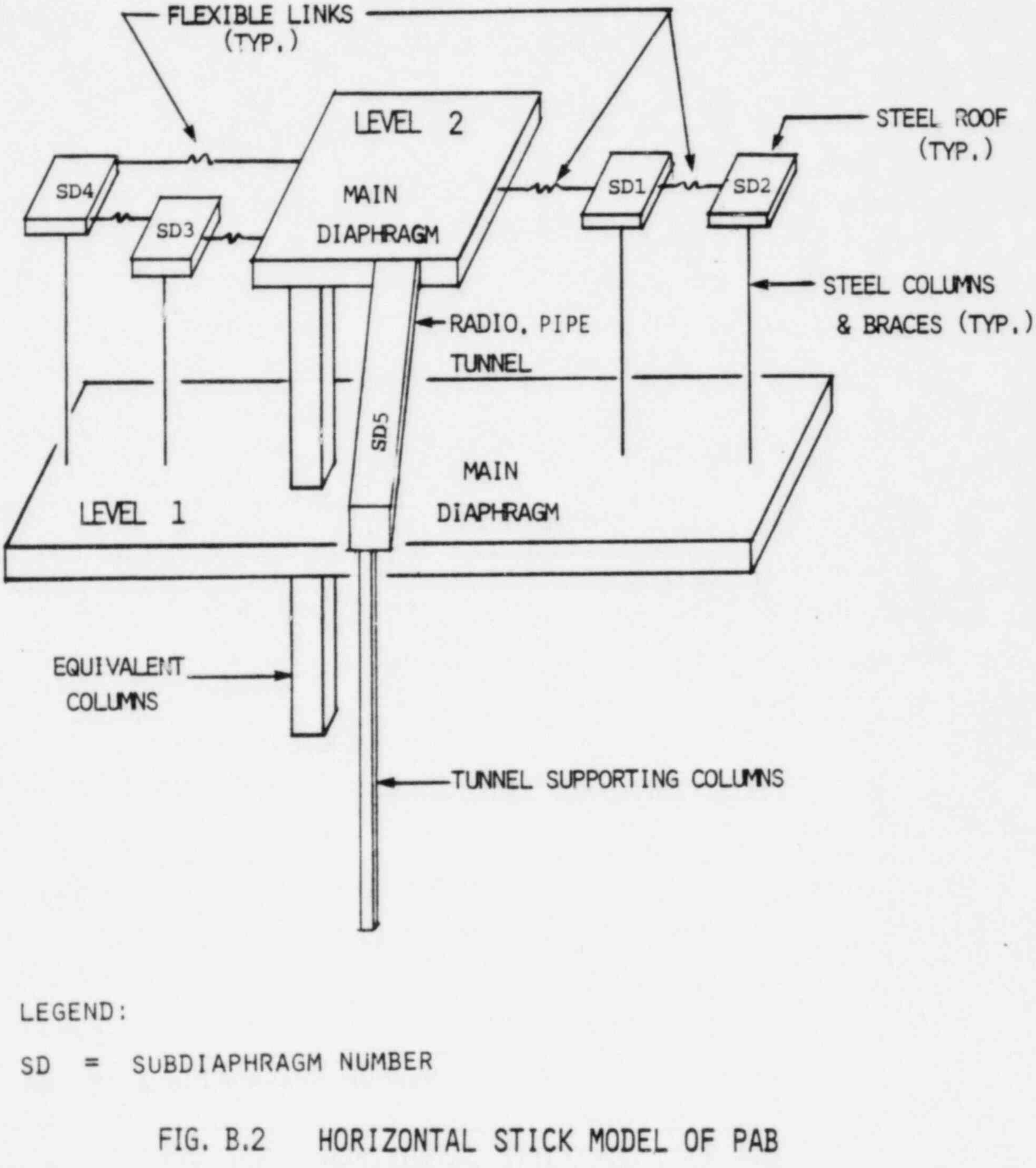
$L_1 + L_2 + L_3 = L$

TOTAL AREA = $A_1 + A_2 + A_3 = L_1T + L_2T + L_3T = LT = A$

TOTAL MOMENT OF INERTIA = $I_1 + I_2 + I_3 =$

$$\left(\frac{A1}{A}\right)I + \left(\frac{A2}{A}\right)I + \left(\frac{A3}{A}\right)I = I$$

FIG. B.1(B) SECTIONAL PROPERTIES OF EQUIVALENT COLUMNS



LEGEND:

SD = SUBDIAPHRAGM NUMBER

FIG. B.2 HORIZONTAL STICK MODEL OF PAB

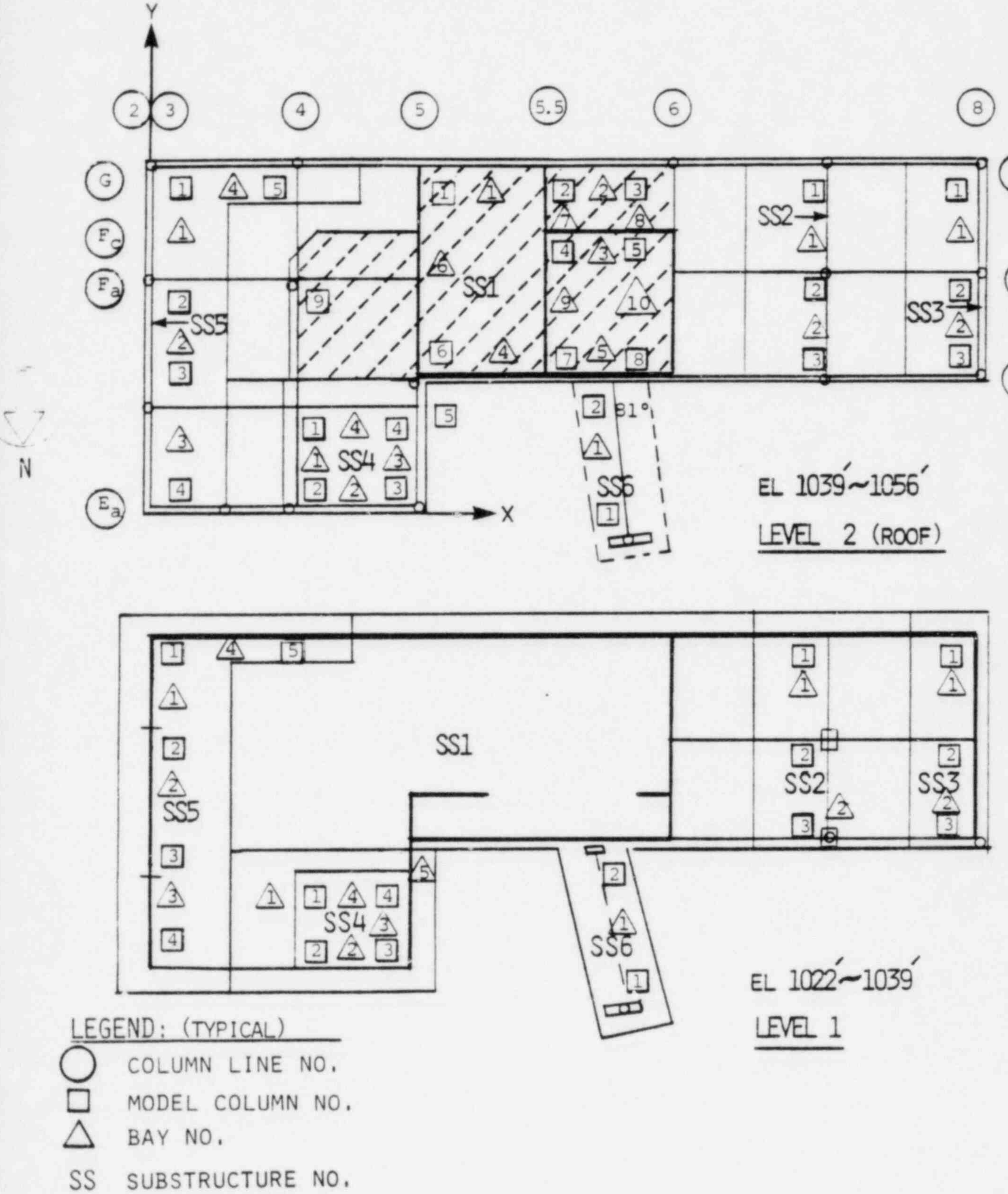
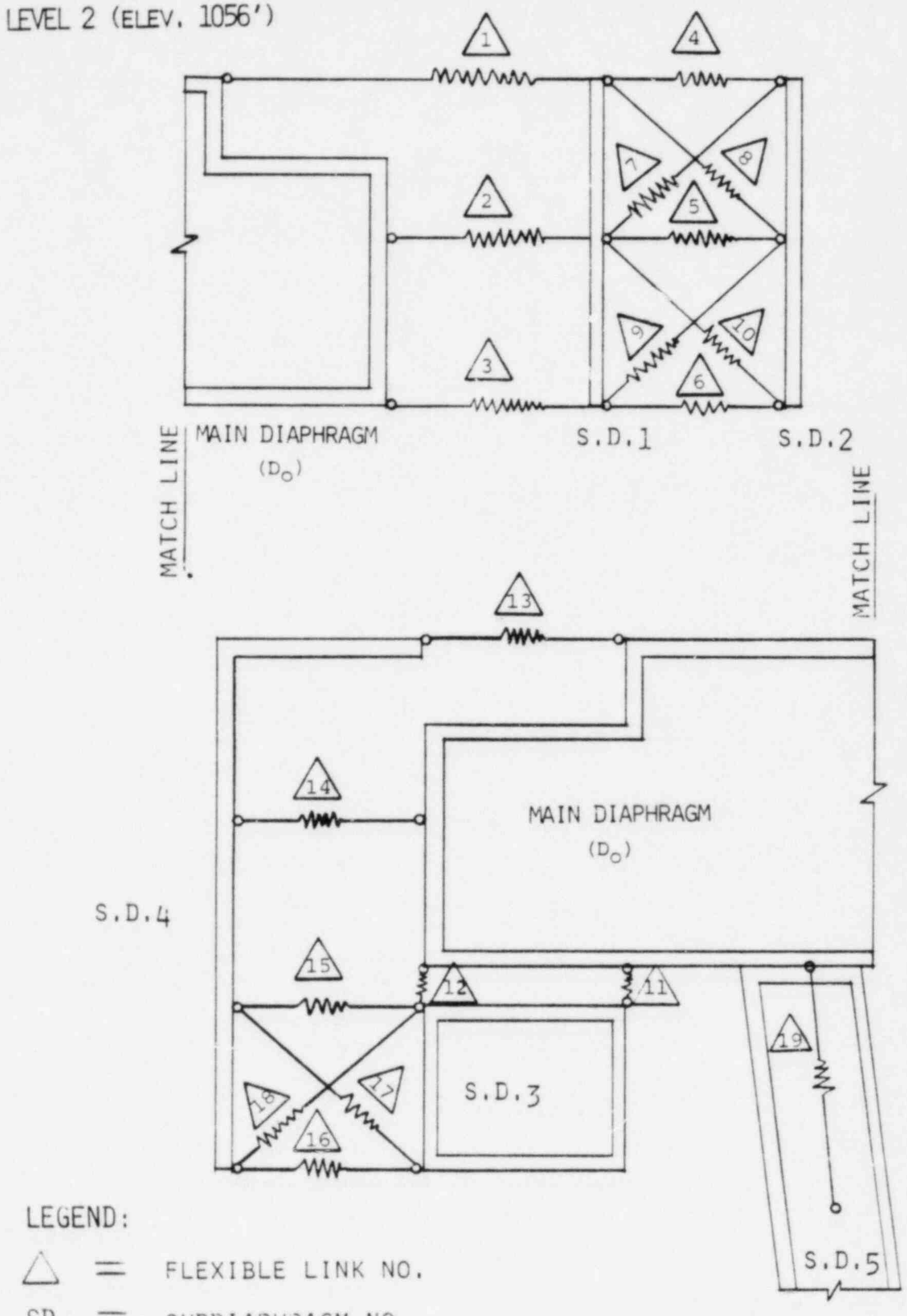


FIG. B.3 FRAMING OF SUBSTRUCTURES



LEGEND:

△ = FLEXIBLE LINK NO.

