URS/JAB 8319

URS

SEISMIC STRUCTURAL EVALUATION OF THE MAIN CONTROL BOARD

November, 1983

West Peabody, Massachusetts 01960

SEISMIC STRUCTURAL EVALUATION OF THE MAIN CONTROL BOARD

R.E. GINNA
NUCLEAR POWER STATION

submitted to

Rochester Gas and Electric Corporation
89 East Avenue
Rochester, New York 14649

submitted by

URS/John A. Blume & Associates, Engineers
83 Pine Street
West Peabody, Massachusetts 01960

FORM 4.1-1

URS/BLUME DOCUMENT APPROVAL SHEET

DOCUMENT TITLE: Seismic Structural Evaluation of the Main Control Board

DOCUMENT TYPE: Criteria ☐ Interface ☐ Report ☒ Specification ☐ Other ☐

PROJECT NAME: R.E. Ginna Control Board

JOB NO.: 8319

CLIENT: Rochester Gas and Electric Company

Initial issue (Rev. 0)

Prepared by: [signature] Date: 11/18/83

Reviewed by: R. Martin Czarnecki Date: 11/18/83

Approved by: Dilip P. Kharon Date: 11/22/83

REVISION RECORD:

Revision No.	Prepared By/Date	Reviewed By/Date	Approved By/ Date	Description of Revision

Rev. 9
5/14/82

URS/Blume

CONTENTS

APPENDICES

TABLES

FIGURES

1. INTRODUCTION

URS/John A. Blume & Associates, Engineers, has performed a seismic qualification analysis and evaluation of the Main Control Board (MCB) located at elevation 289'-0" of the control building of the R.E. Ginna Nuclear Power generating facility. The structural integrity of the MCB was evaluated for gravity and seismic loads for the safe-shutdown earthquake (SSE) as part of the Systematic Evaluation Program (SEP) review of the Ginna Facility.

The MCB is located in the southeast corner of the control room at elevation 289'-0" of the Control Building of the Ginna facility. Figure 1 illustrates the orientation and shape of the MCB and defines the nomenclature used to identify various parts of the board. The coordinate system used for the structural analysis, which coincides with the coordinate system used for the seismic analysis of the Control Building, is also illustrated in Figure 1.

A typical cross-section through the panel, illustrated in Figure 2, shows that the MCB is constructed of structural steel plates and shapes. Main structural framing members are 2 1/2 inch by 2 1/2 inch by 1/4 inch structural angles which are used to attach the roof plates to the vertical plates and the roof plates to each other. Typically, these attachments are welded connections consisting of about one inch of fillet welds assumed to be spaced at approximately ten inches on center. The exterior shell of the MCB consists of a 3/16 inch steel plate except for the roof which is a 1/8 inch steel plate. Holes have been cut in the vertical and bench sections to accommodate the instruments and control devices which are mounted on the panel with sheet metal screws and bolts. Rectangular steel stiffeners, 2 inch by 3/8 inch reinforce the plates of the MCB in the vicinity of some of the heaviest instruments forming a lattice of horizontal and vertical stiffeners.

The MCB consists of three individual units; right, center, and left sections, which are attached together by bolted connections. The end panel of the right and left panels have door openings which are reinforced with structural angles.

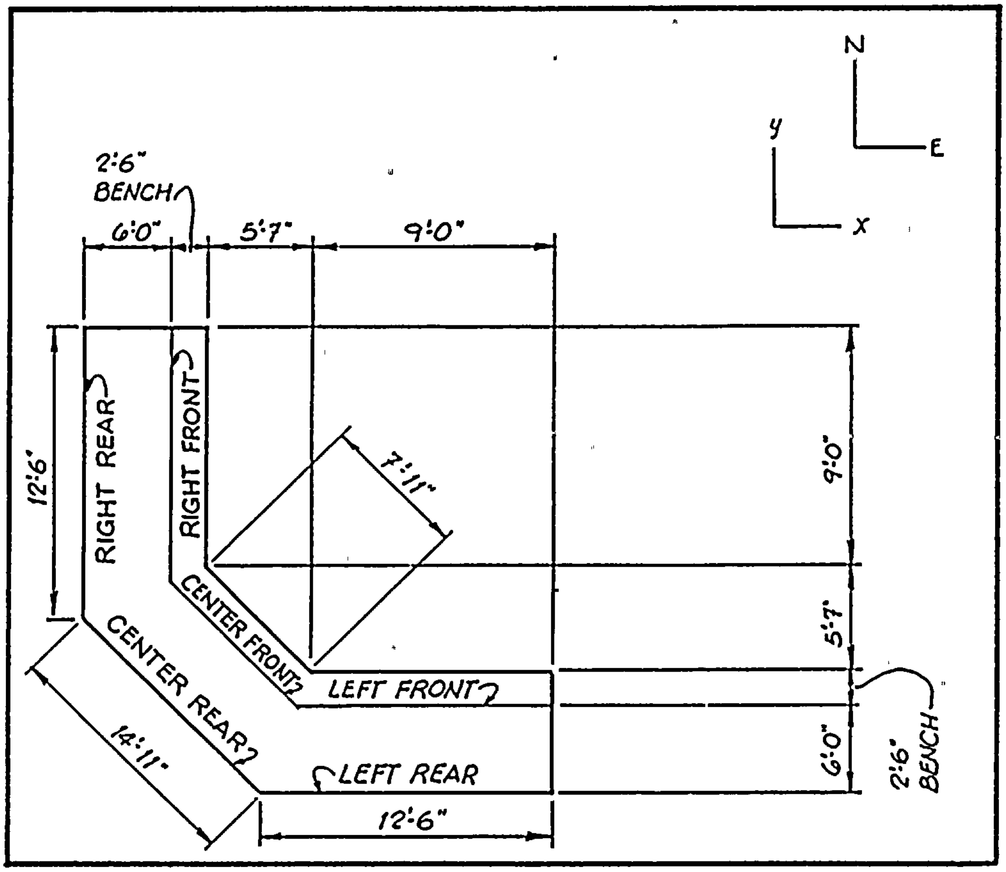


FIGURE 1

MAIN CONTROL BOARD PLAN VIEW

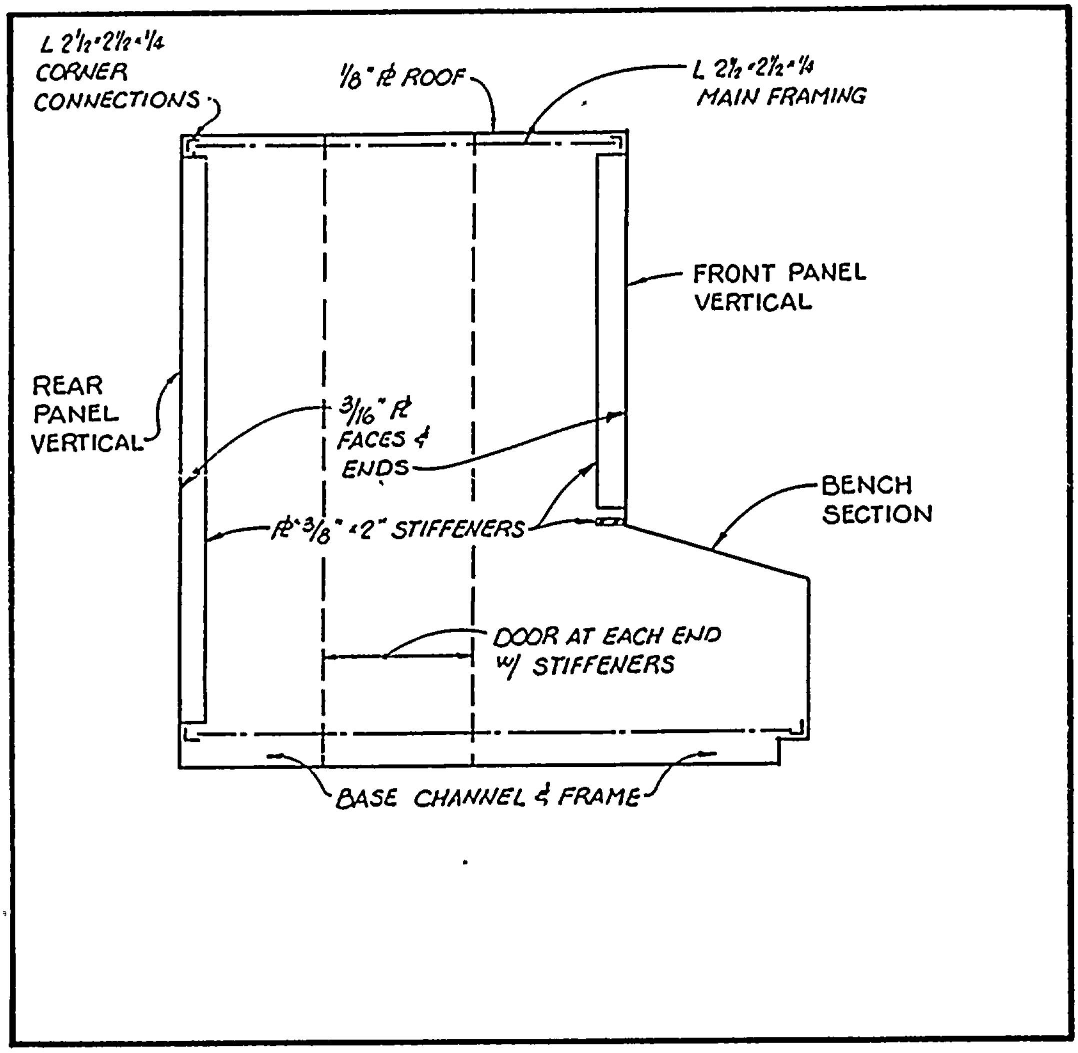


FIGURE 2

TYPICAL CROSS-SECTION OF
MAIN CONTROL BOARD

The evaluation of the MCB was conducted in three phases: in-situ modal testing, data processing, and structural analysis and evaluation.

The in-situ modal testing phase was conducted at the Ginna facility between April 5 and April 15, 1983. A small electromechanical vibration generator was used to dynamically load the MCB and the acceleration response was measured throughout the equipment. Raw test data was recorded on magnetic tapes and analyzed in the subsequent phase of the project.

The data processing phase involved analysis of the test recordings to identify the frequencies and mode shapes of the significant modes of vibration of the MCB.

The structural analysis and evaluation involved calculations of the accelerations, displacements, inertia forces, and stresses in the MCB for the combined effect of gravity and SSE loads. The stresses resulting from these calculations were compared to an acceptance criteria to evaluate the integrity of the structure.

The results of the evaluation indicate that the MCB is expected to survive the SSE with some local yielding of the structural steel cover plates and stiffeners. Yielding is mainly confined to the rear right, center and left vertical panels, and the center front bench panels. Local stiffening of these areas may be desirable to enhance the structural integrity of the MCB.

2. IN-SITU MODAL TESTING

The seismic structural evaluation of the Ginna MCB was performed based on the in-situ modal testing results. The testing was used as an easy means of determining the normal modes of vibrations of the MCB.

The practical significance of normal modes is that the dynamic response of an n degree-of-freedom linear elastic structure may be considered as the combination of the responses of n single degree-of-freedom systems (modal superposition). These single degree-of-freedom systems are the normal modes of vibration, characterized by a natural frequency, mode shape, and damping ratio. The following is a brief description of the test procedure, data processing, and results.

2.1 Testing Procedures

The in-situ modal test of the Ginna MCB involved the measurement and analysis of low-level vibrations that occur when the MCB was dynamically loaded with a small electromechanical shaker. The low-level vibrating force of approximately 30 lbs. peak value and 10 lbs. rms value, was input at a number of locations (driving points) on the structure while the vibration response was measured at a number of other points (measurement points). The driving and measurement points were carefully chosen to identify the natural modes of vibration of the structure.

Prior to the testing, the points of force application (driving points) and measurement grid points were marked on the MCB and recorded for reference purposes. Other pertinent information such as gain and sensitivity of instruments, instrument serial numbers, location, date and type of test was also recorded.

The electromechanical shaker was supported such that the armature could be attached to the MCB. A response accelerometer was placed at the first grid point. The random force was then input into the structure by the shaker covering a frequency range of 0 to approximately 30 hz. The accelerometer was advanced to the next grid location on the structure after the data had been checked and stored. This process was repeated until data had been

collected for the entire grid of the structure. This testing was performed only for the out-of-plane direction of each plate comprising the MCB.

2.2 Data Processing

The dynamic response of a linear elastic structure may be considered as the combination of the responses of the normal modes. By recording and analyzing the dynamic response of a linear elastic structure, it is possible to identify the properties of the contributory modes.

Modal information may be extracted from recorded test data by either time-domain analysis or frequency-domain analysis. The latter (adopting the transfer function method) was used for this work because it is more appropriate and convenient. The transfer function method is based upon the use of digital signal processing techniques and the FFT (Fast Fourier Transform) algorithm to calculate transfer functions between various points on the structure. Transfer functions are obtained from measurements of a random input force and the appropriate response acceleration signal. The function is thus defined as the complex ratio of the acceleration to force in the frequency domain.

The transfer function has characteristics which makes it extremely useful in modal parameter identification. Modal frequencies correspond to peaks in the imaginary part of the transfer function and crossings of the zero axis in the real part of the transfer function. The other modal properties, which are the damping and mode shape, are determined by the least squared method of curve fitting a polynomial of the transfer function for a single mode of vibration to a portion of the recorded transfer function between selected frequency values.

From a single measurement pair (i.e., recorded input force and response time-histories), it is possible to identify the frequency and damping of a mode of vibration. The width of the modal peak is related to the damping of the mode (the wider the peak, the higher the damping). Numerous measurement pairs from various locations on the structure being surveyed are required to calculate the mode shapes. The mode shapes are obtained by assembling the peak values (in the imaginary part of the transfer functions)

from all measurements, and by comparing their relative amplitude at the same frequency.

Analysis of the recorded data was accomplished using a Hewlett-Packard 5423A Structural Dynamics Analyzer. This device has the capability to perform various frequency domain claculations plus store and display various types of data. The analysis begins by inputting various test data such as the geometry of the test specimen, the coordinates of the measurement grid and driving points, and the fourier spectra transfer functions previously calculated from the force and acceleration time-histories. The aim of the analysis is to curve-fit the measured data, and subsequently extract the modal properties.

Mode (or characteristic) shapes are not unique in value, but the components of the vector have a unique relationship with respect to one another. Once generated, mode shapes can be scaled by a number of different methods.

2.3 Summary of Test Results

The results obtained from the modal testing and subsequent data processing of the MCB are given in Appendix A. In general, the test results indicated that the right, center, and left sections respond dynamically as independent units in their respective out-of-plane directions. The front and back panel faces also tend to respond independently. The only nóticeable exception to this general trend is the first mode of the left front panel which occurs at the same frequency as the left and center rear panels.

The fundamental mode frequencies of the panels in the out-of-plane direction range from 6.9 hz to 11.1 hz, a frequency range which corresponds to the range of maximum spectral responses for the Control Building. The center front panel has three modes between 0 hz and 33 hz, and the right rear has eight modes below 22 hz. These two cases represent the extremes in terms of the number of individual modal responses identified in the frequency range of interest.

In-plane modal responses of the panels were not measured during the in-situ tests. However, the instruments cantilevered from the vertical plates

resulted in frequencies in the 8 hz to 15 hz range. An example of this behavior was found on the left rear panel. In this case, the frequency of a cantilevered device in the east-west direction (i.e., in-plane with respect to the panel) coincides with the north-south (out-of-plane) response of the panel. These results indicate the need to use the peak of the horizontal spectra for the evaluation of the cantilevered devices.

A review of the plotted mode shapes reveal that nearly all the modes involve motion in the interior of the panels while the boundaries (top, bottom, connection to an adjacent panel, and connection to the end panel for the right and left sections) remain motionless. The lowest modes generally involve single curvature in the horizontal and vertical direction. The second and third modes of each panel typically involve single curvature in the vertical direction and double or triple curvature in the horizontal direction.

The damping values measured by in-situ modal testing are not considered to be appropriate for a SSE evaluation. However, the measured damping does provide some insight into the expected level of damping for higher motion. Most of the measured damping given in Appendix A is in the range of one to three percent with a few measurements in the range of three to five percent.

3. EVALUATION METHODS AND ACCEPTANCE CRITERIA

The purpose of the structural analysis of the MCB was to evaluate its structural integrity for gravity and seismic loads due to the safe-shutdown earthquake. The general procedure for this evaluation involves the calculation of the combined seismic and gravity stress and comparison to an allowable stress level.

The gravity stresses for this type of equipment which are due to the weight of the MCB structure and the weight of attached instruments and devices are usually quite small compared to the allowable values.

The seismic stresses were calculated using the modal response properties of the MCB determined by testing. A response spectrum analysis was used to calculate the seismic inertial load in each significant mode for three mutually perpendicular directions of earthquake motion. The inertial loads were then used in a static analysis to determine forces, moments and stresses in critical elements of the seismic load path of the MCB. The evaluation was finally performed by comparing predicted gravity plus seismic stresses to an acceptable stress level.

3.1 Seismic Analysis Procedure

The following is a brief description of the methods and procedures used for the seismic analysis of the MCB.

Floor Response Spectra. The seismic input for this analysis was the floor response spectra for the safe shutdown earthquake at elevation 289'-0" of the control building; dated February 1, 1980 (see Figures 3, 4, and 5). The zero-period acceleartion (ZPA) in both horizontal directions is 0.51g and the vertical ZPA is 0.3g. The X-response spectra have been used for the seismic motions in the east-west direction, and Y-response spectra have been used for seismic motions in the north-south direction (see Figure 1).

