ATTACHMENT 2

SYSTEMATIC EVALUATION PROGRAM

Review of the Seismic Re-Evaluation Program
of the Yankee Nuclear Power Station

September 1982

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1. INTRODUCTION

The U.S. Nuclear Regulatory Commission (NRC) is conducting the Systematic Evaluation Program (SEP) which consists of a plant-by-plant safety reassessment of a few older operating plants. Lawrence Livermore National Laboratory (LLNL) has been providing technical assistance to the NRC staff in performing SEP seismic reviews.

As part of the SEP, the Yankee Atomic Electric Company (YAEC) was requested[1] to perform a seismic re-evaluation of the Yankee Nuclear Power Station at Rowe, Massachusetts. LLNL and its consultant, EG&G/San Ramon Operations, reviewed the licensee's seismic re-evaluation program plan and submitted a summary letter report to NRC on December 2, 1981.[2] The program plan review was primarily concentrated on the methodology and criteria the licensee is committed to follow in their seismic re-evaluation. An updated review summary of the program plan is presented in Table 1.

Due to the time constraint of the SEP integrated assessment, the licensee and its consultant, CYGNA, are currently concentrating their seismic re-evaluation effort on completing all analysis work and on identifying all necessary modifications. Therefore, the documentation of their seismic re-evaluation cannot be completed in time for NRC review as only preliminary information[3-5] was available. To facilitate the review of seismic re-evaluation of SEP Group II plants, of which the Yankee plant is one, the staff[6] implemented a procedure for performing the review in parallel with the licensee's re-evaluation effort. The basic concept of this procedure is to hold informal and formal working-level review meetings among the NRC, NRC consultants, licensee, and licensee consultants to complete the reviews and to resolve any questions.

Table 1. Review summary of the seismic re-evaluation program plan.

Item	Addressed	Adequate
Soil and Foundation		
Rock site	n/a	n/a
Soil site		
Foundation input	yes	no (1)
Generation of time history	yes	no (2)
Modeling technique	no	--
Computer codes	no	--
Description of foundation	yes	no (3)
Free field input spectrum	yes	yes (4)
Stuctural		
List and description of Category I structures or structures affecting Category I systems or components	Yes	(5)
Modeling techniques		
Damping	yes	yes
Stiffness modeling	yes	no (1)
Mass modeling	yes	yes
Consideration of 3-D effects	yes	yes
Seismic analysis methods		
Response spectrum, time history or equivalent static analysis	yes	yes
Selection of significant modes	yes	yes
Relative displacements	yes	yes
Modal combinations	yes	yes
Three-component input	yes	yes
Floor spectra generation	yes	no (1,2)
Peak broadening	yes	yes
Load combination	yes	yes
Analytical criteria		
Codes and criteria, including AISC, ACI, and NUREG/CR-0098	yes	no (6)
Computer codes		
Description and verification	yes	(7)

Comments

1. Soil-structure interaction effects were neglected because studies performed previously have shown these effects to have a negligible effect on stresses. However, possible effects on floor spectra have not adequately been addressed for the reactor support structure.

2. Further justification is required for certifying that the duration of the artificial time history generated by matching NRC site-specific spectrum with 10% damping is sufficient to generate conservative in-structure response spectra.

3. Except for the reactor support structure, foundations are not described in the Program Plan.

4. The licensee proposes using a spectrum other than that specified by the site-specific spectrum program for systems other than the hot shutdown system.

5. NRC staff will determine the completeness of the list.

6. Tensile stresses up to 95% of yield are, in general, acceptable; however, stresses up to 95% of the calculated buckling load may not be acceptable. Threaded rods appear to be a special case according to AISC. Special considerations should be applied. Block wall criteria does not meet SRP.

7. It is not known whether or not all the comptuer codes mentioned have been officially verified.

For the Yankee Nuclear Power Station, three of these review meetings were held on April 5-6, May 25-26, and August 3-4, 1982. Trip reports for the first two meetings are attached as Appendices A and B. The following sections present LLNL and EG&G's evaluation of the licensee's seismic re-evaluation results based on the presentations by CYGNA and the discussions of the meeting participants. A set of the viewgraphs[7-12], which are considered preliminary information, were provided by the CYGNA with the approval of YAEC on August 4, 1982.

NCT Engineering, under contract with LLNL, performed an independent seismic analysis of the Yankee plant steel vapor container.[13] The results of this independent analysis provides a bench mark for the evaluation of the licensee's re-evaluation results and were used in this evaluation effort.

B. Bresler of Wiss, Janney, Elstner and Associates, Inc. (WJE) provided special consultation on the review of the connections between the concrete columns and the concrete reactor support structure. Particular attention was focused on the performance of the reinforcing bars in the connections under the cyclic influence of a seismic load. His review is mainly based on a presentation by CYGNA on July 16, 1982. The viewgraphs of this presentation were extracted from the May 25-26, 1982 meeting.[9] The results and conclusions of this review[14] were also included in this report.

2. PLANT STRUCTURES

Figure 1 shows the general arrangement of the Yankee plant structures. Figure 2 is a cutaway drawing of the plant. Following is a list of structures which are considered to be seismic safety related and are included in SEP seismic re-evaluation.

1. Steel vapor container*.
2. Concrete reactor support structure*.
3. Turbine building and turbine pedestal, including control room* and auxiliary bay.
4. Primary auxiliary building* and radioactive tunnel.
5. Diesel generator building*, accumulator enclosure and Annex.
6. Steel-frame structure* which supports MS/FW line.
7. Spent-fuel pool and spent-fuel chute.
8. Field-erected tanks and buried piping or tunnels.

The asterisks (*) indicate structures that house or support the hot-shutdown system and are essential for safe hot shutdown of the plant. The fire water tank is also related to hot shutdown.

The waste disposal building, screen well and pump house, switchyard structure, office area and service building are not safety related and, therefore, were not evaluated. The ion exchange building itself is not safety related. However, it is connected to the primary auxiliary building and, therefore, it is included in the seismic analysis model of the primary auxiliary building.

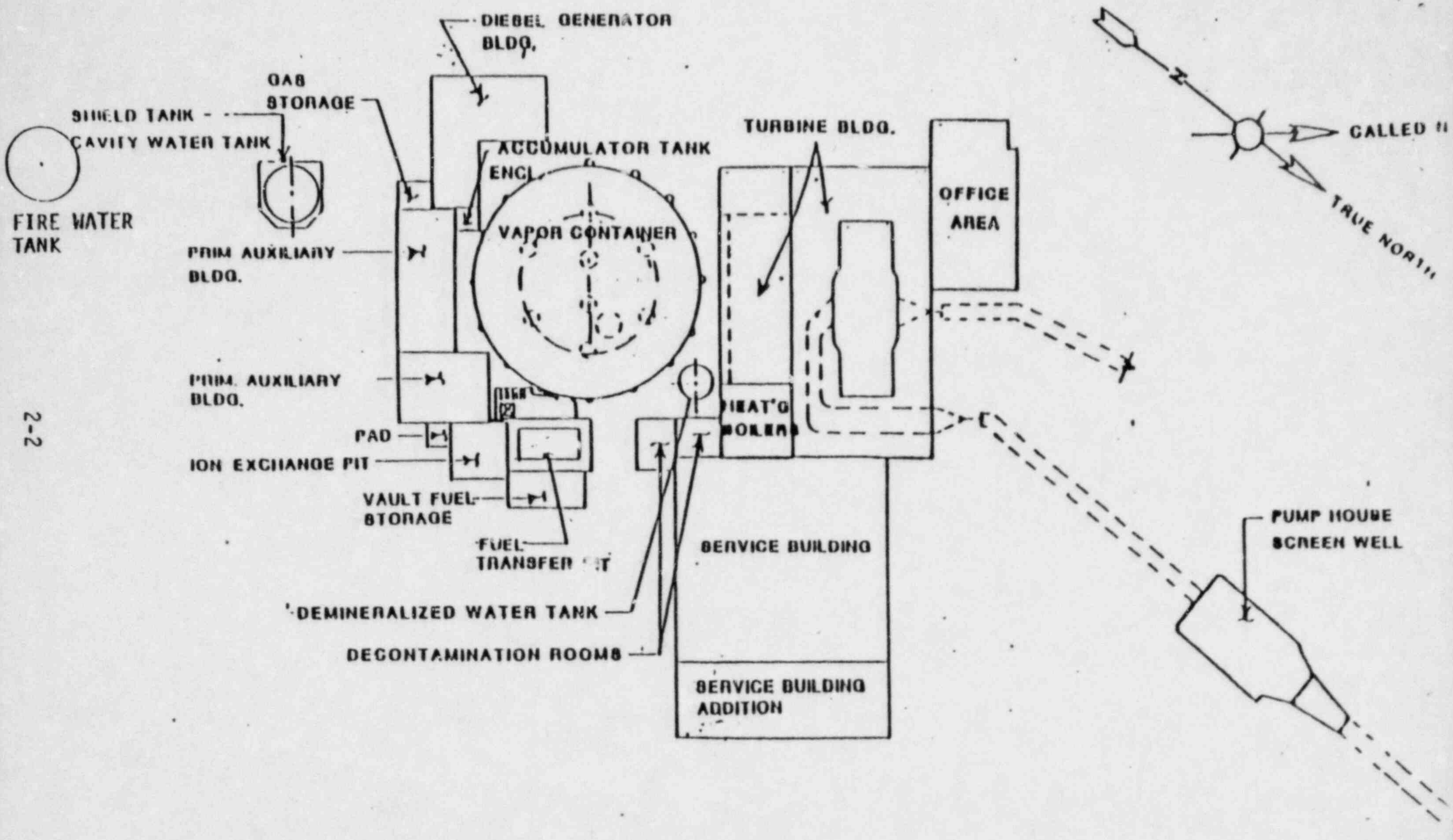


Figure 1. Layout plan of Yankee Nuclear Power Station.

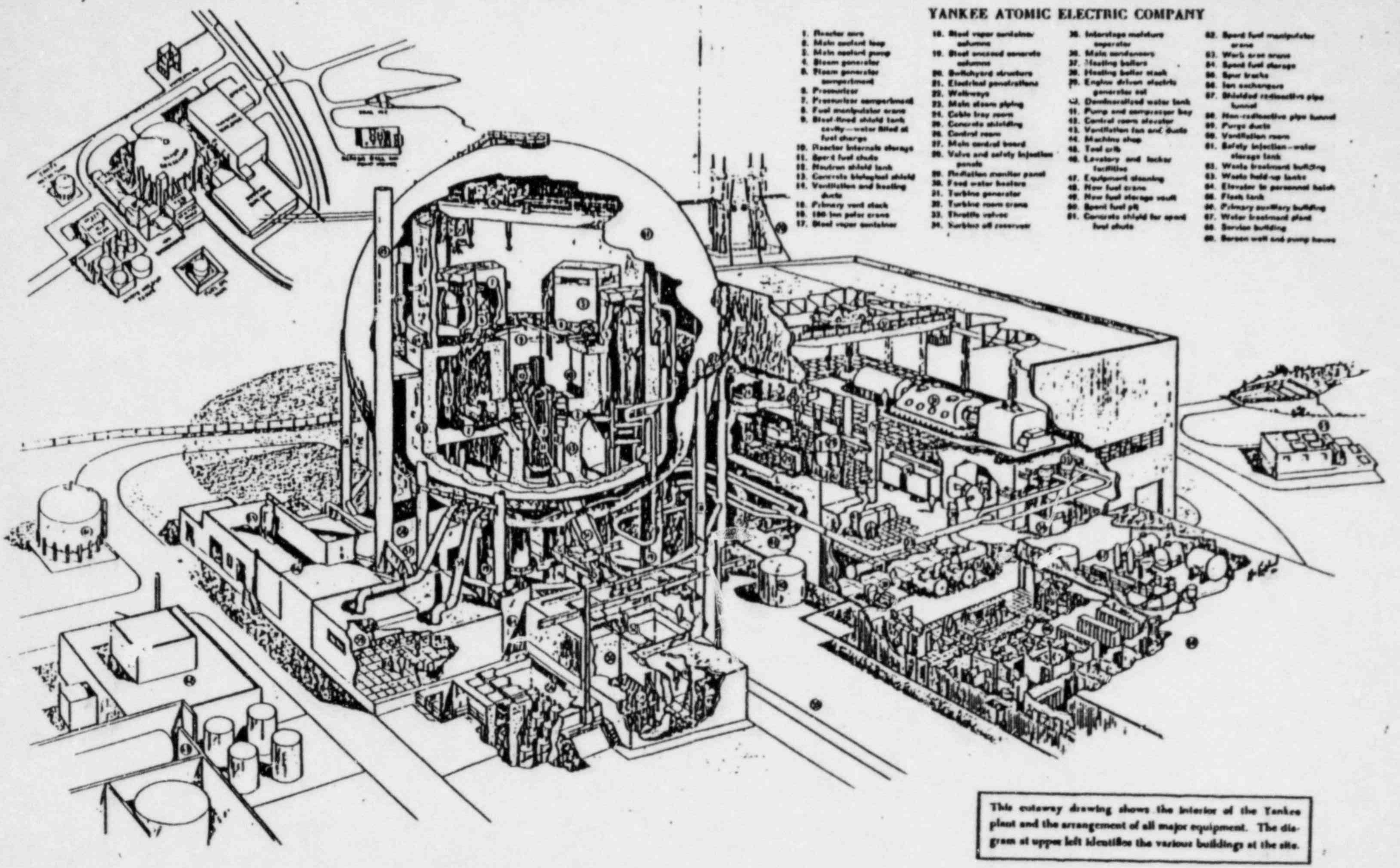


Figure 2. Cutaway drawing of plant.

3. SEISMIC INPUT AND REVIEW

3.1 SEISMIC INPUT

There are four ground response spectra (Figure 3) that have been used in evaluating the Yankee facilities: 1) NRC site-specific spectra, 2) Yankee composite (YC) spectra, 3) Interim design basis spectra (IDBS), and 4) R.G. 1.60 spectra scaled to 0.2-g peak ground acceleration. The IDBS was used to evaluate the Yankee plant structures for the purpose of interim safe operation of the plant while the SEP seismic re-evaluation was carried out. The other three spectra were used in the Yankee seismic re-evaluation. While NRC was developing site-specific spectra for several SEP plants in the eastern United States, Yankee Atomic Electric Company initiated the seismic re-evaluation using the YC spectra. At this moment, the licensee's consultant (CYGNA) has completed the seismic evaluation for most of the safety related structures using YC spectra and for structures housing hot shutdown systems using NRC spectra. The spent-fuel pool and spent-fuel chute were evaluated, based on NRC R.G. 1.60 spectra scaled to 0.2-g peak ground acceleration.

3.2 REVIEW AND DISCUSSION

Two major seismic levels described by the Yankee composite spectra and the NRC site-specific spectra were used in the seismic re-evaluation of Yankee structures as described above. The spent fuel pool and spent fuel chute were evaluated based on 0.2-g R.G. 1.60 spectra. In general, the NRC spectrum envelops YC spectrum which, in turn, envelops the IDBS. The licensee is currently committed to upgrade all safety-related Yankee structures to YC spectra if they are not yet at this level. Furthermore, the structures housing the hot-shutdown system will be qualified to the higher seismic level described by the NRC spectra.

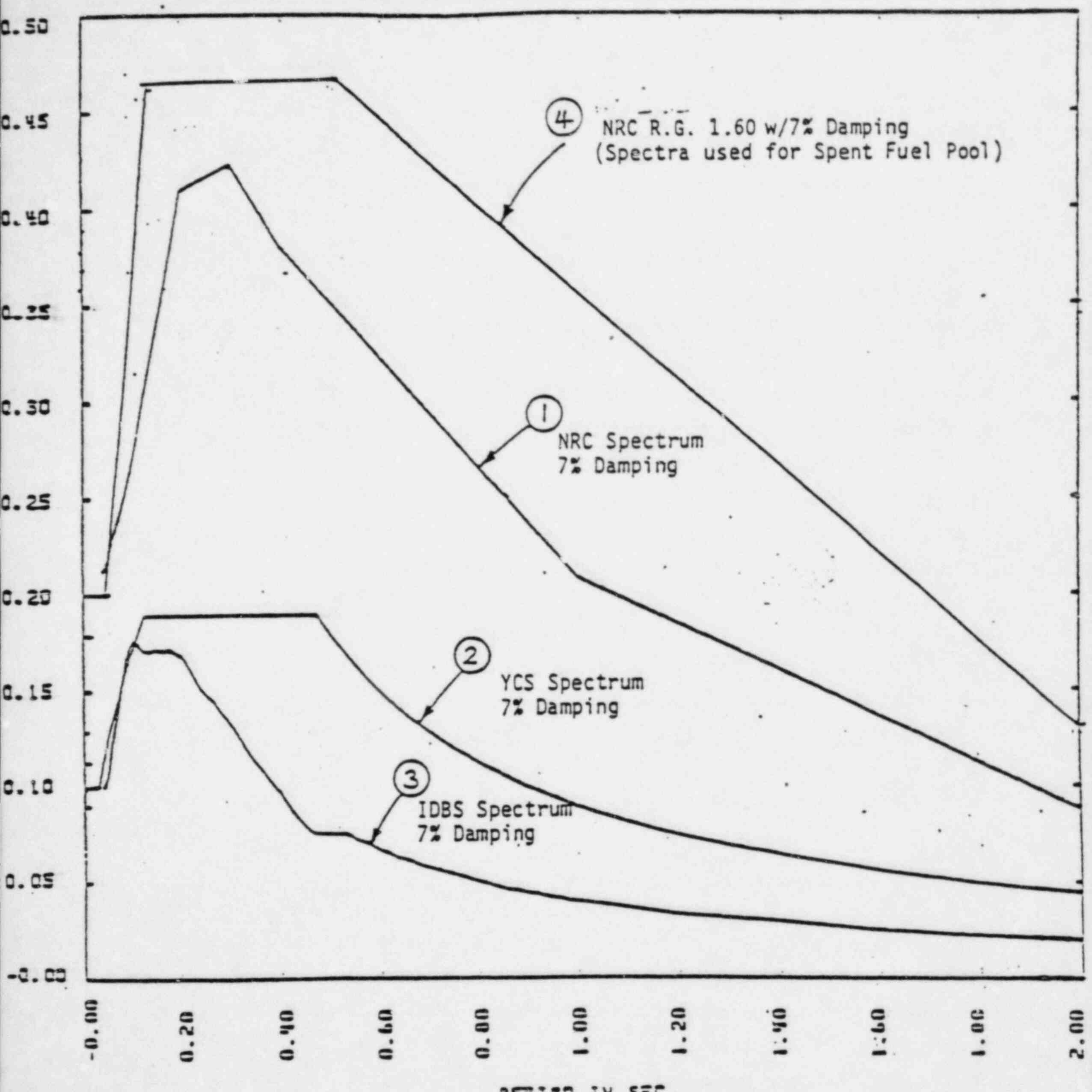


Figure 3. Ground response spectra.

4. STRUCTURAL EVALUATIONS

4.1 VAPOR CONTAINER

4.1.1 Description of Structure

The steel vapor container of the Yankee plant is a spherical pressure boundary that houses the concrete reactor support structure, main coolant loop equipment, reactor pressure vessel, and all other pressurized parts of the main coolant system (Figures 4 and 5). The vapor container is an ASTM A-300, Class A-201, Grade B, carbon-steel sphere having a diameter of 125 ft and a thickness varying from 7/8 in. to 1-1/4 in. The portions comprising the various thicknesses are shown in Figure 4.

Penetrations are provided for electrical conduits, piping, concrete support columns, personnel hatch, equipment hatch, fuel-transfer tube, and other minor appurtenances. With the exception of the hatches and the piping penetrations, significant loadings are prevented from being transferred to the shell either by the small size of the penetrating object or by bellows which serve to transfer the loads to the concrete inner structure while maintaining leak tightness around the shell opening.

The sphere is supported on 16 ASTM-A283, Grade C, steel columns having a 3 ft 6 in. outside diameter and a 7/8 in. thickness. The columns are braced with horizontal steel panel-ties of the same material as the column. The panel ties are 2 ft 9 in. in outside diameter and 1 in. in thickness. Additional bracing is provided by AISC C-1020 steel tie-rod cross braces as shown in Figure 4.