SD = SUBDIAPHRAGM NO.

FIG. B.4 FLEXIBLE LINKS

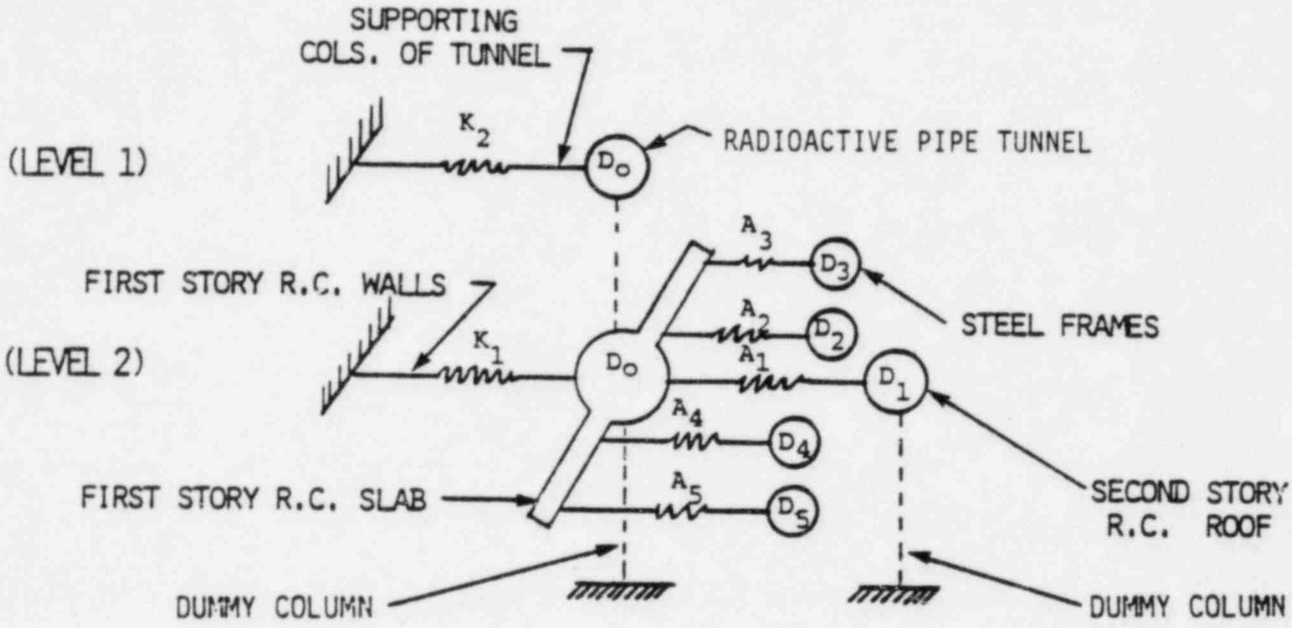


FIG. B.5 VERTICAL MODEL

APPENDIX C

EARTHQUAKE SPECTRA

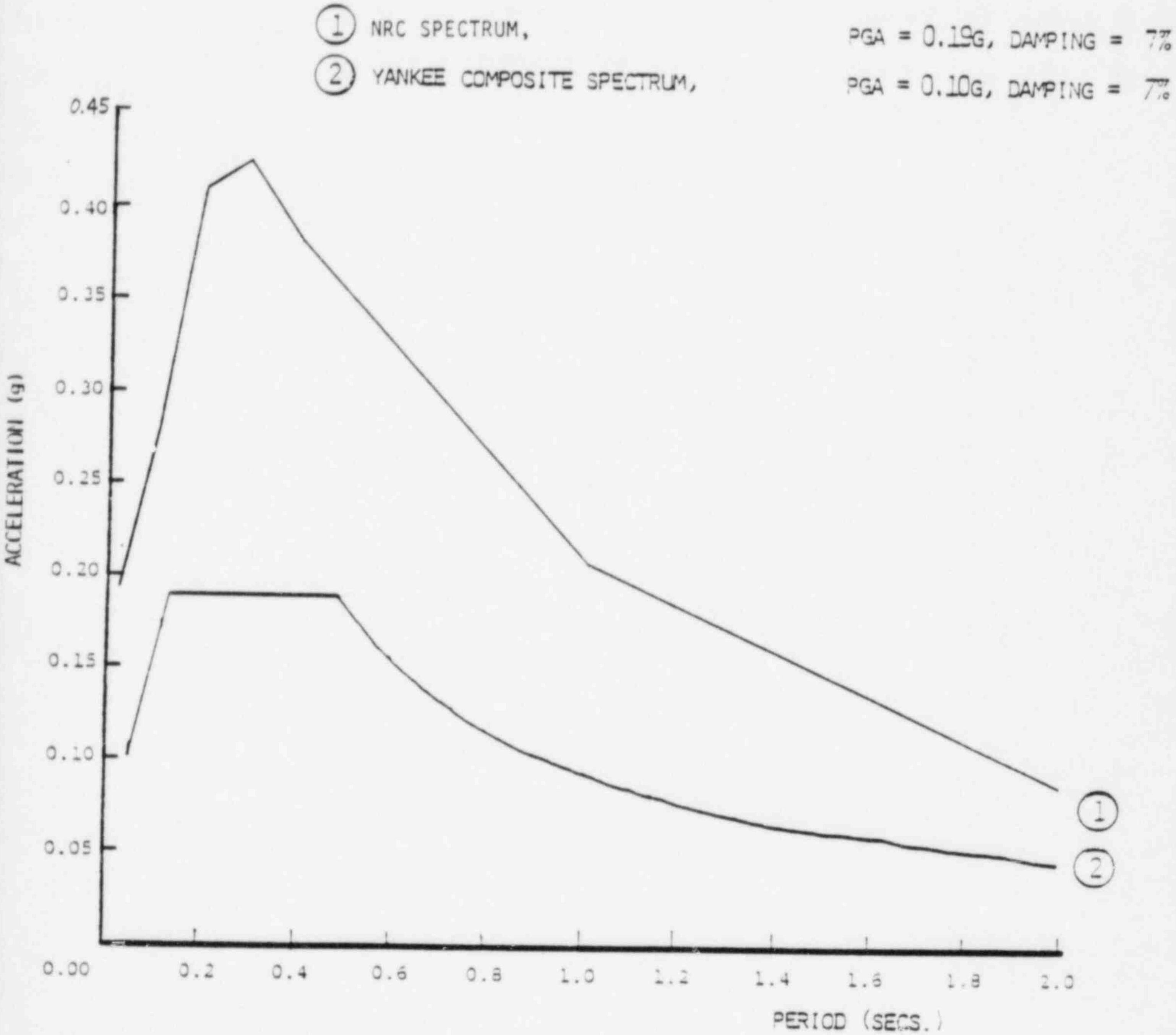


FIG. C.1 EARTHQUAKE SPECTRA

APPENDIX D

TABLE OF FREQUENCIES, MODE SHAPES AND MODAL MASSES

TABLE D.1

HORIZONTAL FREQUENCIES AND MODE SHAPES OF PAB

Mode No.	Freq. (Hz)	Description of Mode Shape
1	3.58	N-S movement of steel frames in C.L. 7 and 8
2	7.03	N-S movement of steel frame in C.L. 3
3	7.55	E-W movement of Radioactive Pipe Tunnel and N-S movement of steel frame in C.L. 3
4	8.63	E-W movement of steel frames in C.L. Ec and 3
5	9.28	N-S movement of steel frames in C.L. 8 and 7
6	13.37	E-W movement of steel frames in C.L. 8 and 7
7	14.59	N-S movement of PAB and Radioactive Pipe Tunnel
8	19.32	E-W movement of PAB and Radioactive Pipe Tunnel
9	24.94	E-W movement of PAB and Radioactive Pipe Tunnel
10	25.57	N-S and E-W movement of steel frame in C.L. 8
11	31.60	E-W movement of steel frame in C.L. 3
12	33.89	N-S movement of first story slab and N-S movement of Radioactive Pipe Tunnel
13	37.58	N-S movement of steel frame in C.L. Ec
14	42.32	E-W movement of steel frames in C.L. 7 and 8
15	45.06	Second E-W vibration mode of PAB (excluding Radioactive Pipe Tunnel)
16	45.57	E-W movement of steel frames in C.L. Ec

Note: C.L. denotes Column Line

TABLE D-2

HORIZONTAL MODAL MASSES

Mode No.	Freq. (Hz)	Direction	Modal Mass (K-Sec²/in)	Percent Mass (%)	Cumulative Percent Mass (%)
1	3.58	E-W (X)	0.	0.	0.
		N-S (Y)	0.24	1.94	1.94
2	7.03	E-W	0.13	1.07	1.07
		N-S	0.17	1.34	3.28
3	7.55	E-W	2.69	21.36	22.43
		N-S	0.20	1.62	4.90
4	8.63	E-W	0.13	1.07	23.50
		N-S	0.	0.	4.90
5	9.28	E-W	0.05	0.36	23.86
		N-S	0.03	0.23	5.13
6	13.37	E-W	0.71	5.62	29.48
		N-S	0.03	0.26	5.39
7	14.59	E-W	4.51	35.83	65.90
		N-S	9.54	75.76	81.15
8	19.32	E-W	4.51	35.83	65.90
		N-S	0.01	0.08	81.23
9	24.94	E-W (X)	1.85	14.66	80.56
		N-S (Y)	0.12	0.94	82.17
10	25.57	E-W	0.03	0.25	80.81
		N-S	0.002	0.02	82.19
11	31.60	E-W	0.39	3.14	83.95
		N-S	0.04	0.28	82.47
12	33.89	E-W	0.15	1.15	85.10
		N-S	2.10	16.67	99.14
13	37.58	E-W	0.59	4.65	89.75
		N-S	0.01	0.09	99.23
14	42.32	E-W	0.05	0.40	90.15
		N-S	0.	0.	99.23
15	45.06	E-W	0.82	6.52	96.67
		N-S	0.05	0.37	99.60
16	45.57	E-W	0.03	0.25	96.92
		N-S	0.003	0.024	99.63