Damping. For welded steel structures at or just below the yield point, Reference 1 recommends 5 percent to 7 percent damping. The corresponding values for bolted steel structures are 10 percent to 15 percent damping. The structure of the MCB consists of a combination of welded and bolted

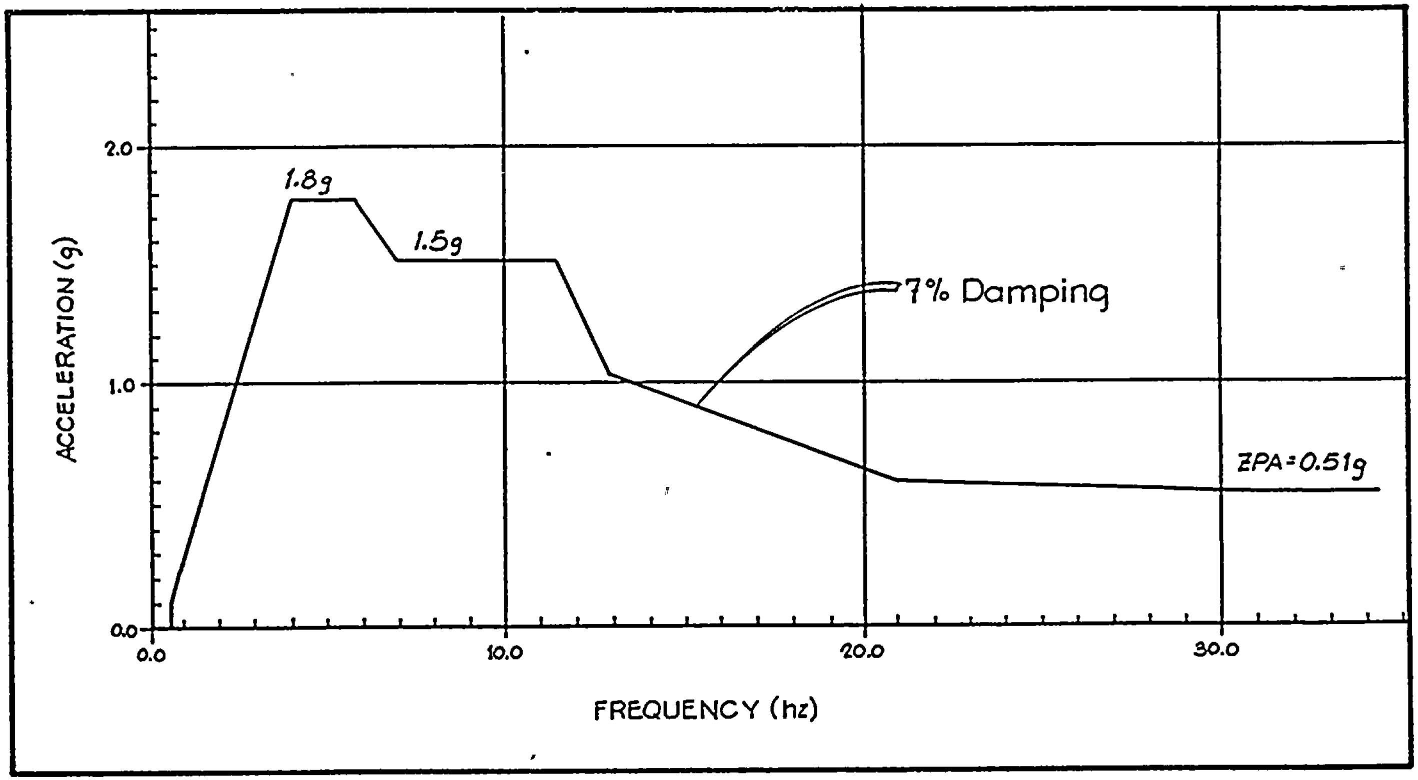


FIGURE 3

GINNA NUCLEAR POWER STATION UNIT NO. 1 - SSE BROAD BAND FLOOR RESPONSE SPECTRA - CONTROL BUILDING

ELEV 289'-0" X-RESPONSE (FIGURE SSE-7 B-X, FEBRUARY 1, 1980)

URS/Blume

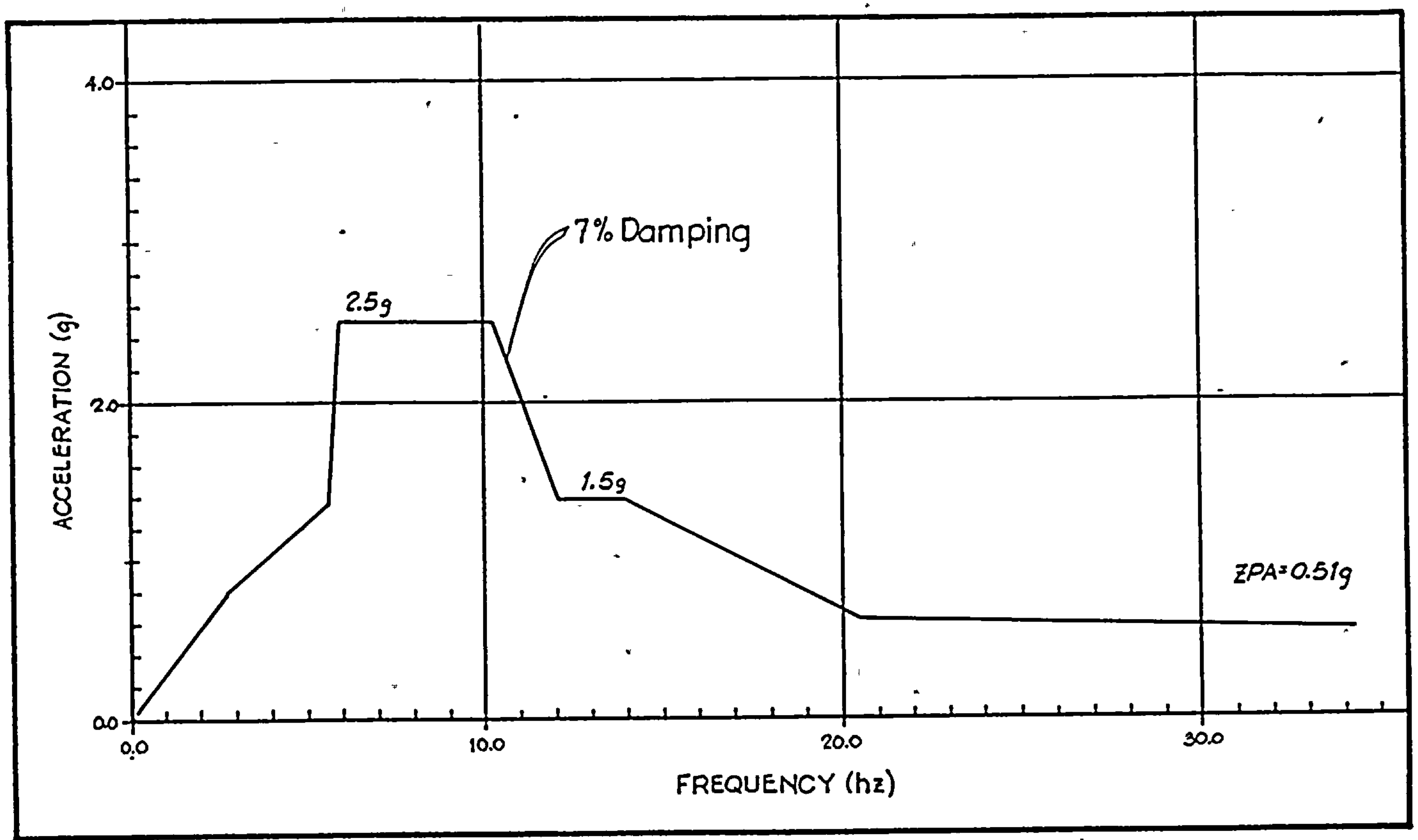


FIGURE 4

GINNA NUCLEAR POWER STATION UNIT NO. 1 - SSE BROAD BAND FLOOR RESPONSE SPECTRA - CONTROL BUILDING

ELEV. 289'-0" Y-RESPONSE (FIGURE SSE-7 B-Y, FEBRUARY 1, 1980)

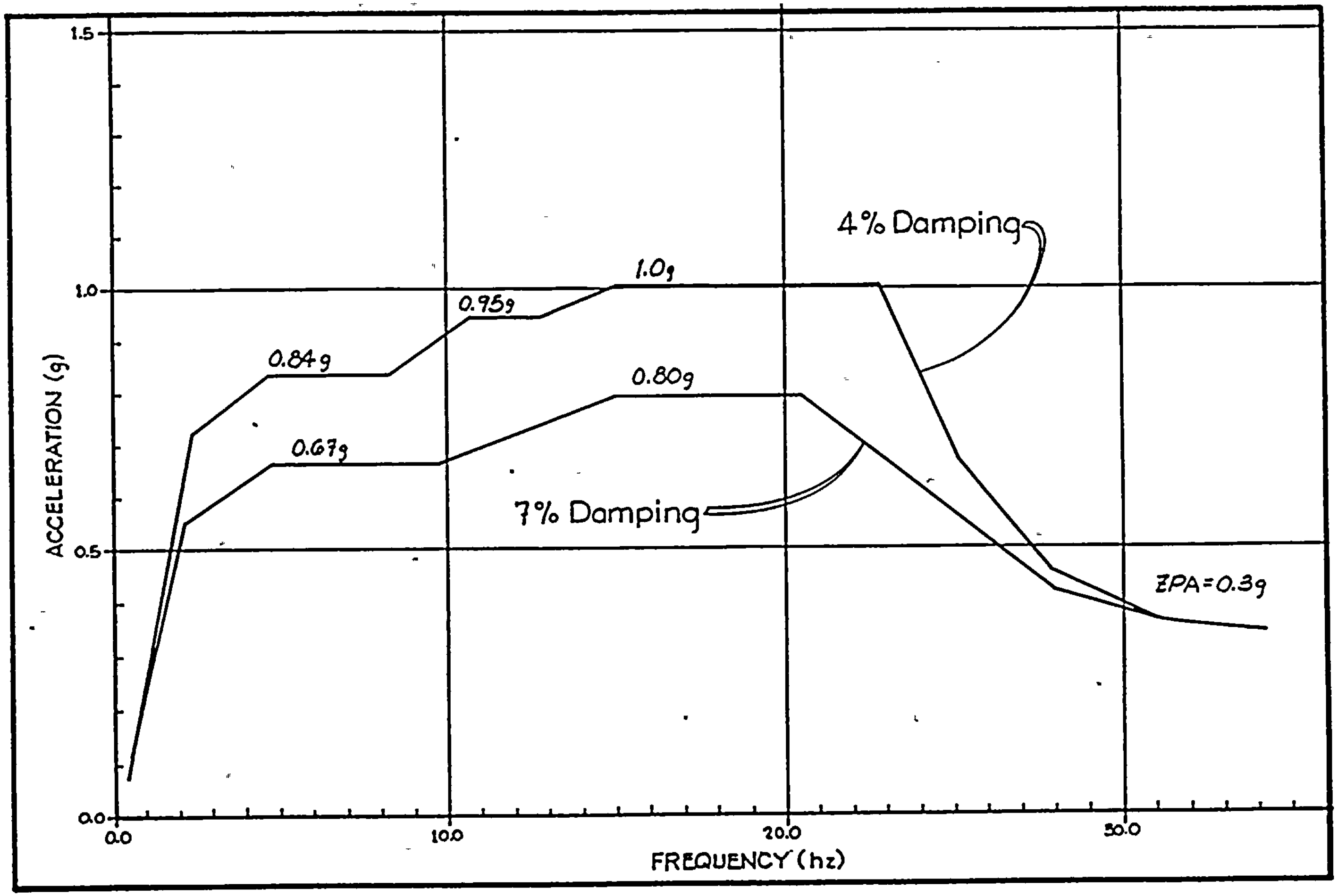


FIGURE 5

GINNA NUCLEAR POWER STATION UNIT 1 - SSE BROAD BAND FLOOR RESPONSE SPECTRA - CONTROL BUILDING

ELEV. 289'-0" VERT-RESPONSE (FIGURE SSE-7 B-V, FEBRUARY 1, 1980)

steel plates and shapes. Considering this hybrid construction, the range of damping values given in Reference 1, and the amplitude of stresses expected for the SSE evaluation, a damping value of seven percent (7%) has been used for this evaluation. Additional justification for 7% damping is as follows:

- The MCB contains numerous electrical devices which are attached to the sheet steel with sheet metal screws. Each device attachment may allow a small amount of slippage, and this will tend to absorb energy during an earthquake and increase damping.

- The MCB contains a large number of loosely attached cables. Testing of electrical raceway systems have shown that loose cables tend to increase damping up to about 25% of critical[2]. A similar behavior may be expected for the MCB.

Thus, the use of 7% damping for the SSE analysis of the MCB is justified. For the vertical earthquake analysis of the MCB roof, 4% damping was used since no instruments are bolted to the roof plate.

Out-Of-Plane Earthquake. The following analysis procedure was used:

(1) The in-situ modal testing results (frequencies and mode shapes) were reviewed and judged to be an appropriate basis for the out-of-plane earthquake analysis. The measurement grids used for the modal testing (Figures 6 through 11) were adopted as the basis of the structural analysis.

(2) The mass tributary to each point on the measurement grid was calculated. The mass included the panel structure in addition to the instruments and devices.

(3) The mass distribution and measured mode shapes were used to calculate the participation factor for each mode.

(4) A spectral acceleration for each mode was obtained based upon the measured frequency and appropriate floor response spectrum.

(5) The mass distribution, mode shapes, frequencies and spectral acceleration were used to calculate the seismic accelerations, displacements and inertial forces at each location of the measurement grid for each mode of interest.

(6) The seismic inertial forces were applied as static loads to the lattice of stiffeners. Each stiffener was treated as a simply supported beam and its maximum seismic induced moment calculated. An effective section modulus was then determined for the composite section consisting of the stiffener and the panel plate. The effective modulus was calculated by adjusting the local stiffness so as to obtain the same modal displacement as that calculated in step 5. The effective modulus was used for stress analysis of the stiffener. This stiffness correlation procedure was performed for each mode using the modal inertial forces and displacements for panels which were relatively highly stressed. However, when the stresses were low, less than one-half of the allowable, this stiffness correlation

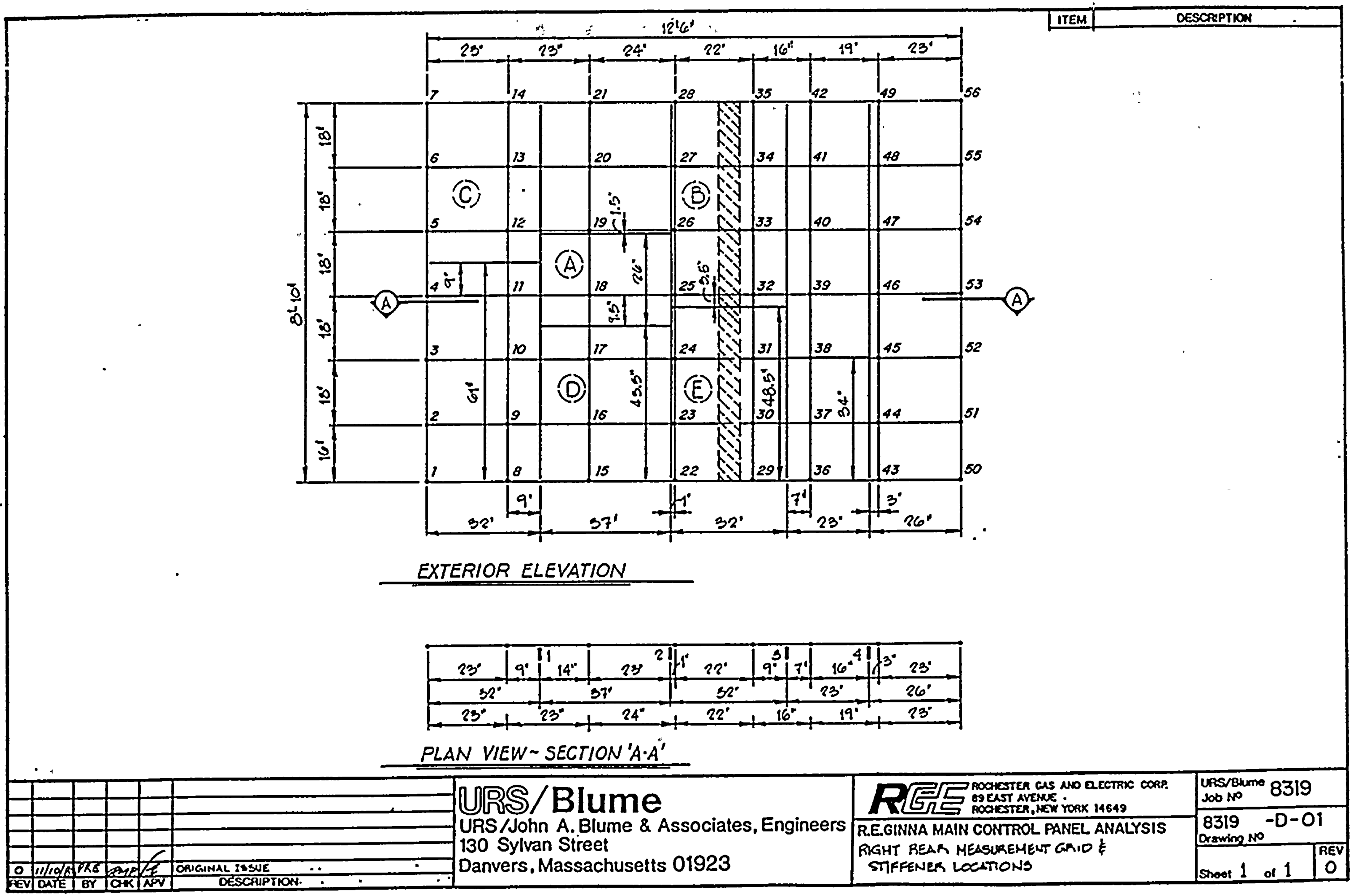
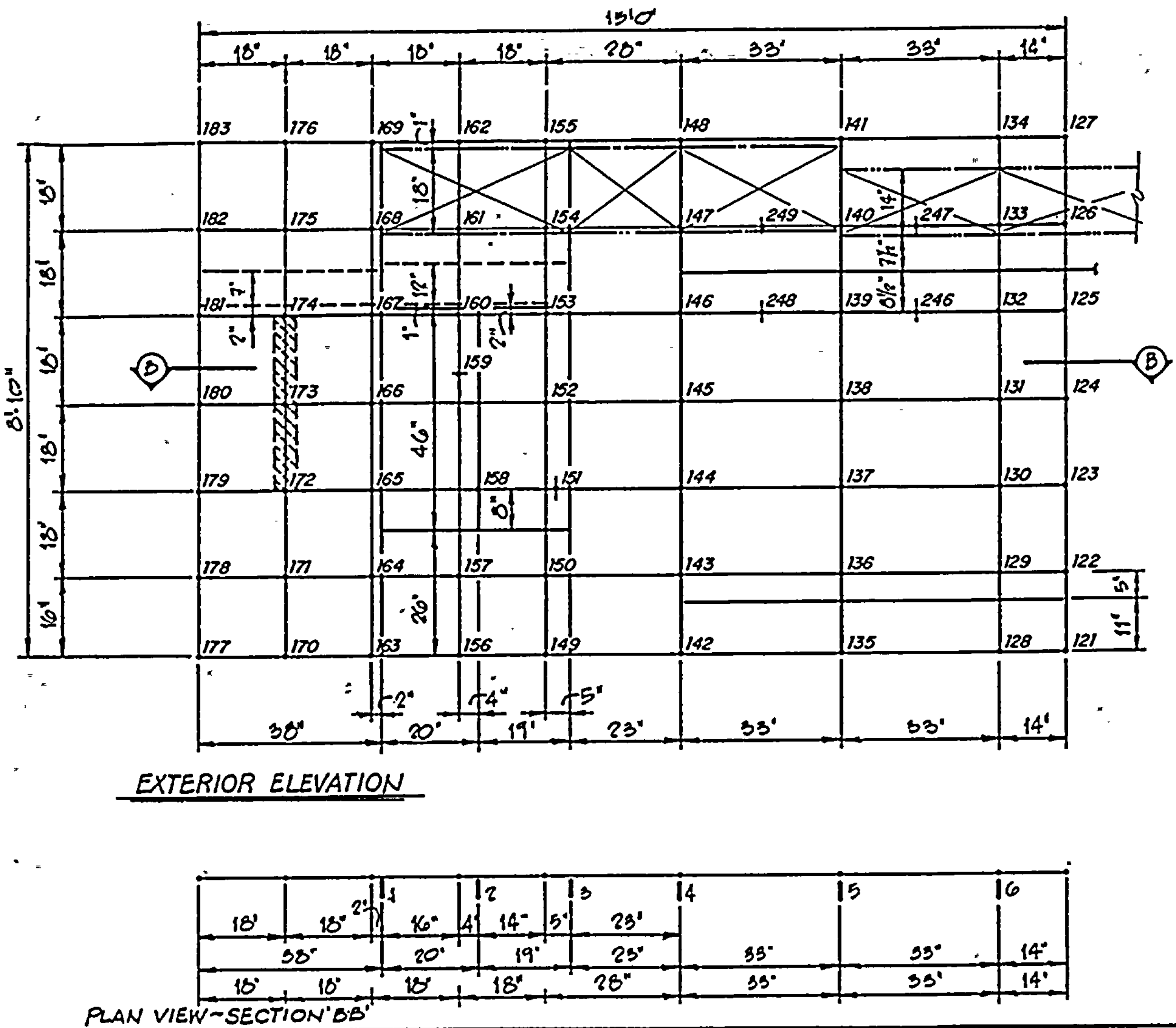


FIGURE 6

RIGHT REAR PANEL SHOWING MEASUREMENT GRID AND STIFFENER LOCATION



ITEM	DESCRIPTION

REV	DATE	BY	CHK	APV	DESCRIPTION
0	11-10-83	PRE	RHP		ORIGINAL ISSUE

URS/Blume
URS/John A. Blume & Associates, Engineers
130 Sylvan Street
Danvers, Massachusetts 01923