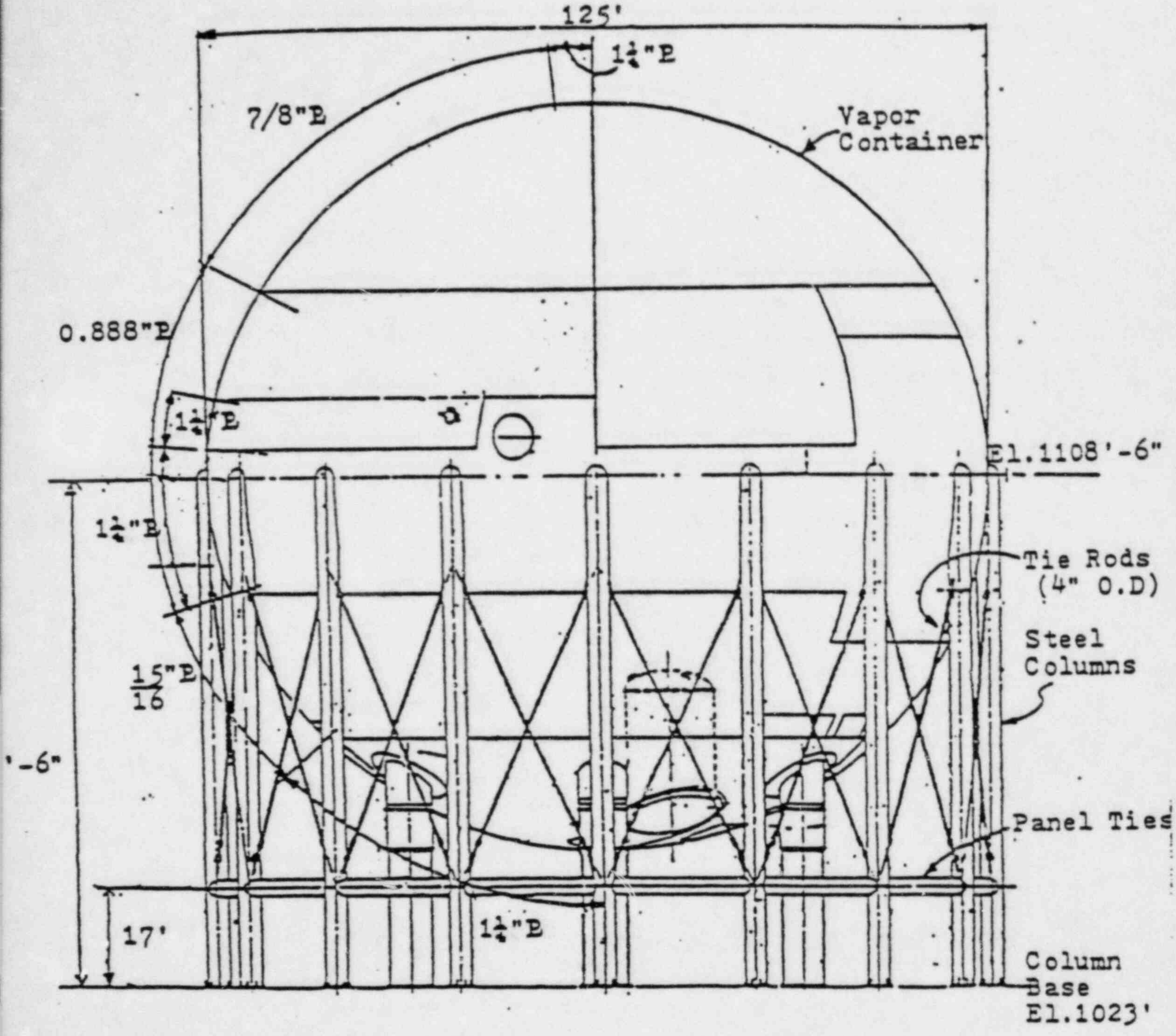


Figure 4. The Yankee plant containment building.

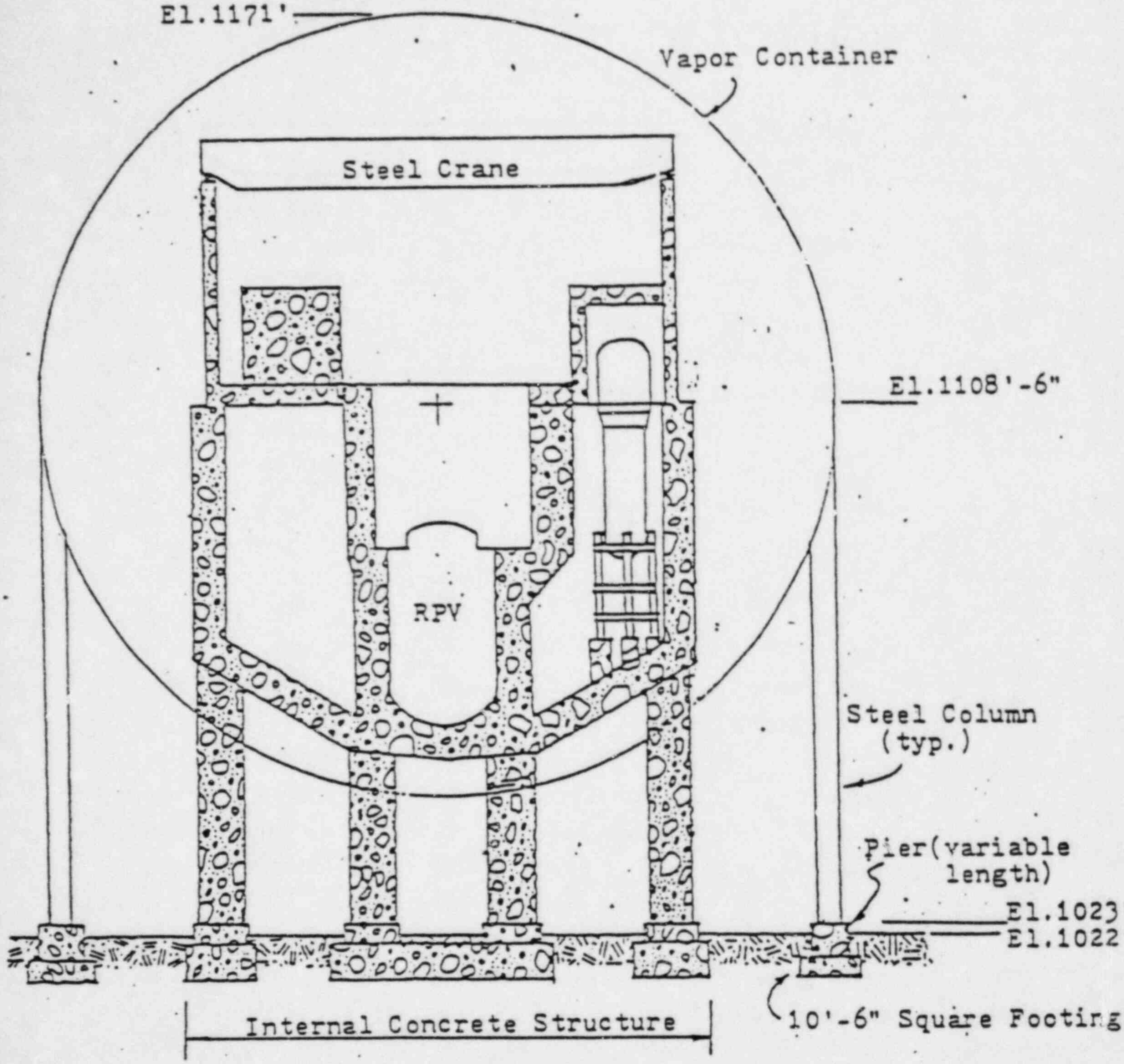


Figure 5. Elevation cross section of the Yankee plant containment building.

Each steel column is supported on a cylindrical concrete pier 5 ft 4 in. in diameter. The length of the pier varies from 4.5 ft to 12.75 ft. Each pier is supported on a concrete footing which is 10 ft 6 in. x 10 ft 6 in. in plan and 2 ft 6 in. in thickness.

4.1.2 Seismic Re-Evaluation of Vapor Container

A finite-element model of the vapor container (Figure 6) was constructed by CYGNA for analysis using EESAP which is a modified version of the SAPIV computer code. Plate elements were used for the steel spherical shell and beam elements were used to represent columns, tie-rods, and panel-ties. The equipment hatch was also included in the model. Fine mesh elements were used around the shell openings or penetrations where sharp variations in force or moment are anticipated. Because of the variations in containment shell thickness, the thickness of the plate elements were varied. Tie-rods, which can carry tensile force only, were modeled by beam elements with half of the effective cross-sectional area. Therefore, the tie rods were evaluated based on two times the loads calculated.

The model is fixed at the base of the steel columns. No soil-structure interaction (SSI) effect is considered. NCT Engineering, in an independent seismic analysis of the Yankee Vapor Container[13], compared the cases of with and without SSI. The results of the comparison indicated that the SSI effect is insignificant for the Yankee Vapor Container. This substantiates CYGNA's approach of neglecting SSI. The dominating frequencies and mode shapes of CYGNA's analysis model were in close agreement with those from the independent analysis by NCT Engineering.

The response-spectrum method was used in CYGNA's seismic analysis. A damping value of 4%, consistent with R.G.1.61, was used. The individual directional responses were calculated for each component of seismic motion and were combined by the SRSS method in accordance with R.G. 1.92. The load combinations considered are consistent with NRC Standard Review Plan Section 3.8.2. The equation is as follows:

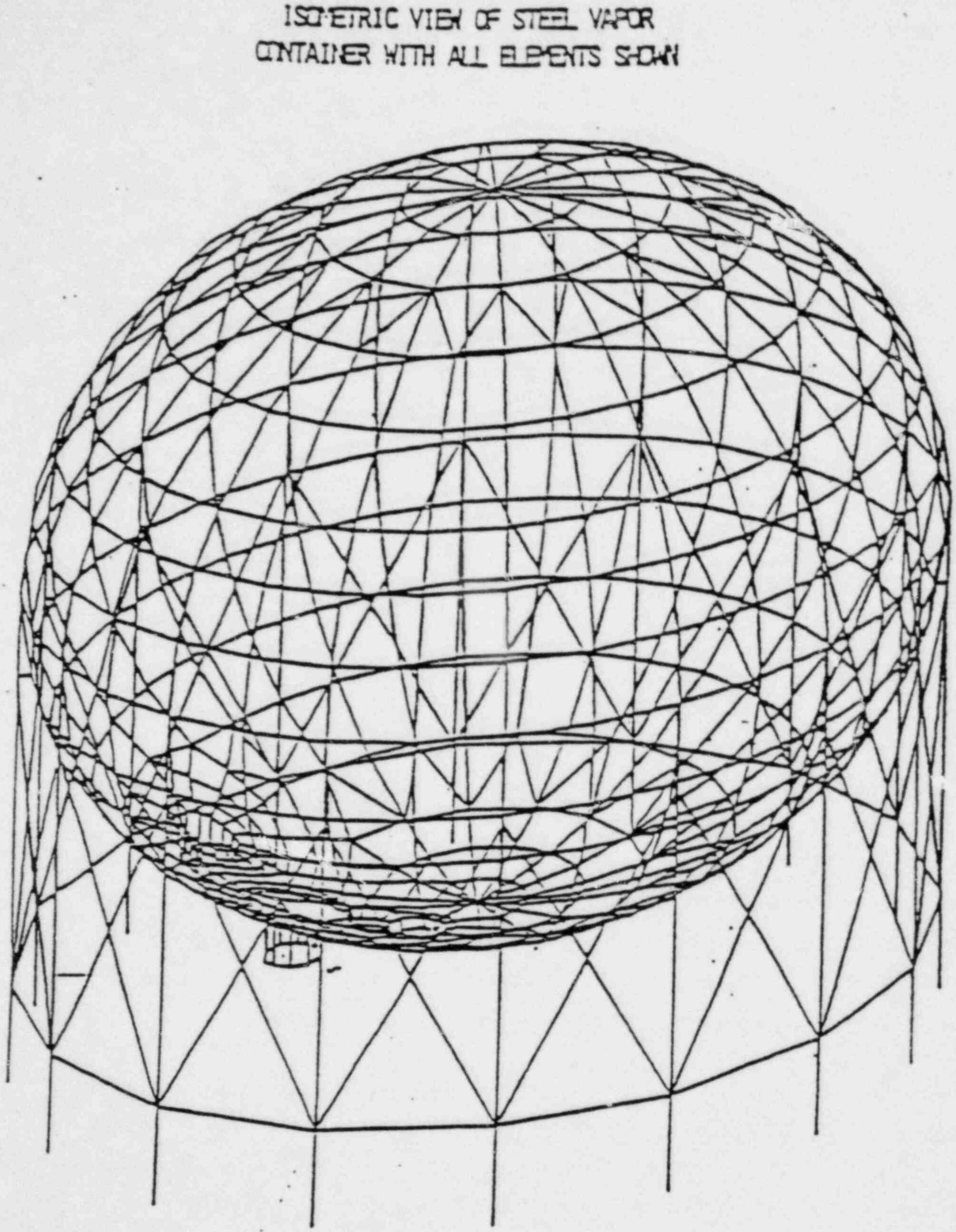


Figure 6. Computer model geometry plot.

$$D + L + T_a + R_a + P_a + E'$$

where: D = Dead loads

L = Live loads

T_a = Thermal loads under thermal conditions generated by the postulated design basis accident.

R_a = Pipe reactions under thermal conditions generated by the postulated design basis accident.

P_a = Pressure equivalent static load generated by the postulated design basis accident

E' = Loads generated by the safe shutdown earthquake.

The live loads, L, and the pipe reactions, R_a, were neglected in the analysis.

The stresses in the shell, columns, and panel-ties are within allowable stresses. Overstress at the threads of one tie-rod was found.

4.1.3 Review and Discussion

The structural-analysis model of the vapor container appears sufficient for seismic re-evaluation after comparing the dominating frequencies and mode shapes with the independent seismic analysis performed by NCT Engineering.

The damping value of 4% is reasonable for the evaluation of the vapor container, even though the stress level is low, since it is our understanding that no equipment was attached to the vapor container and no floor response spectra were generated.

Neglecting SSI is appropriate for the Yankee vapor container since the structure is lightweight and is supported by rather flexible columns with small individual footings. This was verified by the independent seismic analysis performed by the NCT Engineering.

The load combination is consistent with NRC SRP Section 3.8.2 and is therefore acceptable. The appropriateness of the pressure and thermal loads due to a postulated design basis accident were not evaluated since it is considered to be out of the present scope of work.

The overstress in three of the tie-rods under seismic condition is acceptable since the overstress is not great and redistribution of horizontal load to other tie-rods would occur.

4.2 CONCRETE REACTOR SUPPORT STRUCTURE (RSS)

4.2.1 Description of Structure

The concrete reactor support structure (RSS) (Figure 5) supports and provides radiation shielding for the nuclear steam supply system (NSSS). It consists of two concentric concrete ring walls supported by a semi-conical 5-ft.-thick bottom slab. Several radial walls connect and form compartments with the concentric ring walls for various NSSS components.

The bottom slab is supported on eight steel-encased concrete columns. Six of the columns are equally spaced under the outer ring wall and are 7 ft in diameter. These columns are standing on footing pedestals which, in turn, are supported on a circular concrete ring foundation. Inside the ring foundation, a square foundation supports the remaining two columns which have a diameter of 7 ft 6 in. All eight columns penetrate the steel vapor container through bellows to avoid interactive forces.

The exterior columns (Figure 7) are constructed of a 3/8-in. steel shell with concrete infill. The concrete is reinforced only at the top with 44 No. 14 bars. The minimum length of anchorage is estimated as 24 in., and about one-half of the 44 bars have length of anchorage less than 42 in. The reinforcing bars are confined with No. 7 hoops @ 12 in. on centers. In addition, 7/8-in. steel studs are welded to the steel shell at

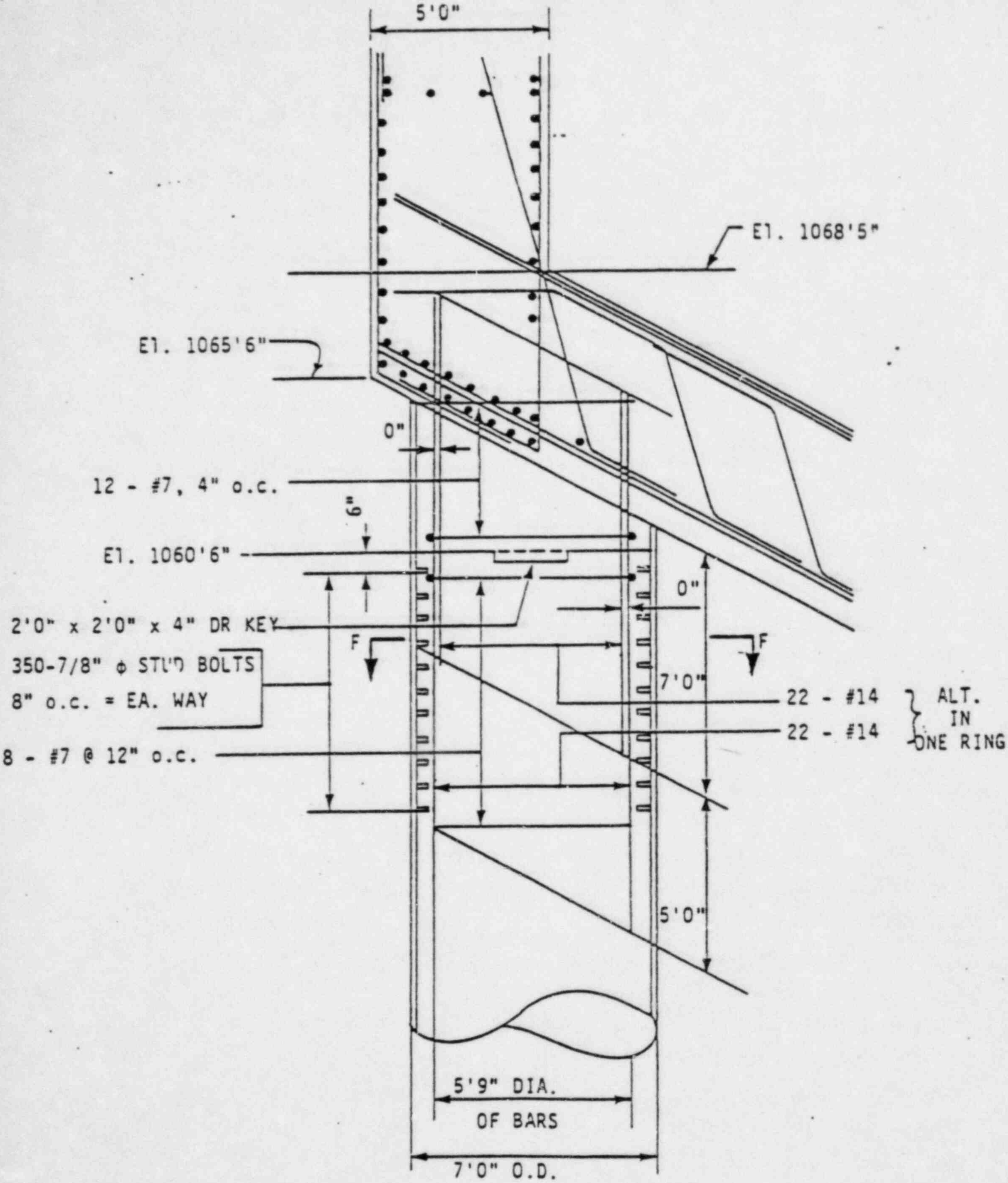


Figure 7. Exterior column.

the top to help transfer force between the concrete and steel shell. The two interior columns (Figure 8) are of the same construction as the exterior columns except that the steel shells are slightly thicker at 1/2 in, and 64 No. 14 bars with minimum length of anchorage of 42 in. were used.

The exterior column bases were recently modified to ensure that more than adequate strength exists for transferring the forces or moments from the columns to the ring foundation; i.e., the column-to-base connection was designed to be stronger than the capacity of the column.

4.2.2 Seismic Re-Evaluation of Concrete Reactor Support Structure

Two analytical models were developed. One is a linear elastic lumped-mass multi-degree-of-freedom stick model (Figure 9). The other is a two-dimensional nonlinear model (Figure 10) in which the concrete columns have bilinear hysteretic stiffness. The soil-structure interaction (SSI) effect for both models was neglected. A sensitivity study[12] was carried out to show that the effect of SSI on the concrete reactor support structure is insignificant.