TABLE D-3

VERTICAL FREQUENCIES AND MODE SHAPES OF PAB

Mode No.	Freq. (Hz)	Description of Mode Shape
1	21.9	Vertical movement of Radioactive Pipe Tunnel
2	29.8	Vertical movement of steel frame in Col. Line 7
3	30.4	Vertical movement of steel frame in Col. Line 8
4	31.6	Vertical movement of steel frame in Col. Lines 4 and 5
5	31.8	Vertical movement of steel frame in Col. Lines 2 and 3
6	53.2	In-phase vibration of concrete slabs in Levels 1 and 2
7	110.2	Out-of-phase vibration of concrete slabs in Levels 1 and 2

TABLE D-4

VERTICAL MODAL MASSES

Mode No.	Freq. (Hz)	Modal Mass $\left(\frac{K\text{-}Sec^2}{in}\right)$	Percent Mass (%)	Cumulative Percent Mass (%)
1	21.9	2.583	20.8	20.8
2	29.8	0.371	3.0	23.8
3	30.4	0.192	1.55	25.35
4	31.6	0.341	2.75	28.1
5	31.8	0.098	0.8	28.9
6	53.2	8.266	66.6	95.5
7	110.2	0.563	4.5	100.0

APPENDIX E

TABLE OF ELEMENT STRESSES

TABLE E-1

SHEAR FORCE IN R.C. WALLS UNDER YCS - 7% DAMPING

Level No.	Location	Shear Area (in^2)	Total Shear Force (kips)	Shear Stress (psi)	Stress Ratio Shear Stress / $2\sqrt{f'c}$ = 110 psi
1	Col. Line 2	16700.	192.2	11.5	0.10
	Col. Line 5	6670.	78.4	11.8	0.11
	Col. Line 6	8640.	125.9	14.6	0.13
	Col. Line 8	5760.	106.6	18.5	0.17
	Col. Line Ea	10220.	88.0	8.6	0.08
	Col. Line Ec	13270.	50.3	3.8	0.03
	Col. Line G	28990.	170.2	5.9	0.05
2	Col. Line 5	10800.	140.3	130.0	0.12
	Col. Line 5.5	9250.	115.7	12.5	0.11
	Col. Line 6	6190.	125.3	20.2	0.18
	Col. Line Fc	6590.	20.5	3.1	0.03
	Col. Line Ec	12240.	205.3	16.8	0.15
	Col. Line G	6750.	65.8	9.8	0.09

TABLE E-2

SUMMARY OF FORCES UNDER YCS - 7% DAMPING
DIAGONAL BRACES IN "X" PAIR

Element No.	Location	Area (in^2)	Axial Force (kips) X-EQ	Axial Force (kips) Y-EQ	Axial Force (kips) SRSS of X & Y EQ	Axial Stress (ksi)	Total Axial Stress to be taken by Tension Mem. F_t (ksi)	Stress* Ratio $\frac{F_t}{F_a}$
Flex. Link 7	Roof Col. Lines 7 & 8	2.09	1.422	5.246	5.435	2.60	5.03	0.19
Flex. Link 8	B & Fb		1.903	4.718	5.087	2.43		
Flex. Link 9	Roof Col. Lines 7 & 8	2.09	1.235	4.628	4.790	2.29	5.04	0.19
Flex. Link 10	Fb & Ec		2.118	5.336	5.741	2.75		
Flex. Link 17	Roof Col. Lines 3 & 4	2.09	5.887	4.560	7.447	3.56	3.36	0.13
Flex. Link 18	Eb & Ea		5.820	4.546	7.385	3.53		
SS4 Diag. Brace 1	Level 2	2.09	3.459	0.614	3.513	1.68	3.36	0.13
SS4 Diag. Brace 2	Col. Line Ea		3.457	0.613	3.511	1.68		
SS5 Diag. Brace 2	Level 2	2.09	5.634	5.037	7.557	3.62	7.24	0.27
SS5	Col. Line 3		5.634	5.037	7.557	3.62		

* Fa = 1.33 x 0.6 Fy = 26.4 ksi

TABLE E-3

AXIAL FORCE IN SINGLE DIAGONAL BRACE UNDER YCS - 7% DAMPING

Element No.	Location	Area (in^2)	Axial Force (kips) X-EQ	Axial Force (kips) Y-EQ	Axial Force (kips) SRSS of X & Y EQ	Axial Stress (ksi)	Allowable Compression Stress Fa (ksi)	Stress Ratio Axial Stress / Fa
SS3 Diag. Brace 1	Level 2 Col. Line 8	4.18	3.21	24.86	25.07	6.00	4.34*	1.38
SS5 Diag. Brace 1	Level 2 Col. Line G	6.72	2.10	0.59	2.18	0.32	16.97	0.02

*Based on $\frac{Kl}{r}$ = 214. Note that the maximum $\frac{Kl}{r}$ ratio for the secondary member allowed by UBC is 200.

TABLE E-4

AXIAL FORCE IN STEEL COLUMN (LEVEL 2) UNDER YCS - 7% DAMPING

Location	Area (in^2)	Due to Dead Load (ksi)	Axial Force (kips) Due to EQ Loads			Axial Stress Due to EQ Loads (ksi)	Total Axial Stress F_t (ksi)	Allowable Comp. Stress F_a (ksi)	Stress Ratio $\frac{F_t}{F_a}$
			X-EQ	Y-EQ	SRSS of X & Y EQ				
Col. Lines 2 & G	41.85	0.20	1.39	0.39	1.44	0.03	0.23	17.16	0.01
Col. Lines 3 & Fa	9.12	1.87	0.	0.	0.	0.	1.87	12.76	0.15
Col. Lines 3 & Ea		0.94	4.29	3.83	5.75	0.63	1.57		0.12
Col. Lines 4 & G		0.94	0.	0.	0.	0.	0.94		0.07
Col. Lines 4 & Eb		1.00	0.	0.	0.	0.	1.00		0.08
Col. Lines 4 & Ea		1.00	2.51	0.45	2.55	0.28	1.28		0.10
Col. Lines 5 & Ea		1.00	0.	0.	0.	0.	1.0		0.08
Col. Lines 7 & G	9.12	1.40	0.	0.	0.	0.	1.40	12.76	0.11
Col. Lines 7 & Fb	14.40	1.77	0.	0.	0.	0.	1.77	14.92	0.12
Col. Lines 7 & Ec	9.12	1.40	0.	0.	0.	0.	1.40	12.76	0.11
Col. Lines 8 & G	9.12	1.07	0.	0.	0.	0.	1.07	12.76	0.08
Col. Lines 8 & Fb	9.12	2.15	2.41	18.64	18.80	2.06	4.21	12.76	0.33

TABLE E-5

FORCES DEVELOPED IN STEEL BEAM -
CONCRETE WALL CONNECTION UNDER YCS - 7% DAMPING

Location	Flexible Link No.	Connection Shear (kips) X-EQ	Y-EQ	SRSS of X & Y EQ	Allowable Shear Stress Va (kips)	Stress Ratio Conn. Shear / Va
Col. Lines 5.5 & G	1	6.31	7.58	9.86	0.	N.A.
Col. Lines 6 & Fb	2	9.85	4.60	10.87	6.6	1.65
Col. Lines 6 & Ec	3	8.52	10.14	13.24	6.6	2.01
Col. Lines 5 & G	13	5.46	1.49	5.66	6.6	0.86
Col. Lines 5 & Ec	11	4.75	4.08	6.26	8.05	0.95

TABLE E-6

MOMENTS IN RADIOACTIVE PIPE TUNNEL AND SUPPORTING COLUMNS UNDER YCS - 7% DAMPING

	COMP.	Moment (K-in) X-EQ	Moment (K-in) Y-EQ	Moment (K-in) SRSS of X & Y EQ	Crack Moment Mcr (K-in)	Ultimate Moment Mu (K-in)	Stress Ratio Against Crack $\frac{M_1}{Mcr_1} + \frac{M_2}{Mcr_2}$	Stress Ratio Against Failure $\frac{M_1}{Mu_1} + \frac{M_2}{Mu_2}$
Rad. Tunnel	About Z-Axis	47227.	13682.	49169.	1.92×10^5	2.52×10^5	0.26	0.20
Column	About X-Axis	75.	345.	353.	6967.*	16567.*	0.56	0.23
	About Y-Axis	6030.	1700.	6265.	12297.*	29925.*		

*Evaluated under axial compression = D.L. - Vert. YCS = 529 kips

TABLE E-7

SHEAR FORCE IN R.C. WALLS UNDER YCS - 7% DAMPING

Level No.	Location	Shear Area (in^2)	Total Shear Force (kips)	Shear Stress (psi)	Stress Ratio Shear Stress / $2\sqrt{f'c}$ = 110 psi
1	Col. Line 2	16700.	318.0	19.1	0.17
	Col. Line 5	6670.	131.3	19.7	0.18
	Col. Line 6	8640.	207.0	24.0	0.22
	Col. Line 8	5760.	175.0	30.4	0.28
	Col. Line Ea	10220.	148.7	14.6	0.13
	Col. Line Ec	13270.	88.1	6.6	0.06
	Col. Line G	28990.	293.8	10.1	0.09
2	Col. Line 5	10800.	231.3	21.4	0.19
	Col. Line 5.5	9250.	188.7	20.4	0.19
	Col. Line 6	6190.	206.9	33.4	0.30
	Col. Line Fc	6590.	35.9	5.5	0.05
	Col. Line Ec	12240.	343.6	28.1	0.26
	Col. Line G	6750.	114.0	16.9	0.15

TABLE E-8

SUMMARY OF FORCES UNDER NRC SPECTRUM - 7% DAMPING
DIAGONAL BRACES IN "X" PAIR

Element No.	Location	Area (in^2)	Axial Force (kips) X-EQ	Axial Force (kips) Y-EQ	Axial Force (kips) SRSS of X & Y EQ	Axial Stress (ksi)	Total Axial Stress to be taken by Tension Mem. F_t (ksi)	Stress* Ratio $\frac{F_t}{F_a}$
Flex. Link 7	Roof Col. Lines 7 & 8	2.09	2.319	11.578	11.808	5.65	10.84	0.41
Flex. Link 8	B & Fb		3.09	10.405	10.854	5.19		
Flex. Link 9	Roof Col. Lines 7 & 8	2.09	2.038	10.217	10.418	4.98	10.84	0.41
Flex. Link 10	Fb & Ec		3.428	11.767	12.256	5.86		
Flex. Link 17	Roof Col. Lines 3 & 4	2.09	9.976	7.868	12.705	6.08	12.11	0.46
Flex. Link 18	Eb & Ea		9.865	7.842	12.602	6.03		
SS4 Diag. Brace 1	Level 2	2.09	5.585	1.044	5.682	2.72	5.44	0.21
SS4 Diag. Brace 2	Col. Line Ea		5.582	1.043	5.679	2.72		
SS5 Diag. Brace 2	Level 2	2.09	9.62	8.725	12.987	6.214	12.43	0.47
SS5	Col. Line 3		9.62	8.725	12.987	6.214		

* Fa = 1.33 x 0.6 Fy = 26.4 ksi

TABLE E-9

AXIAL FORCE IN SINGLE DIAGONAL BRACE UNDER NRC SPECTRUM - 7% DAMPING

Element No.	Location	Area (in^2)	Axial Force (kips) X-EQ	Axial Force (kips) Y-EQ	Axial Force (kips) SRSS of X & Y EQ	Axial Stress (ksi)	Allowable Compression Stress Fa (ksi)	Stress Ratio $\frac{\text{Axial Stress}}{\text{Fa}}$
SS3 Diag. Brace 1	Level 2 Col. Line 8	4.18	5.20	55.12	55.36	13.25	4.34*	3.05
SS5 Diag. Brace 1	Level 2 Col. Line G	6.72	3.86	1.09	4.01	0.60	16.97	0.04

*Based on $\frac{Kl}{r}$ = 214. Note that the maximum $\frac{Kl}{r}$ ratio for the secondary member allowed by UBC is 200.