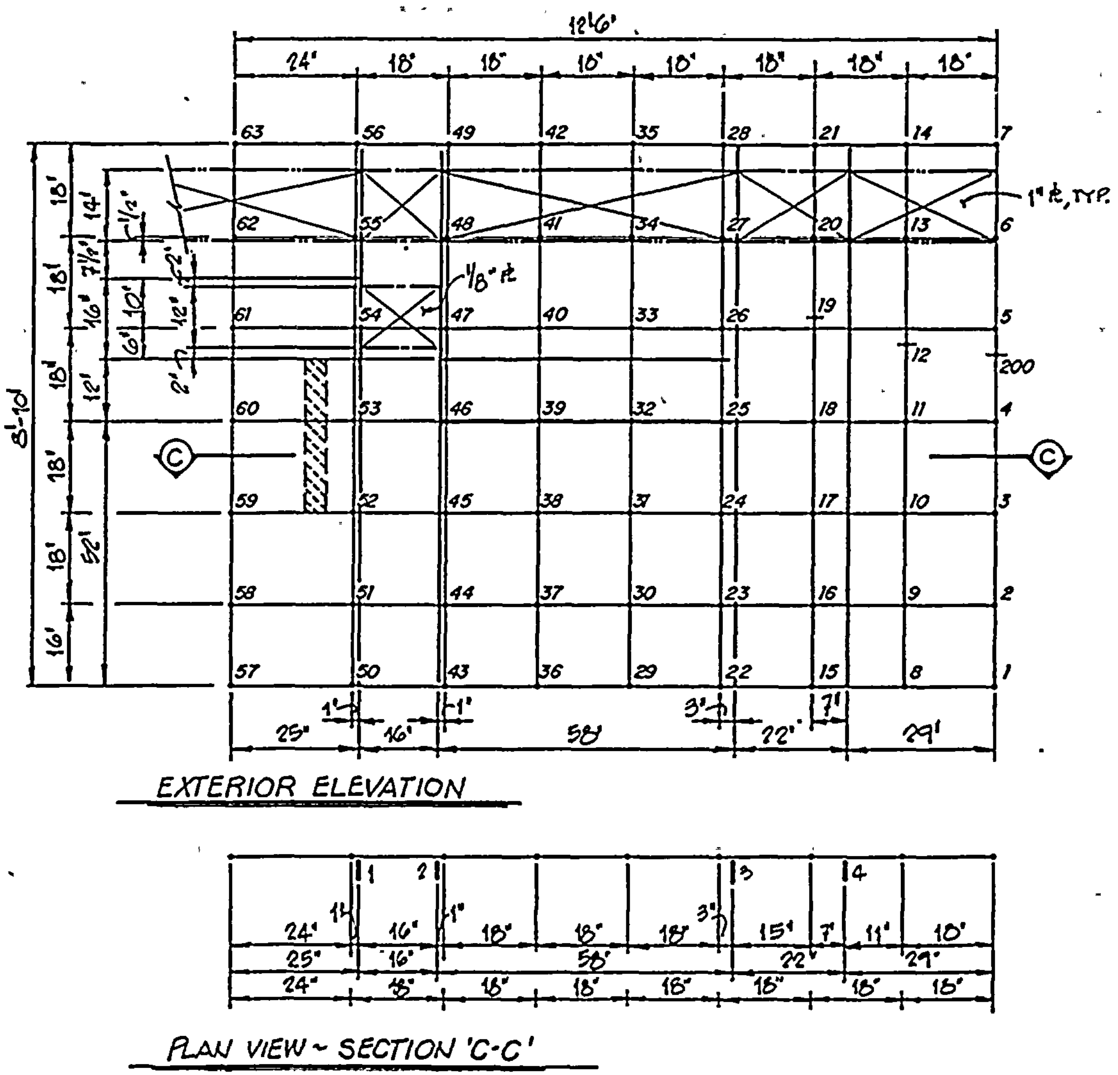
RG&E ROCHESTER GAS AND ELECTRIC CORP.
89 EAST AVENUE
ROCHESTER, NEW YORK 14649

R.E.GINNA MAIN CONTROL PANEL ANALYSIS
CENTER REAR MEASUREMENT GRID & STIFFENER LOCATION

URS/Blume Job No 8319
8319 .-D-02 Drawing No
Sheet 1 of 1
REV 0

FIGURE 7

CENTER REAR PANEL SHOWING MEASUREMENT GRID AND STIFFENER LOCATION



ITEM	DESCRIPTION

REV	DATE	BY	CHK	APV	DESCRIPTION
0	11-10-83	PRE	RMP		ORIGINAL ISSUE

URS/Blume
URS/John A. Blume & Associates, Engineers
130 Sylvan Street
Danvers, Massachusetts 01923

RG&E ROCHESTER GAS AND ELECTRIC CORP.
89 EAST AVENUE
ROCHESTER, NEW YORK 14649

R.E.GINNA MAIN CONTROL PANEL ANALYSIS
LEFT REAR MEASUREMENT GRID & STIFFENER LOCATIONS

URS/Blume Job No 8319
Drawing No 8319 -D-03
Sheet 1 of 1
REV 0

FIGURE 8

LEFT REAR PANEL SHOWING MEASUREMENT GRID AND STIFFENER LOCATION

URS/Blume

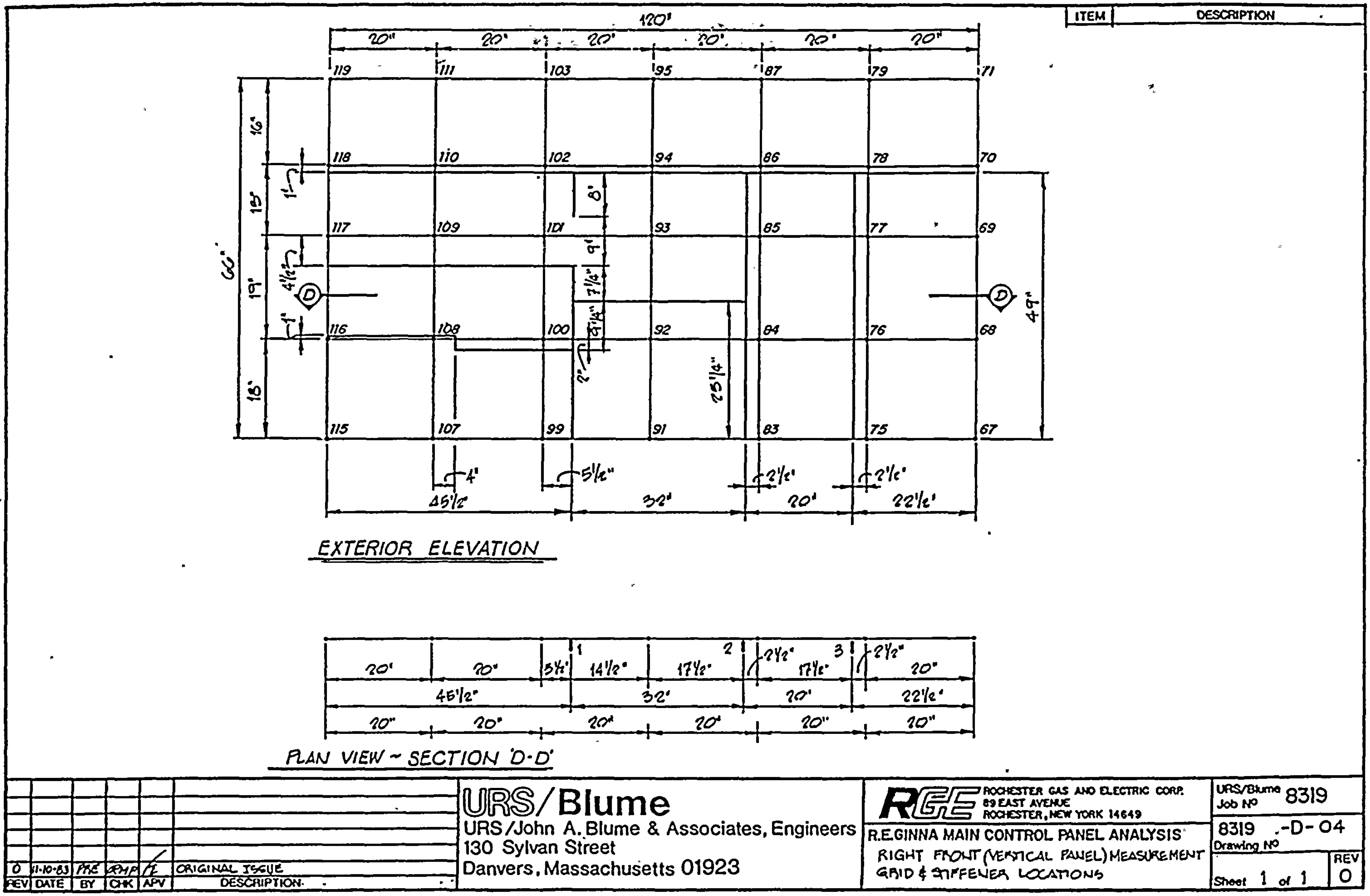


FIGURE 9

RIGHT FRONT PANEL (VERTICAL PANEL) SHOWING MEASUREMENT GRID AND STIFFENER LOCATION

URS/Blume

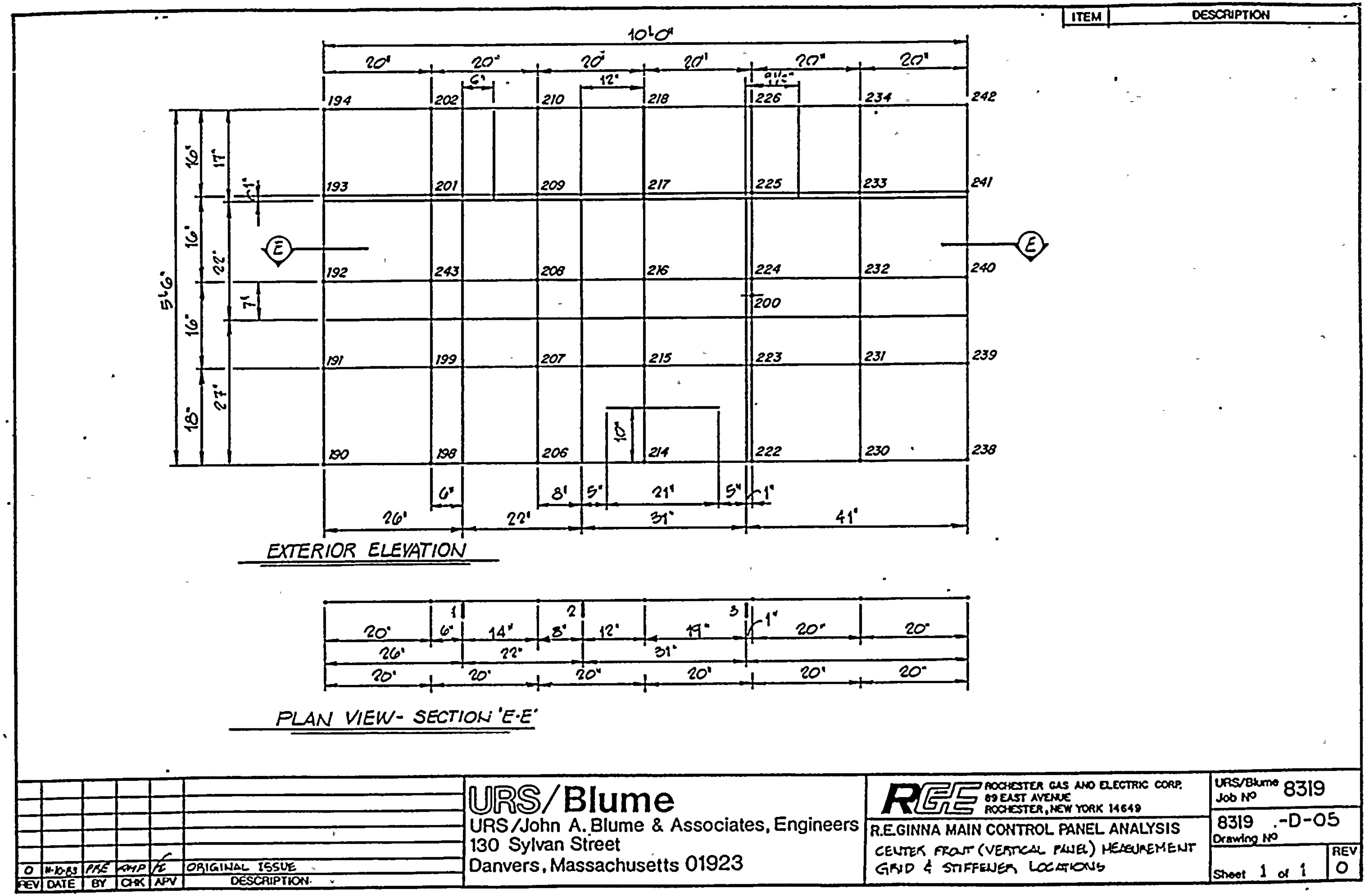


FIGURE 10

CENTER FRONT PANEL (VERTICAL PANEL) SHOWING MEASUREMENT GRID AND STIFFENER LOCATION

URS/Blume

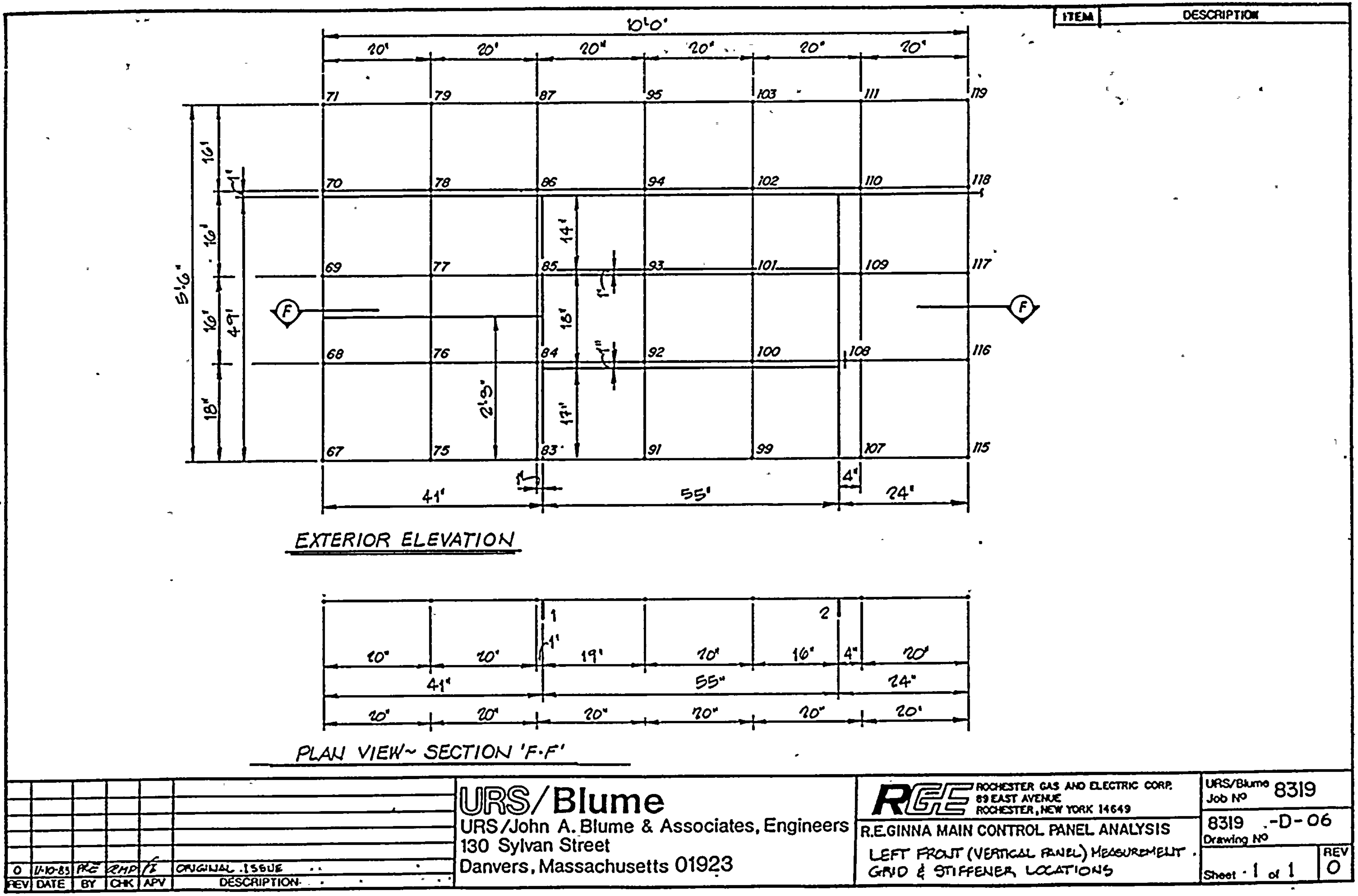


FIGURE 11

LEFT FRONT PANEL (VERTICAL PANEL) SHOWING MEASUREMENT GRID AND STIFFENER LOCATION

was performed using the SRSS combination of the modal seismic inertial forces and displacements for all measured modes.

(7) A static stress analysis was performed using the seismic inertial forces and adjusted panel stiffnesses. The 3/16 inch panel plates were considered to span between stiffeners. One-way or two-way action was used to calculate the stresses in the plates, depending upon the aspect ratio and the location and number of cut-outs. The stiffeners were considered to behave as simply supported beams spanning between the panel boundary (top and bottom) or between other stiffeners. Stresses were calculated in the stiffeners due to tributary inertial loads. The roof of the panel was considered to resist the top reaction of the stiffeners which span vertically. The roof was treated as a deep horizontal beam (i.e., diaphragm action) spanning between the end panels of the left and right sections. Shear and bending stresses in the roof diaphragm were calculated. Any benefit due to the shape of the roof diaphragm was conservatively ignored. The end panels of the right and left section were considered to transfer the reactions from the roof to the floor. The end panels were treated as steel shear walls. Shear and bending stresses were calculated.

(8) Stresses in the above mentioned elements were calculated on a modal basis. The fundamental mode generally contributes the majority of the stress in a given element, and higher modes were ignored when their contribution was less than 10% of the response. When higher modes were important, the modal responses were combined by the square-root-of-the-sum-of the-squares (SRSS) method. For panels with significant closely spaced modes, the double sum method of modal combination was used. This method is detailed in Reference 3.

In-Plane Earthquakes. The stresses due to the two in-plane direction earthquakes were based upon the equivalent static load method. The static load was determined using a factor of 1.5 applied to the peak of the appropriate floor response spectrum. The stresses due to horizontal in-plane earthquake were calculated by dividing the total in-plane inertial force above a given section by the cross sectional area of the 3/16 inch plate available to resist the shear forces. The vertical in-plane earthquake was considered to cause axial stresses in the vertical stiffeners and the 3/16 inch plate.

Directional Combinations. Co-directional responses due to the three components of ground motion have been combined by the SRSS method. The resultant has been added absolutely to the dead load response.

Cantilevered Devices. Cantilevered devices are instruments or recorders which are mounted flush to the cover plate of the MCB and which protrude

into the inside of the MCB some significant distance (more than 5 or 6 inches, see Figure 12). Since the center of gravity of these devices is located some distance from the plate mounting, in-plane earthquake components (with respect to the mounting plate) cause a local bending moment on the mounting plate. This moment is resisted by equal and opposite forces applied at each side of the device. These forces induce local stresses in the mounting plate.

These stresses have been evaluated using the quivalent static load method. The load was determined using a factor of 1.5 applied to the peak of the appropriate floor response spectrum. The reaction on all four sides of the device was calculated based upon its geometry as indicated in Figure 12. It was assumed that the device anchorage is capable of transfering the loads to the MCB plates. These reaction loads were considered to be resisted by bending and twisting of the plate strips of the MCB plates between the devices. These plate stresses were combined with those due to the earthquake normal to the panel and dead load.

The MCB contains many more cantilevered devices than can be practically analyzed. Thus, the approach to this analysis was to concentrate on the heavier devices with large cantilevered distances located in areas of relatively high stress due to the out-of-plane earthquake.