The linear elastic model was first developed for the seismic analysis using the Yankee composite spectra. The stresses calculated at various locations of the structure are within elastic limits of the material. This justifies the use of a linear elastic model. When this elastic model was subjected to the NRC spectra, the stresses in some of the rebars, which connect the columns to the conical slab, exceeded the yield strength of the steel reinforcing bars. This lead to the development of the 2D nonlinear analysis model.

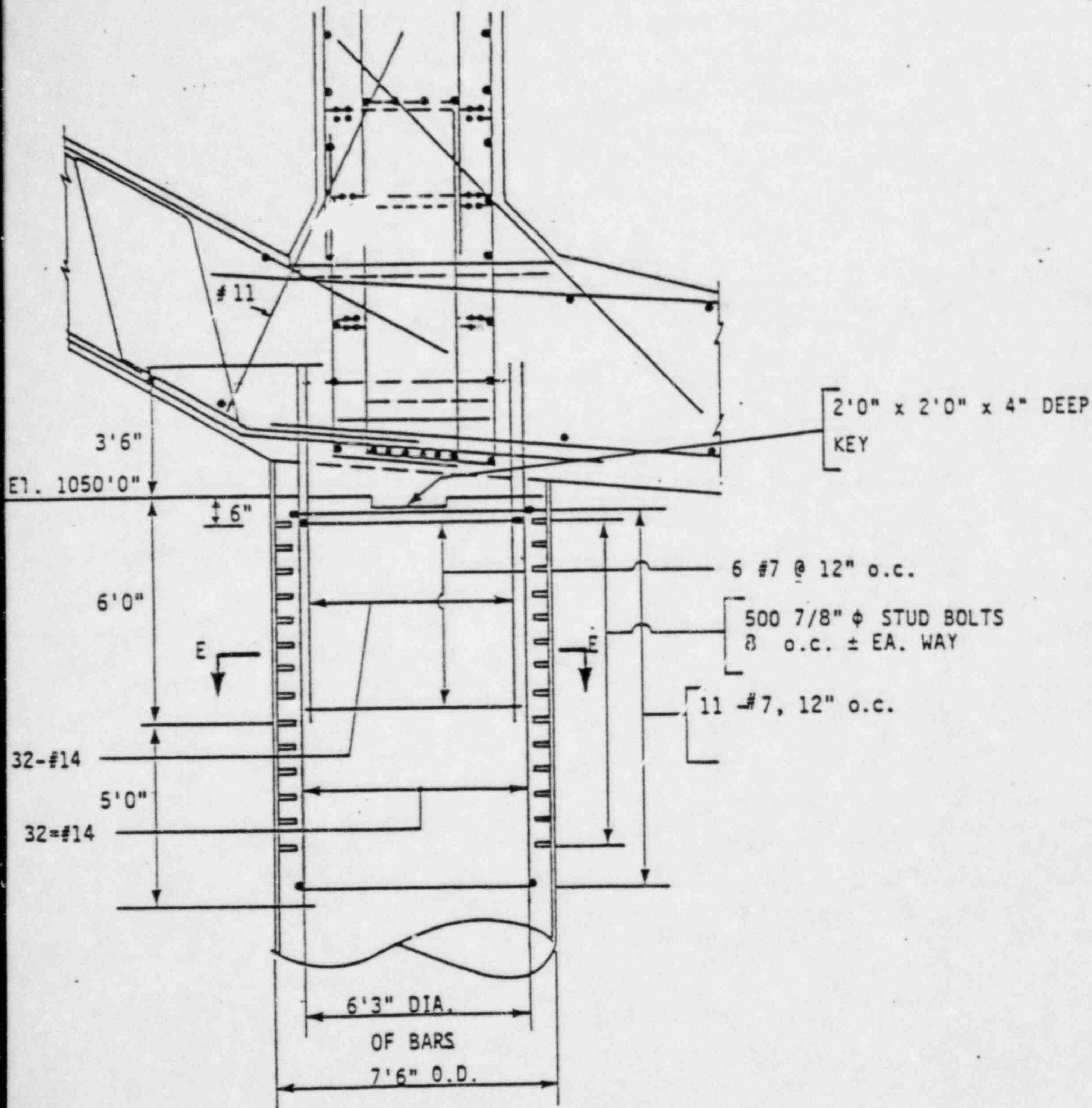


Figure 8. Interior column.

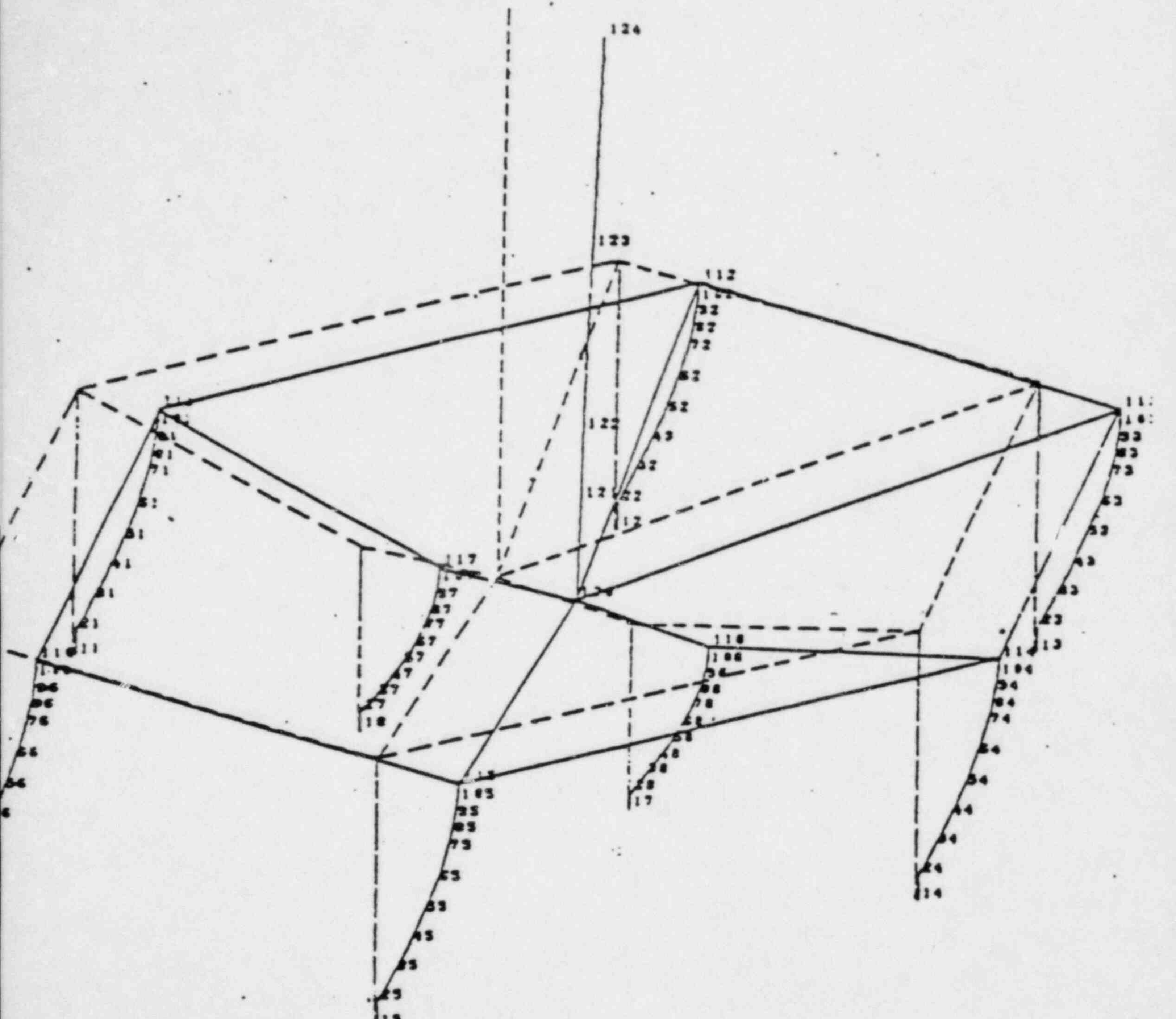


Figure 9. Containment structure model Y-2M mode shape No.1.

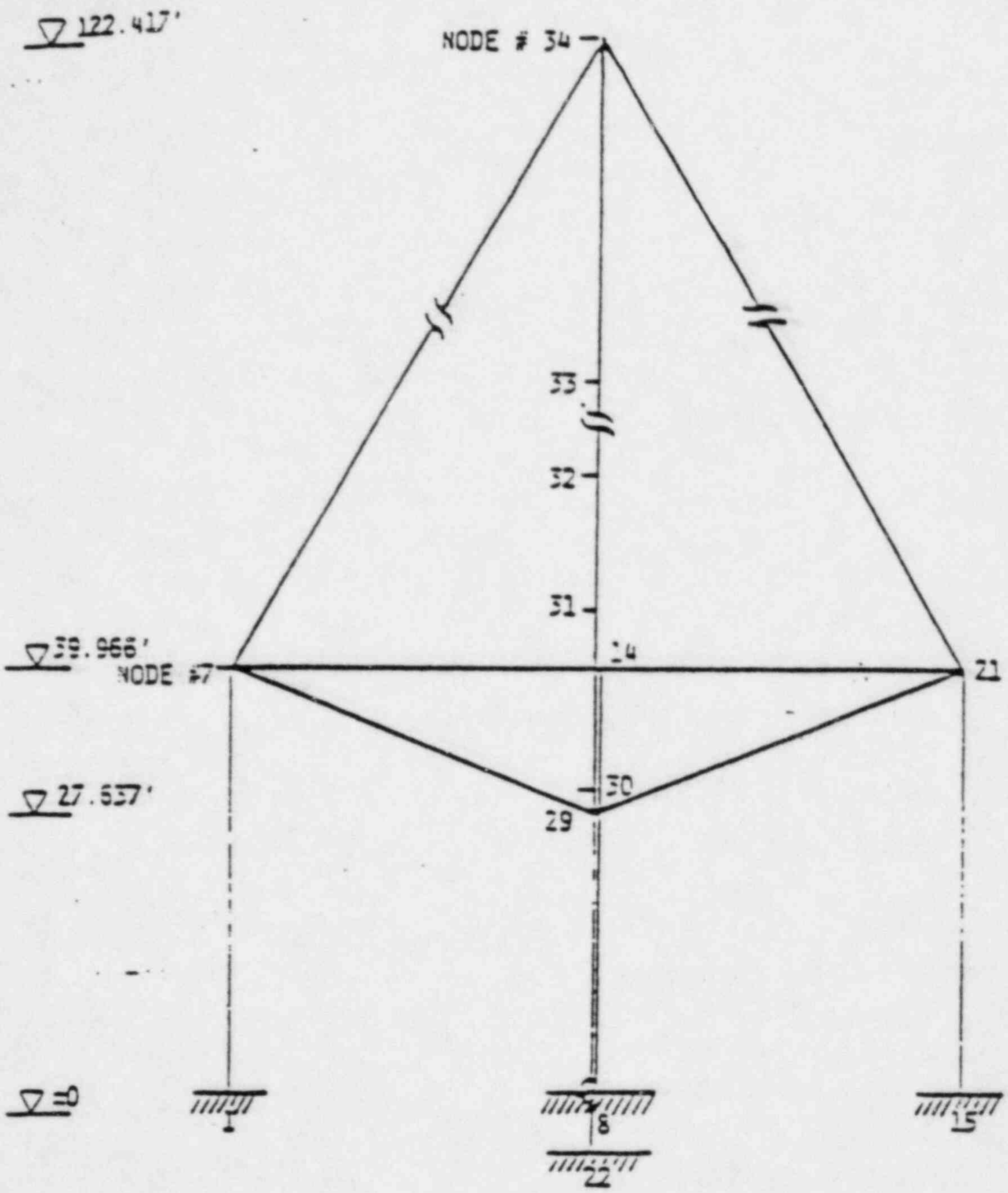


Figure 10. RSS non-linear model.

4.2.2.1 Linear Elastic Model

The superstructure of the concrete internal structure was modeled as a lumped mass stick model. Each column was modeled as a series of vertical beam elements. The conical slab which supports the superstructure was assumed to be rigid. Therefore, a series of rigid beams were used to connect the tops of the columns with the base of the superstructure stick mode. The columns were assumed to be fixed at the base. SSI effects were neglected.

Two seismic analyses methods were used for the linear elastic model. The modal-response-spectrum approach is used to assess the structural integrity of the concrete structure while a modal time history approach is used to develop in-structure response spectra. For the response-spectrum approach, modes are combined according to the U.S. NRC R.G. 1.92. The SRSS method is used to combine the effect of the three components of seismic motion.

The in-structure response spectra (ISRS) were developed using time histories obtained at nodes close to the equipment to be evaluated. This approach automatically considered the fact that the equipment are sometimes located far away from the lumped-mass locations of the model.

The stresses in the superstructure due to seismic loads are low. The area of concern is in the capacity of the columns which support the superstructure, especially in the column-to-slab connections.

The embedment length of the dowels at the top of the column is considerably less than that required by current code for full development of bar strength. A special study regarding bond and pull-out behavior of the reinforcing bars at the top of the columns was conducted by CYGNA. The conclusions from this study are:

1. The simplified bond slip model produces reasonable results in agreement with experimental observations.

2. The No. bars with 42-in. embedment at Yankee Atomic Electric Company plant will rupture before pulling out if subjected to monotonic loads.

3. The bond-slip relation for these bars will be as shown in Figure 8.

4. Under cyclic load reversals, the behavior is stable if yield stress in the steel is not exceeded. If yield stress in the steel is exceeded, there is a marked deterioration of the bond stress capacity and large bond slip is obtained in subsequent cycles at low steel stress levels.

The calculated stresses in the reinforcing bars using YC spectra were below yield. However, the stresses in some of the reinforcing bars exceeded yield when the NRC spectra were applied.

4.2.2.2 Two-Dimensional Nonlinear Model

The intent of the nonlinear analysis is to have a better understanding of the dynamic behaviors of the concrete columns when subjected to NRC spectrum. The concrete reactor support structure was represented by a two-dimensional lumped-mass stick model. The nonlinear stiffness of the concrete columns were obtained from computer analysis of the column sections using the computer program RCCOLA[15], which evaluate the flexural characteristics of reinforced concrete cross-sections subjected to monotonic loadings of axial forces and non-axial bending moments. The nonlinear dynamic analyses of the structure were performed by a modified version of the computer program DRAIN-2D[16]. To account for the second horizontal component of the ground motion which could not be included in the two dimensional model, the horizontal ground motions input for the analysis were increased by a factor of 1.1. From the flexural characteristics obtained from RCCOLA for column sections under monotonic loadings and the assumption of kinematic hardening, hysteric loading and unloading curves were derived for column stiffnesses. The other properties of the DRAIN-2D model were obtained by the standard procedure of structural modeling. The structure was assumed to have 7% damping.

Artificial time-history ground motions, with a duration of 10 seconds, which envelop 1.1 times the site-specific spectrum were used for the analysis. The results showed that the interior columns were under the most severe loads, the extreme bars just reached the strain hardening level. The rotations at the connection were on the order of 2.8×10^{-4} rad. The same nonlinear model was also analyzed for the El Centro earthquake. No strain hardening in rebars occurred.

4.2.3 Review and Discussion

Both the linear and nonlinear models were reviewed and were found to be appropriate. The stresses in the superstructure were low for both the YC and NRC spectra and, therefore, are acceptable.

In response to NRC questions regarding soil-structure interaction (SSI) raised during the April 5-6, 1982 meeting (Appendix A), CYGNA[12] studied single-degree-of-freedom systems simulating RSS with or without SSI. The absolute horizontal response time histories (Figures 11 and 12) show that the strong motion response of the structure occurs during the last two seconds of an 8-second duration. This indicates that the artificial ground motion time history used might not be long enough to yield conservative in-structure response spectra (ISRS). Therefore, the in-structure response spectra comparison may not be valid. Furthermore, care should be taken in using ISRS generated from the above described artificial ground motion time history for the evaluation of piping systems and equipment.

The adequacy of the connections between the concrete columns and the slab of RSS was evaluated by B. Bresler of Wiss, Janney, Elstner and Associates, Inc. His judgment is that there is no likelihood of a catastrophic collapse of the RSS due to failure of the supporting structure in a seismic event. However, the following issues should be resolved:

1. What is the steel grade and what is the appropriate stress-strain curve for the No. 14 bar dowels? Values of F_y = 40 ksi and F_y = 50 ksi have been used in different reports.

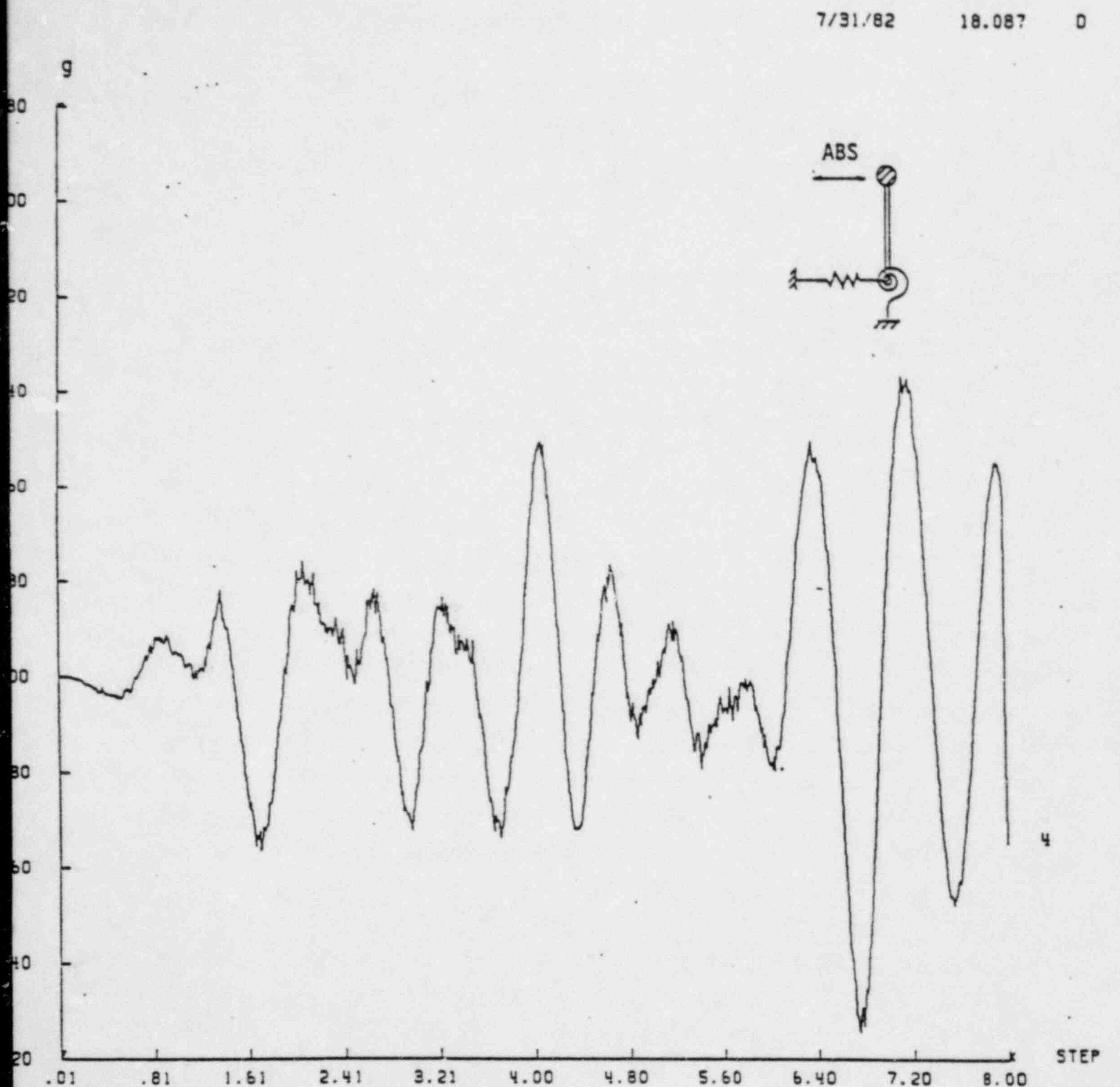


Figure 11. Simplified RSS-soil model (horizontal and rocking).