TABLE E-10

AXIAL FORCE IN STEEL COLUMN (LEVEL 2) UNDER NRC SPECTRUM - 7% DAMPING

Location	Area (in^2)	Due to Dead Load (ksi)	Axial Force (kips) Due to EQ Loads			Axial Stress Due to EQ Loads (ksi)	Total Axial Stress F_t (ksi)	Allowable Comp. Stress F_a (ksi)	Stress Ratio $\frac{F_t}{F_a}$
			X-EQ	Y-EQ	SRSS of X & Y EQ				
Col. Lines 2 & G	41.85	0.20	2.54	0.72	2.64	0.06	0.26	17.16	0.02
Col. Lines 3 & Fa	9.12	1.87	0.	0.	0.	0.	1.87	12.76	0.15
Col. Lines 3 & Ea		0.94	7.32	6.64	9.88	1.08	2.02		0.16
Col. Lines 4 & G		0.94	0.	0.	0.	0.	0.94		0.07
Col. Lines 4 & Eb		1.00	0.	0.	0.	0.	1.00		0.08
Col. Lines 4 & Ea		1.00	4.06	0.76	4.13	0.45	1.45		0.11
Col. Lines 5 & Ea		1.00	4.09	0.76	4.18	0.46	1.46		0.11
Col. Lines 7 & G	9.12	1.40	0.	0.	0.	0.	1.40	12.76	0.11
Col. Lines 7 & Fb	14.40	1.77	0.	0.	0.	0.	1.77	14.92	0.12
Col. Lines 7 & Ec	9.12	1.40	0.	0.	0.	0.	1.40	12.76	0.11
Col. Lines 8 & G	9.12	1.07	0.	0.	0.	0.	1.07	12.76	0.08
Col. Lines 8 & Fb	9.12	2.15	3.9	41.33	41.51	4.55	6.7	12.76	0.53

TABLE E-11

FORCES DEVELOPED IN STEEL BEAM -
CONCRETE WALL CONNECTION UNDER NRC SPECTRUM - 7% DAMPING

Location	Flexible Link No.	Connection Shear (kips)			Allowable Shear Stress Va (kips)	Stress Patio Conn. Shear / Va
		X-EQ	Y-EQ	SRSS of X & Y EQ		
Col. Lines 5.5 & G	1	10.20	16.32	19.25	0.	N.A.
Col. Lines 6 & Fb	2	15.9	8.8	18.17	6.6	2.75
Col. Lines 6 & Ec	3	13.81	22.09	26.05	6.6	3.95
Col. Lines 5 & G	13	10.31	2.89	10.71	6.61	1.62
Col. Lines 5 & Ec	11	8.05	6.86	10.58	8.05	1.31

TABLE E-12

MOMENTS IN RADIOACTIVE PIPE TUNNEL AND SUPPORTING COLUMNS UNDER NRC SPECTRUM - 7% DAMPING

	COMP.	Moment (K-in) X-EQ	Y-EQ	SRSS of X & Y EQ	Crack Moment Mcr (K-in)	Ultimate Moment Mu (K-in)	Stress Ratio Against Crack $\frac{M_1}{Mcr_1} + \frac{M_2}{Mcr_2}$	Stress Ratio Against Failure $\frac{M_1}{Mu_1} + \frac{M_2}{Mu_2}$
Rad. Tunnel	About Z-Axis	78596.	22776.	81830.	1.92×10^5	2.52×10^5	0.45	0.32
Column	About X-Axis	126.	560.	574.	6967.*	16567.*	0.93	0.38
	About Y-Axis	10032.	2834.	10425.	12297.*	29925.*		

*Evaluated under axial compression = D.L. - Vert. NRC Spectrum = 470 kips

APPENDIX F

PLOTS OF AMPLIFIED RESPONSE SPECTRA

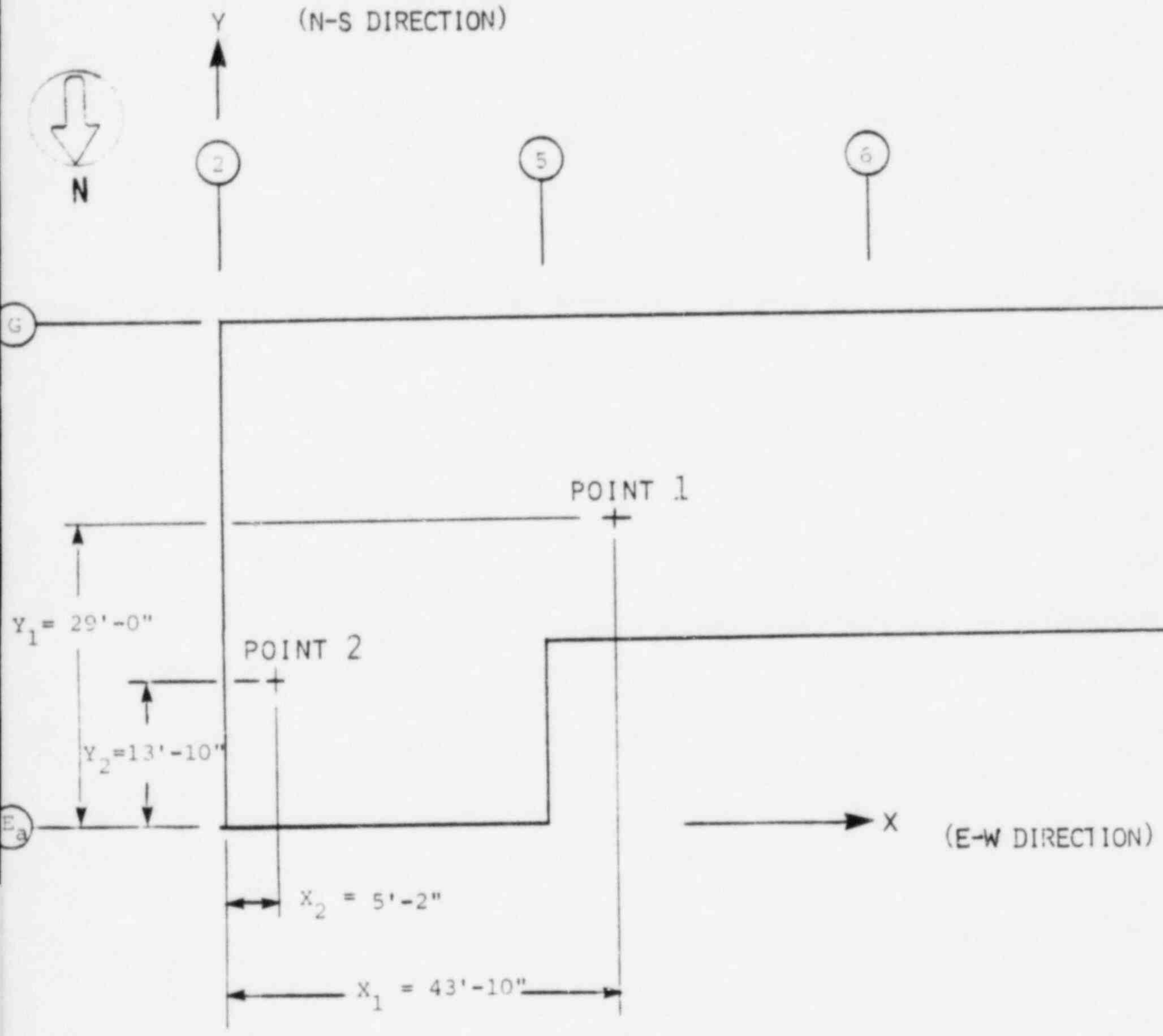


FIG. F.1 LOCATION FOR ARS GENERATION

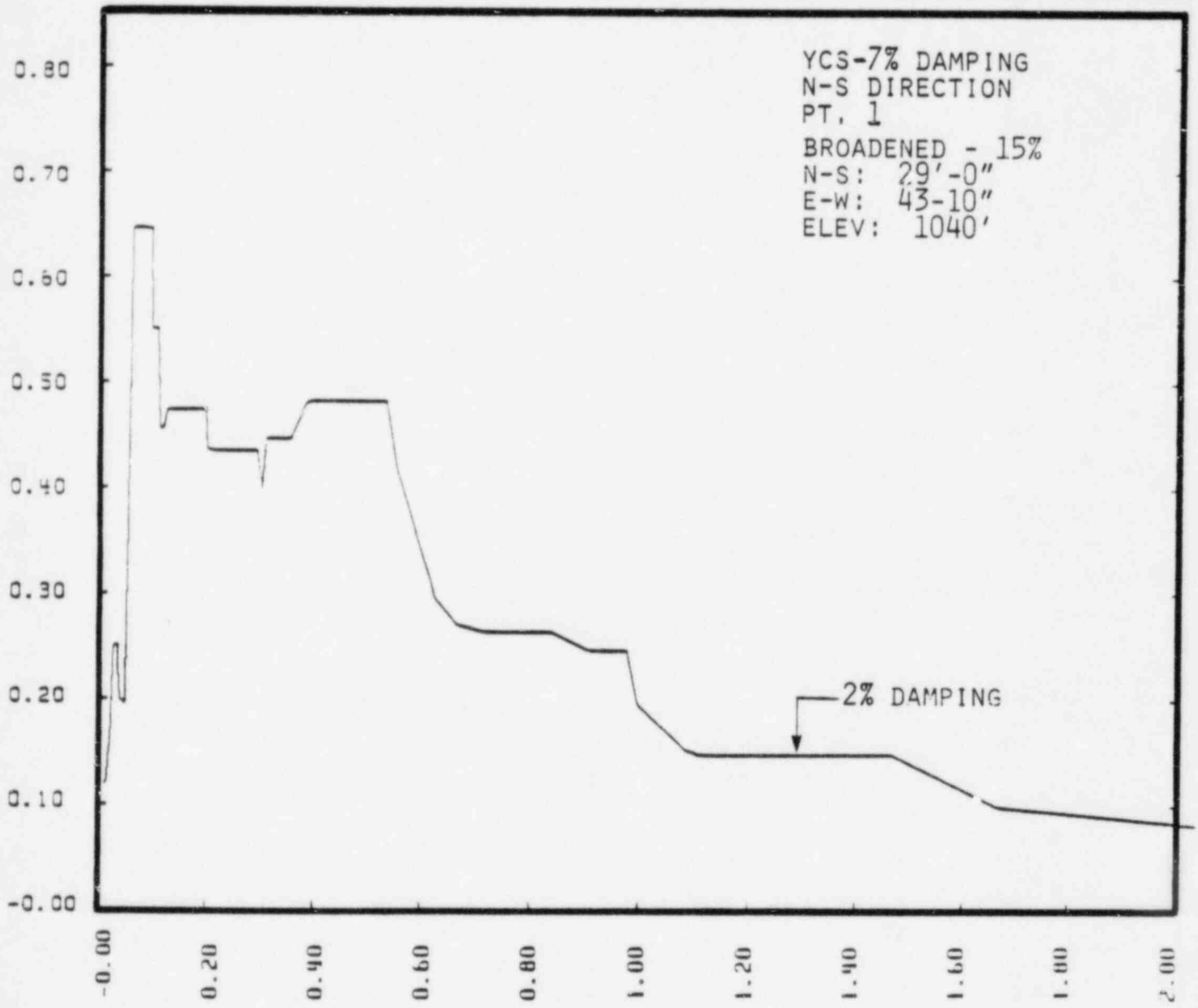


FIG. F.2 AMPLIFIED RESPONSE SPECTRUM
(SHEET 1)