3.2 Acceptance Criteria

The MCB is constructed of structural steel plates and shapes. There is no documentation concerning the actual materials of construction, and the manufacturer of the panel is no longer in business. In 1966 when the control panels were purchased and installed, A36 steel was the most commonly produced steel for hot-rolled structural shapes and plates. It is unlikely that cold-rolled steel was used because of the high cost of cold-rolled shapes of the sizes found in the panel, and the relative difficulty associated with welding this material. However, if cold rolled material was used, the yield strength would probably be higher than A36. Therefore, for the purpose of this evaluation, the following material properties have been assumed:

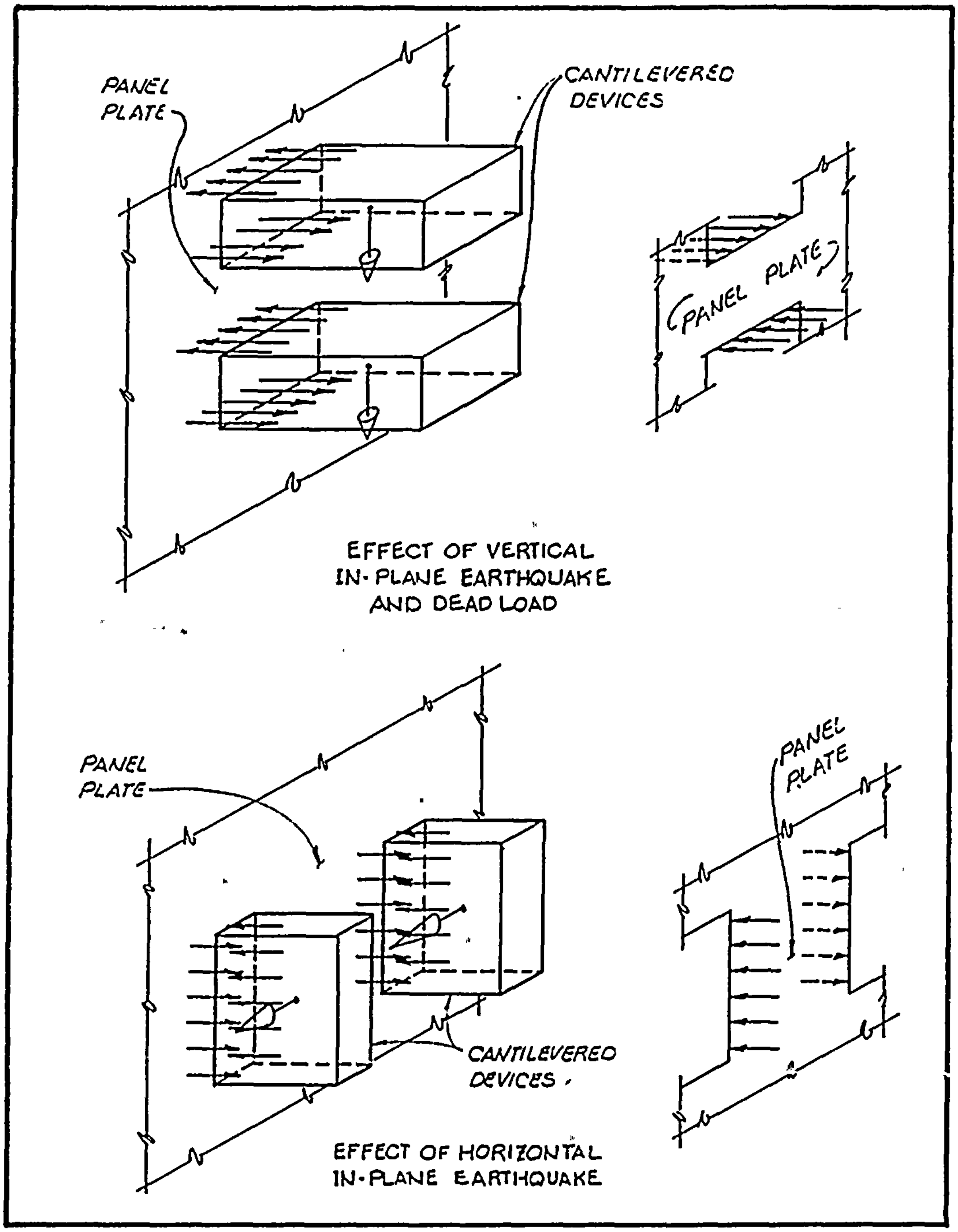


FIGURE 12

EVALUATION METHOD OF STRESSES INDUCED BY CANTILEVERED DEVICES

ASTM A-36 Steel
Yield Point 36,000 psi
Modulus of Elasticity = 29×10^6 psi
Poissons Ration = 0.25

Acceptable stress limits were as defined in NUREG 0800 Section 3.8.4[4]. For the SSE load case of steel structures, the acceptable stress limit of 1.6 times the elastic design strength defined in PART 1 of the AISC Specification[5] was used but not exceeding 0.9Fy for axial and bending stresses and 0.58 Fy for shear stresses. For thin metal sections, the AISI design manual[6] was also considered.

4. RESULTS OF STRUCTURAL EVALUATION

This chapter summarizes the results of the seismic structural evaluation of the MCB at R.E. Ginna. The evaluation was first performed for the controlling out-of-plane seismic motion for each of the panel plates comprising the MCB. The effect of the in-plane seismic motions was then evaluated for each of the panel plates and the significant cantilevered instruments. The stresses were then combined for the three earthquakes and added to the dead load stresses.

4.1 Out-of-Plane Seismic Motion

The evaluation results described in this section pertain only to stresses resulting from out-of-plane seismic motion. An evaluation of the stresses resulting from in-plane seismic motions and dead loads is discussed separately.

Right Rear Panel. The right rear panel contains the heaviest instruments of the six panels evaluated. An exterior view of the panel showing the structural dimensions and equipment locations is provided on RG&E's Drawing No. 33013-763, Rev. 2. The combined total weight of the panel and its equipment is approximately 2,000 lbs. The measurement grid used for the in-situ testing of the right rear panel is shown in Figure 6. Superimposed on the grid are the locations of the existing structural stiffeners.

The dynamic properties of the right rear panel in the out-of-plane direction are given in Table 1. Eight modes, with frequencies ranging from 6.8 hz to 21.2 hz, were obtained from the test data. The participation factors were determined based upon the calculated mass distribution and the measured mode shapes. The spectral accelerations, which form the basis of the subsequent analysis, were obtained from the 7% damped spectra in the east-west direction at the modal frequencies shown in the table.

The seismic response of the right rear panel due to the out-of-plane earthquake is summarized in Table 2. Maximum values for deflection, acceleration, and inertia forces are approximately 0.5 inches, 2.3g, and 230 pounds respectively. Maximum values of deflection and acceleration occur

Mode	Frequency (Hz)	Participation Factors	Spectral* Acceleration (g)
1	6.861	1.454	1.540
2	8.971	-.166	1.510
3	11.618	.534	1.459
4	15.119	-.334	.893
5	16.339	.253	.810
6	17.671	.656	.735
7	19.699	.401	.642
8	21.348	.088	.609

*Based on elevation 289'-0" of the Control Building east-west spectra for 7% damping.

TABLE 1

DYNAMIC PROPERTIES OF RIGHT REAR PANEL

DEFLECTION* (inch)							
.0061	.0094	.0085	.0441	.0126	.0047	.0096	.0066
.0094	.1407	.2232	.2467	.2317	.1293	.0720	.0057
.0114	.2485	.3640	.4255	.3802	.2122	.1247	.0047
.0136	.2878	.4206	.4668	.3662	.2718	.1367	.0044
.0145	.2566	.3541	.3461	.3162	.2445	.1059	.0020
.0060	.1138	.1455	.1677	.1500	.1034	.0465	.0015
.0009	.0009	.0014	.0014	.0009	.0014	.0009	.0005

ACCELERATION* (g)							
.03	.05	.05	.23	.07	.02	.06	.03
.05	.75	1.12	1.22	1.18	.65	.47	.03
.06	1.29	1.77	2.09	1.89	1.07	.82	.02
.08	1.54	2.04	2.28	1.79	1.35	.94	.03
.11	1.52	1.80	1.71	1.57	1.22	.69	.01
.04	.67	.75	.83	.77	.52	.33	.01
.00	.00	.01	.01	.00	.01	.00	.00

INERTIAL FORCE* (pound)							
.17	.55	.52	2.58	.60	.20	.55	.19
.54	21.87	53.47	53.47	42.03	15.18	22.08	.56
.98	54.09	162.61	152.15	94.42	29.40	51.39	.62
1.59	121.89	226.56	193.04	129.04	32.83	69.02	.77
1.64	96.10	219.91	165.10	120.14	32.59	49.33	.23
.45	22.92	25.77	36.30	43.18	10.70	8.35	.11
.02	.04	.07	.07	.04	.05	.04	.01

* The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 2

RESPONSE OF THE RIGHT REAR PANEL DUE TO

OUT-OF-PLANE EARTHQUAKE

at measurement point 25, while the maximum inertial force is at point 18 (see Figure 6).

The stress analysis was performed by first evaluating the structural adequacy of the 3/8 inch by 2 inch stiffeners. The 3/16 inch plate was then evaluated by selecting critically loaded plates spanning between stiffeners.

Stiffener Evaluation: The inertial forces at the measurement grid points corresponding to the first mode (6.86 hz) of the right-rear panel response were used to evaluate the stiffeners.

Stiffener 2 (see Figure 6) was found to have the largest inertial forces across its length. Conservatively treating the stiffener as a simply supported beam, the maximum moment was determined to be 17,406 in-lb. An effective section modulus, S, was then determined for the stiffener such that the maximum deflection of the idealized simply supported stiffener matched the first mode deflection. The resultant maximum bending stress of 23,208 psi was comfortably below the allowable stress of $0.9F_y$, or 32,400 psi.

Evaluation of the stresses resulting from consideration of higher mode responses indicated that the higher modes have a negligible effect. The higher mode responses were therefore neglected in this evaluation.

Plate Evaluation: Plate sections B, D and E, as shown in Figure 6, were selected for evaluation. These plate sections were selected because they had relatively long unstiffened spans compared to other plate sections and contained fairly heavy equipment.

The evaluation considered the out-of-plane bending responses of the strips of plate between cut-outs spanning between stiffeners. Typically, the minimum width of these strips is approximately 1 inch to 1 1/2 inches. The evaluation was performed for the most critically loaded strips within each of the plate sections and considered both horizontal and vertical strips.

The inertial forces along these strips were determined by scaling the tributary distributed equipment weights by the out-of-plane acceleration of the first mode response. The strips were conservatively evaluated as simply supported.

All of the strips evaluated were found to have bending stresses within the allowable stress limit of $0.9F_y$ with the exception of a vertical strip passing through the center of plate section B and E. These strips were found to have a maximum bending stress of 40,328 psi and 47,339 psi respectively. It was therefore concluded that an additional vertical stiffener is required in the highly stressed region of plate section B and E as indicated by the shaded area in Figure 6.

Center Rear Panel. An exterior view of the center rear panel is shown in RG&E's Drawing No. 33013-762, Rev. 0. The in-situ test measurement grid and the locations of the existing structural stiffeners are shown in Figure 7. The total weight of the center rear panel is approximately 1,740 lbs. The panel has two large 200 lb. DC distribution panels which are framed in by vertical stiffeners 4, 5 and 6 as shown in Figure 7. Three 50 lb. and one 100 lb. temperature recorders and one 100 lb. switch unit are framed in by stiffeners 1, 2 and 3. Nine 15 lb. instruments are located to the left of stiffener 1.

The dynamic properties of the center rear panel in the out-of-plane direction are given in Table 3. Three modes, with frequencies ranging from 8.3 hz to 16.6 hz were obtained from the test data. The participation factors were determined based upon the calculated mass distribution and the measured mode shapes. The out-of-plane direction of the center rear panel is at a 45 degree angle with respect to the global X and Y directions. Hence, the SRSS combination of the X and Y 7% damped spectra was used in the evaluation.

The seismic response of the center rear panel due to the out-of-plane earthquake is summarized in Table 4. The maximum values of deflection, acceleration, and inertia force are approximately 0.76 inches, 5.7g, and 318 pounds. Maximum values of deflection and acceleration for these

Mode	Frequency (hz)	Participation Factor	Spectral Acceleration* (g)
1	8.250	1.214	2.500
2	11.120	0.905	1.888
3	16.560	0.276	0.915

*Based on elevation 289'-0" of the Control Building. SRSS of east-west and north-south spectra for 7% damping.

TABLE 3

DYNAMIC PROPERTIES OF THE CENTER REAR PANEL

DEFLECTION* (inches)								
.0159	.0168	.0187	.0197	.0199	.0234	.0154	.0139	.0118
.0166	.1232	.3046	.2779	.2720	.1969	.1279	.0649	.0117
.0405	.3035	.5091	.5580	.4641	.3357	.2578	.1368	.0186
.0405	.7555	.6822	.5907	.5519	.3116	.2105	.1112	.0146
.0138	.3020	.5735	.4569	.4310	.2703	.1532	.0693	.0067
.0008	.1141	.3171	.2778	.2107	.1698	.1011	.0280	.0019
.0002	.0002	.0030	.0015	.0030	.0015	.0007	.0009	.0007

ACCELERATION* (g)								
.11	.12	.13	.14	.14	.18	.12	.11	.08
.12	.87	2.14	1.93	1.98	1.66	1.15	.60	.08
.29	2.18	3.59	3.88	3.33	2.84	2.32	1.29	.16
.30	5.74	4.87	4.12	3.97	2.69	1.97	1.09	.12
.09	2.14	4.06	3.18	3.12	2.34	1.40	.70	.06
.01	.89	2.28	1.94	1.51	1.44	.95	.34	.02
.00	.00	.02	.02	.02	.01	.00	.01	.00

INERTIAL FORCE* (pounds)								
.48	1.03	1.12	1.19	1.59	2.59	1.78	1.15	.29
.99	14.87	55.87	33.46	43.45	35.78	17.60	8.88	.56
2.53	64.10	116.68	122.71	178.89	102.23	50.70	23.61	1.06
2.62	281.07	283.12	318.02	274.59	193.59	130.51	43.49	.77
.85	138.22	201.29	271.22	159.59	123.40	93.41	28.21	.40
.08	14.43	37.19	85.41	31.09	57.22	51.35	11.48	.16
.01	.02	.18	.15	.25	.15	.07	.11	.01

*The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 4

RESPONSE OF THE CENTER REAR PANEL DUE TO OUT-OF-PLANE EARTHQUAKE

URS/Blume

response parameters occur at measurement point 173, while the maximum inertia force is at point 159 (see Figure 7).

Stiffener Evaluation: Based upon a review of the first mode inertial force distributions, it was determined that the heaviest loaded stiffeners are 1 and 3. Considering contributions to the inertial forces from the first three modes, it was found that stiffeners 1 and 3 had maximum bending stresses of 36,840 psi and 26,173 psi, respectively. Since the stress in stiffener 1 is in excess of the allowable value of 32,400 psi, it is necessary to increase its capacity. This can be performed by adding an external vertical stiffener to the center rear panel along the line of action of stiffener 1. This stiffener could be stitch welded to the panel's plate. All other stiffeners were found to meet the acceptance criteria.

Plate Evaluation: From a review of the stiffener arrangement in Figure 7, and RG&E Drawing No. 33013-762, Rev. 0 containing equipment locations and weights, it is observed that:

- Reinforced cutouts for the AC and DC distribution panels are located to the right of stiffener 3. No equipment cutouts exist in this area and plate stresses, therefore, are expected to be low.
- Three 50 lb. and one 100 lb. temperature recorders and a 100 lb. switch unit are located between stiffeners 1 and 3. These large pieces of equipment are supported by both vertical and horizontal stiffeners along their edges. This section is judged to be sufficiently reinforced by stiffeners such that the majority of the load transfer occurs through the stiffeners rather than through the 2 to 3 inch wide strips of plate separating the equipment. It is therefore expected that the plate stresses in this region will be well within the allowable.
- Between stiffener 1 and the left edge of the panel where the nine 15 lb. instruments are located, the resultant SRSS accelerations and inertial forces are large.

Based upon the above observations, the plate of the center rear panel is judged to be qualified with stress levels well below the allowable limit except for the region to the left of stiffener 1. The most highly stressed area within this region was determined to be a 1 1/2 inch wide vertical strip spanning between the two horizontal stiffeners (nodes 174 and 172 of Figure 7) and located between the 15 lb. instruments. An evaluation

of this 36 inch long strip resulted in a stress value of 42,300 psi which is in excess of the allowable value. Thus, stiffening of this area is recommended.

Left Rear Panel. The dimensions of the left rear panel, along with the equipment locations and weights are shown on RG&E Drawing No. 33013-764, Rev. 0. The grid used to obtain in-situ measurements and the locations of the existing stiffeners are shown in Figure 8. The total weight of the panel and its associated equipment is approximately 1,090 lbs. which is about half that of the right rear panel.

The dynamic properties of the left rear panel are given in Table 5. Five modes, with frequencies ranging from 8.8 hz to 16.4 hz, were obtained from the test data. The participation factors were determined based upon the calculated mass distribution and the measured mode shapes. The spectral accelerations, which form the basis for the subsequent analysis, were obtained from the 7% damped spectra in the north-south direction at the modal frequencies shown in the table.

The seismic response of the left-rear panel due to the out-of-plane earthquake is shown in Table 6. The maximum values of deflection, acceleration, and inertial force are approximately 0.5 inches, 4.1g, and 119 pounds. All maximum values occur at measurement point 31 (see Figure 8). The inertial forces for the left rear panel are considerably less than the center and right rear panels, while the number and arrangement of stiffeners is similar.

Stiffener Evaluation: A review of the panel mounted equipment indicated that very light weight equipment only (e.g., voltmeters, rheostats and ammeters) varying in weight from one to three pounds, are located to the right of the measurement grid line extending vertically between nodes 36 and 42. Thus, stiffener 2 (Figure 8) was chosen for evaluation due to the high inertial loads to which it is subjected.

A stress evaluation was performed for the stiffener considering it to be simply supported at its ends. The resultant maximum stress was determined

Mode	Frequency (hz)	Participation Factors	Spectral Acceleration* (g)
1	8.800	4.114	2.510
2	11.100	.446	1.951
3	11.700	.654	1.537
4	15.000	.974	1.163
5	16.400	.443	.975

*Based on elevation 289'-0" of the Control Building north-south spectra for 7% damping.

TABLE 5

DYNAMIC PROPERTIES OF THE LEFT REAR PANEL

DEFLECTION* (inches)								
.0079	.0144	.0092	.0118	.0054	.0119	.0159	.0105	.0105
.0131	.1570	.1946	.2482	.2840	.1556	.0913	.0477	.0092
.0092	.2976	.3787	.4076	.4121	.2660	.1450	.0640	.0092
.0066	.1009	.4049	.3984	.3795	.2821	.1381	.0647	.0066
.0092	.3802	.3345	.4519	.4958	.2478	.0964	.0237	.0040
.0040	.1648	.1647	.2170	.2103	.1239	.0490	.0066	.0013
.0001	.0013	.0026	.0013	.0013	.0013	.0013	.0002	.0000
ACCELERATION* (g)								
.06	.12	.07	.09	.05	.10	.14	.09	.08
.10	1.25	1.55	2.06	2.49	1.30	.84	.47	.07
.07	2.38	3.01	3.25	3.37	2.19	1.33	.67	.08
.05	1.38	3.22	3.16	3.02	2.31	1.26	.64	.06
.07	3.07	2.66	3.68	4.07	2.01	.87	.27	.03
.03	1.33	1.32	1.78	1.73	1.00	.41	.06	.01
.00	.01	.02	.01	.01	.01	.01	.00	.00
INERTIAL FORCE* (pounds)								
.36	1.15	.64	.81	.40	.85	1.17	.73	.72
1.20	25.15	26.58	35.36	42.83	22.34	15.14	9.24	.41
.84	47.77	51.73	55.85	70.79	44.33	30.03	14.57	.34
.61	42.70	158.43	107.14	111.14	64.94	49.71	34.23	.95
.86	78.08	69.29	83.17	118.92	60.36	30.20	5.39	.29
.35	25.25	21.45	28.99	28.24	16.35	7.51	.95	.09
.01	.09	.16	.08	.09	.09	.08	.03	.00

*The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 6

RESPONSE OF THE LEFT REAR PANEL DUE TO OUT-OF-PLANE EARTHQUAKE

URS/Blume

to be 23,234 psi which is comfortably below the allowable 32,400 psi. The remaining stiffeners are thus also qualified because they have lower inertial forces and similar span lengths.

Plate Evaluation: Visual inspection of the panel drawing no. 33013-764, Rev. 0, and field visits indicate that the panel contains very light pieces of equipment ranging from one to three pounds over the right two-thirds section of the panel. The heaviest pieces of equipment - two 25 lb. recorders and a small distribution panel - are located to the left of stiffener 1 (Figure 8). The response accelerations and inertia forces in that region are small due to the proximity of the end boundaries.

A comparison of the maximum panel equipment weights and maximum acceleration values with those panels previously qualified indicated that the plate sections in this panel will be well below the allowable stress values.

Right, Center, and Left Front Panels (vertical panels). The measurement grids and stiffener locations of the right, center, and left front panels are shown on Figure 9, 10 and 11. The dynamic properties in the out-of-plane direction of the panels are summarized in Table 7. Compared to the rear panels, the frequencies of the front panels are slightly greater, but the spectral accelerations are approximately the same due to the shape of the spectral curves.

The responses of the three center panels are summarized in Tables 8, 9, and 10. Some of the acceleration values of the front panels are of the same magnitude as the values in the rear panels. However, the displacements and inertial forces on the front are generally less than the rear. The spans of the stiffeners and plates and the inertial forces on the front panels are less than those of the rear panels. Therefore, it is reasonable to expect that the stresses in the front would be less than those in the back.