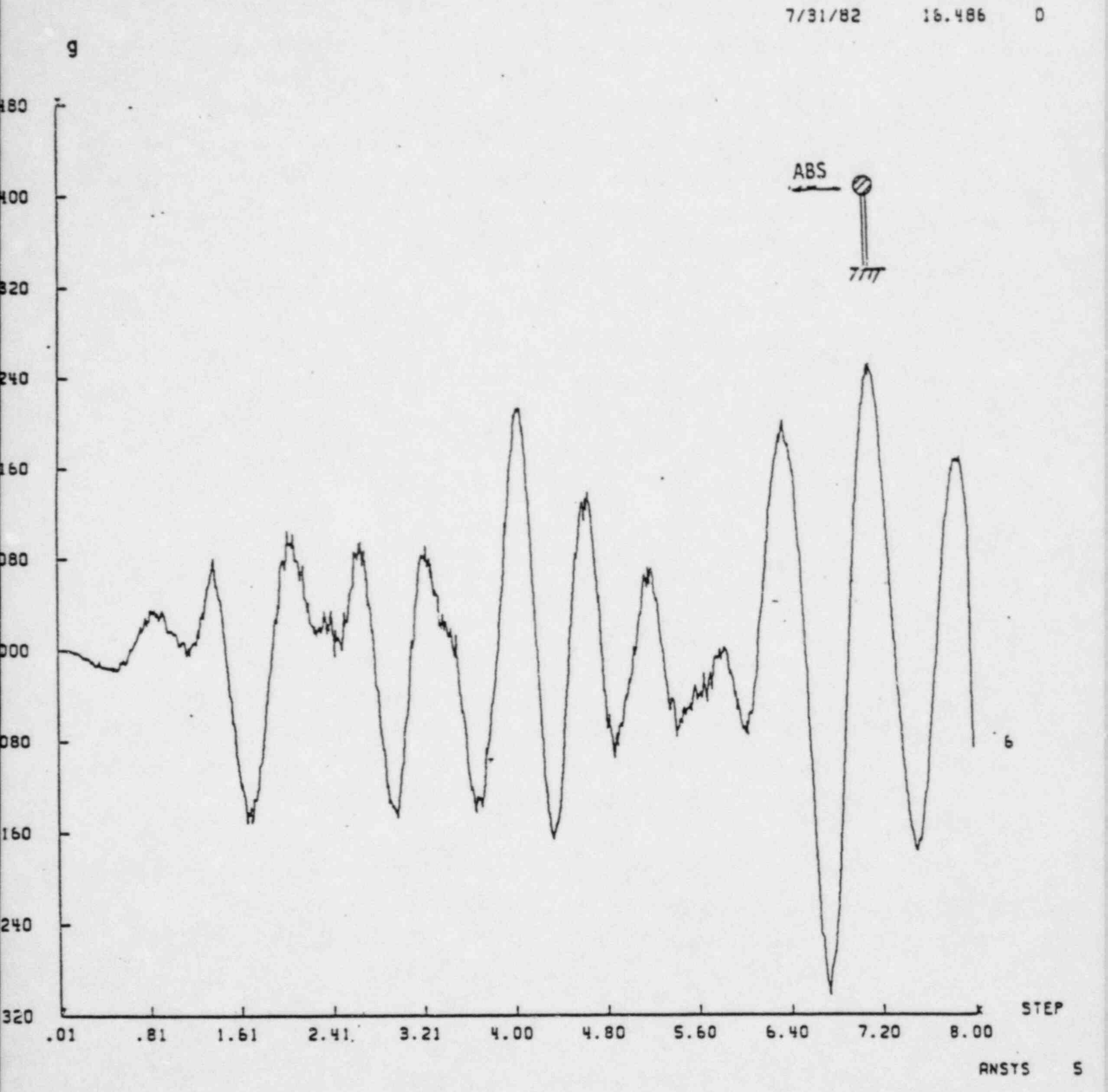


Figure 12. Simplified RRS model.

What effect does the steel yield strength and the shape of the stress-strain diagram have on the hysteretic behavior of the joint?

2. The size and type of studs should be identified and an analysis of load transfer from the steel shell through the studs and into the concrete and the dowel bars should be carried out. In particular, consequences of semi-ductile or brittle behavior on the local load transfer should be evaluated.

3. What is the maximum bond stress which can be reasonably expected in the connection? The value of 1000 psi used in the CYGNA analysis appears to be somewhat high. Justifications of the appropriate maximum bond stress value requires further documentation of the geometry of the surface deformations in the actual dowel bars. This information would then allow a better estimate of bond strength under monotonic loads.

   In arriving at bond strength, credit has been taken for the effects of aging on the concrete compressive strength. However, concrete compressive strength at the top of a column is likely to be below average because of the normal amount of bleeding and dilution of paste.

   Below the construction joint, circular ties are spaced at 12 in., and the dowel bars in the interior columns are too closely spaced. Local cracking around the stud bolts may further reduce the ultimate bond strength.

   Degradation in bond strength due to cyclic loading, particularly when maximum tension stress in the reinforcement reaches yield strength, must also be taken into account.

On the positive side, there is the confinement effect of the steel shell on the bond strength and on the overall capacity of the connection. Under these conditions, and in the absence of directly applicable test results, it is not possible to predict reliably the value of maximum bond strength. However, it would be prudent to evaluate the effect of a lower bound bond stress value on the behavior of the structure.

4. Anchorage length of 42 inches is assumed in developing a bilinear force-displacement curve for the No. 14 bar dowel. This anchorage length is based on the arrangement of steel in the interior columns. Some bars in the exterior columns have an embedment length of only about 24 in. The following questions must be resolved: Can yield strength of No. 14 bar be developed in a 24-in. anchorage length and with reduced bond stress, and what effect, if any, does this condition have on the overall behavior of the structure?

5. Values of 7% and 10% of damping used in the seismic analyses appear to be excessive, in light of low stresses throughout most of the structure and the confinement of concrete columns provided by the steel shells. Verification of the concept that foundation lift-off and associated behavior may justify the higher damping values requires a detailed review of foundation behavior and of its effect on the general behavior of the structure during a seismic event.

4.3 TURBINE BUILDING

4.3.1 <u>Description of Structures</u>

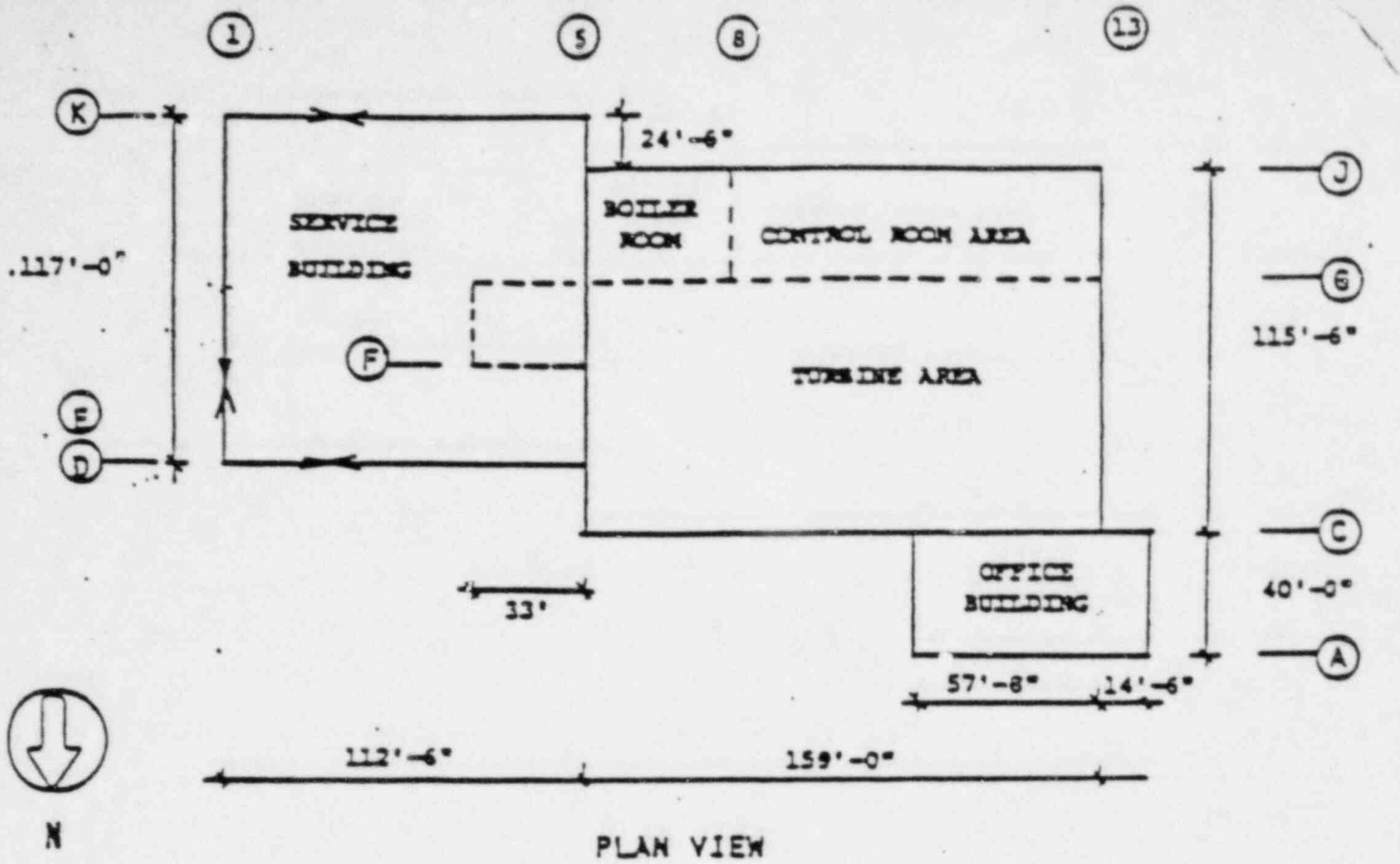
The turbine building is basically a steel frame structure with reinforced concrete floors. The turbine building is adjacent to the service building and the office building, and is connected to the reinforced concrete control-room building (Figure 13). The exterior walls of the upper part of turbine building consist of steel frames, bracing systems with back-to-back channels, and steel siding. The walls of the control room area, which houses critical equipment on the side facing the reactor, are constructed of 4-ft-thick solid concrete. This concrete construction serves both as building walls and as shielding in case unusual radioactivity should develop. The exterior walls of the lower part of the turbine building are of hollow concrete-block construction. Interior partitions are all concrete blocks of the standard hollow type, but grouted solid where shielding is required. Steel decking is used for the roof, except over the control room where 4-ft-thick reinforced concrete is used to provide shielding.

The turbine pedestal, which supports the turbine generator, is separated from the turbine building floors and can be considered as an independent structure.

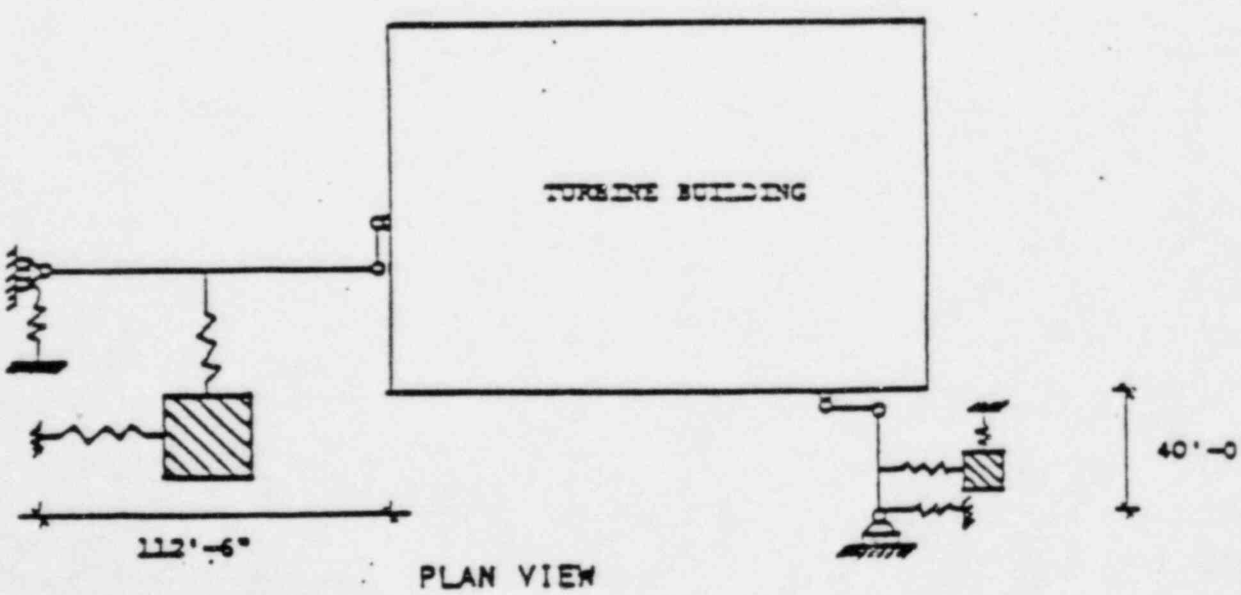
4.3.2 Seismic Re-Evaluation of Turbine Building

The structural models used for the seismic analysis of the turbine building are linear lumped-mass stick models (Figure 14). To evaluate the effects of block walls, models with and without the effects of the block walls included were analyzed. The final results were taken from the enveloped results of the two models. The study of the vertical response used three models: (1) Model 1 for the control and switch gear rooms; (2) Model 2 for the turbine area; and (3) Model 3 which represents a typical column in signal line G. The turbine building and the turbine pedestal were assumed to be uncoupled. The turbine pedestal was assumed to act as a one-story moment-resisting space frame (Figure 15).

For structural evaluations, the method of response-spectrum analysis was used. The seismic inputs were the Yankee composite spectrum and the NRC spectrum with 7% critical damping. The computer program used was a CYGNA in-house program, BATS. The model responses and responses to ground motion components were combined by the SRSS method.



a. Arrangement of attached buildings.



b. Equivalent modeling.

Figure 13. Modeling of attached buildings.

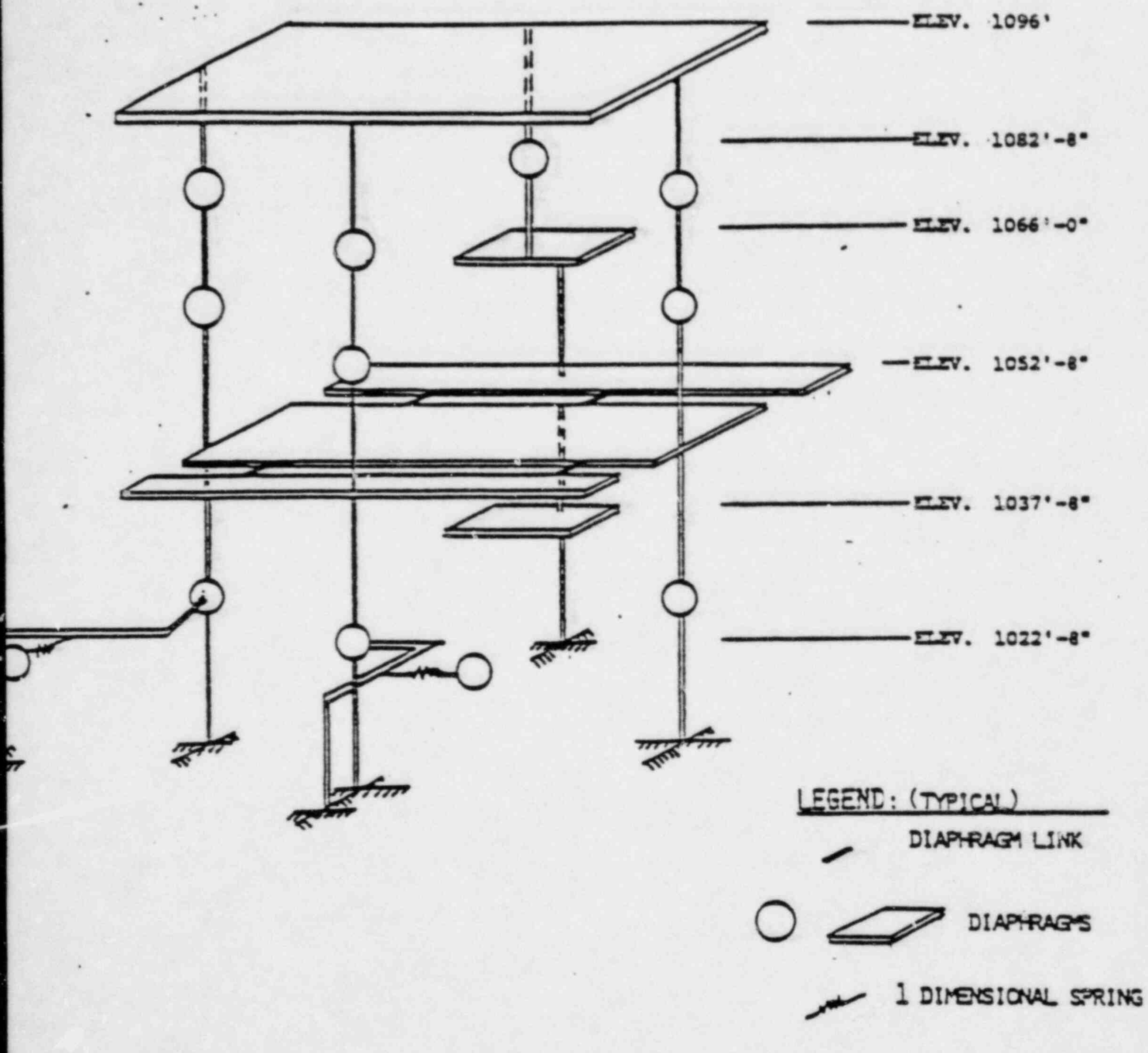
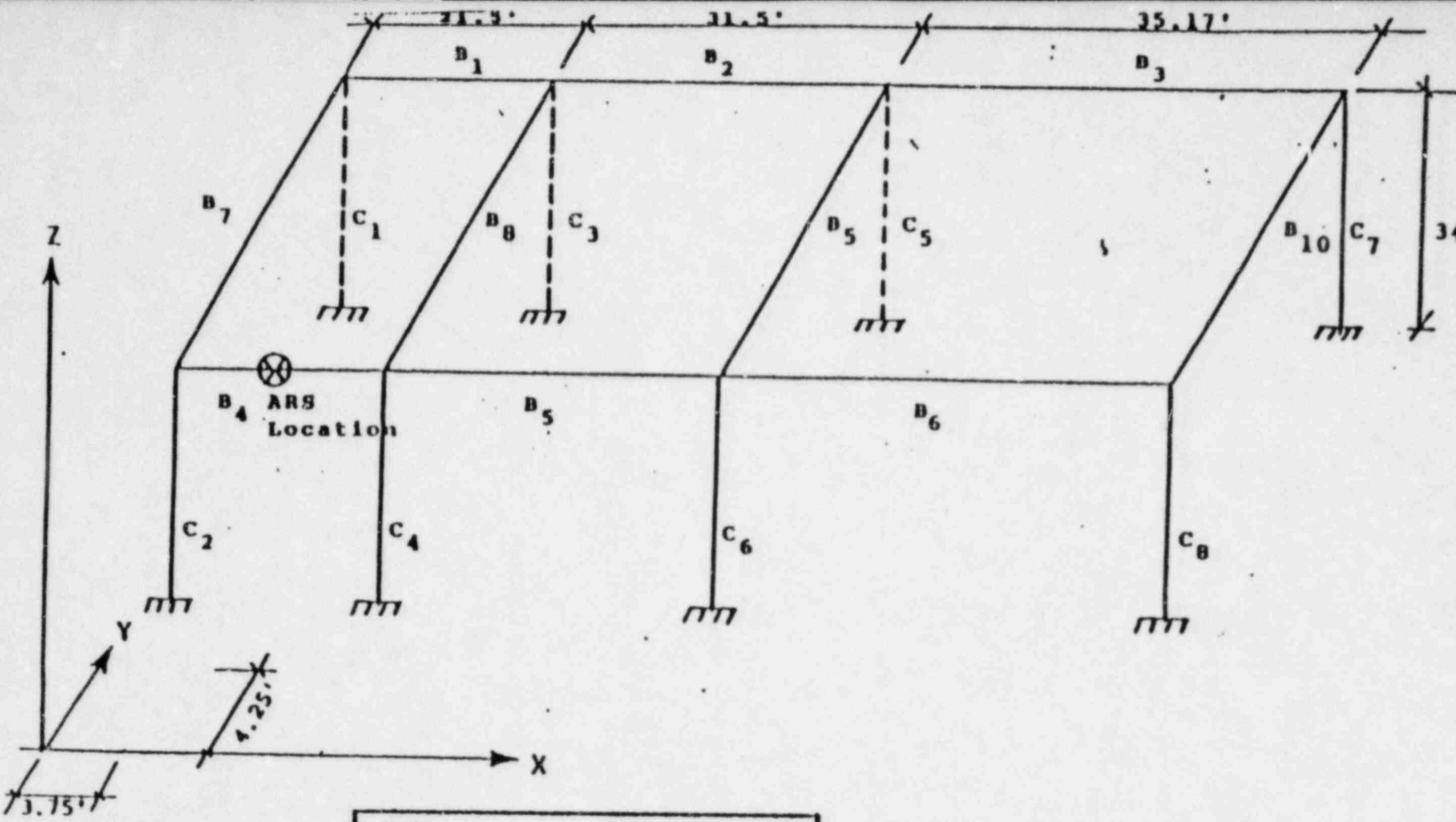


Figure 14. Schematic representation turbine building stick model.