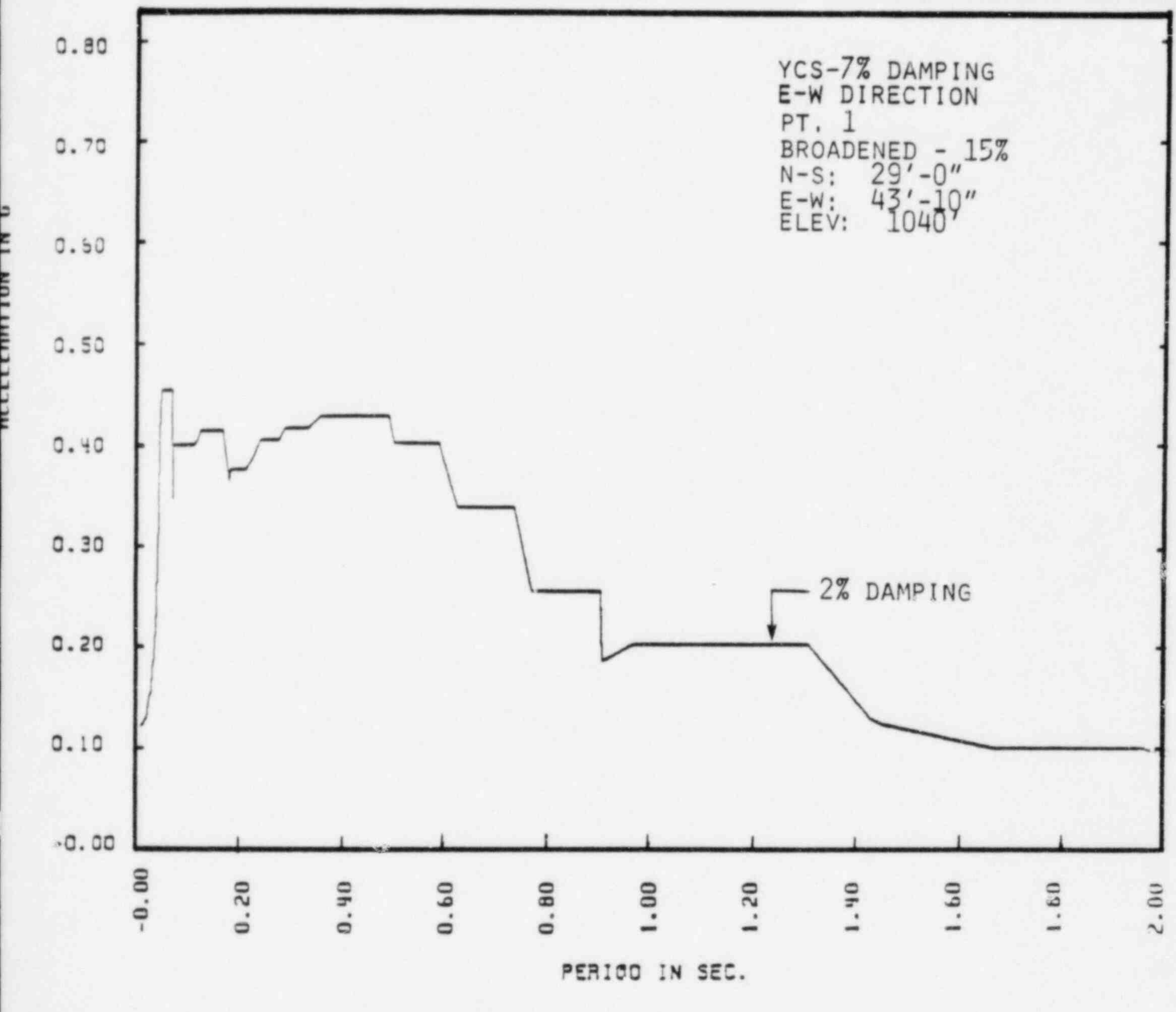


FIG. F.3 AMPLIFIED RESPONSE SPECTRUM
(SHEET 2)

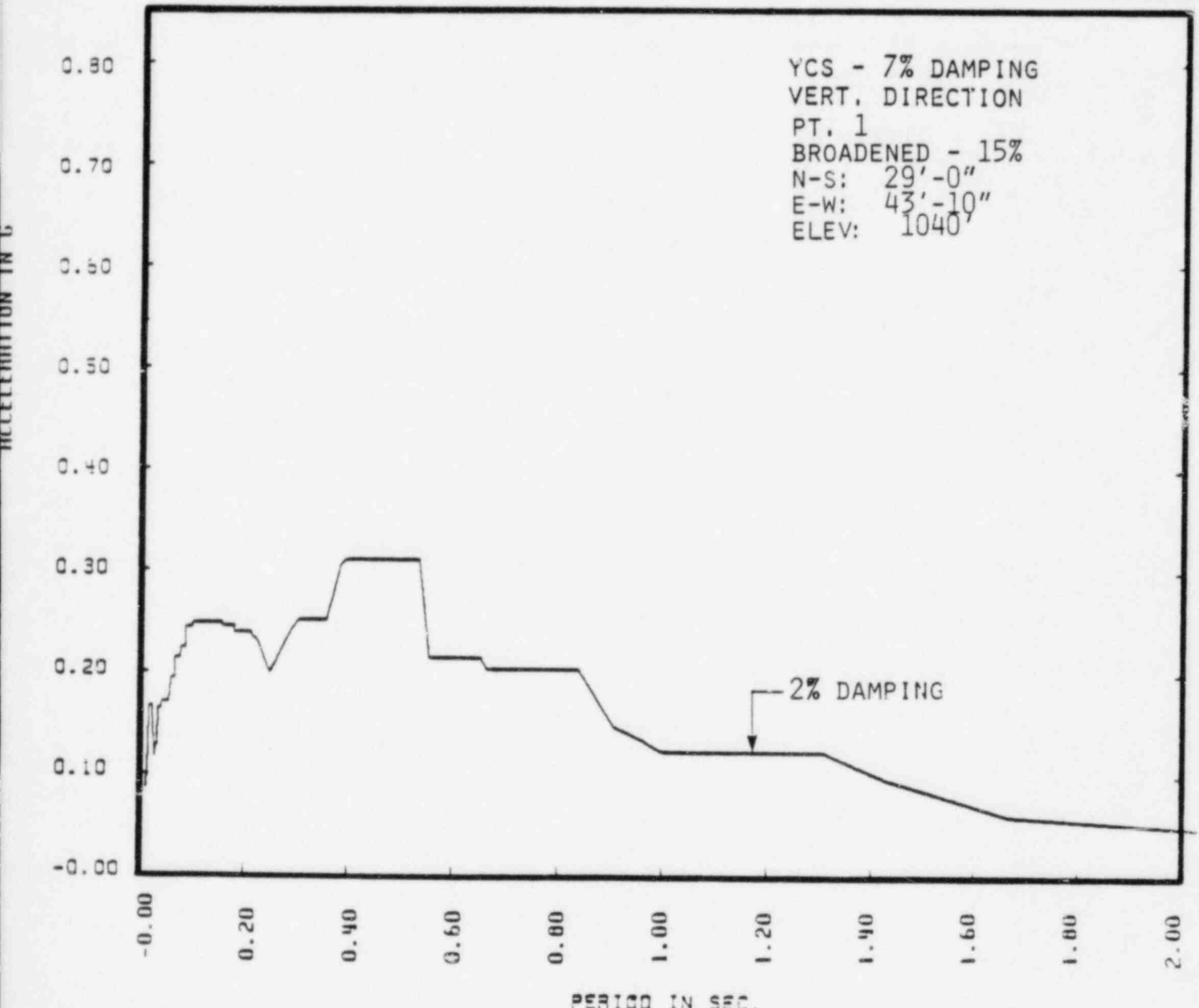


FIG. F.4 AMPLIFIED RESPONSE SPECTRUM
(SHEET 3)

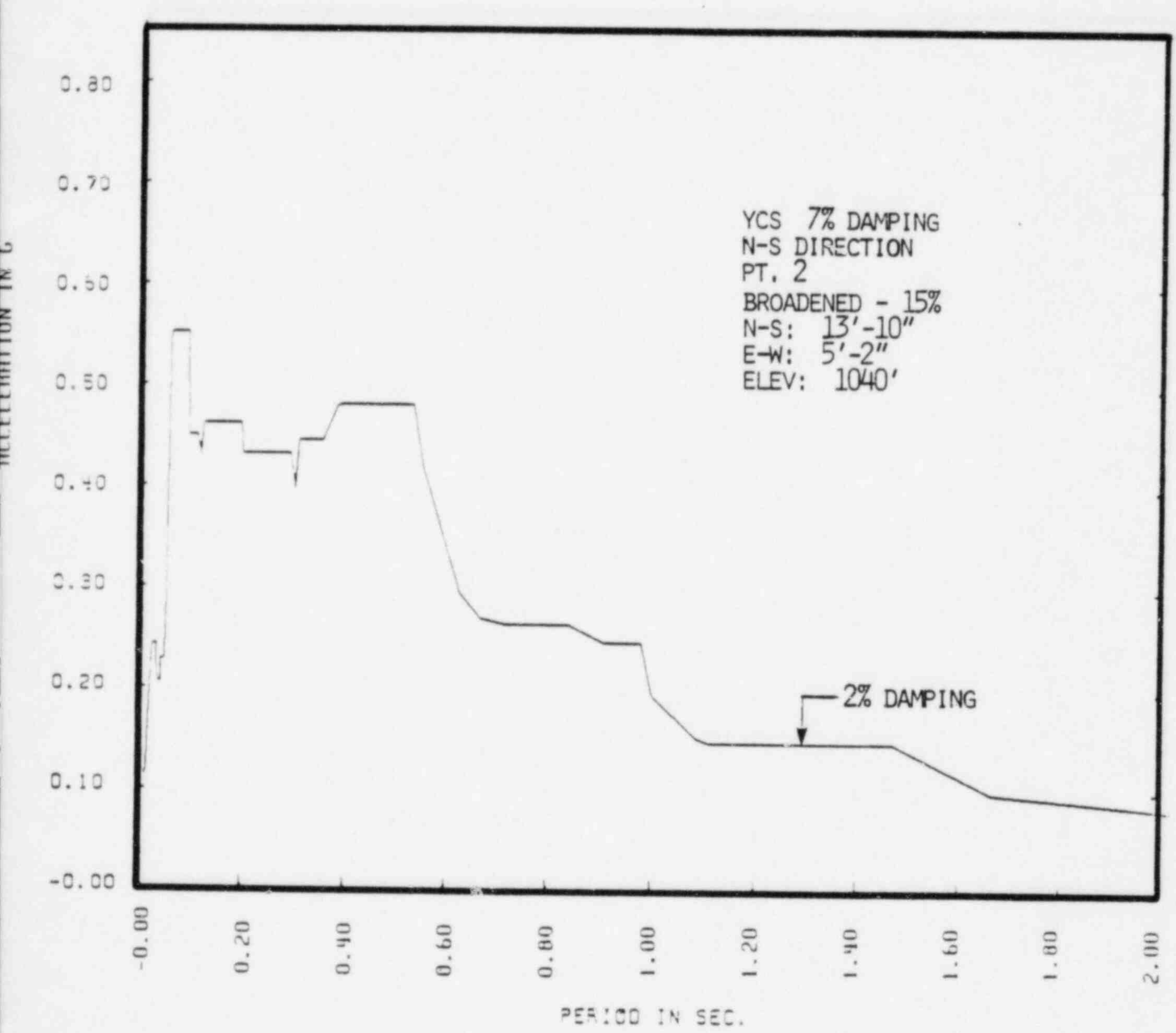


FIG. F.5 AMPLIFIED RESPONSE SPECTRUM
(SHEET 4)