This expectation was verified by calculations of the maximum stiffener and plate stresses in the right, center and left panel sections. The maximum stiffener stress value of 20,900 psi was found in stiffener 3 of

RIGHT FRONT			
Mode	Frequency (hz)	Participation Factors	Spectral[1] Acceleration (g)
1 2 3	9.430 15.830 25.750	1.834 .529 .513	1.510 .843 .552
CENTER FRONT			
Mode	Frequency (hz)	Participation Factors	Spectral[2] Acceleration (g)
1 2 3 4 5	10.700 12.100 16.200 18.300 26.300	2.495 1.509 1.580 1.397 -.328	2.784 1.874 1.298 1.057 .777
LEFT FRONT			
Mode	Frequency (hz)	Participation Factors	Spectral[3] Acceleration (g)
1 2 3 4 5	11.100 14.600 21.000 24.700 26.600	2.281 3.894 3.109 .074 .385	1.951 1.226 .640 .577 .551

1. Based on Elevation 289'-0" of the Control Building east-west spectra for 7% damping.

2. Based on Elevation 289'-0" of the Control Building SRSS of east-west and north-south spectra for 7% damping.

3. Based on Elevation 289'-0" of the Control Building north-south spectra for 7% damping.

TABLE 7

DYNAMIC PROPERTIES OF RIGHT, CENTER AND LEFT FRONT PANELS

DEFLECTION* (inches)						
.0037	.0037	.0049	.0037	.0046	.0043	.0040
.0110	.1091	.1632	.1889	.1549	.0865	.0068
.0088	.1635	.3053	.2628	.1347	.0728	.0043
.0061	.0750	.1808	.1412	.0674	.0391	.0037
.0027	.0098	.0125	.0079	.0058	.0043	.0021
ACCELERATION* (g)						
.03	.03	.05	.03	.04	.04	.04
.10	1.04	1.53	1.75	1.46	.83	.06
.08	1.50	2.81	2.39	1.27	.73	.04
.06	.70	1.66	1.29	.65	.41	.04
.03	.09	.11	.07	.06	.04	.02
INERTIAL FORCE* (pounds)						
.27	.71	.92	.64	.86	.89	.30
1.18	29.01	42.47	50.78	38.85	23.49	.72
.69	27.82	67.05	66.88	20.73	17.88	.45
.96	28.15	67.29	39.35	36.10	14.44	.58
.34	2.70	3.17	1.66	1.65	1.29	.28

*The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 8

RESPONSE OF THE RIGHT FRONT PANEL DUE TO OUT-OF-PLANE EARTHQUAKE

DEFLECTION* (inches)						
.0235	.0247	.0274	.0246	.0340	.0273	.0264
.0363	.2494	.3889	.4324	.3470	.2198	.0344
.0263	.2120	.3301	.3940	.3006	.2172	.0283
.0238	.1016	.2944	.2028	.1813	.1209	.0293
.0207	.0265	.0256	.0384	.0338	.0329	.0240
ACCELERATION* (g)						
.31	.33	.36	.32	.43	.36	.34
.45	3.36	5.19	5.75	4.61	2.93	.44
.34	2.70	4.32	5.18	4.00	3.04	.37
.32	1.32	3.75	2.68	2.42	1.57	.38
.27	.34	.36	.48	.42	.41	.31
INERTIAL FORCE* (pounds)						
2.51	6.83	6.84	6.36	8.24	7.44	2.73
5.57	103.09	156.81	162.23	128.05	86.54	5.50
3.13	56.70	96.68	110.93	89.64	60.72	3.49
3.49	61.83	186.61	69.50	51.53	44.74	6.07
2.88	13.63	20.21	24.38	11.34	10.57	4.18

*The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 9

RESPONSE OF CENTER FRONT PANEL DUE TO OUT-OF-PLANE EARTHQUAKE

DEFLECTION* (inches)						
.0139	.0143	.0178	.0092	.0121	.0160	.0132
.0164	.1265	.2104	.2392	.2274	.1496	.0184
.0146	.1260	.1959	.1948	.1843	.1137	.0149
.0145	.1739	.1556	.1323	.1042	.0839	.0114
.0065	.0110	.0138	.0150	.0143	.0132	.0096
ACCELERATION* (g)						
.18	.18	.23	.14	.21	.21	.17
.21	1.62	2.69	3.07	3.26	2.24	.24
.19	1.64	2.51	2.51	2.52	1.61	.19
.20	2.25	1.98	1.69	1.39	1.22	.15
.09	.16	.17	.20	.19	.17	.12
INERTIAL FORCE* (pounds)						
1.44	3.83	4.40	2.65	4.13	4.33	1.37
2.60	50.02	76.80	85.86	96.55	77.37	3.52
1.76	31.42	28.39	52.10	54.59	47.61	3.05
2.79	93.85	88.81	52.67	41.11	53.30	4.04
1.13	5.29	5.66	5.15	5.75	6.63	2.19

*The arrangement of this table is the same as the lay-out of the measurement grid.

TABLE 10

RESPONSE OF LEFT FRONT PANEL DUE TO OUT-OF-PLANE EARTHQUAKE

the center panel. The maximum plate stress value of 18,600 psi was calculated in the right front section. Both these values are well below the allowable stress limits.

Bench Panels. A review of the equipment layout drawing for the three horizontal bench panels indicates that the center bench supports the heaviest equipment followed by the left and right benches. The bench plates are also reinforced by 2 inch by 3/8 inch stiffeners similar to the vertical plates. A layout of the existing stiffeners is shown in Figure 13. The three benches are actually slightly inclined at a 16° angle, however, they were evaluated as if they were perfectly horizontal. The vertical SSE response spectrum for 7% damping was used.

The in-situ testing resulted in five modal frequencies for the center bench between 9.2 and 24 hz in its out-of-plane direction. It exhibited some out-of-plane motion in the third out-of-plane mode of the center front vertical panel at 16 hz (see Appendix A). The in-situ testing did not result in detailed mode-shapes for the benches, but did indicate the expected frequencies. Thus, the equivalent static load method with a 50% increase of the peak of the 7% damped vertical spectrum was conservatively used and the inertial loads added absolutely to the dead loads.

The center bench was evaluated by considering three simply supported plates loaded with uniform pressure distribution. The three plates span between the edges of the bench panel and the stiffeners spanning the entire width of the panel. The pressure value used for each plate was that of the most highly loaded strip of the entire plate. The left plate was the most highly stressed with a maximum value of 46,180 psi. Since this stress is larger than 0.9 F_y, a modification of the center bench is required. This modification could consist of extending the left most stiffener, 1 foot 9 1/8 inches long, to the upper edge of the center bench. This modification would reduce the span length of the rectangular plate and thus reduce the bending stresses to an acceptable level. The remaining two plates of the center bench are acceptable without any modifications. All the stiffeners were also evaluated and found acceptable after increasing the length of the left stiffener. The largest stiffener stress thus obtained was 30,720 psi.

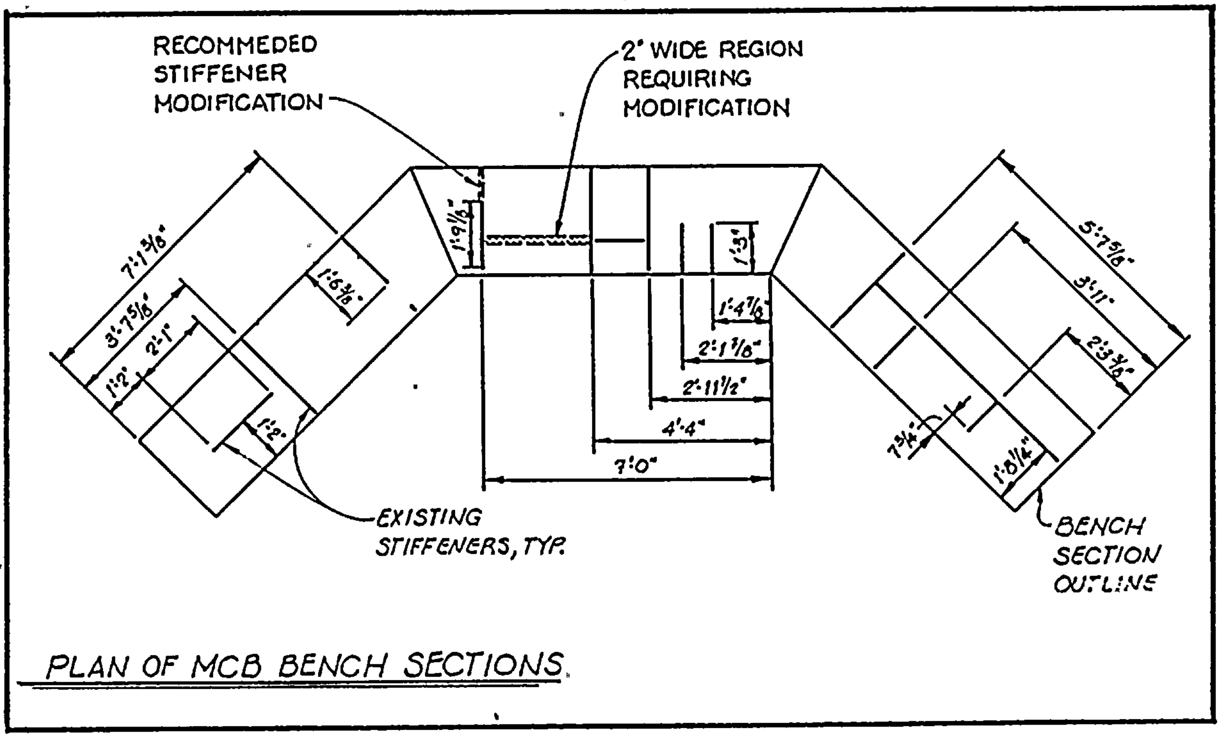


FIGURE 13

LAYOUT OF STIFFENER LOCATIONS OF THE MCB BENCH SECTIONS

The right and left benches were evaluated by analyzing critical horizontal plate strips as simply supported beams subjected to equivalent static load similar to the center bench. The stresses due to the in-plane and out-of-plane earthquakes were calculated and found acceptable. The maximum stresses in the right and left bench panels were 15,700 psi, and 18,300 psi respectively. Both values are less than the allowable stress of 32,400 psi. The stiffeners were also found to be acceptable.

Roof Diaphragm and End Panels. The roof of the MCB is a critical element of the seismic load path because it transfers the top edge reaction loads of the vertical panels due to out-of-plane earthquake to the end panels of the right and left section. The roof diaphragm consists of three 1/8 inch thick plates for the left, center and right boards. Each plate is reinforced with three stiffeners spanning between the front and back panels. The stiffeners are structural angles with dimensions of 2-1/2 by 2-1/2 by 1/4 inches. A layout of the stiffeners is indicated in Figure 14.

The vertical SSE with 4% damping plus dead load was considered for an out-of-plane analysis of the roof diaphragm using the equivalent static load method. The analysis considered a simply supported plate, 3 feet 11 inches by 6 feet 0 inches. These dimensions correspond to the typical dimension between roof stiffeners and the width of the panel. A total uniform load of 2.5g times the plate weight was applied to the plate. The maximum out-of-plane bending stress was calculated to be 6,230 psi.

The roof diaphragm was also analyzed to determine the in-plane shear stresses due to the top edge reaction loads of the vertical panels (front and back). For this evaluation, it was assumed that the roof diaphragm behaves as a straight, simply supported beam spanning 420 inches between the end panels of the right and left sections (see Figure 15). This analysis conservatively ignores any benefit gained from the shape of the board and any shear load transferred to the ground at the miter bends of the board.

The top edge reaction of the center rear, which resulted in the largest reaction loads, conservatively assumed to apply everywhere for the

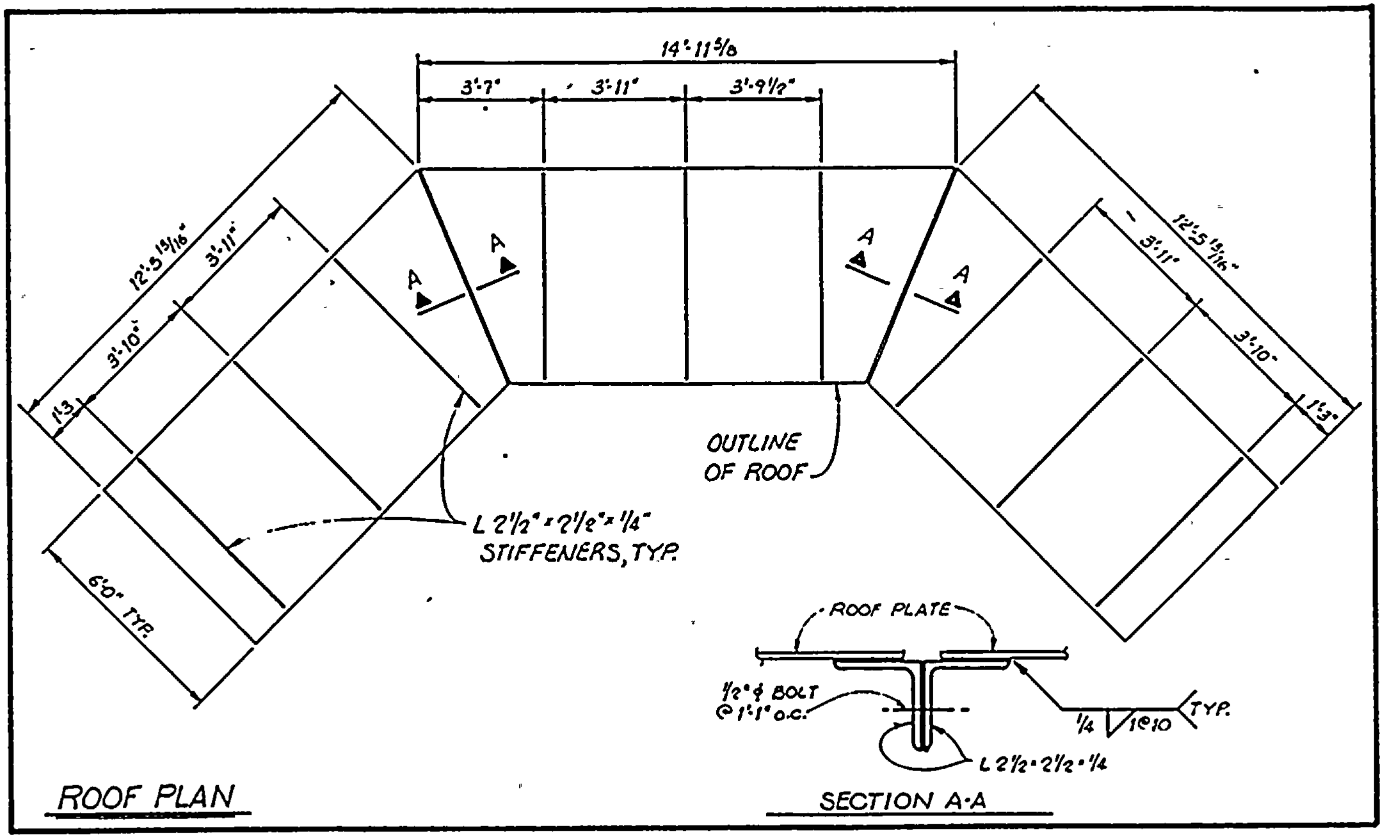


FIGURE 14

CEILING PLAN

MAIN CONTROL BOARD

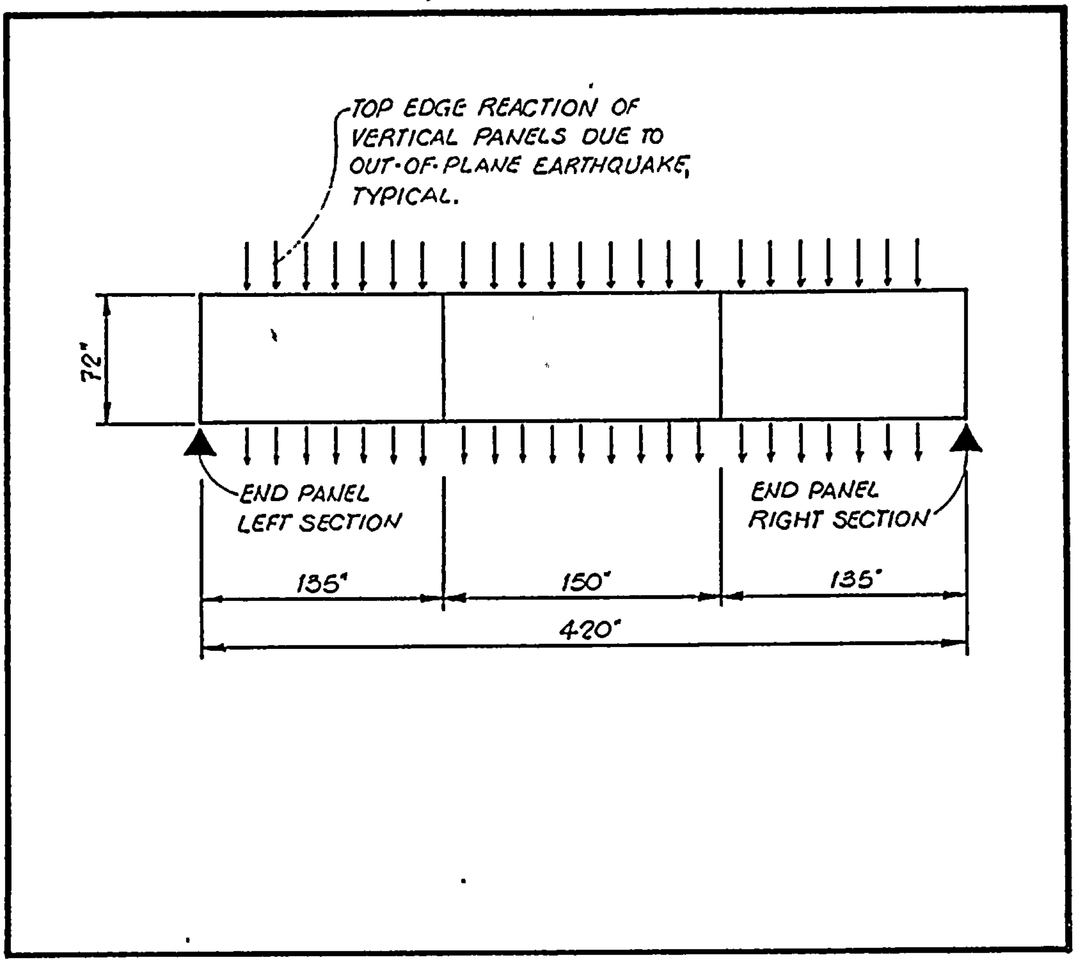


FIGURE 15

ANALYSIS OF ROOF DIAPHRAGM

diaphragm analysis. This load is equal to 7.67 lbs/inch and was assumed to also apply for the front panels. Thus, the diaphragm was analyzed for two times this value or 15.3 lbs/inch. Based upon a 420 inch span length, the maximum shear and moment are 3221 lbs. and 338,247 lbs-inch. The moment is considered to be resisted by axial tension and compression in the 2-1/2 by 2-1/2 by 1/4 inch angles on the front and back edges of the diaphragm (see Figure 2). Since the depth is equal to 72 inches, the chord force and axial stress are 4698 pounds and 3948 psi respectively. This stress is much less than the allowable value of 32,400 psi. Compression buckling is not a problem due to the plates attached to the angle legs. Since the roof plate is 1/8 inch thick, the maximum shear stress is 358 psi. Since the allowable shear stress was calculated at 400 psi[6] it was concluded that the roof will carry the loads to which it is subjected.

The end panels transfer the reaction loads (3221 lbs.) from the roof diaphragm to the base. Shear and bending stresses were calculated on both sides of the door opening and the maximum values were 358 psi (shear) and 1789 psi (bending). Both of these values are less than the allowable values, and the end panels will properly transfer the loads to the floor anchors.

Finally, the connections of the vertical plates, both front and back, to the roof diaphragm and the connections of the roof to the end side panels were also evaluated and found to be acceptable. These connections consist of structural angles assumed to 2-1/2 by 1-1/2 by 1/4 inch welded with 1/4 inch fillet welds one inch long at ten inch spacing. Thus, it is concluded that the roof diaphragm and end side panels are qualified and provide an acceptable load path to the floor slab for the earthquake and dead loads in the out-of-plane direction of each of the plates which comprise the MCB.

4.2 In-Plane Seismic Motion. The stresses in the panels due to the in-plane motions are in two categories. The first category is pure shear and axial stresses in the plates while the second is torsional shear and bending stresses due to the moments induced by the attached cantilevered equipment as shown in Figure 12.