Column	X (in.)	Y (in.)
1	3.75	31.25
2	3.75	4.25
3	25.00	31.25
4	25.00	4.25
5	61.00	31.25
6	61.00	4.25
7	96.17	31.25
8	96.17	4.25

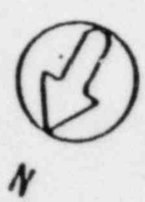


Figure 15. Three-dimensional model for lateral analysis of turbine pedestal.

4.3.3 Review and Discussion

Preliminary results indicate that the stresses in the turbine pedestal were estimated to be very low. The pedestal is expected to remain within the elastic range for both Yankee composite-spectra and NRC site-specific spectra.

Results of the turbine-building analysis showed that the frames, bracings, and joints were overstressed at several locations. Also the blockwalls under the control room were overstressed. Structural modification of the turbine building is required.

YAEC has decided to upgrade the frames, bracing, joints, and blockwalls of the turbine building to design allowables using the Yankee composite spectra. As shown in Tables 2 through 4, the upgraded turbine building will also meet the SEP criteria for NRC site-specific spectra.

4.4 PRIMARY AUXILIARY BUILDING (PAB)

4.4.1 Description of Structure

This is a two-story structure with a reinforced concrete lower story and a steel frame upper story. This building shares a common wall with the ion-exchange building. Interior partitions of the PAB are all concrete blocks of the standard hollow type, but are grouted solid where shielding is required. The roof is metal deck and the foundation is composed of the spread footings. The structure is connected to the shielded radioactive pipe tunnel.

4.4.2 Seismic Re-Evaluation of Primary Auxiliary Building

The effects of soil-structure interaction were neglected in the analysis. The structure was modeled by a stick model with lumped masses.

STORY LEVEL	FRAME LINE	BRACE ID's	P SEISMIC (KIPS)	f_A (KSI)	F_A, AS PER AISC 1.5.2	$\frac{f_A}{F_A}$
1096'-0"	5	C-D	210	17.9	17.1	1.05
1096'-0"	5	G-F	144	12.3	17.1	0.72
1082'-8"	5	D-F	270	18.4	18.6	0.99
1066'-0"	5	F-D	245	16.6	18.6	0.89
1052'-8"	5	F-G	78	3.0	18.2	0.16
1037'-8"	5	E-D	188	16.0	22.6	0.71
1037'-8"	5	E-F	180	15.3	22.6	0.68
1082'-8"	5	G-F(N)	145	10.7	13.1	0.82
1066'-0"	5	F-G(N)	180	13.3	13.1	1.02
1052'-8"	5	H-G(N)	51	4.4	9.3	0.47
1052'-8	5	H-J(N)	40	3.5	9.3	0.38
1052'-8"	5	D-E(N)	75	6.5	13.4	0.49
1052'-8"	5	F-E(N)	101	8.8	13.4	0.67
1096'-0"	G	9-8	106	9.0	18.9	0.48
1082'-8"	G	8-7	156	13.3	19.4	0.69
1052'-8"	G	12-11	36	3.1	19.4	0.16
1037'-8"	G	11-10	27	2.3	19.4	0.12
1096'-0"	G	9-10(N)	131	9.7	10.8	0.90
1082'-8"	G	8-9(N)	138	10.2	12.7	0.80
1037'-8"	G	11-12(N)	28	2.4	9.1	0.26
1096'-0"	13	C-D	183	15.6	17.1	0.91
1096'-0"	13	G-F	142	12.1	17.1	0.71
1082'-8"	13	F-D	253	17.2	18.6	0.92
1066'-0"	13	D-F	230	15.6	18.6	0.84
1052'-8"	13	F-G	112	9.5	18.2	0.52
1037'-8"	13	F-G	63	5.4	18.2	0.30
1082'-8"	13	F-G(N)	138	8.9	13.1	0.68
1066'-0"	13	F-G(N)	161	10.5	13.1	0.80
1037'-8"	13	G-F(N)	58	5.0	9.3	0.54
1096'-0"	C	12-11	93	7.9	17.3	0.46
1082'-8"	C	11-10	154	13.1	18.0	0.73
1066'-0"	C	10-9	154	13.1	18.0	0.73
1052'-8"	C	9-8	103	8.8	18.4	0.48
1037'-8"	C	8-9	106	9.0	18.4	0.49
1096'-0"	C	10-11(N)	65	5.7	8.9	0.64
1082'-8"	C	9-10(N)	145	10.7	10.5	1.02
1066'-0"	C	8-9(N)	153	11.3	13.7	0.82
1052'-8"	C	7-8(N)	102	8.9	9.3	0.96
1037'-8"	C	8-7(N)	114	9.9	9.7	1.02

NOTE: Brace ID W(N) denotes New Members.

Table 2. Stresses in diagonal braces for NRC spectrum, w/fixes.

CONNECTION LEVEL ABOVE	FRAME LINE	COLUMN ID AT TOP	BRACED LOAD (KIPS) $P_{SEISMIC}$	CONNECTION CAPACITY AS PER AISC 1.5.2	STATUS
1096'-0"	13	C	183	201	OK
1096'-0"	G	9	106	167	OK
1082'-8"	13	F	253	436	OK
1082'-8"	G	8	156	235	OK
1066'-0"	5	F	245	235	OK
1066'-0"	C	10	154	302	OK
1052'-8"	13	F	112	302	OK
1037'-8"	5	E	188	176	7% OVERSTRESSED SAY OK
1037'-8"	5	E	180	176	2% OVERSTRESSED SAY OK
1037'-8"	13	F	63	235	OK

Table 3. Loads in brace to beam column joint connection for NRC spectrum, w/fixes.

	DEAD LOAD, 1					COMBINATION			
		OVERLYING							
FTG. ID	COL.	FTG.	SOIL	LIVE LOAD, 2	SEISMIC LOAD, 3	1+2+3	1-3	FTG. CAPACITY	REMARKS
C-13	39	31	41	277	119	507	-8	678	UP-OK
D-13	81	25	26	247	90	469	42	623	
F-13	81	22	19	247	146	515	-24	623	UP-OK
G-13/RC	268 + 331	88	79	355	250	1371	516	1781	
G-12	561	76	53	638	72	1400	618	1830	
G-11	566	63	46	625	21	1321	654	1621	
G-10	566	76	53	625	12	1332	683	1830	
G-9	566	86	104	625	68	1449	688	1921	
G-8/RC	278 + 331	111	107	630	662	2119	165	2035	4% OL-OK
G-7	126	45	34	550	83	838	122	1145	
G-5	101	21	29	379	265	795	-114	797	UP-OK
J-8/RC	28 + 2380	252	262	82 + 221	1559	4784	1363	6455	
J-7	28	8	6	82	0	124	42	215	
J-5	29	6	5	56	29	125	11	170	
F-5	108	30	25	301	161	625	2	848	
E-5	7	4	4	22	15	52	0	130	
D-5	108	41	43	301	6	499	186	954	
C-5	32	25	23	250	134	464	-54	596	UP-OK
C-12	87	38	35	419	57	636	103	742	
C-11	87	36	33	419	66	641	90	795	
C-10	87	60	62	419	20	648	189	848	
C-9	87	78	100	419	39	723	226	1272	
C-8	87	54	56	419	79	695	118	919	
C-7	87	62	61	419	120	749	90	795	
J-13	2380	229	262	221	1309	4401	1562	6074	

NOTES:
1. Seismic Load Refer to Computer Binder = 81061/2.1.F/A
2. Col. Dead & Live Loads are from S&W DWG #FS-2B
3. Ftg W'T Includes Concrete Pedestal & Grade Beam Within Ftg. area.
4. Overlying Soil W't = $0.1K_{CF}$ X (A_{ftg} - $A_{pedestal + grade bm}$) x (1022'-8" - T.O. Ftg. El.)
5. Ftg capacity = 10.6 FSF x ftg area.
6. Remarks: OL denotes Overload; UP denotes Uplifting
7. See sheet attached for investigation of Uplifting on Fig. G-5.

Table 4. Foundation load for NRC spectrum, w/fix.

The model included the mass and stiffness effects of the ion-exchange building. The metal deck roof was simulated by elastic links (Figures 16 through 19). The method of response-spectrum analysis was used for calculating structural responses. The computer program used was CYGNA's in-house program, BATS. The seismic inputs were the Yankee composite and the NRC spectra with 7% critical damping. The model responses and responses to ground motion components were combined by the SRSS method.

4.4.3 Review and Discussion

The predicted stresses in general are low. However, at some connections of steel beams to concrete walls, overstresses can occur.

Minor fixes are required to upgrade the primary auxiliary building in order to meet the Yankee composite spectra. The upgraded structure is expected to meet the NRC spectra. Figure 20 shows some of the fixes suggested by CYGNA for the primary auxiliary building.

4.5 DIESEL-GENERATOR BUILDING

4.5.1 Description of Structure

The existing building is mainly a steel-frame structure with bracings. There are also some block walls which are not reinforced and not grouted. An annex of the building is currently being added to the structure. The structure includes the building, annex, nitrogen storage supporting frame, and accumulator tank.

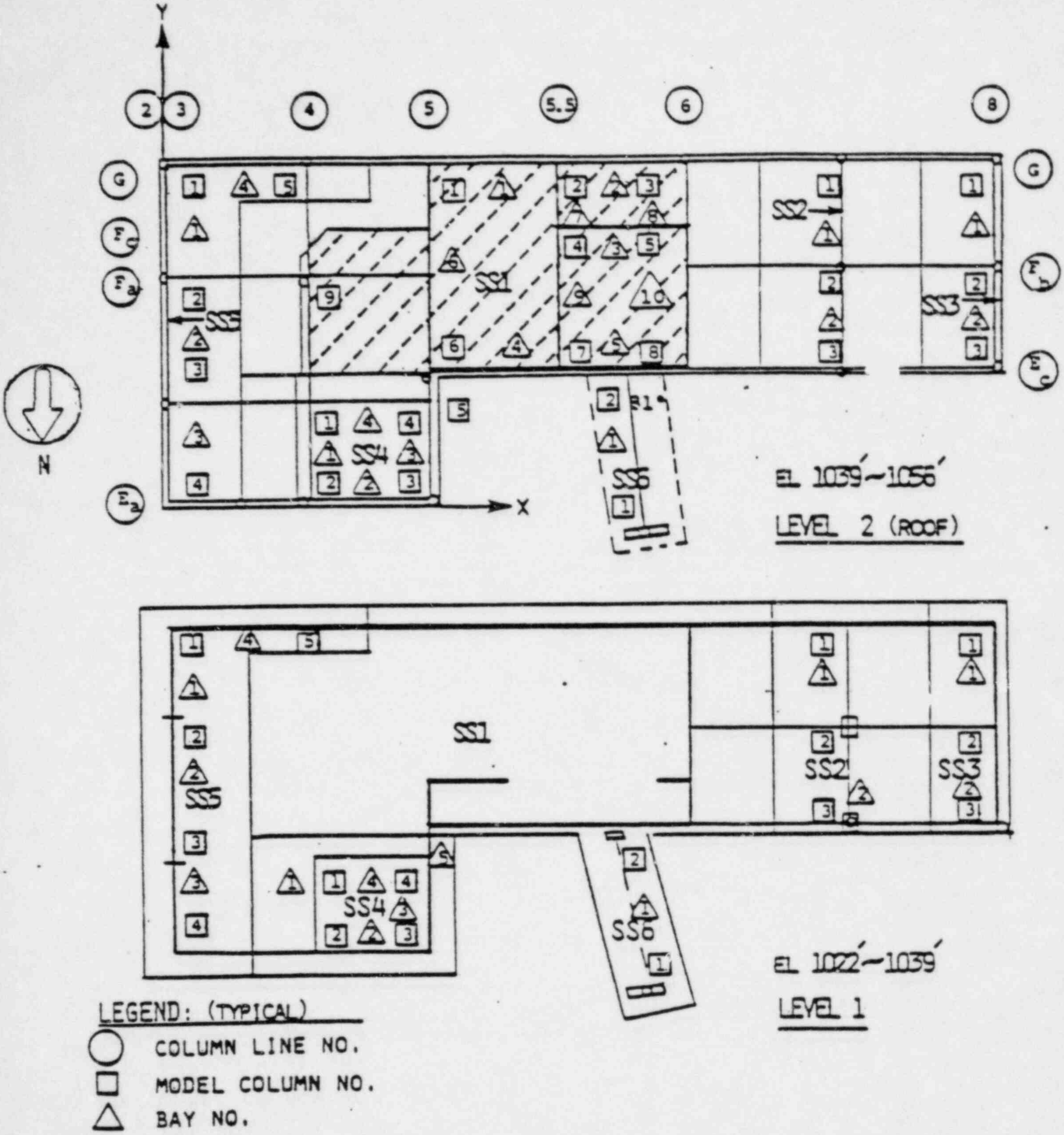


Figure 16. Framing of substructures.

LEVEL 2 (ELEV. 1056')

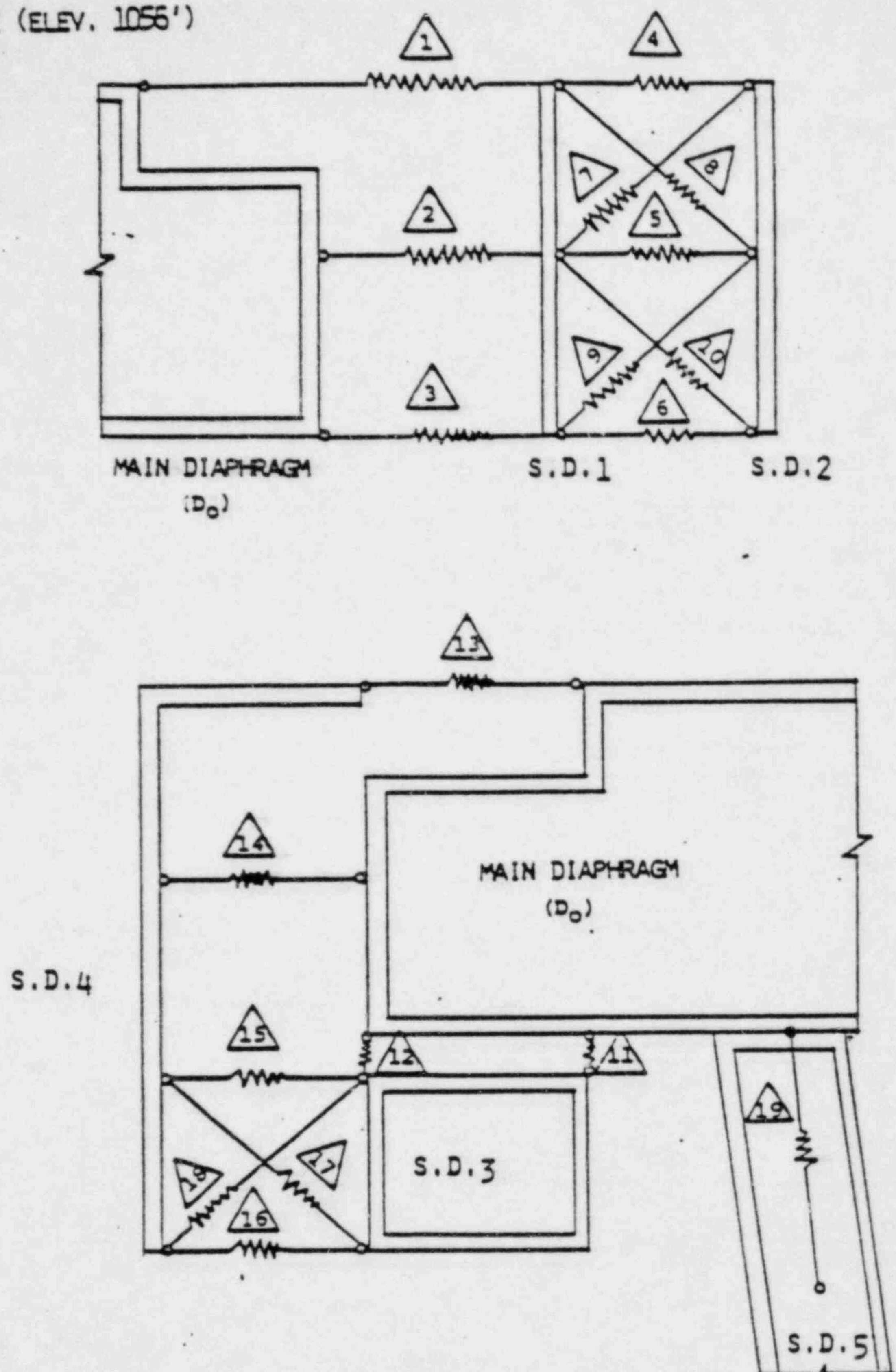


Figure 17. Flexible links.

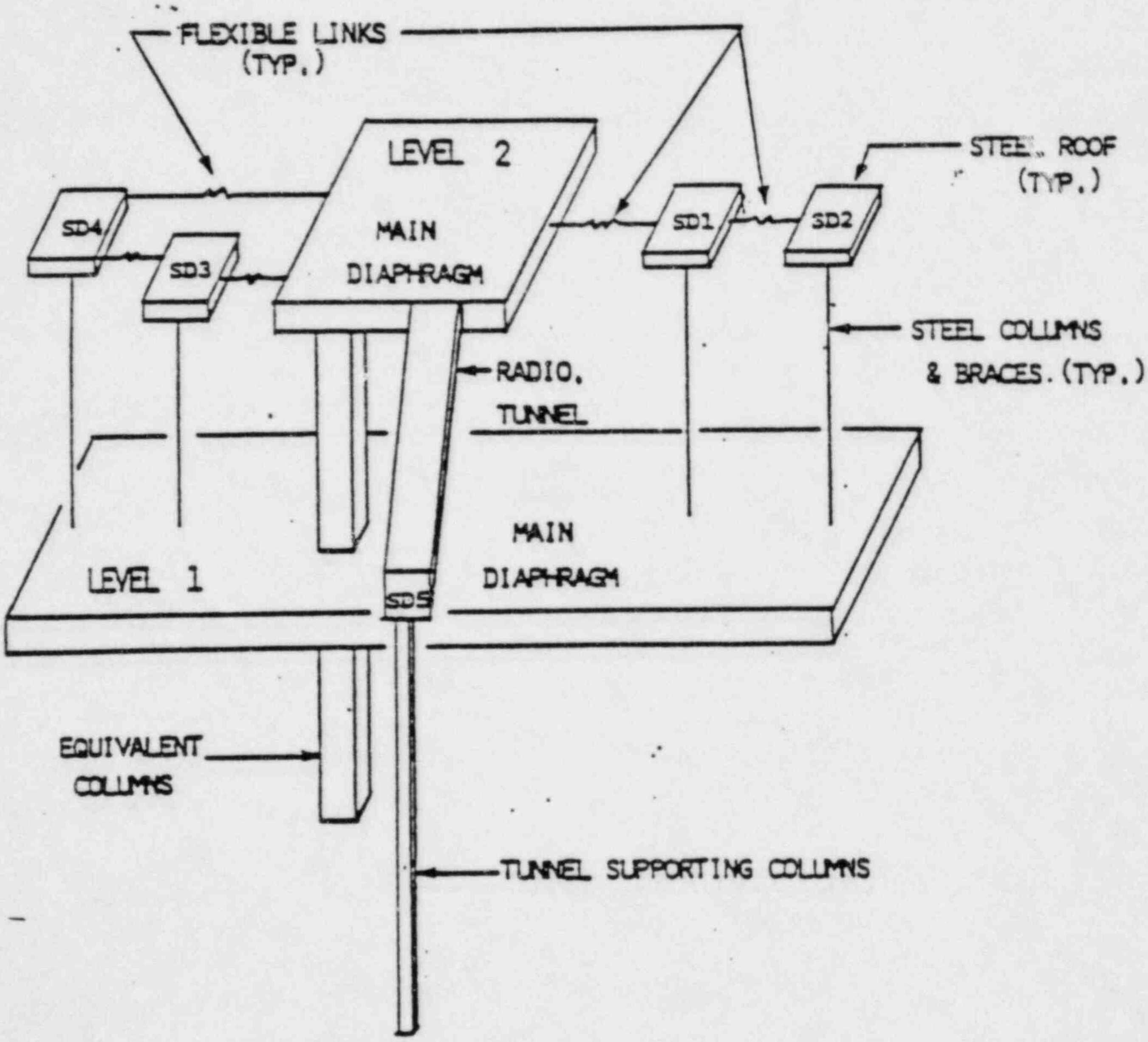


Figure 18. Horizontal stick model of PAB.