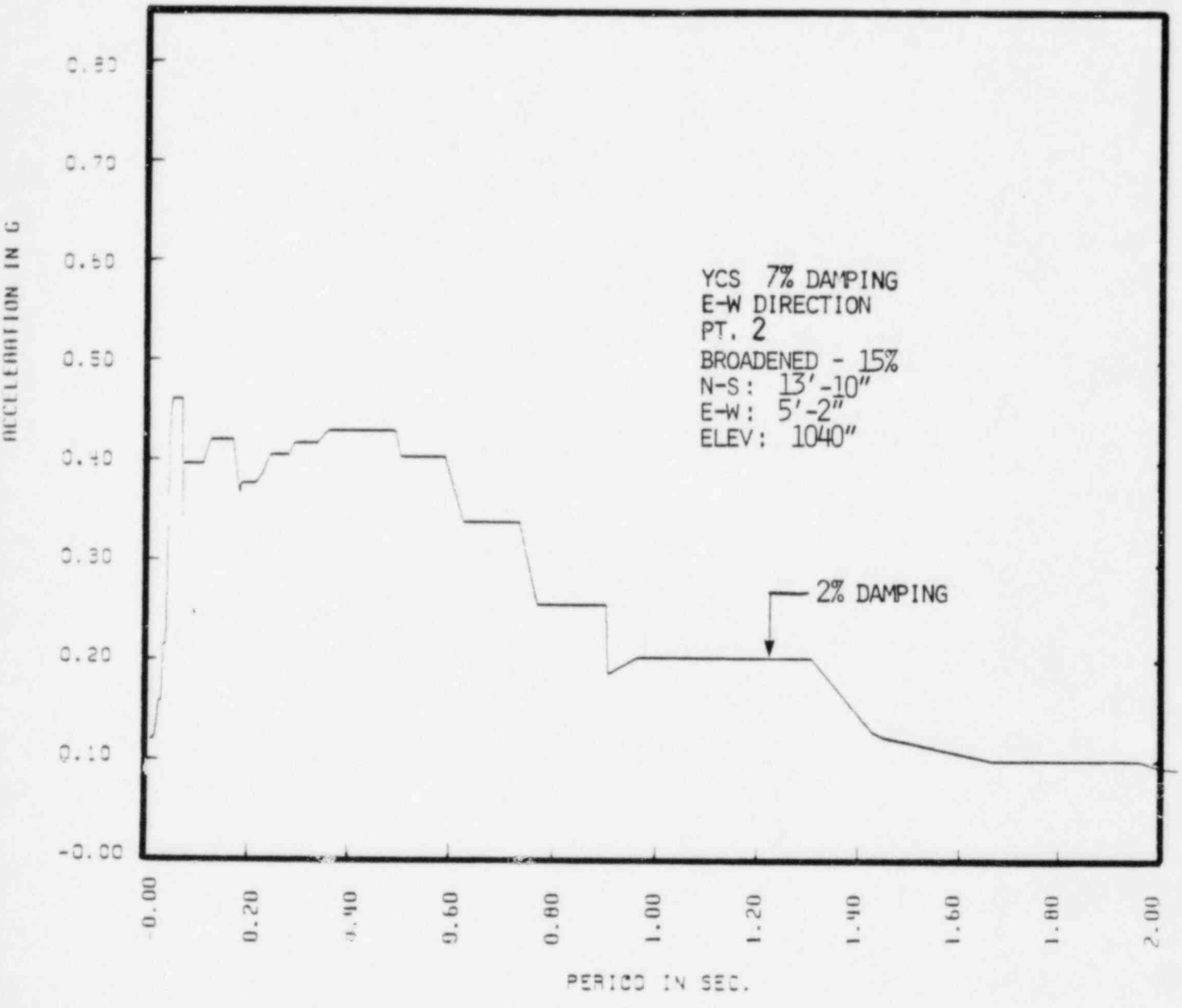


FIG. F.6 AMPLIFIED RESPONSE SPECTRUM
(SHEET 5)

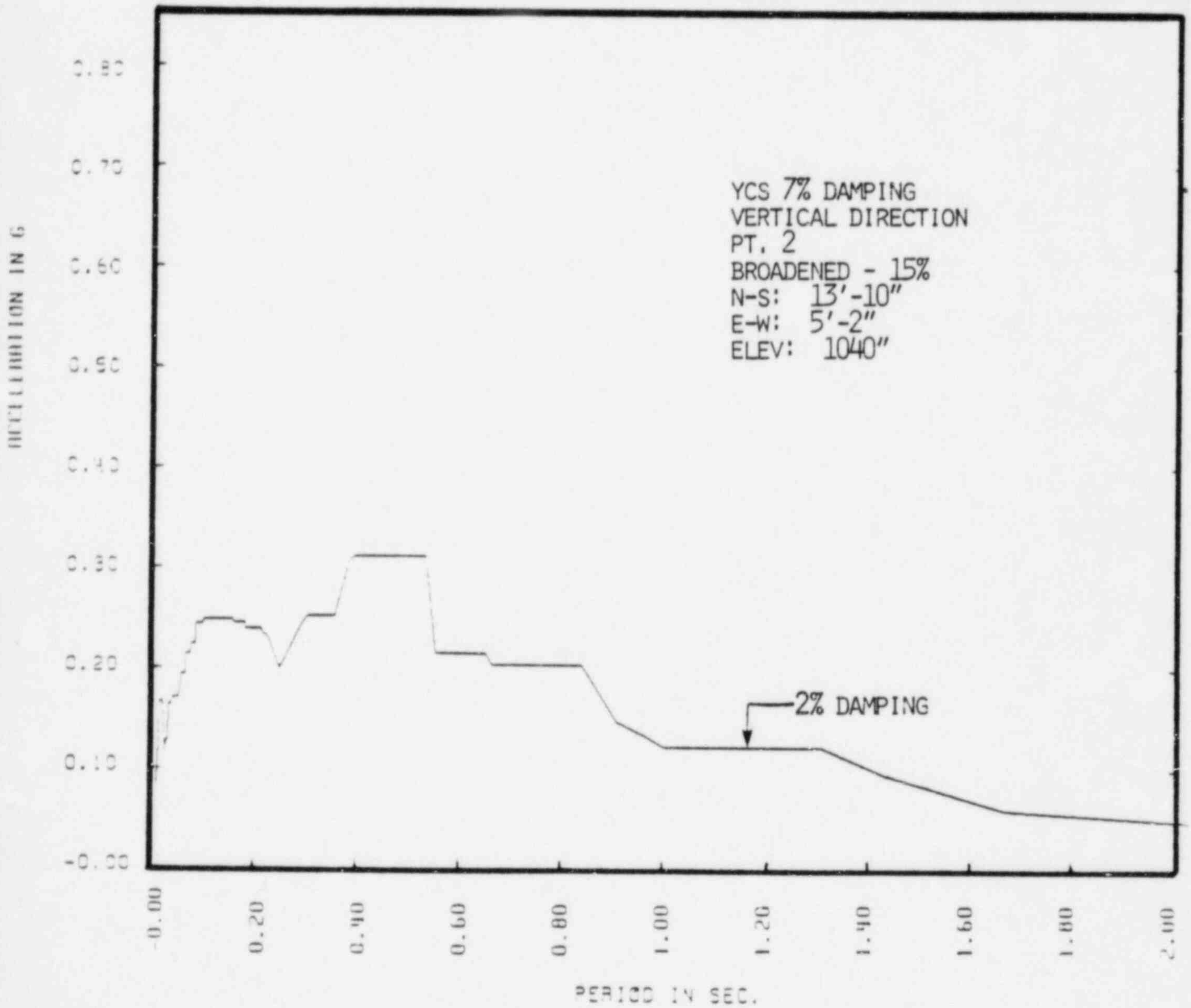


FIG. F.7 AMPLIFIED RESPONSE SPECTRUM
(SHEET 6)