The pure shear and axial stresses have been evaluated on a worst case basis using the equivalent static load method and the heaviest panel weight. The heaviest weight of 2,000 lbs., for the rear right panel, is used with a 3.75g horizontal seismic acceleration and 2.2g vertical acceleration (seismic plus dead load). These loads resulted in acceptable stresses for all in-plane evaluations when combined with the out-of-plane earthquake stresses.

The evaluation of the plate stresses due to the moments induced by the cantilevered equipment was performed for the critical plate strips in each of the six vertical plates and three horizontal benches. The stresses obtained from the out-of-plane direction evaluation using the SRSS combination method. All plates evaluated for these induced moments were found to be acceptable with the exception of the left rear panel and the center bench. These two locations require additional stiffening.

The problem area in the left rear panel is a 2-3/4 inch wide vertical strip located to the left of stiffener 1 (see Figure 8) between two recorders and a distribution panel. This strip is approximately 22 1/4 inches long. It appears that the two recorders might have been moved from a region next to stiffeners to this unstiffened location.

The problem area in the center bench is a two inch horizontal strip, located at the left side of the bench, spanning between the stiffener which is recommended for modification and the second stiffener from the end of the bench (see Figure 13). This strip is 32 inches long and is located between a row of controllers and a row of light switches. The controllers are 15 lbs. each and cantilevered approximately 19 inches.

The modification for these two regions could be either in the form of adding additional stiffeners to the plates or adding additional support to the cantilevered instrument to reduce the induced moment.

5. CONCLUSIONS

The Main Control Board at the R.E. Ginna Station has been shown to meet the acceptance criteria for the SSE postulated at the site after some minor modifications are installed. This earthquake was defined by the response spectra shown in Figures 2 to 4. The load path for the inertial forces has been evaluated and found to be adequate except as noted below. The anchorage of the instruments to the panels and the anchorage of the MCB to the floor was not included in this study. These evaluations were previously performed by Rochester Gas and Electric and proper modifications were installed.

The following modifications are recommended as a result of this study.

1. Addition of a vertical stiffener to a 1-inch wide vertical plate strip in the middle of Sections B and E of the rear right panel (see Figure 6).
2. Increasing the capacity of vertical stiffener 1 and the 1-1/2 inch wide vertical strip to the left of stiffener 1 of the center rear panel (see Figure 7).
3. Re-support the two recorders at the left edge of the left rear panel or stiffen up the 2-3/4 inch wide vertical strip between the recorders and the distribution panel (see Figure 8).
4. Extend the left most stiffener of the center bench the entire width of the bench (see Figure 13).
5. Re-support the controllers on the left side of the center bench between the stiffener described in 4 above and the next one, or stiffen up the 2-inch wide horizontal strip right above the controllers (see Figure 13).
6. Add connection plates between the adjacent sections of the MCB on the vertical panels and the roof plates at the miter junctions of the MCB. These connections are needed to insure the proper support of the roof diaphragm and validate the deep beam method of evaluation for the roof.

The six modifications described above will result in a structural integrity qualification of the MCB. They are not expected to drastically change the fundamental mode (and maybe even the second mode) of vibration of the majority of the MCB as described in the appendix.

REFERENCES

1. Newmark, N.M., and Hall, W.J., *Development of Criteria for Seismic Review of Selected Nuclear Power Plants,* prepared for U.S. Nuclear Regulatory Commission, NUREG/CR-0098, May, 1978.

2. URS/John A. Blume & Associates, Engineers, *Analytical Techniques, Models, and Seismic Evaluation of Electrical Raceway Systems,* prepared for the SEP Owners Group under the direction of KMC, Inc., Washington, D.C., August, 1983.

3. U.S. Nuclear Regulatory Commission, *Regulatory Guide 192, Combining Modal Responses and Spatial Components in Seismic Response Analysis,* Revision 1, Washington, D.C., February 1976.

4. U.S. Nuclear Regulatory Commission, *Standard Review Plan,* Section 3.8.4, Office of Nuclear Reactory Regulation, NUREG 0800, Washington, D.C., July, 1981.

5. American Institute of Steel Construction, *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,* New York, New York, 1969.