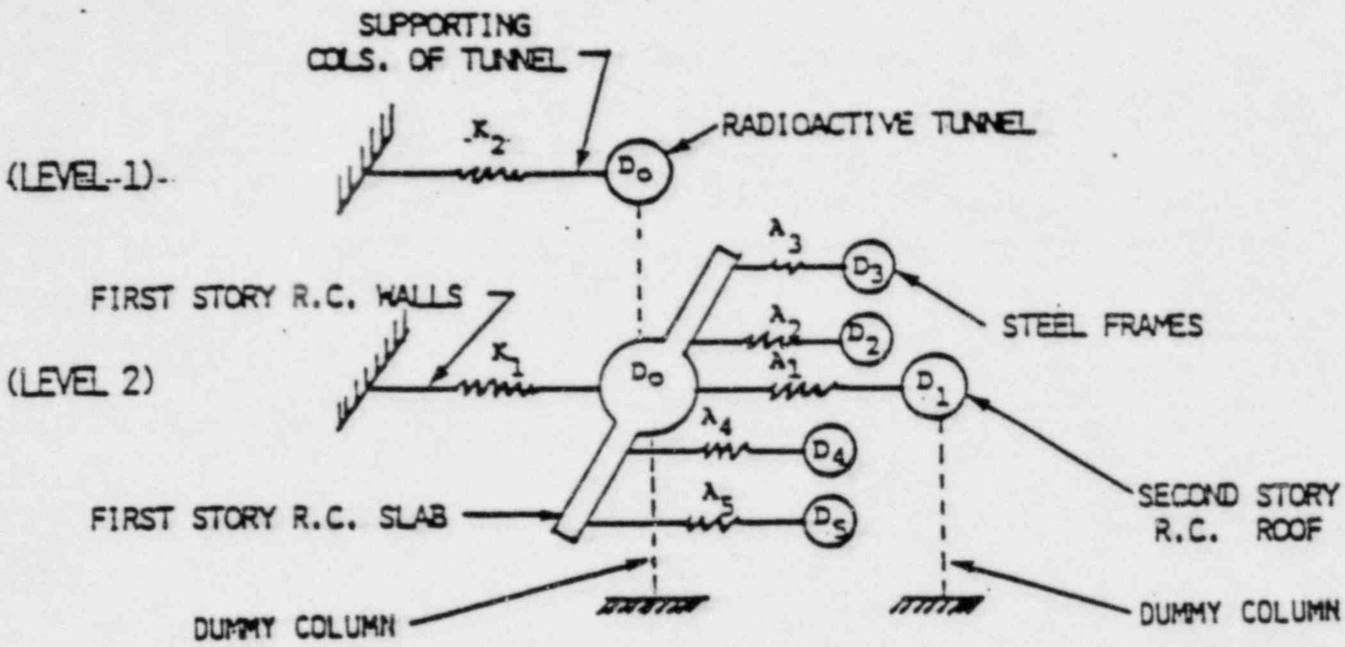


Figure 19. Vertical model.

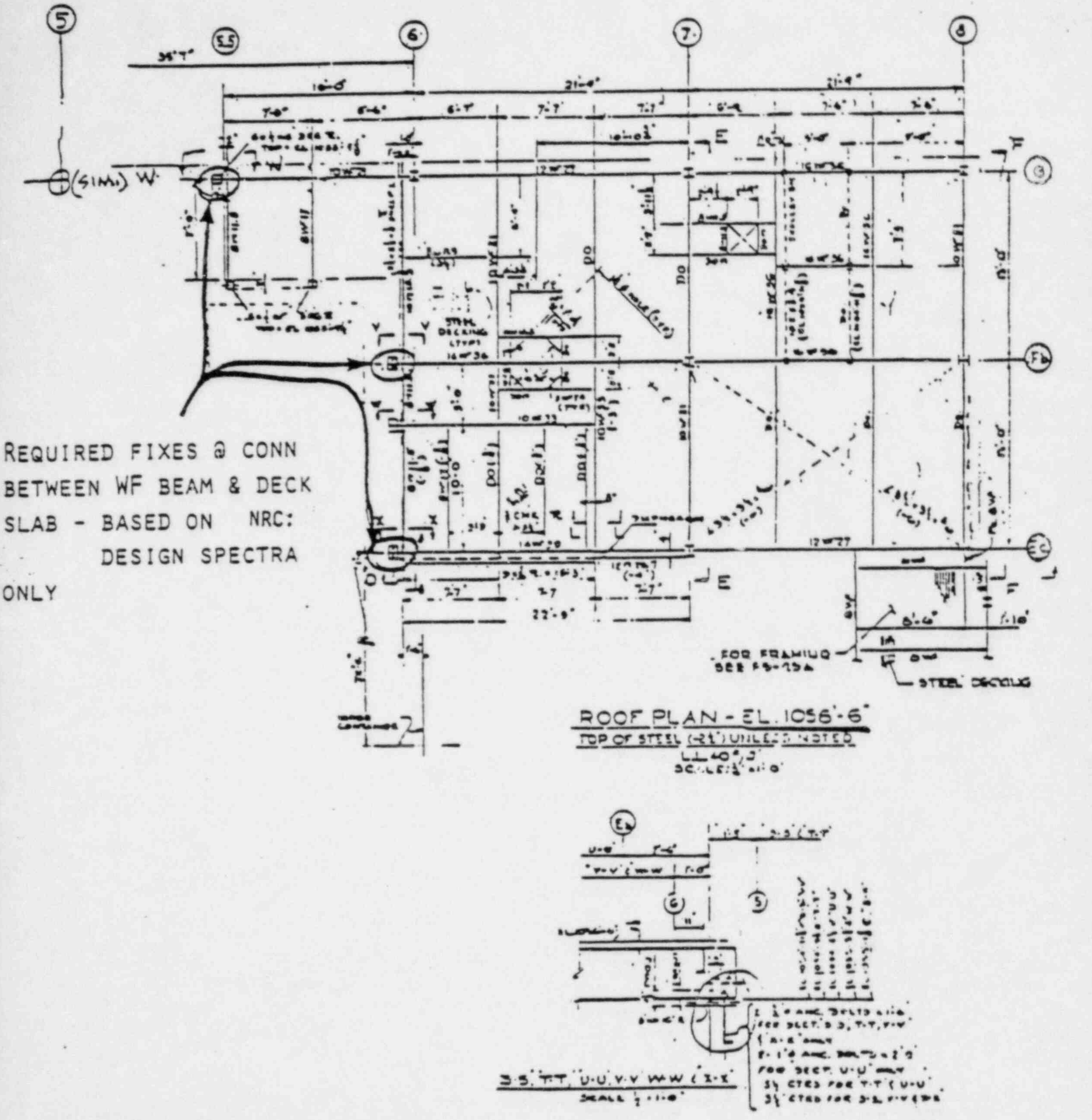
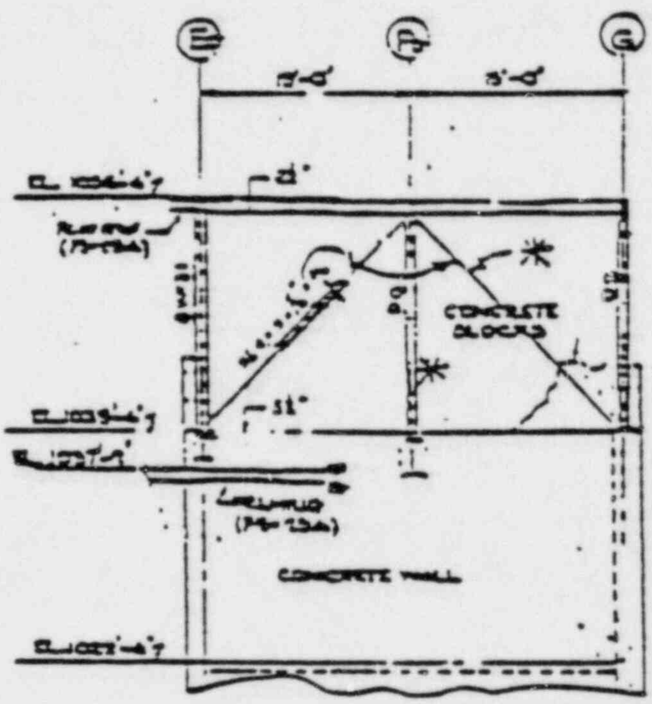


Figure 20. Suggested fixes for the Primary Auxiliary Building.
(Sheet 1 of 2.)

ORIGINAL DRAWING: FS-19B

*THESE MEMBERS, INCLUDING END CONNECTIONS, ARE REQUIRED "FIX" BASED ON THE NRC AND THE YCS.



(GRID LINE 8)

SECTION F-F

Figure 20. Suggested fixes for the Primary Auxiliary Building. (Sheet 2 of 2.)

4.5.2 Seismic Re-Evaluation of Diesel-Generator Building

The three-dimensional space frame model (Figure 21) was used for this structure. No soil-structure interaction was considered in the analysis, and the foundation was assumed to be rigid. The mass, but not the stiffness of the block walls, was included in the model. Both the Yankee composite and NRC spectra were used for the analysis. The response-spectrum method and the code BATS were used for the analysis. Model responses and responses to ground motion components were combined by the SRSS method.

4.5.3 Review and Discussion

Analysis results showed no buckling in bracings, but high bending stresses occurred in some columns. CYGNA indicated that the lateral force resistance of the diesel generator building and annex will be adequate to withstand seismic loads as specified by the Yankee composite and NRC spectra if certain fixes are implemented. Also, the block walls in this building will be upgraded to resist the loads induced by the Yankee composite spectra.

4.6 STEEL-FRAME STRUCTURE

4.6.1 Description of Structures

The steel-frame structure supports the main steam (MS) and feed water (FW) lines between the concrete reactor support structure and the turbine building.

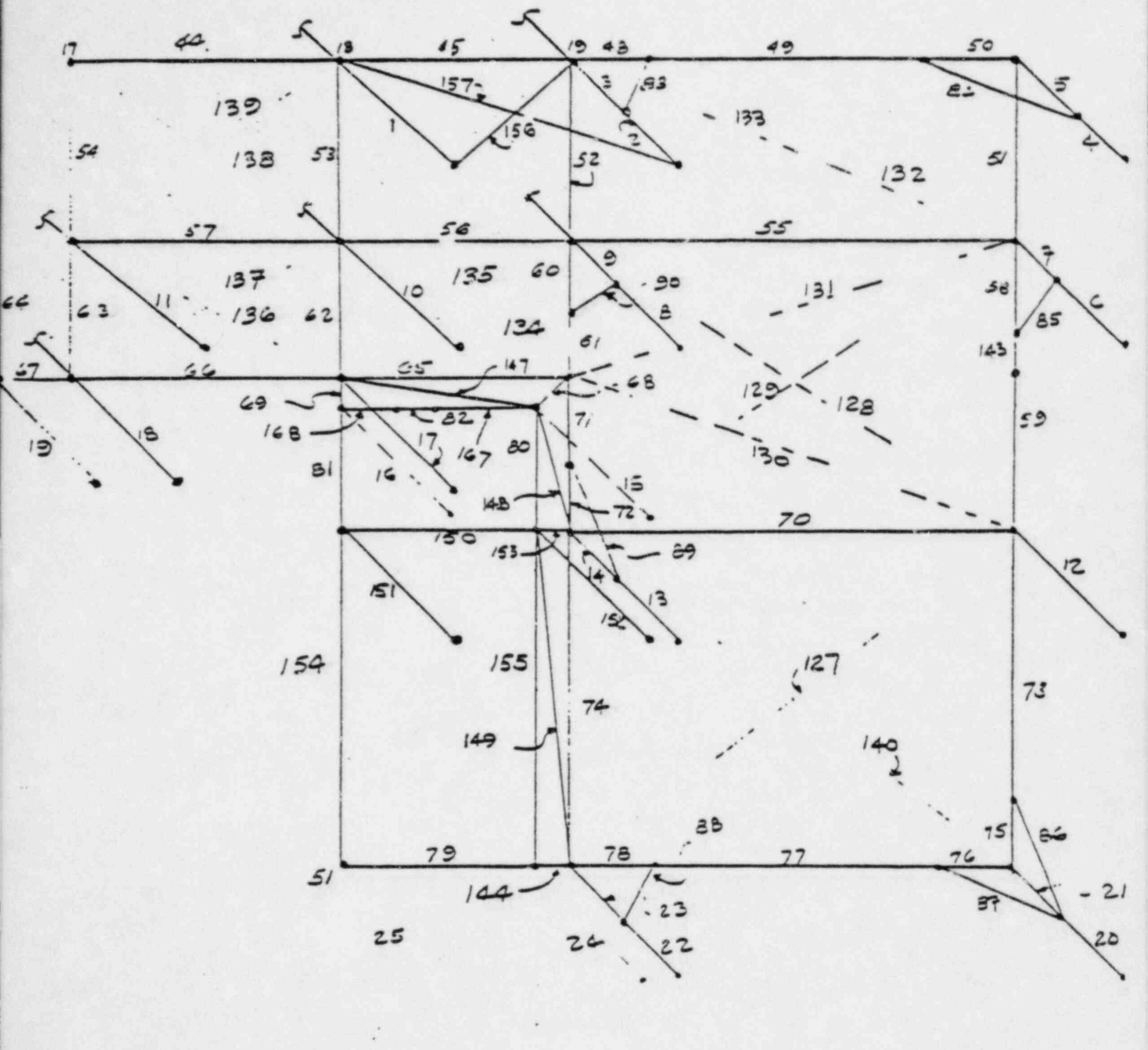


Figure 21. Diesel generator annex nodal points. (Sheet 1 of 2.)

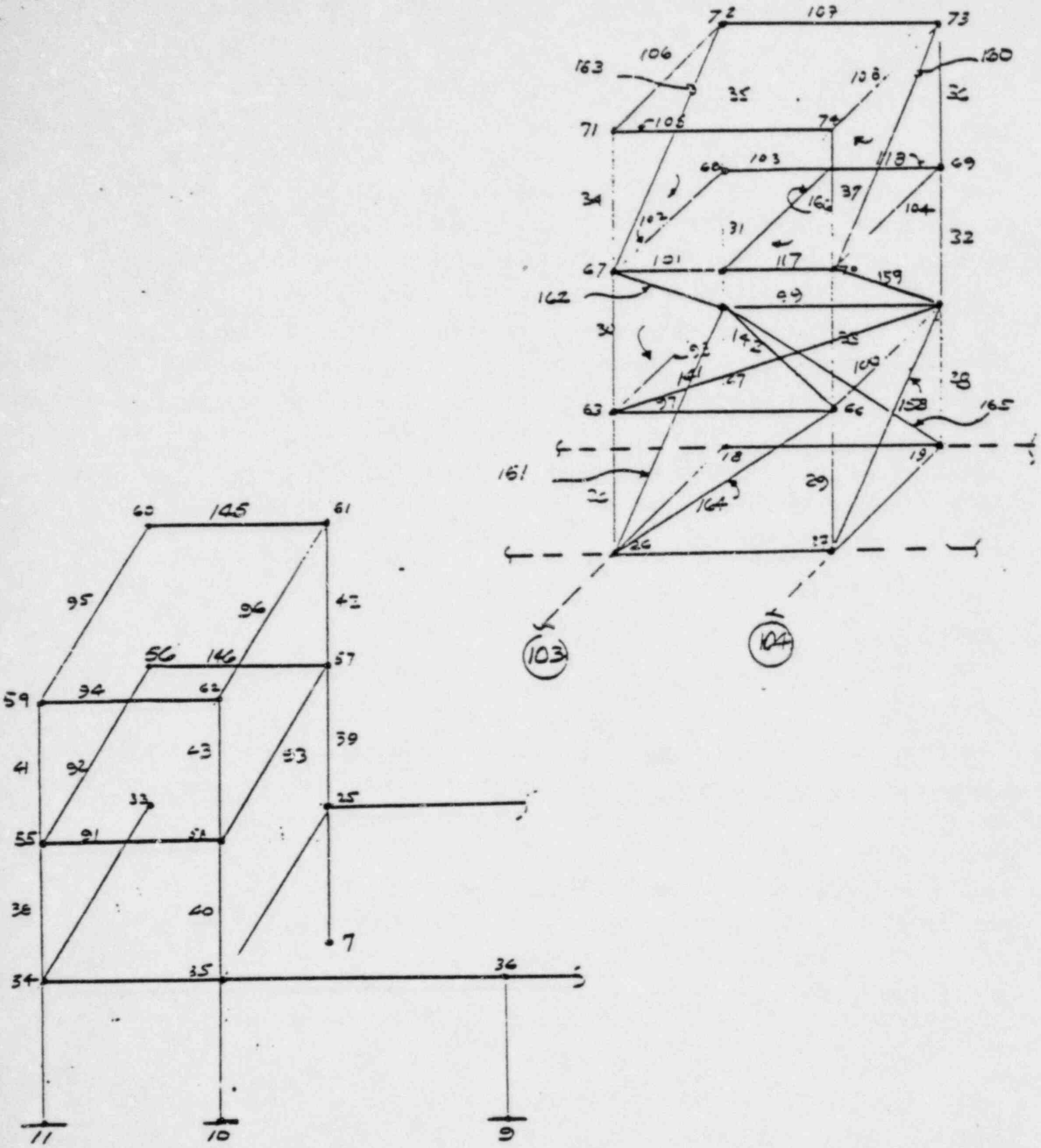


Figure 21. Diesel generator annex nodal points. (Sheet 2 of 2.)

4.6.2 Seismic Re-Evaluation of Steel Frame Structure

The steel-frame structure, which supports the MS/FW lines, is considered a part of the MS/FW system and will be included in the model for the piping evaluation. The MS/FW support structure is accounted for in the analysis as a series of discrete flexible supports at which the tributary mass of the support structure is lumped. The reactor building is modeled as a single-degree-of-freedom system whose stiffness is calculated by putting a unit force at the piping support point and calculating the resulting deflection. The mass is then computed to match the first mode frequency of the building. It is presumed that a similar procedure is used for the turbine building support point. CYGNA will apply a set of three-component artificial earthquake ground-motion time histories which match the NRC spectrum in accordance with the guidelines in SRP Section 3.7.2.

4.6.3 Review and Discussion

The licensee has not completed analyzing the MS/FW support structure, thus no evaluation is given in this report. However, one concern about the proposed approach is the problem of neglecting SSI. In the licensee's proposed approach, both the concrete reactor support structure model and the turbine-building model have fixed bases and the SSI effect is neglected. In addition, it is not clear how the loads induced by the piping and the inertial forces from the support structure itself will be applied to the support structure analysis. The SDOF models for the adjacent buildings do not account for higher mode responses. The effects of neglecting higher modes are not known.

4.7 SPENT-FUEL POOL AND SPENT-FUEL CHUTE

4.7.1 Description of Structure

The spent-fuel pool is a pool-type reinforced concrete structure. The spent-fuel chute is an inclined reinforced concrete channel, connecting the reactor support structure to the spent-fuel pool.

4.7.2 Seismic Re-Evaluation of Spent-Fuel Pool and Spent-Fuel Chute

The pool and the chute were assumed to be uncoupled. No soil-structure interaction was considered in the analyses. The pool was analyzed using a three-dimensional finite element model. The loading conditions included in the analyses were seismic load (including sloshing effect), soil pressure, hydrostatic pressure and thermal load. The dynamic soil effect was represented by the equivalent static soil pressure. (The seismic input for the pool was the 0.2-g R.G 1.60 spectrum with 7% critical damping.) The chute was modeled by a stick model (Figure 22) and was analyzed for seismic and dead loads. The seismic inputs for the chute included the Yankee composite spectrum with peak ground acceleration of 0.10 g and the NRC spectrum of 0.19-g peak ground acceleration. Critical damping of 7% was assumed.