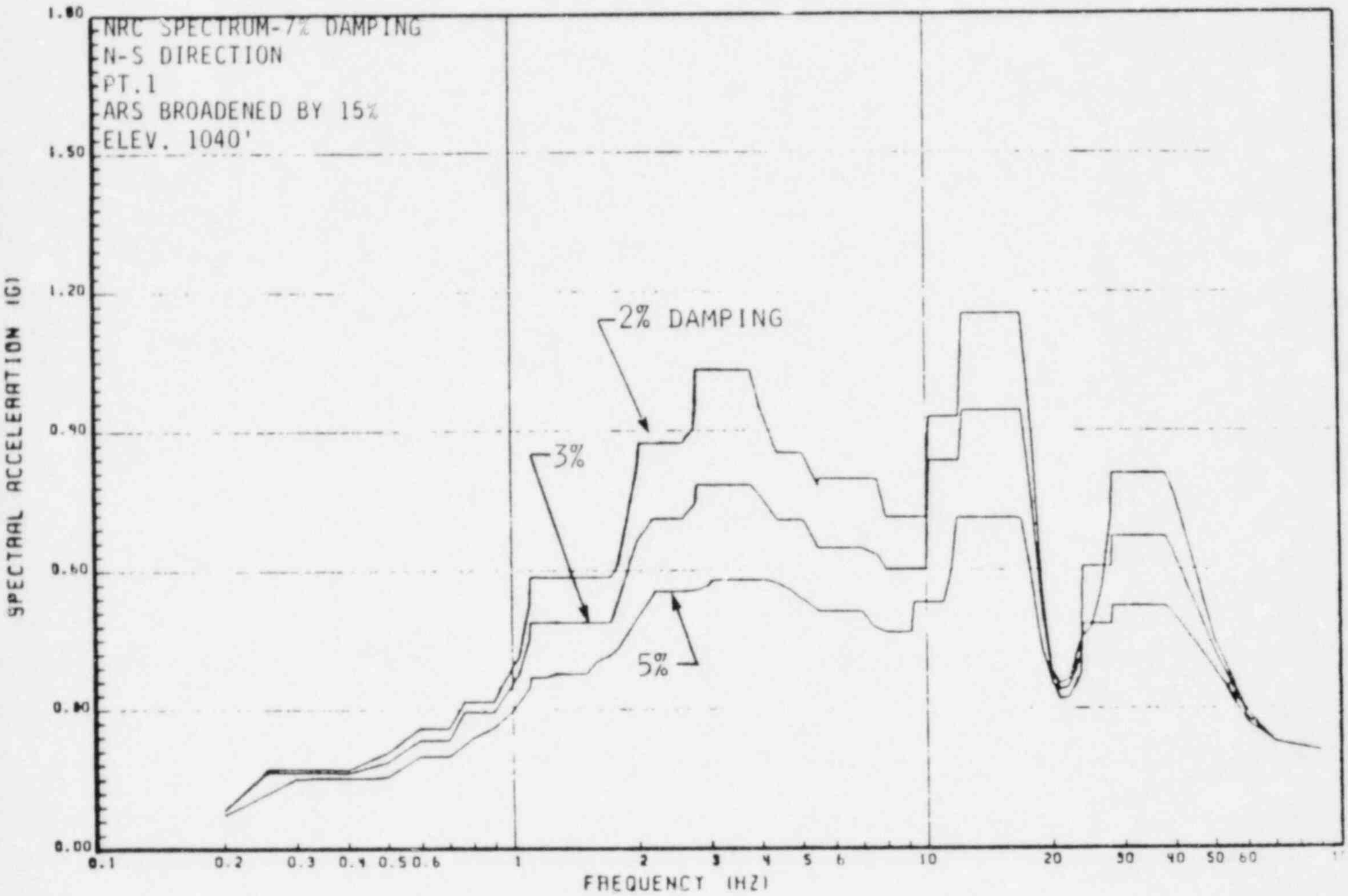


FIG. F-8 ARS AT PT. 1 N-S DIRECTION

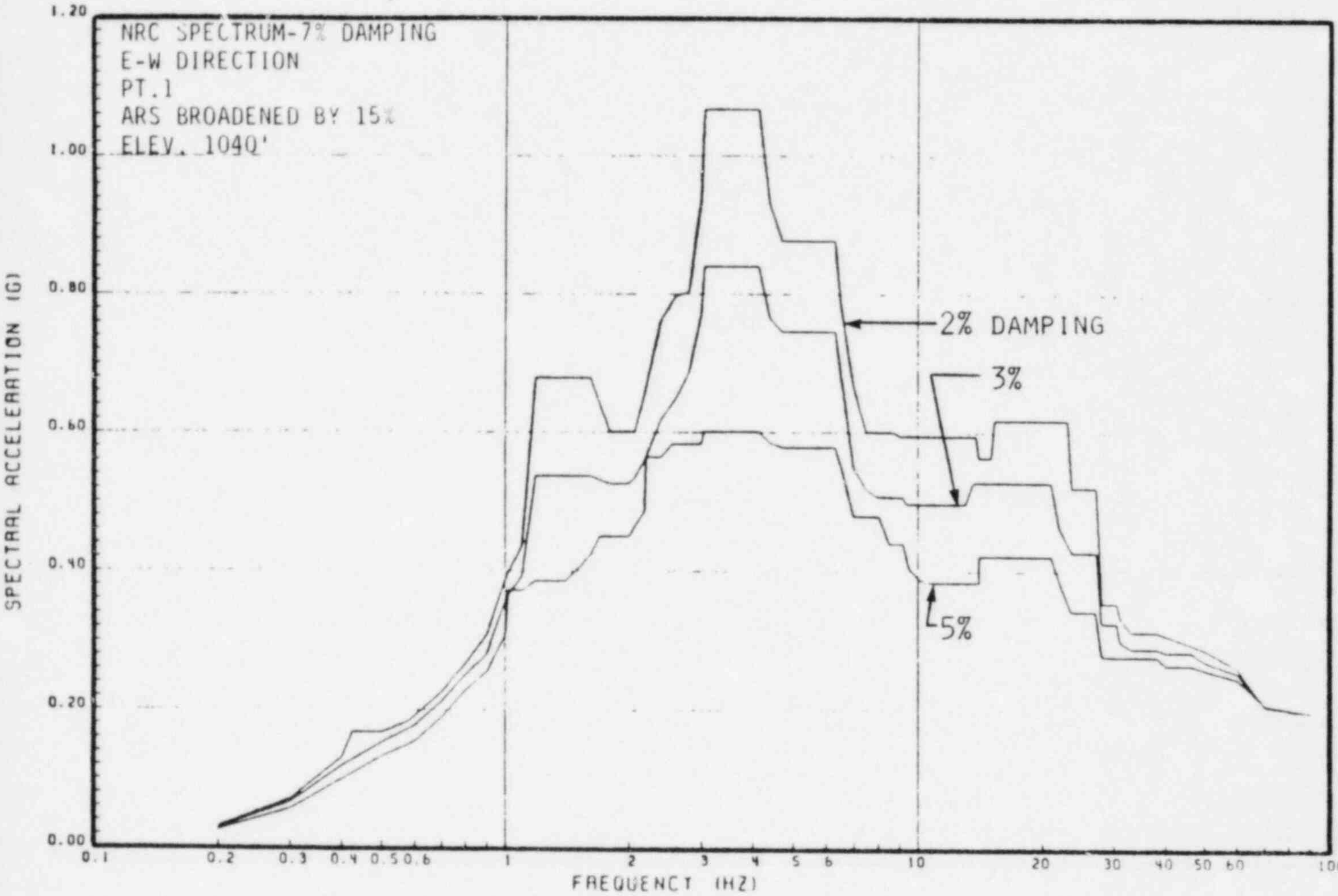


FIG. F-9 ARS AT PT. 1 E-W DIRECTION

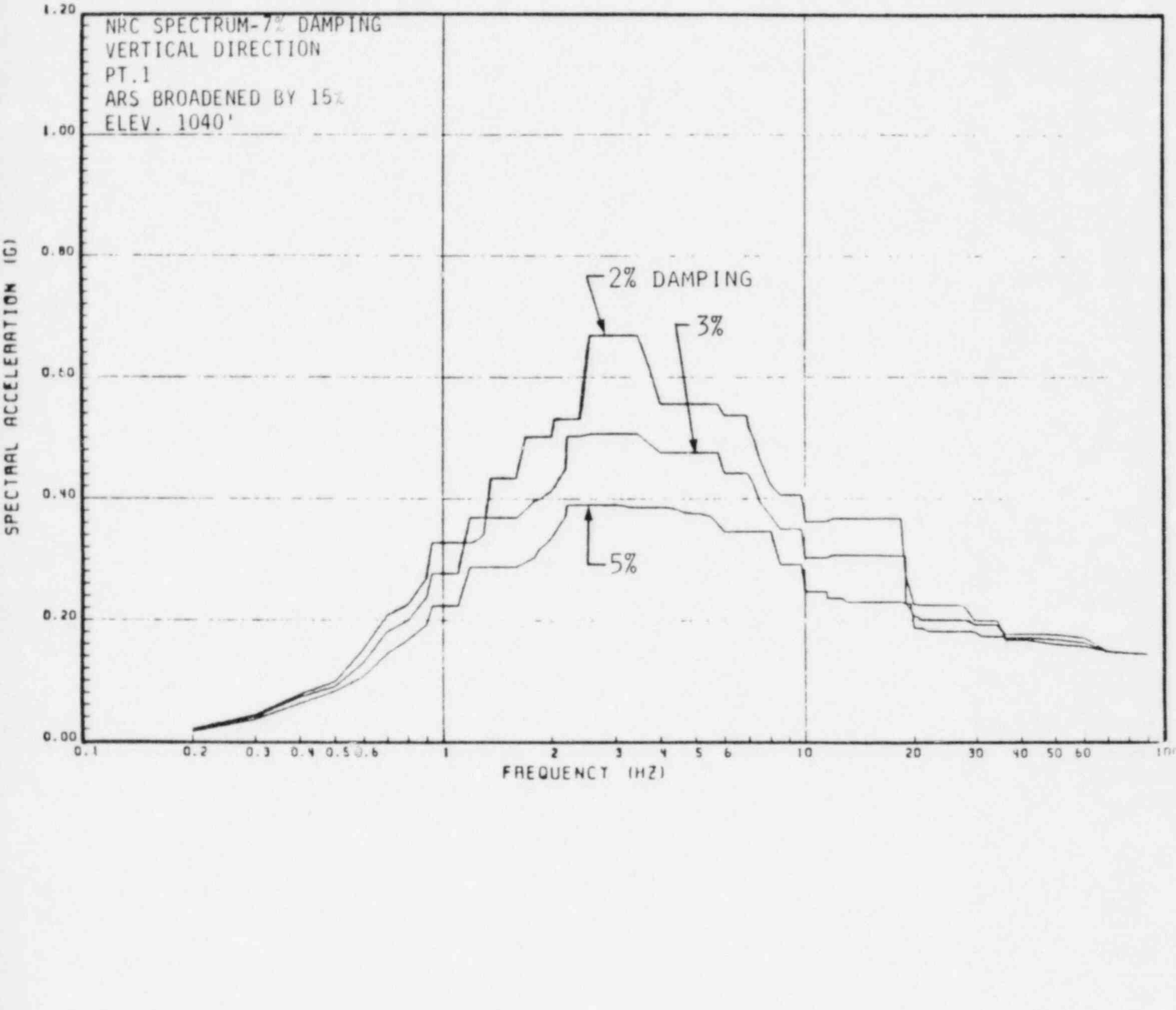


FIG. F-10 ARS AT PT. 1 VERTICAL DIRECTION

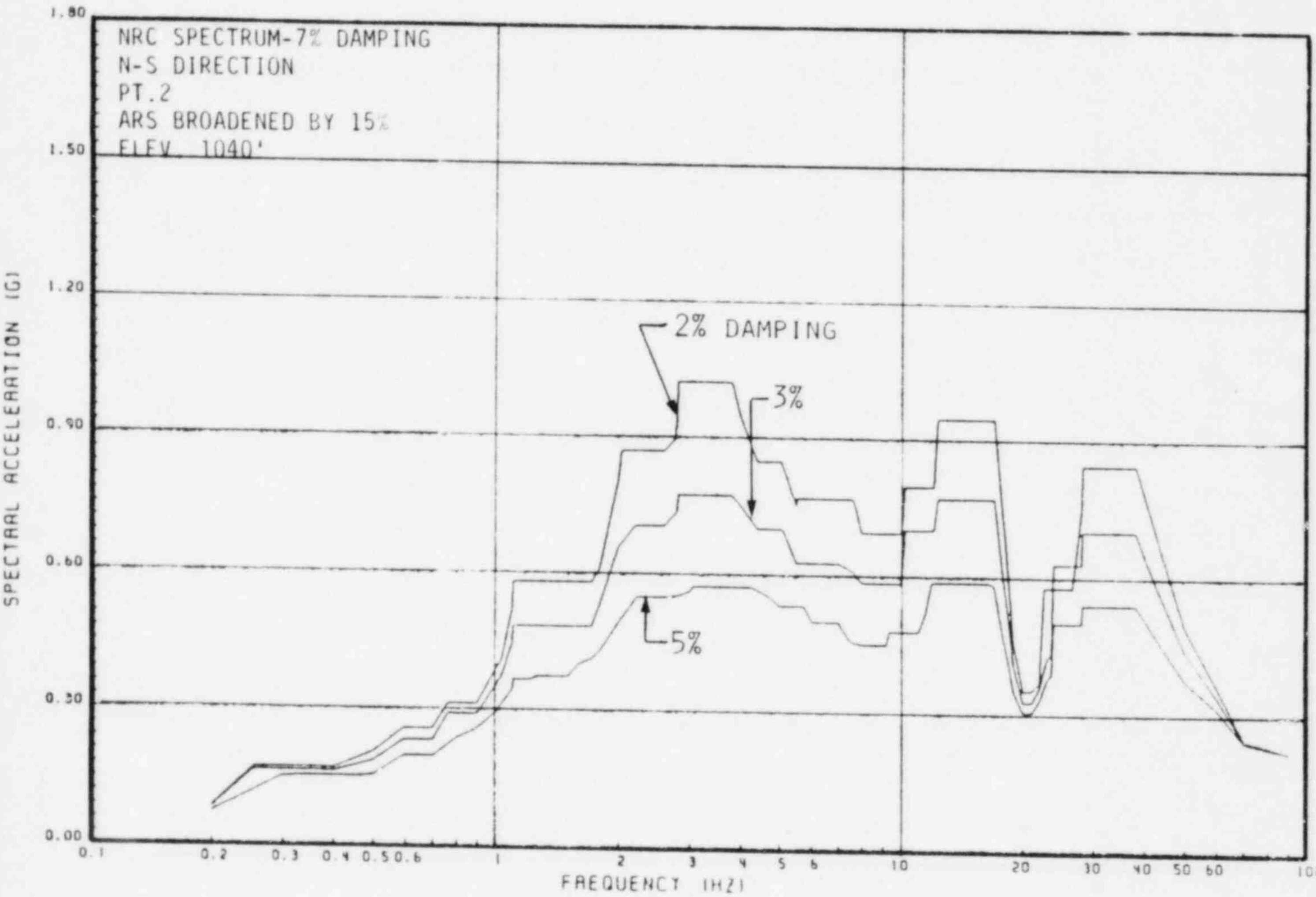


FIG. F-11 ARS AT PT. 2 N-S DIRECTION

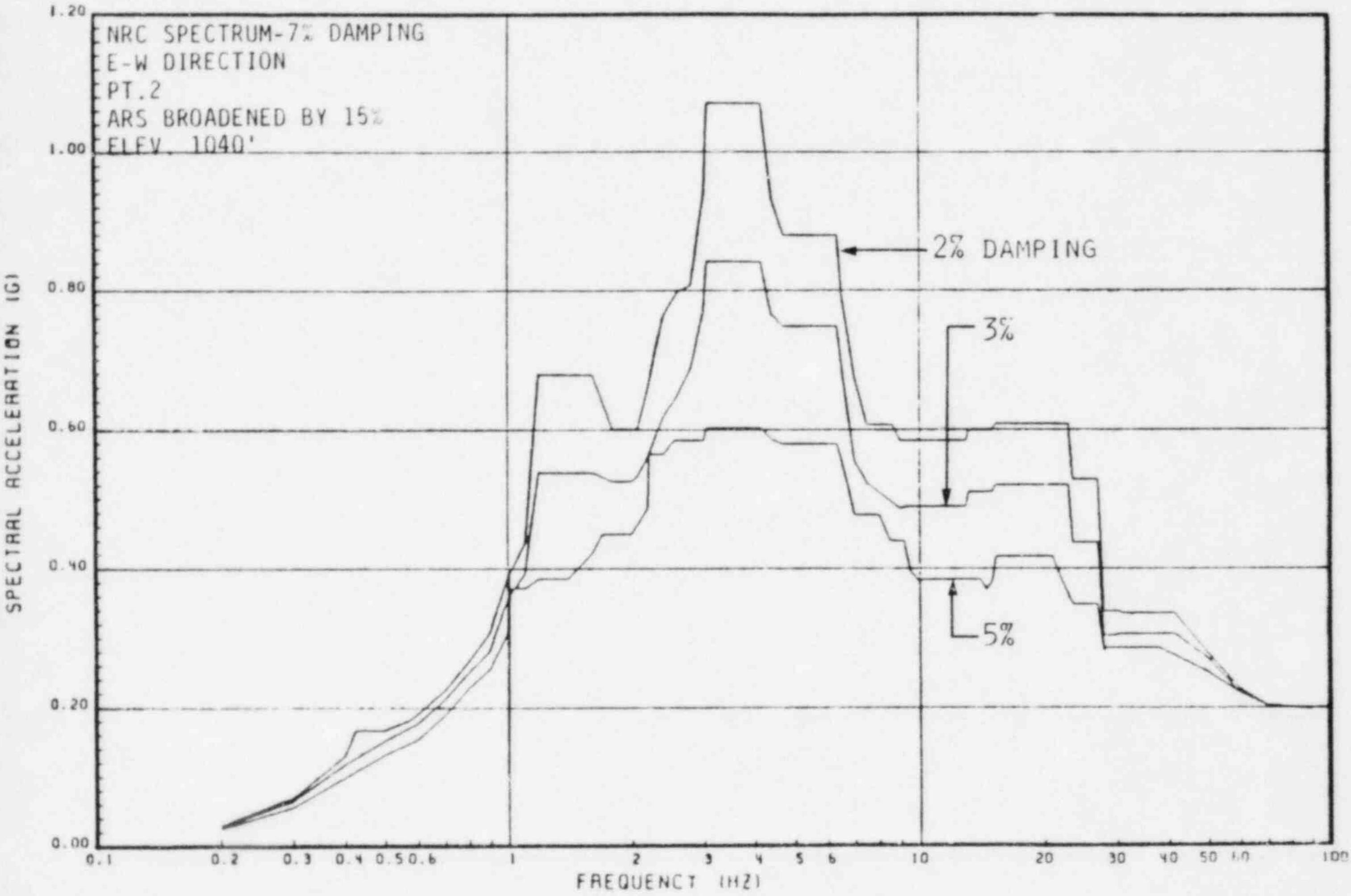


FIG. F-12 ARS AT PT. 2 E-W DIRECTION

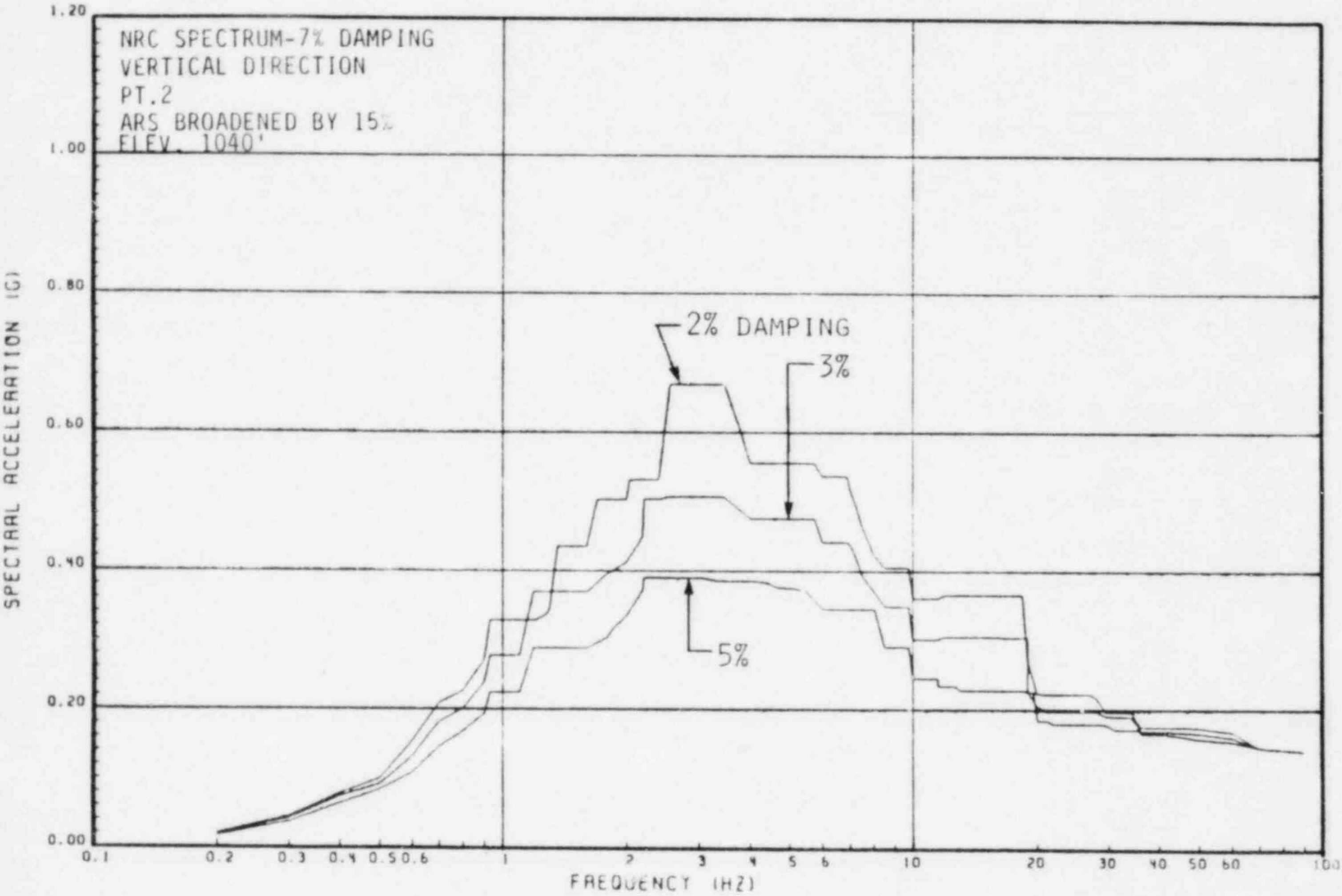


FIG. F-13 ARS AT PT. 2 VERTICAL DIRECTION

APPENDIX G

REFERENCES

APPENDIX G

REFERENCES

1. "Seismic Reevaluation and Retrofit Criteria for Yankee Nuclear Power Station, Rowe, Massachusetts," Cygna Energy Services, August, 1982.

2. "Uniform Building Code," 1979 Edition.

3. "Manual of Steel Construction," Eighth Edition, AISC, 1980.

4. Ozdemir, H. and Lau, P., "BATS, A Computer Program for Analysis of Multistory Frame and Shear Wall Building Systems," Earthquake Engineering Systems, Inc., Feb., 1980.

5. Ozdemir, H. and Lau, P., "MOST, A Computer Program for Mode Superposition Time-History Analysis," Earthquake Engineering Systems, Inc., Oct., 1980.

6. Ozdemir, H., and Lau, P., "INSPEC, A Computer Program for Calculating Spectra," Earthquake Engineering Systems, Inc., Nov., 1979.