6. American Iron and Steel Institute, *Cold Form Steel Design Manual,* 1977 Edition, New York

APPENDIX A

MODAL TESTING RESULTS

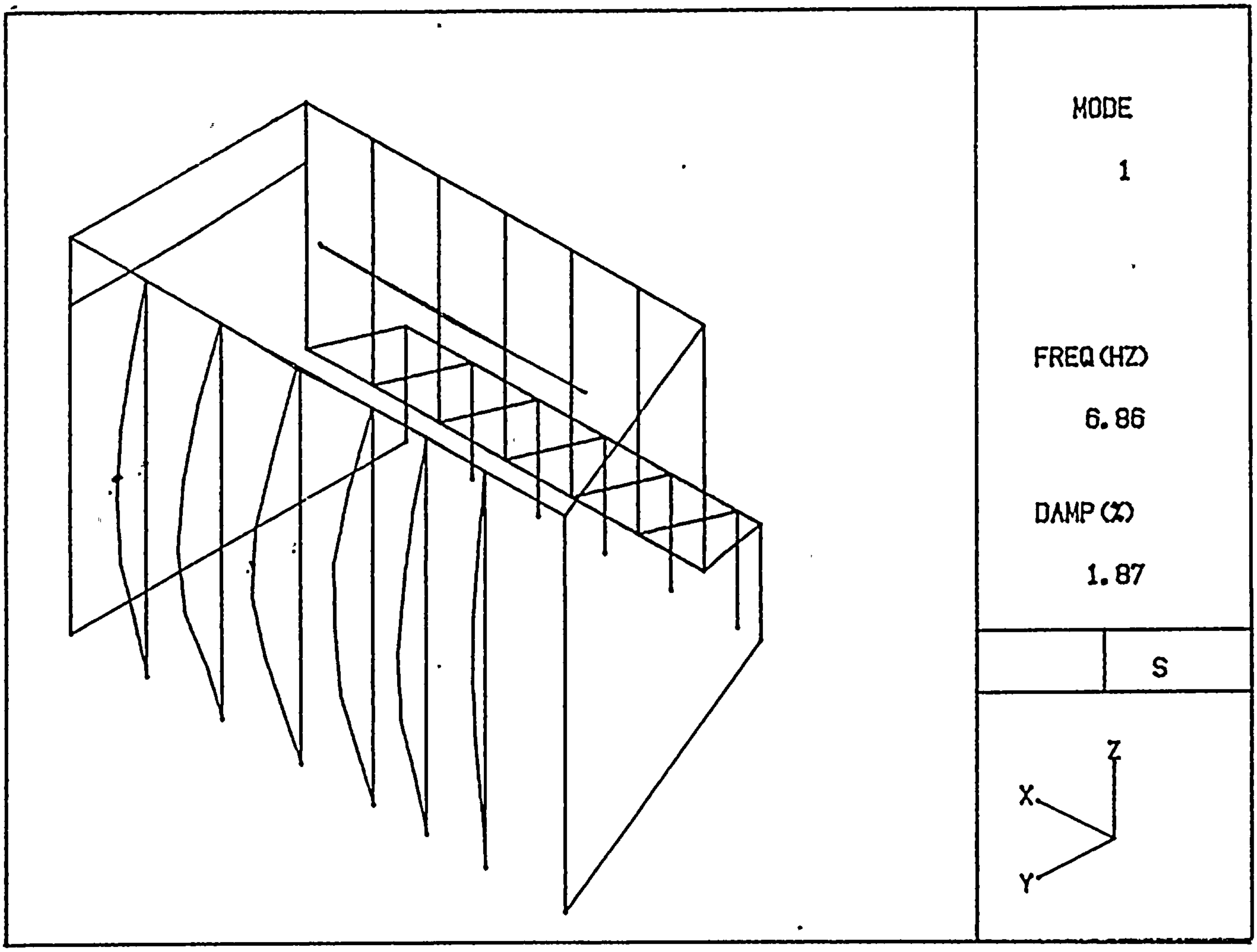


FIGURE A.1

MODE 1 RIGHT REAR MCB

R.E. GINNA PLANT

URS/Blume

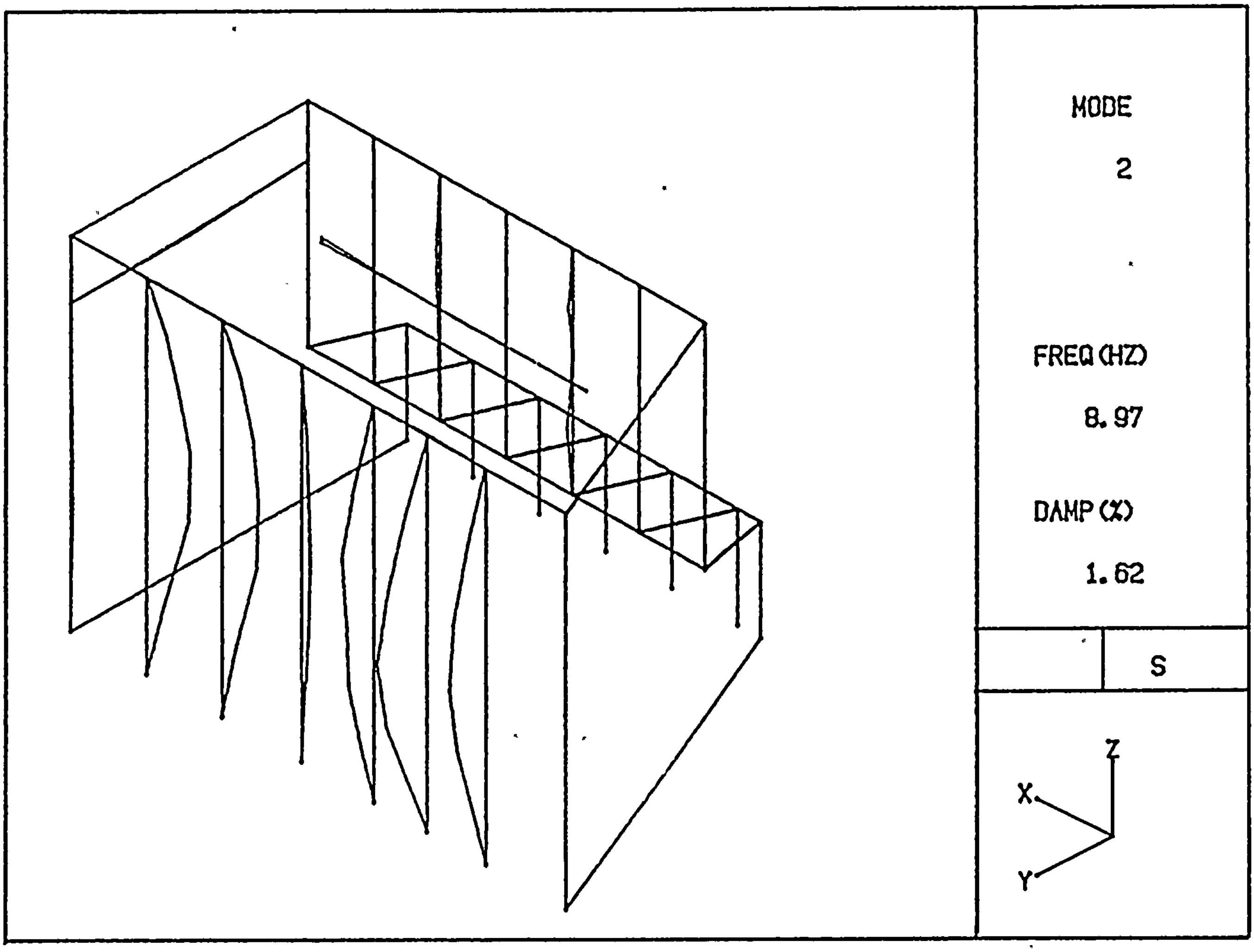


FIGURE A.2
MODE 2 RIGHT REAR MCB
R. E. GINNA PLANT

URS/Blume

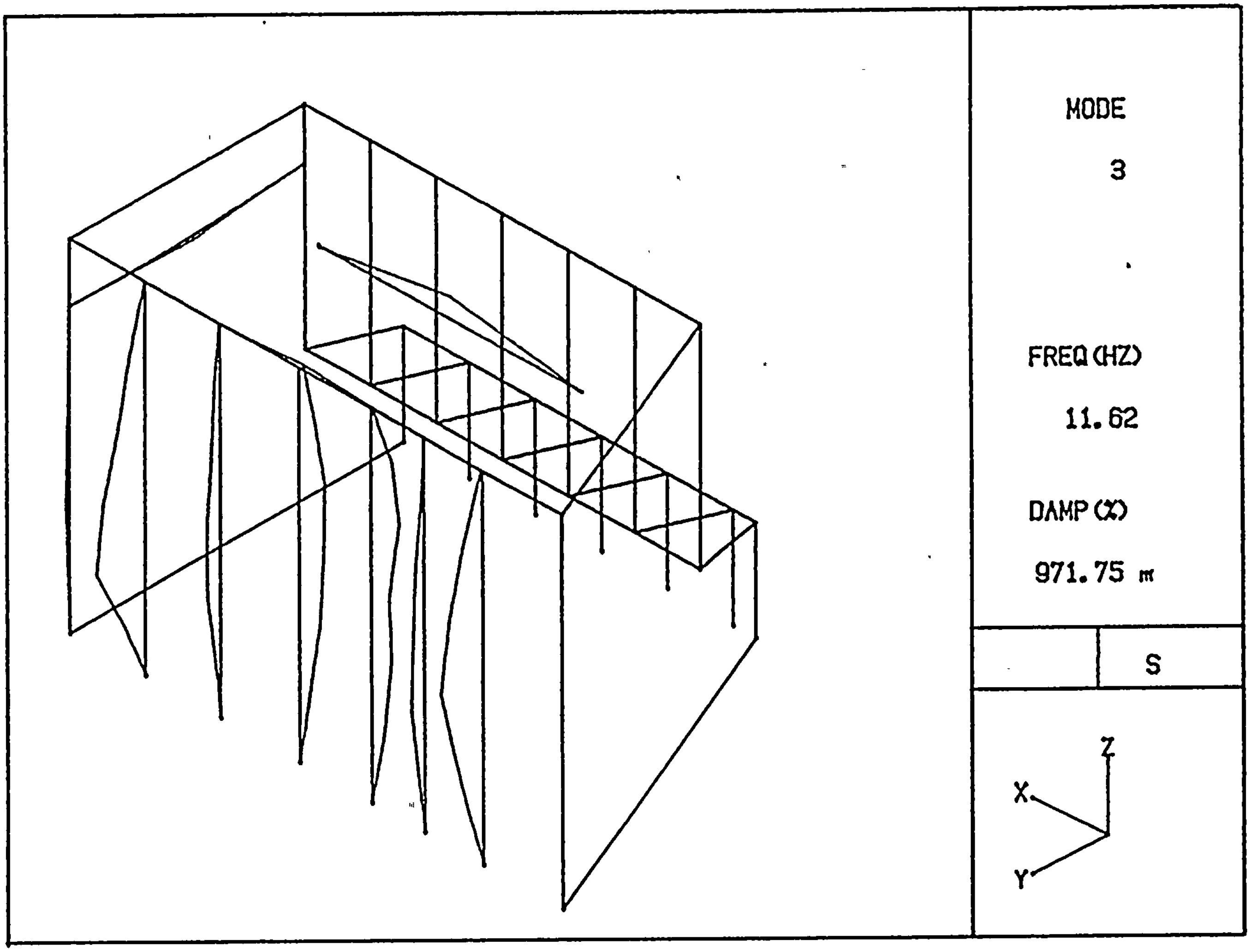


FIGURE A.3

MODE 3 RIGHT REAR MCB

R. E. GINNA PLANT

URS/Blume

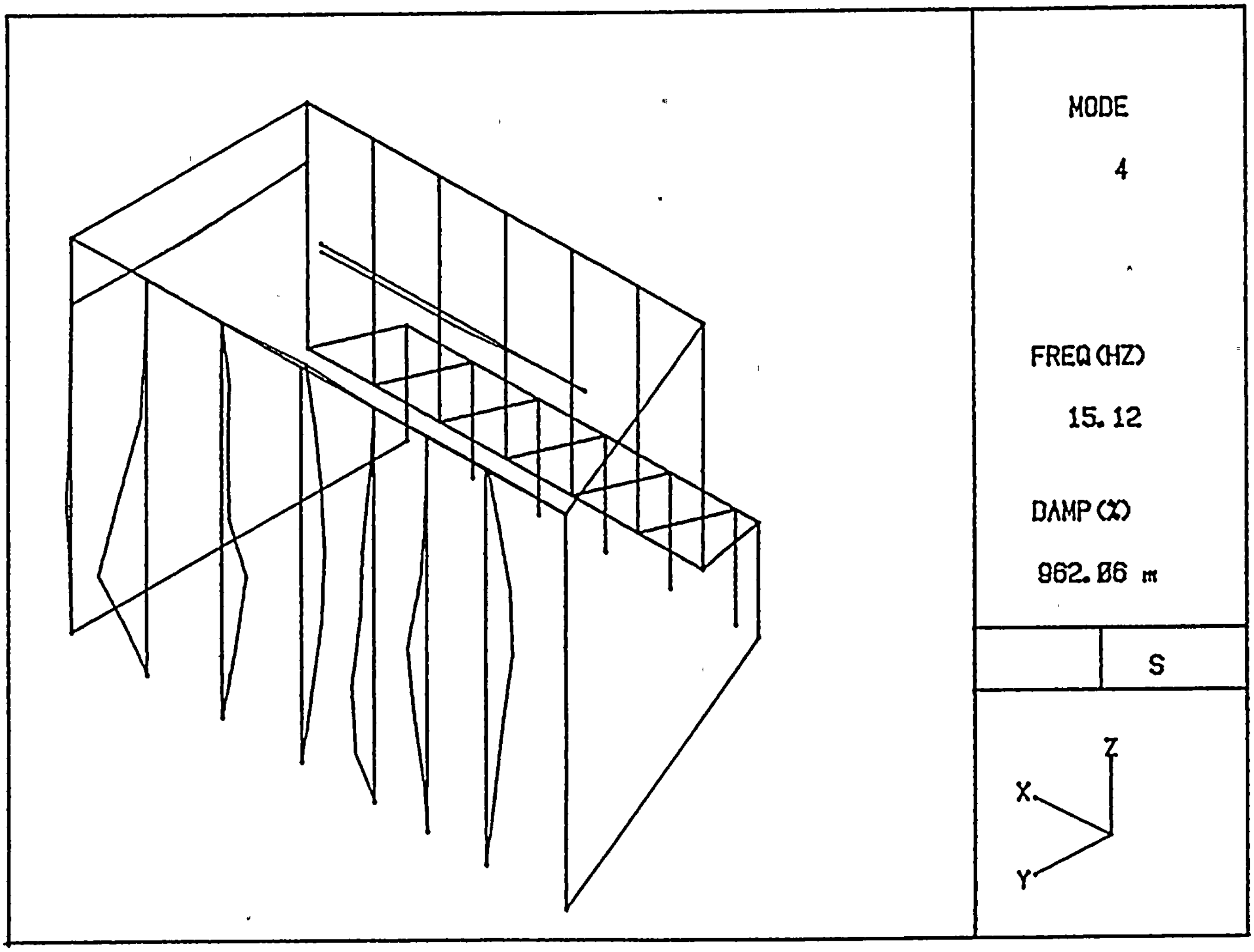


FIGURE A.4

MODE 4 RIGHT REAR MCB

R. E. GINNA PLANT

URS/Blume

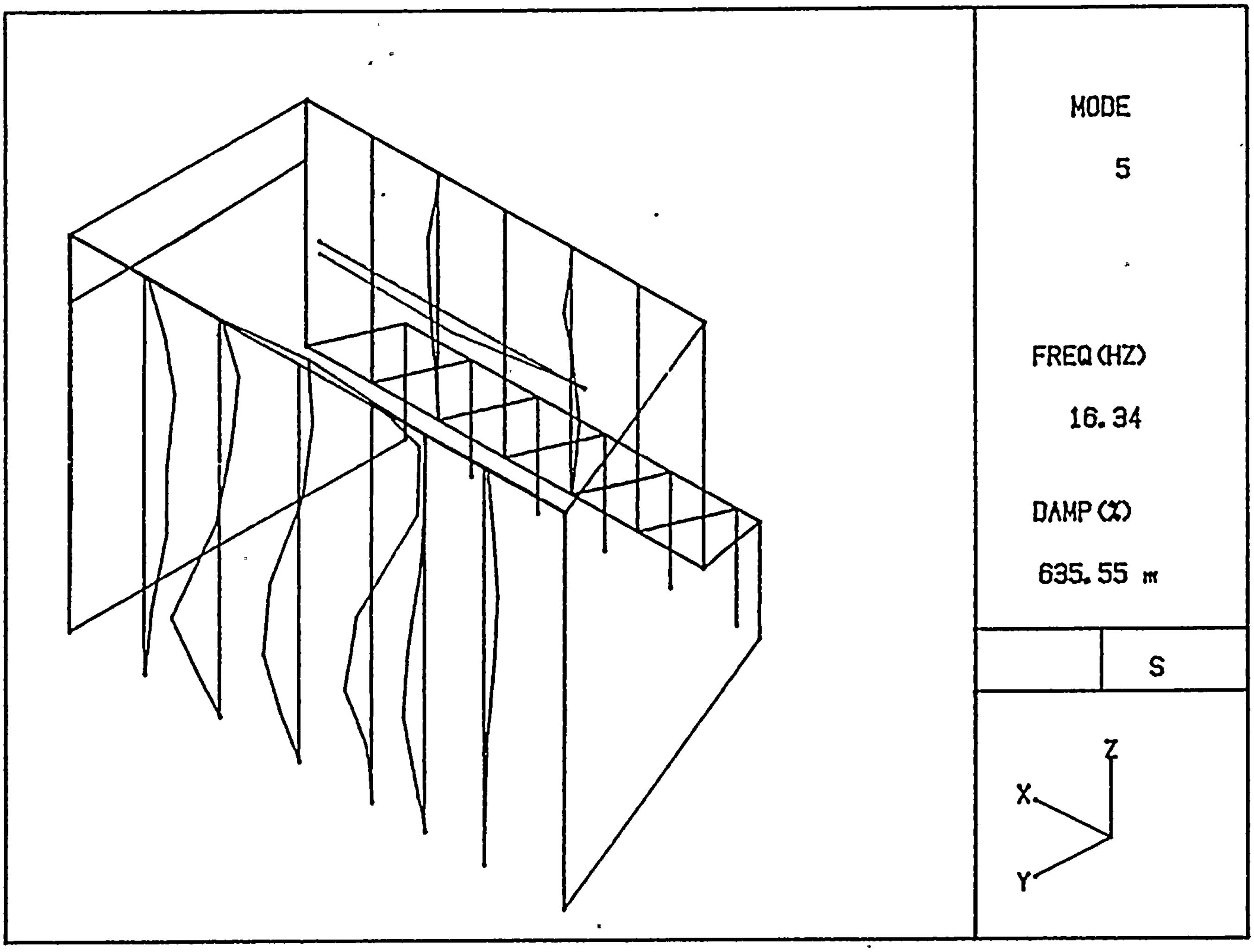


FIGURE A.5
MODE 5 RIGHT REAR MCB
R. E. GINNA PLANT

URS/Blume

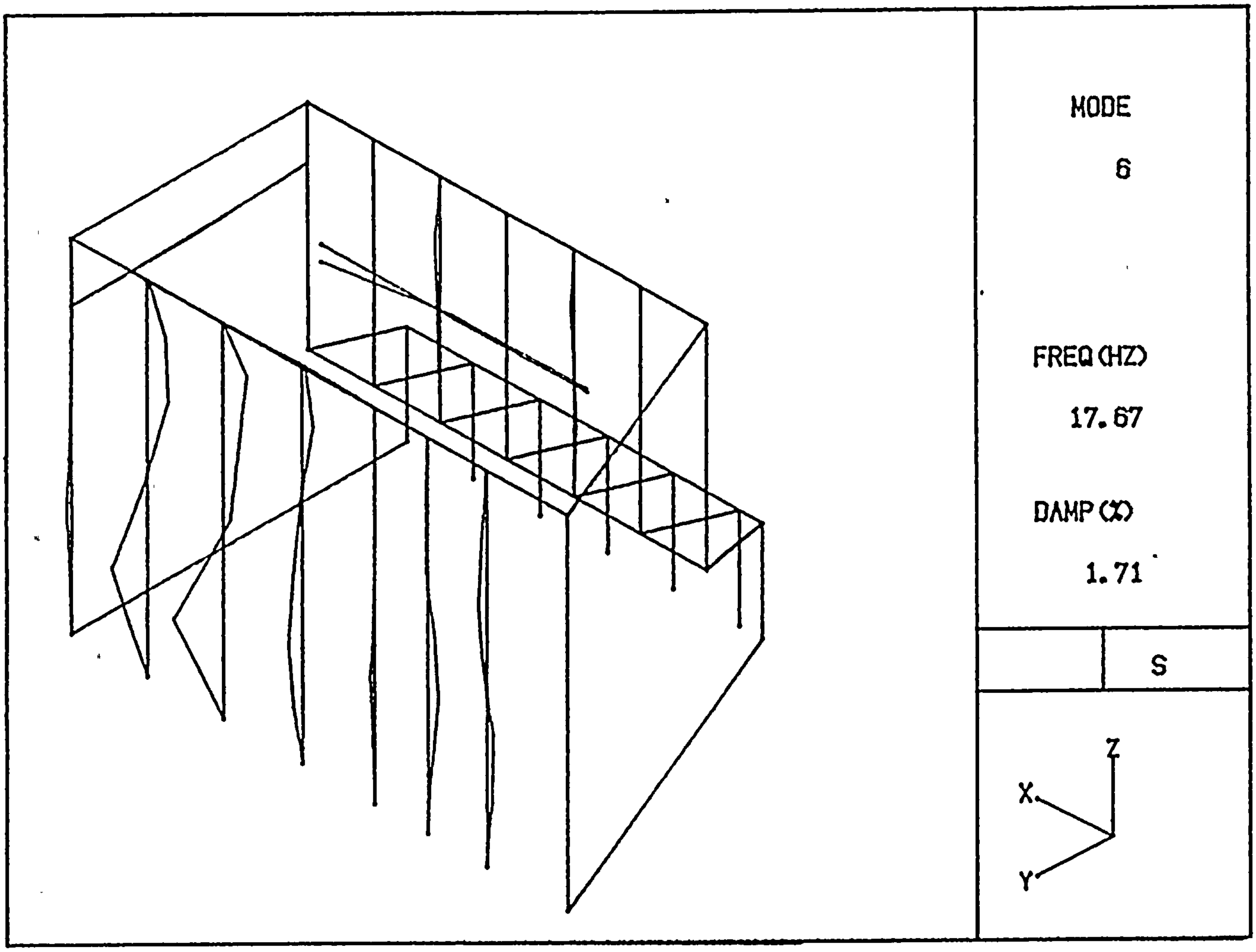


FIGURE A.6

MODE 6 RIGHT REAR MCB

R. E. GINNA PLANT

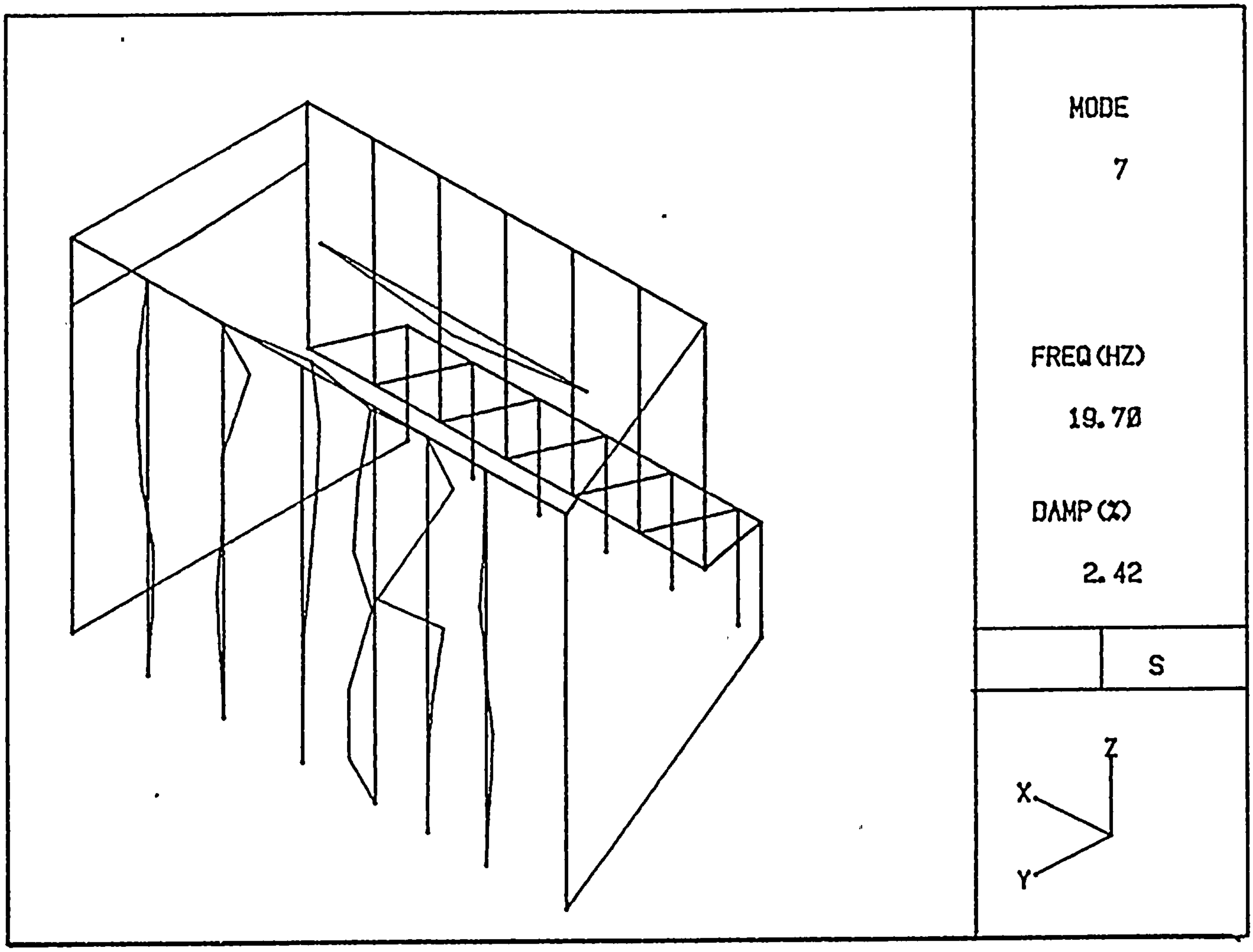


FIGURE A.7

MODE 7 RIGHT REAR MCB

R. E. GINNA PLANT

URS/Blume

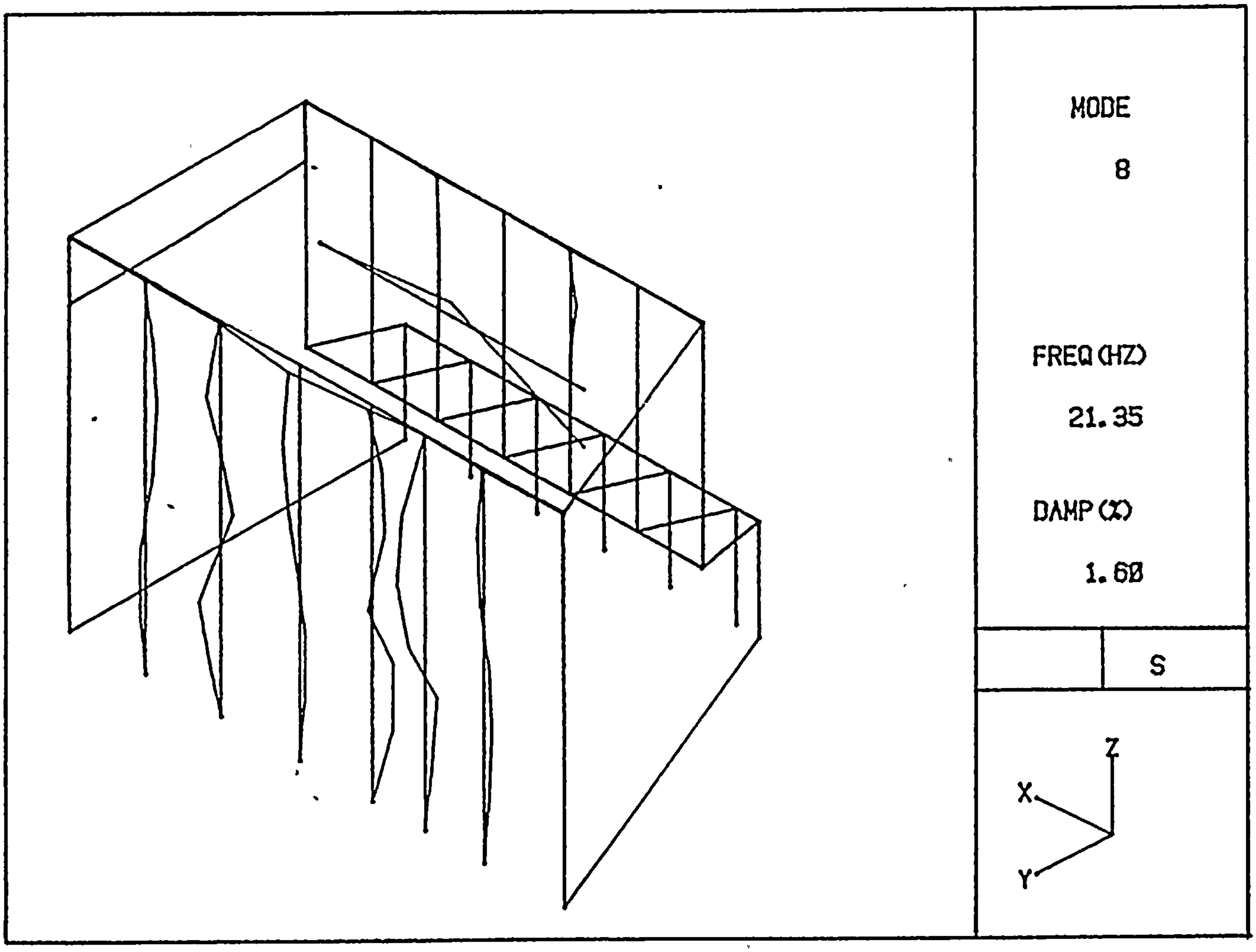


FIGURE A.8

MODE 8 RIGHT REAR CONTROL PANEL

R. E. GINNA PALNT

URS/Blume

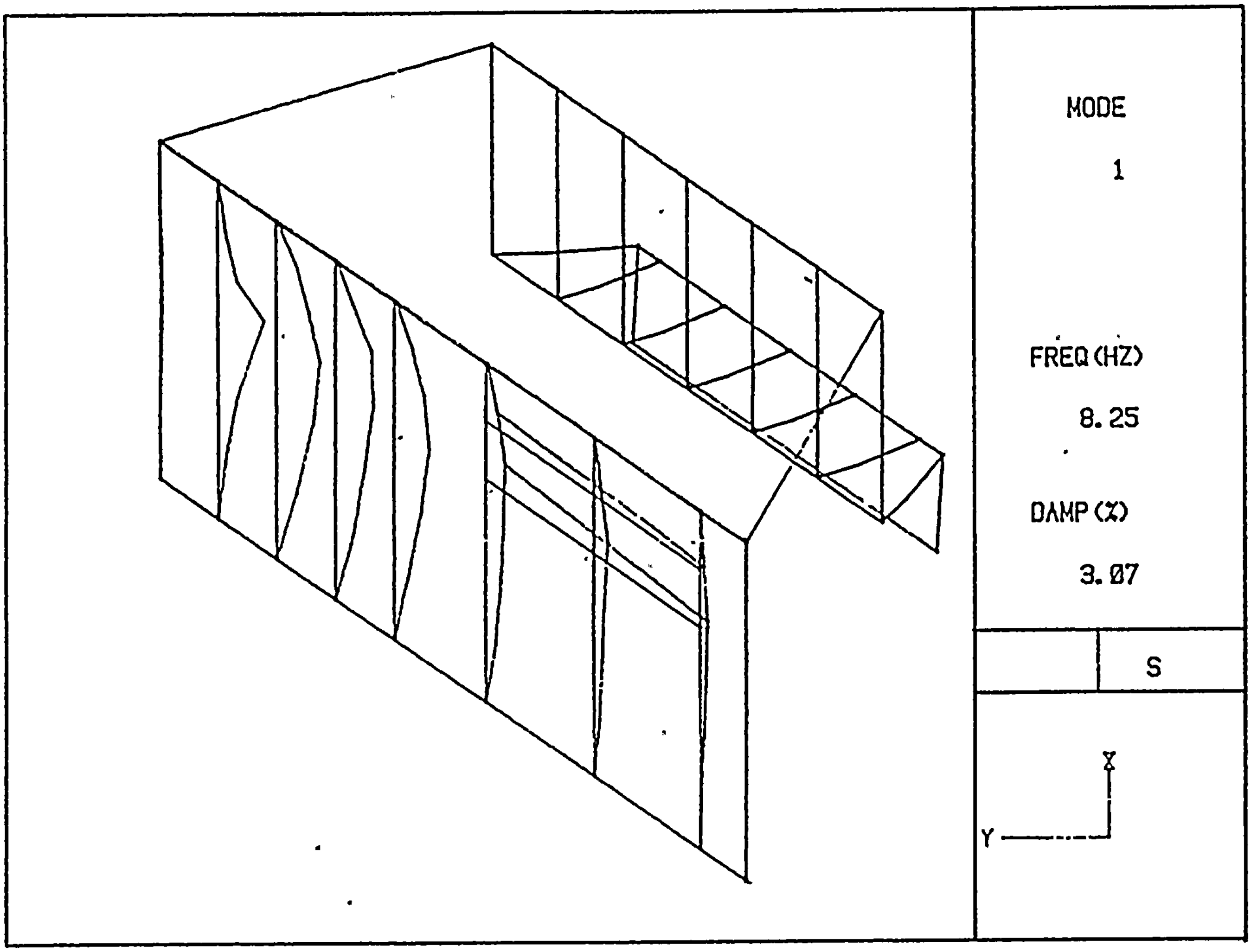


FIGURE A.9

MODE 1 CENTER REAR CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

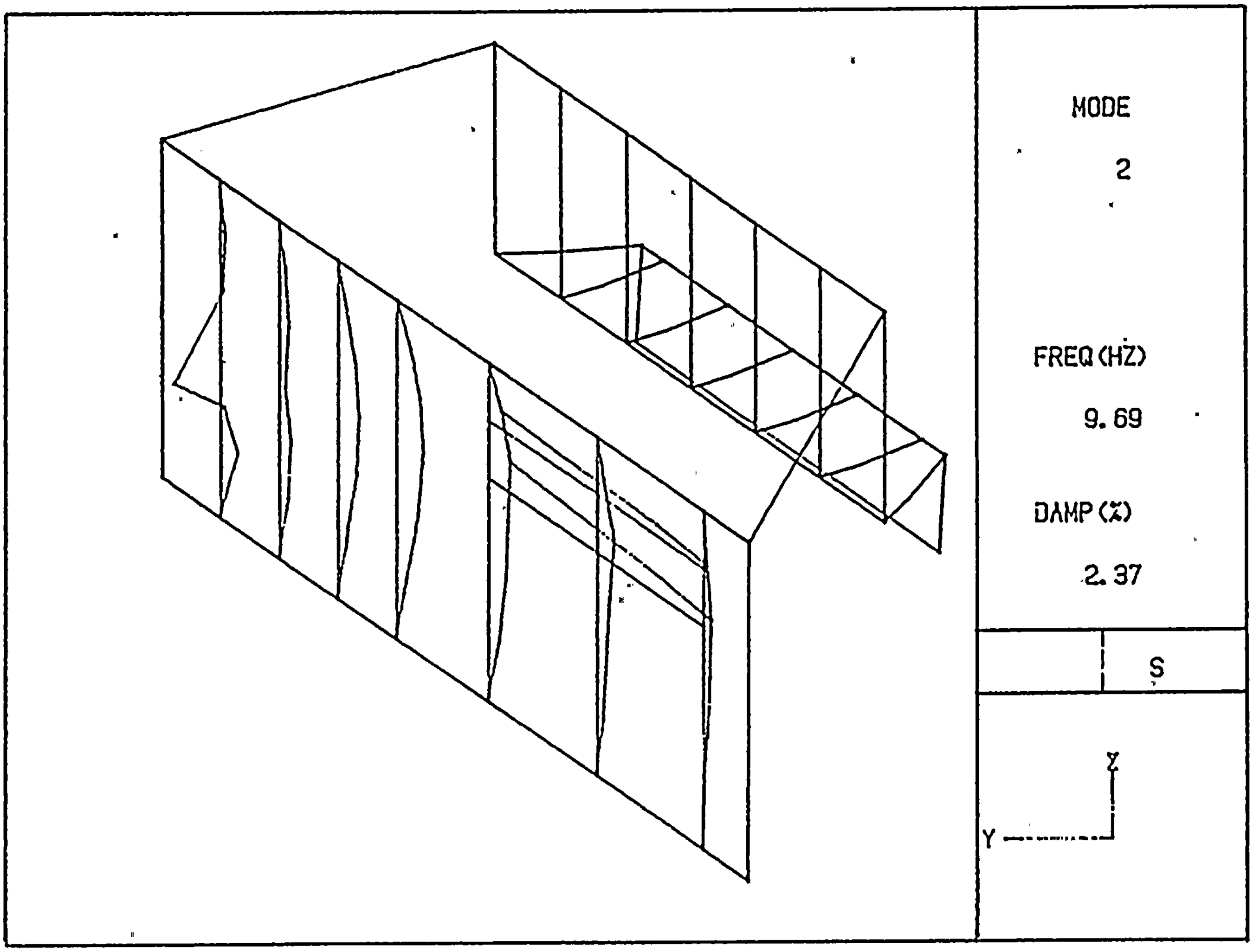


FIGURE A.10
MODE 2 CENTER REAR CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