The SRSS method was used for combining model responses and responses to ground motion components in the analysis of the spent fuel chute.

4.7.3 Review and Discussion

From the analysis, it was concluded that the seismic load was small compared with the static soil pressure. The structures met the SEP criteria for the 0.2-g R.G. 1.60 spectra. They were therefore considered acceptable for the less severe Yankee composite or NRC site-specific spectra. It was concluded that the spent fuel chute met the criteria requirements of both NRC and Yankee composite spectra.

The effect on the spent-fuel chute due to relative motion of the concrete reactor support structure and the spent-fuel pool was considered and was found to be small.

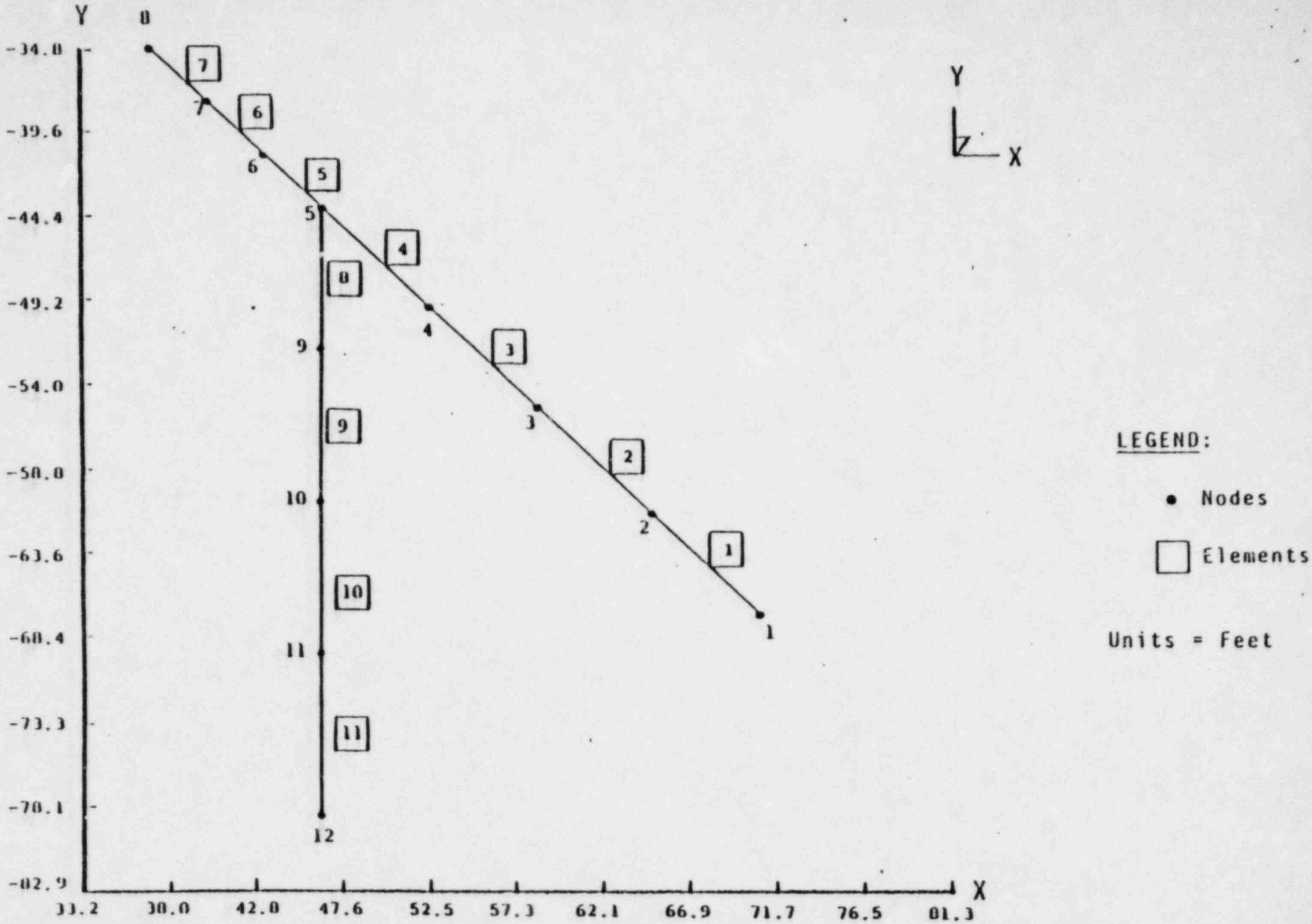


Figure 22. Stick model for the spent fuel chute spectral analysis.

4.8 FIELD-ERECTED TANKS AND BURIED PIPING OR TUNNELS

The licensee stated that there are no safety related buried piping or tunnels at Yankee Nuclear Power Station and the fire water tank is the only safety related field-erected tank. The fire water tank is a circular cylindrical steel structure tied down by 24 anchor bolts to a circular ring footing. The tank was originally designed to the standard of the American Petroleum Institute, and certified by Geotechnical Engineers, Inc. for 0.24-g R.G. 1.60 spectra.

CYGNA's preliminary analysis of the fire water tank indicates that the natural frequency is around 3.8 Hz. The preliminary results also indicate that the roof of the fire water tank is capable of withstanding the sloshing effect of water without buckling or failure.

The final analysis of this tank is not yet completed. It is recommended that the final results be reviewed as soon as they become available.

4.9 MASONRY WALLS

4.9.1 Description of Structures

Among the reinforced and unreinforced masonry block walls at Yankee Power Station, there was an estimated 2640 linear ft of masonry walls which were classified as critical for hot shutdown. The seismic performance of the critical walls would affect hot shutdown components or systems. The critical walls are located in the reactor support structure, primary auxiliary building (including the non-radioactive tunnel), turbine building, and diesel-generator building. Typical wall sections are shown in Figures 23 through 25.

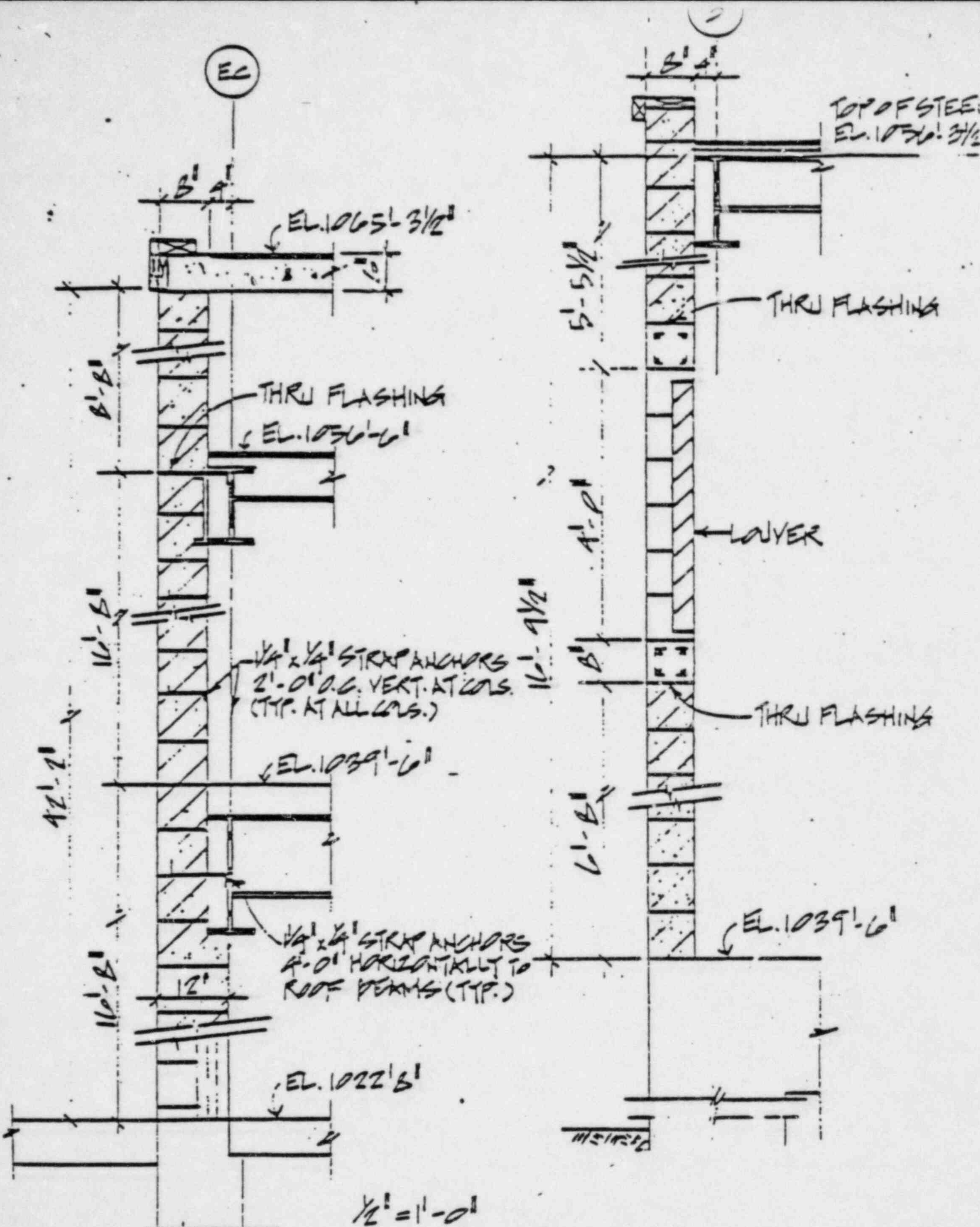


Figure 23. Typical wall sections (existing) primary auxiliary building.

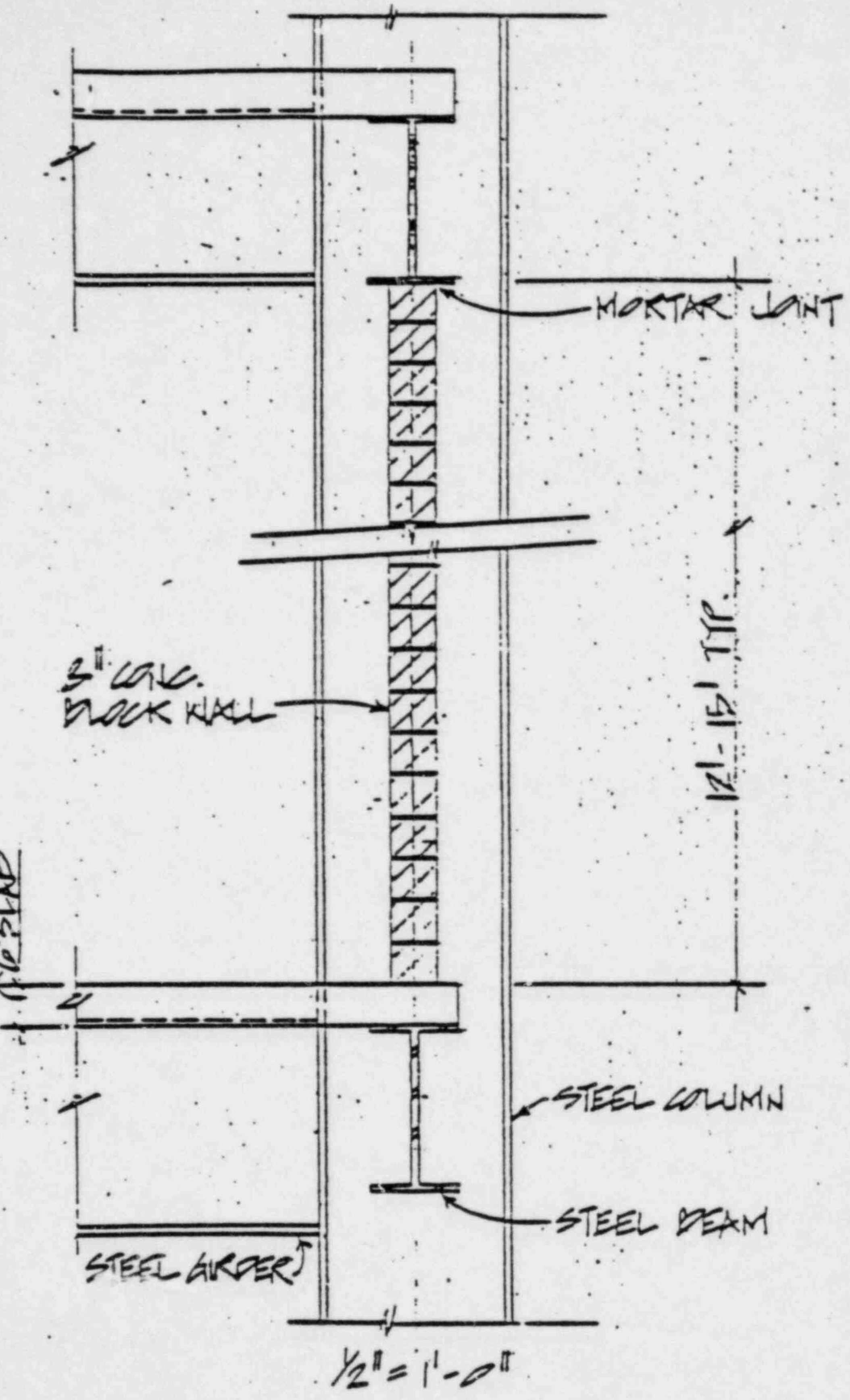


Figure 24. Typical wall section (existing) turbine building.

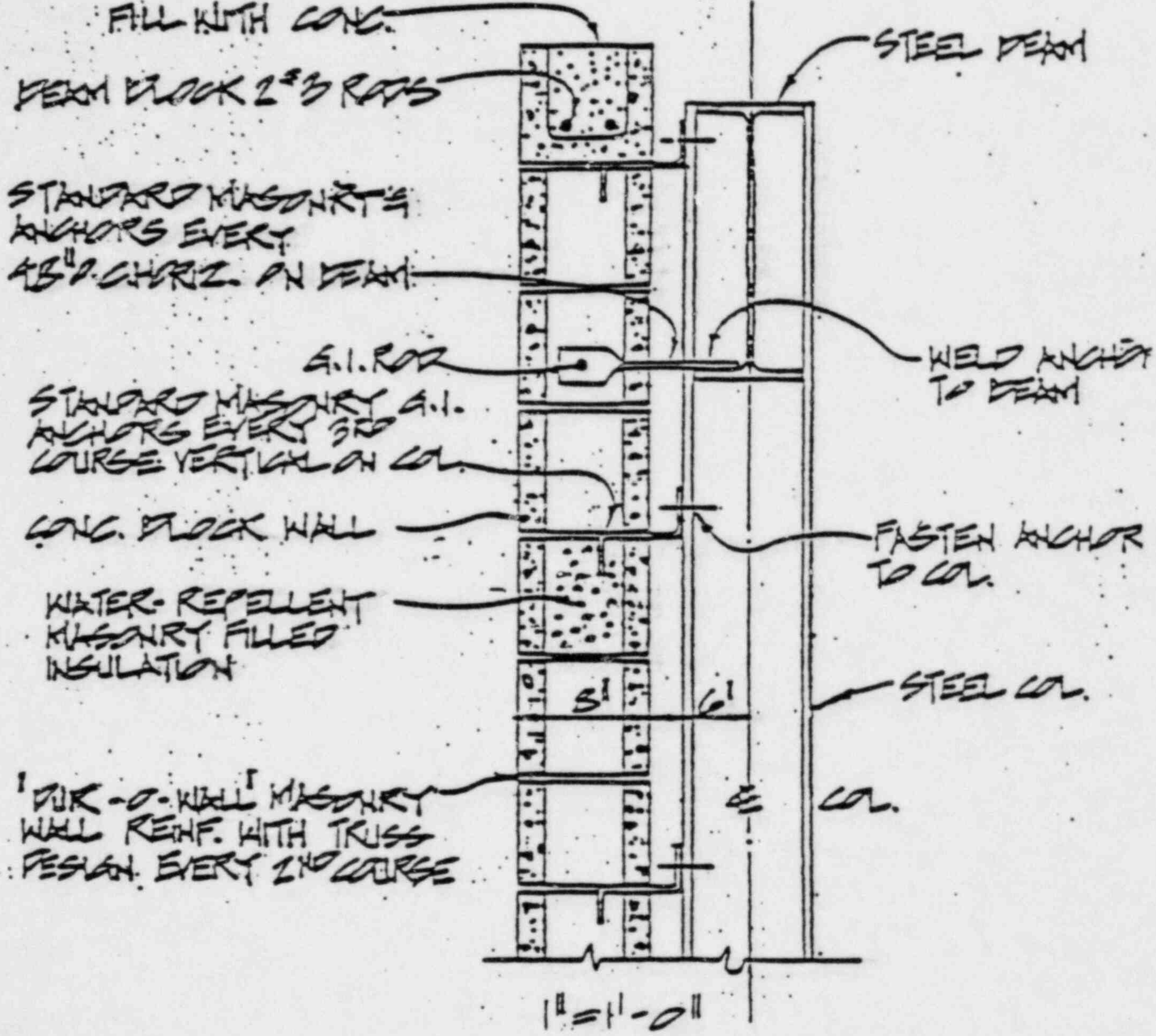


Figure 25. Typical exterior wall detail (existing). Diesel Generator Building.

4.9.2 Seismic Re-Evaluation of Masonry Walls

Linear elastic finite-element analyses were performed for all critical walls. The walls were modeled by plate elements using uncracked section properties in the two major axes. Support conditions for all walls were considered pinned on a minimum of two sides unless otherwise noted in field survey information. Damping values were 5% and 7% of critical damping associated with Yankee composite and NRC spectra, respectively.

In addition to the two seismic load cases, wind loads based on the Uniform Building Code and depressurization loads were also used to evaluate the maximum moments of masonry walls. Table 5 gives the acceptance criteria for the evaluation.

4.9.3 Review and Discussion

An estimated 2157 ft of the critical masonry walls require modification. Table 6 shows the locations of those walls and a preliminary set of solutions.

For evaluating masonry walls, SRP 3.8.4, Appendix A, does not allow tensile stress normal to the bed joint for reinforced masonry, and allows 1.3 times of working stress for unreinforced masonry if the allowable value can be justified by the test program. For shear carried by masonry, 1.3 times of UBC allowable stress is allowed by SRP. These allowable values given by SRP and UBC for masonry without special inspection or test program are lower than the acceptance criteria given in Table 5. Justification for the higher allowables is therefore required for the final evaluation of the modifications.

Table 5. Acceptance criteria for masonry wall evaluation.

ACCEPTANCE CRITERIA

FLEXURE

THE MAXIMUM MOMENT SHALL BE CALCULATED FOR THE FOLLOWING LOAD CASES:

- YCS
- LLL/TERA
- WIND LOAD

THE MOMENT RATIO $MR = \frac{\text{MAXIMUM MOMENT}}{\text{CRACKING MOMENT}}$ SHALL NOT EXCEED THE FOLLOWING VALUES:

FOR LOAD CASE 1: $MR < 0.40$

FOR LOAD CASE 2: $MR < 0.80$

FOR LOAD CASE 3: $MR < 0.80$

THE CRACKING TENSILE STRESS NORMAL TO THE BED JOINT SHALL BE TAKEN AS $2\sqrt{m_o} = 55$ PSI.

IN-PLANE SHEAR

THE ALLOWABLE SHEAR STRESS SHALL BE TAKEN AS $1.35\sqrt{f'_m} = 43$ PSI.

INTERSTORY DISPLACEMENTS

THE TOTAL RELATIVE DISPLACEMENT SHALL NOT EXCEED 0.1% OF THE HEIGHT OF THE WALL.

Table 6. Summary of preliminary selection of solutions.

Solutions \ Building	Primary Auxiliary Building		Turbine Building		Diesel Generator Building	
	# of Walls	Linear Feet	# of Walls	Linear Feet	# of Walls	Linear Feet
No Fix Non-Critical	5	84	23	400	0	0
No Fix Critical Wall OK	0	0	4	93	0	0
Relocate Equipment	1	16	3	41	0	0
Shield Equipment	0	0	2	38	0	0
Replace Wall With Reinforced Masonry	5	75	2	27	0	0
Replace Wall With Metal Siding	6	79	0	0	0	0
Construct Parallel Reinforced Concrete Wall	3	58	9	228	0	0
Tie-Back Bracing With Braced Frame In-Plane	0	0	5	101	0	0
Vertical Ribs With Wall Insolation In-Plane	3	52	15	270	17	302
Vertical Ribs With Braced Frame In-Plane	14	273	13	356	0	0
TOTAL	37	637	76	1653	17	302

5. SUMMARY AND CONCLUSIONS

The licensee and its consultant, CYGNA, have completed the seismic re-evaluation of Yankee Nuclear Power Station structures. However, the documentation of their seismic re-evaluation cannot be completed in time for NRC review. Therefore, review meetings were carried out to evaluate the licensee's re-evaluation results.

It is important to point out that the evaluations presented in this report are based mainly on presentations by CYGNA and discussions by the meeting participants. The presentation material given to the NRC review team at the end of the meeting on August 4, 1982 was considered preliminary.

Three response spectra were used in the Yankee seismic re-evaluation. All safety-related structures were evaluated to at least the seismic level described by the Yankee composite spectrum developed by the licensee. The NRC site-specific spectrum, which envelops the YC spectrum, was used for structures, systems, and components essential for hot shutdown of the plant. An even higher level of seismic input, an R.G. 1.60 spectrum scaled to 0.2-g peak ground acceleration, was used for the evaluation of the spent-fuel pool.

In general, the seismic analysis methodology and acceptance criteria used by the licensee in their seismic re-evaluation of the Yankee structures appear reasonable. A summary of the program plan review is given in Table 1. A list of the conclusions for each structure follows:

SUMMARY AND CONCLUSIONS

1. The analysis of the vapor container is generally acceptable. Although some overstress in three of the rods was predicted, it is believed that there is sufficient strength in other rods to carry the load to preclude failure. There is an open item regarding clevis and turnbuckle stresses. The structural response was verified by an independent study by NCT Engineering. Thus, pending resolution of the clevis and turnbuckle forces, this structure should withstand the earthquake excitation expected for the site as described by the NRC site specific spectrum.

2. The critical elements of the concrete reactor support structure are the connections of the eight supporting columns. The bases of the columns were upgraded for the purpose of continued operation. The calculations for the upgrade have not been reviewed. However, assuming that the upgraded connections can develop the full strength of the column, as stated by the licensee, these connections should be adequate. The capacities of the top-of-column connections are controlled by the bond strength of the dowels. It is believed that there is sufficient capacity to avoid collapse of this structure during the postulated seismic event using the NRC spectrum. However, the actual response of these connections may not have been accurately predicted. Thus, the questions identified herein should be addressed.

3. The Turbine Building was shown to have several overstressed elements. Structural modifications are required. Assuming the modifications are designed and constructed as described by the licensee, this structure should be able to withstand the earthquake characterized by the 0.2-g NRC spectrum.

4. The Primary Auxiliary Building was analyzed for both the 0.1-g Yankee composite spectrum and the 0.2-g NRC spectrum. Some upgrades are required to resist both levels of earthquake. Since this structure apparently houses equipment necessary for hot shutdown, these fixes should be designed to resist postulated loads induced by the 0.2-g NRC spectrum. Based on discussions with the licensee, the suggested upgrades should make the structure adequate to resist the postulated 0.2-g earthquake.

5. The Diesel Generator Building and Annex, similarly, require fixes to resist both levels of earthquake. As for the Turbine Building, this structure should be adequate if the upgrades identified by the NRC spectrum analysis are implemented.

6. The main steam/feedwater support structure remains an open item.

7. The spent-fuel pool and spent-fuel chute was analyzed for a 0.2-g RG-1.60 spectrum. Since they met SEP criteria for this earthquake level, they are considered adequate for the NRC site-specific spectrum.

8. The fire water tank analysis is not complete and is, therefore, an open item.

9. Many masonry walls do not meet acceptance criteria. In some cases, upgrades or replacements are proposed by the licensee and in other cases relocation of equipment are proposed. The fixes proposed for the identified walls appear acceptable. However, the criteria used to identify the critical walls do not meet the SRP criteria. Justification should be provided for the higher allowables used by the licensee.

Several questions and comments need further clarification and discussion:

1. In response to the NRC question regarding soil structure interaction (SSI), CYGNA studied single-degree-of-freedom systems simulating the reactor support structure with and without SSI. A question arose regarding the artificial time histories used in the study. The absolute horizontal structure response time histories show that the strong motion response occurs during the last 2 seconds of an 8-second duration. This indicates that the artificial ground motion time history used might not be long enough to yield conservative in-structure response spectra. Therefore, the in-structure response spectra comparisons may not be valid. Thus, the SSI question is not yet resolved for this structure. Justification should also be provided for using this time history in nonlinear analyses and floor spectra generation.

2. It is necessary to clarify the steel grade and the appropriate stress-strain curve for the No. 14 bar dowels used in the connection between the reactor support structure and its support columns. Values of Fy = 40 ksi and Fy = 50 ksi have been used on different

occasions. What effect does the steel yield strength and the shape of the stress-strain diagram have on the hysteretic behavior of the joint?

3. The size and type of studs used in the reactor support columns must be identified and the load transfer from the steel shell through the studs and into the concrete and the dowel bars must be analyzed. In particular, consequences of semi-ductile or brittle behavior on the local load transfer must be evaluated.

4. What is the maximum bond stress which can be reasonably expected in the connection? The value of 1000 psi used in the CYGNA analysis appears to be somewhat high. Justifications of the appropriate maximum bond stress value requires further documentation of the geometry of the surface deformations in the actual dowel bars. This information would then allow a better estimate of bond strength under monotonic loads.

   In arriving at bond strength, credit has been taken for effects of aging on the concrete compressive strength. However, concrete compressive strength at the top of a column is likely to be below average because of the normal amount of bleeding and dilution of paste.

   Below the construction joint, circular ties are spaced at 12 in. and the dowel bars in the interior columns are too closely spaced. Local cracking around the stud bolts may further reduce the ultimate bond strength.

   Degradation in bond strength due to cyclic loading, particularly when maximum tension stress in the reinforcement reaches yield strength, must also be taken into account.

On the positive side, there is the confinement effect of the steel shell on the bond strength and on the overall capacity of the connection. Under these conditions, and in the absence of directly applicable test results, it is not possible to predict reliably the value of maximum bond strength. However, it would be prudent to evaluate the effect of a lower bound bond stress value on the behavior of the structure.

5. An anchorage length of 42 in. is assumed in developing a bilinear force-displacement curve for the No. 14 bar dowels. This anchorage length is based on the arrangement of steel in the interior columns. Some bars in the exterior columns have an embedment length of only about 24 in. The following questions must be resolved: Can the full yield strength of a No. 14 bar be developed in a 24-in. anchorage length and with reduced bond stress, and what effect, if any, does this condition have on the overall behavior of the structure?

6. Values of 7% and 10% of damping used in the seismic analyses of the reactor support structure appear to be excessive, in light of low stresses throughout most of the structure and confinement of concrete columns provided by the steel shells. Verification of the concept that foundation lift-off and associated behavior may justify the higher damping values requires a detailed review of foundation behavior and of its effect on the general behavior of the structure during a seismic event.

7. An allowable compressive stress of 0.95 Fcr may not be conservative. Further justification is required for members which approach this limit.

8. Proposed modifications need to be implemented.

9. Only preliminary assessment on the fire water tank was done. Final seismic evaluation needs to be reviewed as soon as it becomes available.

10. The MS/FW support frame structure assessment was not completed. Final results need to be reviewed.

11. Evaluation of the steel column base anchorage needs to be clarified. The sample calculation provided is not clear.

12. The calculation for the clevis evaluation shows a criterion of 0.95 times the ultimate load. This is 4.75 S whereas SRP section 3.8 allows only 1.6 S for SSE loading. Further justification is required.

REFERENCES

1. Letter from D. M. Crutchfield of U.S. NRC to J. A. Kay of Yankee Atomic Electric Company dated August 4, 1980.

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3. "Preliminary Seismic Evaluations, Concrete Reactor Support Structure for Yankee Nuclear Power Station", Earthquake Engineering Systems, Inc. for Yankee Atomic Electric Company, June 1979.

4. "Seismic Analysis & Stress Report for the Steel Vapor Container Structure, Yankee Nuclear Power Station", Earthquake Engineering Systems, Inc. for Yankee Atomic Electric Company, March 2, 1979.

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8. Structural presentation, Yankee Rowe Power Station, to NRC review team, by CYGNA Energy Services on April 5 and 6, 1982 (viewgraphs).

9. Reactor support structure presentation, Yankee Rowe Power Station, to NRC review team, by CYGNA Energy Services on May 25 and 26, 1982 (viewgraphs).

10. Structural presentation, Yankee Rowe Power Station, to NRC review team, by CYGNA Energy Services on May 25 and 26, 1982 (viewgraphs)

11. NRC structural presentation, Yankee Rowe Power Station, Rev. 0, Volume 1, August 3 and 4, 1982 (viewgraphs).

12. NRC structural presentation, Yankee Rowe Power Station, Rev. 0, Volume 2, August 3 and 4, 1982 (viewgraphs).

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14. "Review of Concrete Column Joints Supporting Concrete Containment Structure, Yankee Nuclear Power Station," Wiss, Janney, Elstner and Associates, Inc., prepared for Lawrence Livermore National Laboratory, August 10, 1982.

15. "RCCOLA - A Computer Program for Reinforced Concrete Column Analysis - User's Manual and Documentation," Department of Civil Engineering, University of California, Berkeley, August 1977.

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17. "Preliminary Seismic Evaluation Concrete Reactor Support Structure for Yankee Nuclear Power Station," E-Y-YR-80064, Earthquake Engineering Systems, Inc., August 10, 1979, Rev. 1.

APPENDIX A

TRIP REPORT FOR SEP SEISMIC REVIEW MEETINGS OF YANKEE NUCLEAR POWER STATION

MEETING 1

Date: April 5 and 6, 1982

Location: CYGNA Office, San Francisco

Attendees:

YAEC

S. L. Chin
T. M. Cizauskas
J. R. Hoffman
A. Kadak

NRC

R. Caruso
T. Cheng
R. Herman

CYGNA

M. Deguzman
B. Falciani
B. Kacyra
E. V. Stijgeren
J. Vallenas
T. Y. Wang

LLNL

T. Y. Lo
T. A. Nelson

EG&G/SRO

C. Y. Liaw

NCT

T. Tsai

TRIP REPORT

A.1 INTRODUCTORY REMARKS, YANKEE ATOMIC ELECTRIC COMPANY (YAEC)

Kadak of Yankee Atomic Electric Company (YAEC) made an introductory presentation concerning the historical perspective, current status and activities of SEP, and the physical arrangement of the Yankee Nuclear Power Station. The important points included:

a. The Yankee plant was built in 1956 and started operation in 1960.

b. The continuing operation of the Yankee plant was based on a 0.1-g interim design basis spectrum.

c. The bases of the exterior columns of the reactor support structure have been upgraded.

d. Currently, there is no agreement between NRC and YAEC about which earthquake-response spectra should be used for the Yankee site. The YAEC re-evaluation was based on 0.1-g Yankee composite spectra. The spectra suggested by NRC have generally higher spectral values over all the frequencies and have a peak ground acceleration of 0.2 g.

e. The piping and structure analyses of the Yankee plant are expected to be completed by June 30, 1982.

f. YAEC will perform the integrity assessment of the plant in the next few months.

A.2 TECHNICAL PRESENTATION, CYGNA, INC.

CYGNA, the consultant for YAEC, made the technical presentation of the approach and methodology used. This presentation can be summarized as follows:

a. The basic evaluation criteria are based on the regulatory guides and the SRP. The design-allowable-stress (0.95 yield) approach was used for the results of the Yankee composite-spectra analysis.

b. The alternate criteria for evaluation include the considerations of operability of critical systems and the ductility or inelastic responses of structural systems.

c. The following load combinations were included: dead, live, thermal, pressure, earthquake, and wind.

d. The computer programs used for the analysis were ANSYS, BATS, EESAP, and SIMQUAKE.

A.3 DISCUSSION

A.3.1 Reactor Support Structure

a. The effects of soil structure interaction were ignored in the analysis.

b. The upper connections of the concrete columns were evaluated in detail. The pull-out action (or bond-slip relation) of dowels was studied for monotonic and cyclic loads. CYGNA felt that the connections can sustain not only the Yankee composite spectra, but also the NRC spectra.

A.3.2 Vapor-Containment Structure

a. The bracing members were modeled as half-area members to account for the buckling behavior of the members under compression.

b. A high-stress concentration was observed in the mathematic model around the equipment hatch connection. Further refinement of the model in this area is required.

c. No significant stress was observed in the area around the column (shell connections).

d. The buckling criteria for the shell were based on 95% of the theoretical critical buckling loads.

A.3.3 Primary Auxiliary Building

a. No soil-structure interaction was included.

b. The predicted stresses were generally low, except at the steel-beam-to-concrete wall connections.

c. This building has some safety-related equipment.

A.3.4 Diesel-Generator Building

a. There are some block walls in the building which are neither reinforced nor grouted.

b. Some columns and K-braces were predicted to be overstressed. The new annex structure being added may help reduce some of the high stresses.

A.3.5 Turbine Building

a. The control room which houses critical equipment is connected to the turbine building. There are block walls in the control room. The turbine building itself is basically a steel frame structure.

b. The analytical results were selected to envelop those from mathematic models with and without the block walls included.

c. The turbine pedestal is separated from the structure.

d. The frame and bracing systems were predicted to have overstress.

A.3.6 Main Steam/Feedwater Structure

The analysis of this structure is still in progress.

A.3.7 Spent-Fuel Chute and Pool

The pool structure was evaluated for 0.2-g R.G. 1.60 spectra. The spent-fuel chute was evaluated for the Yankee composite spectra. The seismic stresses were small and within the allowable stresses.

A.4 SUMMARY OF PRESENTATION

The YAEC/CYGNA indicated their intent to upgrade the plant to have elastic responses to a seismic event of the level of the Yankee composite spectrum, and to allow inelastic responses in structural components for the NRC spectra.

A.5 PRELIMINARY EVALUATION OF THE NRC STAFF

Analysis techniques used in conducting elastic analysis with the Yankee composite-response spectra appear reasonable except for the following areas which need further clarification and/or justification:

a. The effects of soil-structure interaction on structure response, especially on in-structure response criteria. Particular attention should be addressed to:

   (1) Primary auxiliary building
   (2) Control-room/turbine building complex
   (3) Spent-fuel chute and pool.

   In addition, provide justification for ignoring SSI on the reactor support structure.

b. Buckling Criteria

(1) Justify use of 95% of calculated buckling stress (load) for straight members.

(2) Justify shell buckling criteria. Identify critical locations and provide stress summaries. Include formulae and/or references which form the basis for your criteria.

(3) Explain how the effects of imperfections were considered.

c. Ensure connection details are evaluated for all structures. Consideration should include the following:

(1) AISC criteria for steel connections.

(2) Provide detailed criteria for reactor support building upper-column connections.

(3) Consideration of net section at bolted connections and in threaded rods.

(4) Anchorage of column bases.

(5) Describe the dynamic behavior of the reinforced concrete infill wall in the turbine building. Clarify the shear transfer mechanism.

d. Provide a summary of the analysis results presented during the April 5, 1982 meeting.

APPENDIX B

TRIP REPORT FOR SEP SEISMIC REVIEW MEETINGS OF YANKEE NUCLEAR POWER STATION

MEETING 2

Date: May 25 and 26, 1982

Location: CYGNA Office, San Francisco

Attendees:

YAEC

J. R. Hoffman
A. C. Kadak
A. V. Roudenko

CYGNA

M. Bilginatalay
H. Condreras
B. Falciani
M. de Guzman
P. Joadder
B. Kacyra
J. C. Minichiello
E. V. Stijgeren
J. Vallenas
T. Y. Wang

U.C. Berkeley

S. A. Mahin

NRC

T. Cheng
R. Hermann

LLNL

T. Y. Lo
T. A. Nelson

EG&G/SRO

C. Y. Liaw

EG&G/Idaho

T. L. Bridges
S. L. Busch
T. Thompson

TRIP REPORT

On the first day of the meeting (May 25, 1982), YAEC and CYGNA discussed the hot-shutdown system and the piping analysis of the Yankee plant. The following is a summary of the second meeting (May 26, 1982), when most discussions related to the structural systems.

CYGNA had performed additional structural analyses since the April 5 and 6, 1982 meeting. The additional work included evaluations of structures to the spectra suggested by NRC, further evaluations of critical structures, and time-history analysis of structures to provide input motions for the multiple-support-excitations analysis of piping systems.

B.1 HAZARD LEVEL OF SEISMIC INPUTS

It is the conclusion of YAEC that the hazard levels for the interim design spectra, the Yankee composite spectra, and the NRC site-specific spectra are 10^{-3}, 4.8×10^{-4}, and 10^{-4} per year, respectively.

B.2 UPGRADED STRUCTURES

YAEC and CYGNA indicated that the turbine bulding, primary auxiliary building, diesel-generator bulding, and block walls will be upgraded to meet the criteria of the Yankee composite spectra. They also felt that the upgraded structures can also meet the criteria of NRC site-specific spectra.

B.3 VAPOR CONTAINER

The container structure was analyzed again with a refined finite element mesh around the equipment hatch support. High-stress concentration was not found, as shown in the previous analysis.

Detailed evaluation of the bracing rods showed some overstress at the root of threads. The bracing system has to rely on its ductility.

The critical buckling pressure used for deriving allowable compressive shell stress was based on the following equation:

$$P_{cr} = CE \left(\frac{t}{R}\right)^2, \quad C = 0.15$$

B.4 REACTOR-SUPPORT STRUCTURE

In an earlier study[17], linear analysis was performed for the reactor support structure. Preliminary results indicated that the reinforcing bars at the tops of the columns would develop their yield strength and that once their yield strengths were reached, the embedment bond strength deteriorated under additional cyclic loadings.

Nonlinear analyses were used to study the cyclic behavior of the reinforced concrete connections under seismic excitations.

Artificial time-history ground motions with a duration of 10 seconds were used. The structure was simplified to a two-dimensional system. The structure was assumed to have 7% damping. The nonlinear behavior of the concrete joint was approximated by a bilinear moment rotation relationship. The results showed that the interior columns were under the most severe loads, the extreme bars just reached the strain hardening level. The rotations at the connection were on the order of 2.8×10^{-4} rad.

The same nonlinear model was also analyzed for the El Centro earthquake. No exterior column reached yield in this case. No strain hardening in rebars occurred.

The bond-slip relation was based on the study in Appendix B of Reference 8. A 1.0-ksi uniform distribution was used in the study.

B.5 STRUCTURAL EVALUATION QUESTIONS OF NRC STAFF

a. Evaluate the effect of nonlinear response of the reactor support structure on in-structure response spectra.

b. Justify accounting for biaxial bending of the reactor support structure columns by multiplying the aniaxial/ moment by 1.1.

c. Provide references cited to support the argument that the column dowels are adequate to allow inelastic structural response at the top of the reactor support structure columns. The staff may also request additional data to perform an independent evaluation of these connections.

APPENDIX C

STATUS OF OPEN ITEMS RAISED AT REVIEW MEETINGS

Meeting 1, April 5 and 6, 1982

a. Studies conducted by CYGNA indicate that soil-structure interaction does not appear to have a significant effect on stresses in the structures. However, the possible effects on in-structure spectra for the reactor support structure have not been adequately addressed.

b.

1. 0.95 Fcr needs further justification.
2. Shell buckling criteria is adequate.
3. 0.95 Fcr does not sufficiently allow for imperfections in straight members.

c. Connection details were presented.

1. Reactor support columns are addressed in the body of this report.
2. Evaluation of steel column base anchorage needs further clarification. The sample calculation provided is not clear.
3. The calculation for the clevis evaluation shows a criteria of 0.95 times the ultimate load. This is 4.75 S, whereas the SRP Section 3.8 allows only 1.6 S for SSE loading. A similar problem will be present for turnbuckles. Further justification is required.
4. The infill wall was adequtely described.

d. A summary was provided.

Meeting 2, May 25 and 26, 1982

a. It was stated by CYGNA that the overall response of the reactor support structure was linear even though some localized inelastic response was predicted. This is acceptable pending the evaluation of responses to questions regarding the nonlinear analysis of the reactor support structure.

b. Biaxial bending was adequately addressed.

c. References were provided addressing the bond strength of dowels. Questions resulting from review of this material are contained in this report.