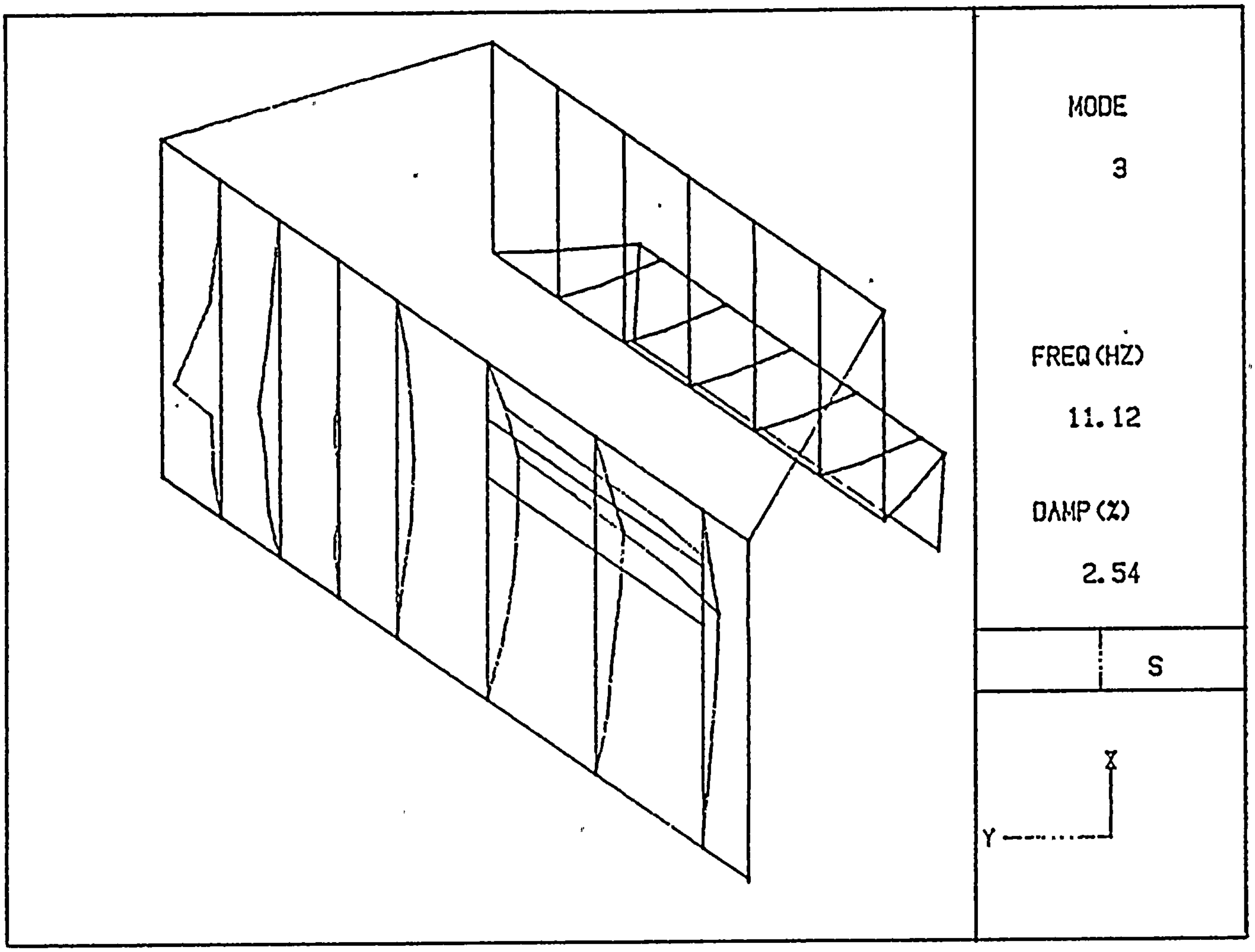


FIGURE A.11
MODE 3 CENTER REAR CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

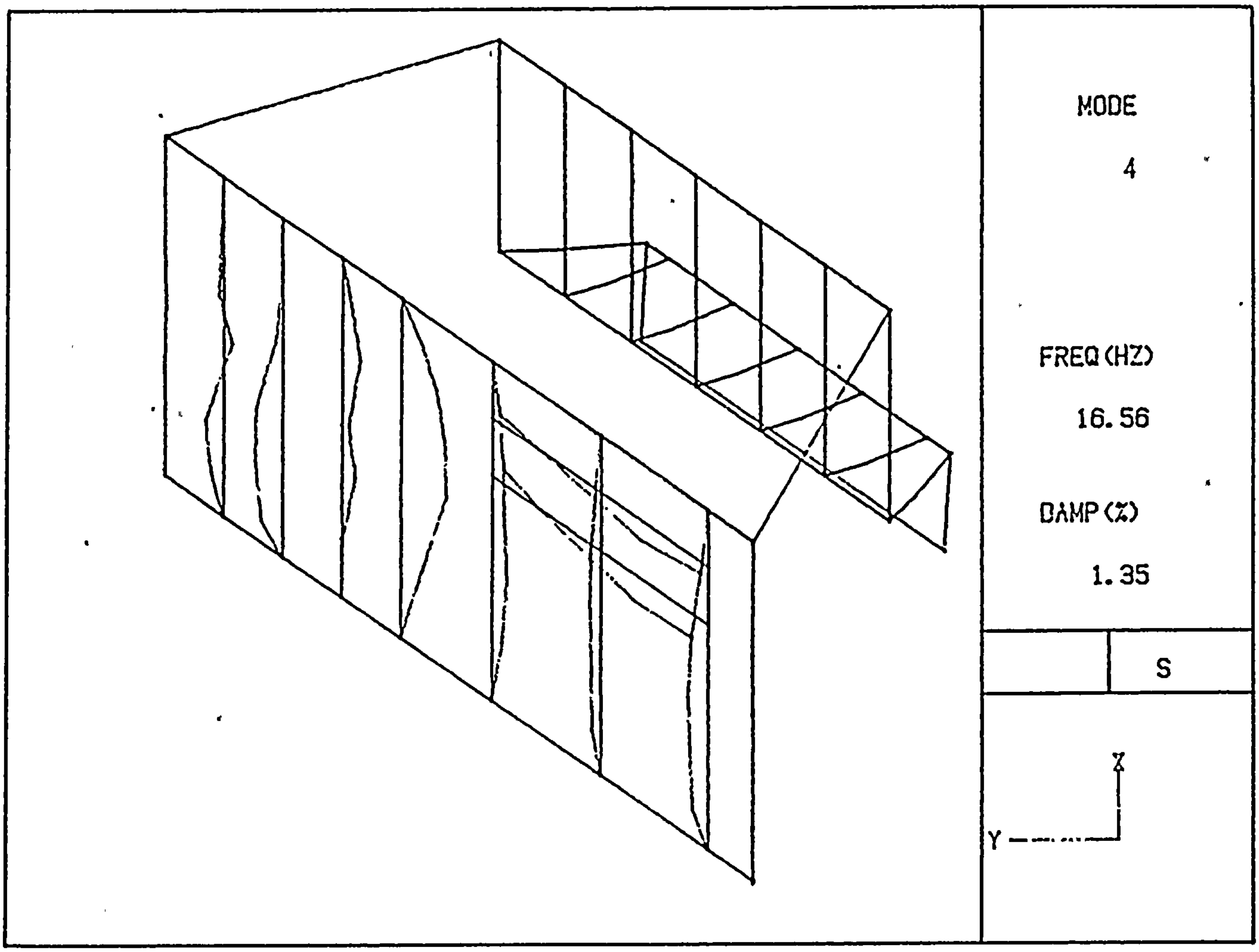


FIGURE A.12

MODE 4 CENTER REAR CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

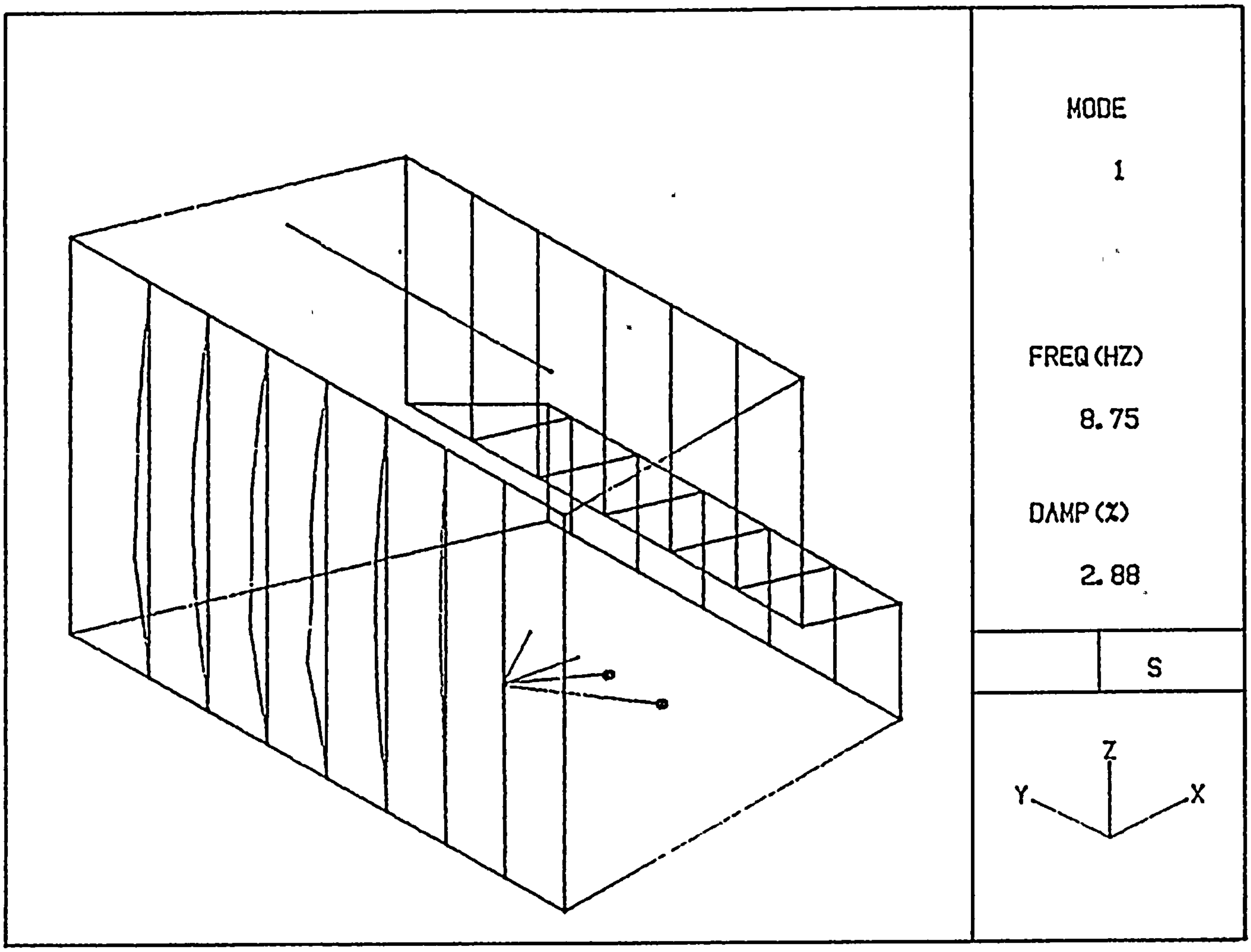


FIGURE A.13

MODE 1 LEFT REAR CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

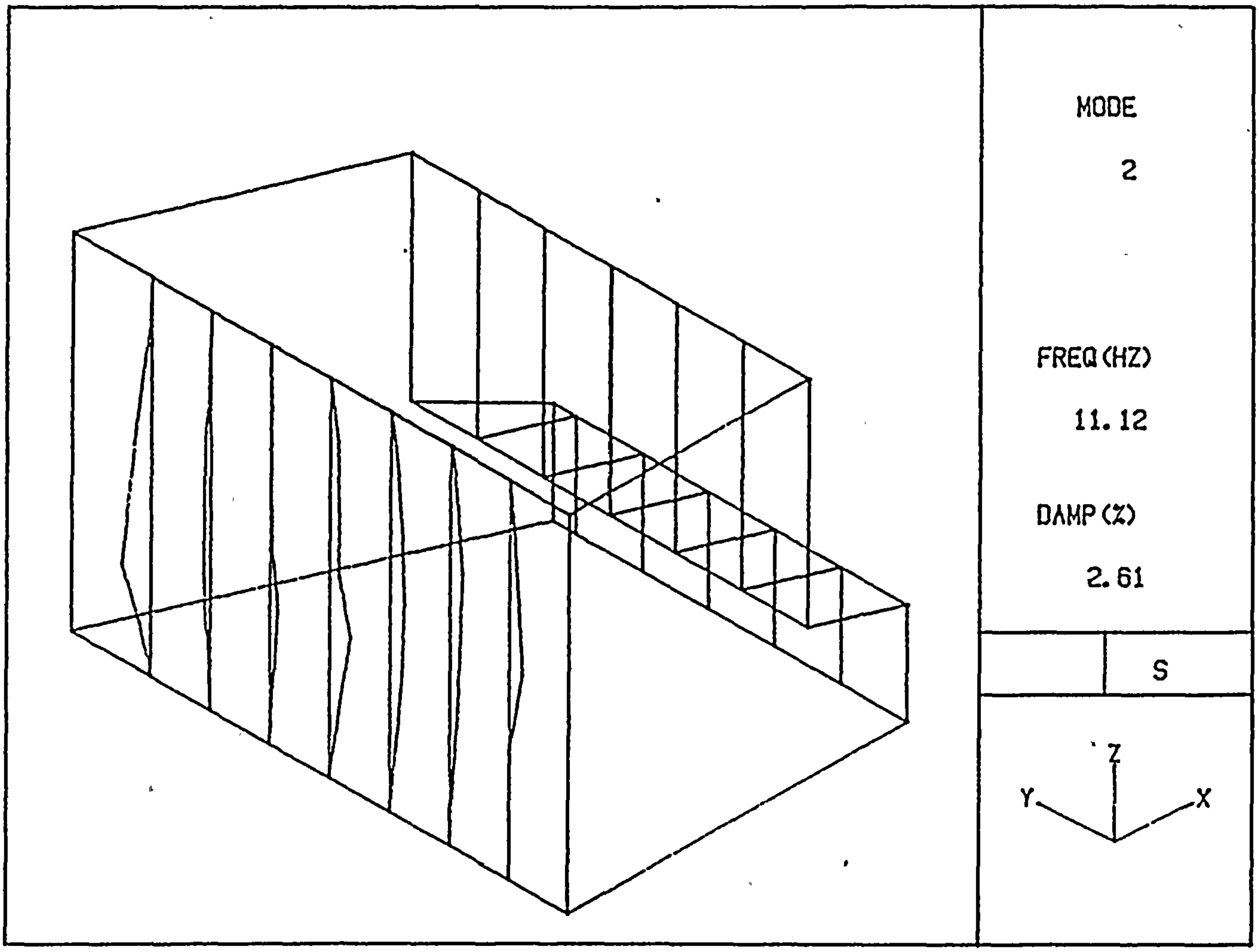


FIGURE A.14
MODE 2 LEFT REAR CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

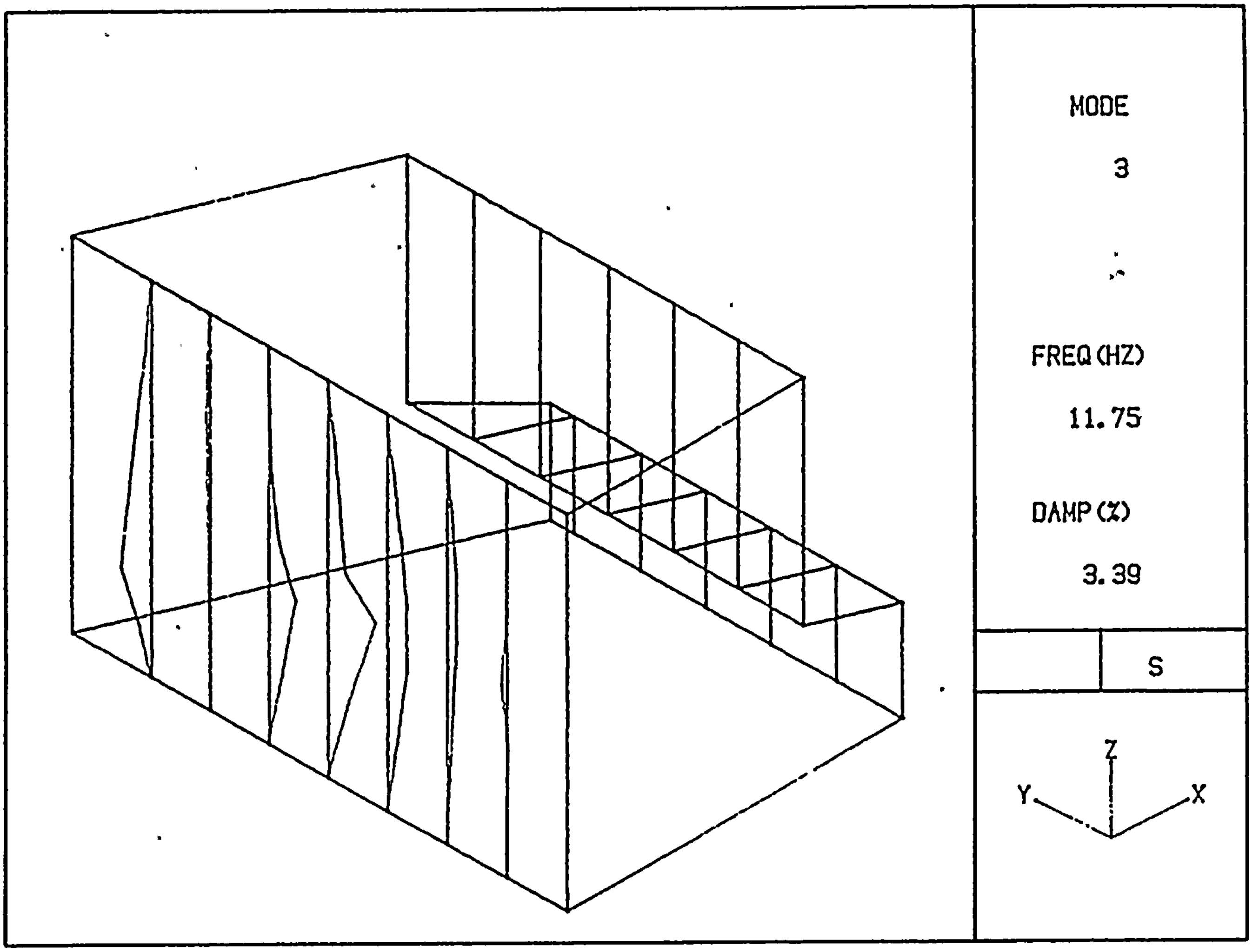


FIGURE A.15
MODE 3 LEFT REAR CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

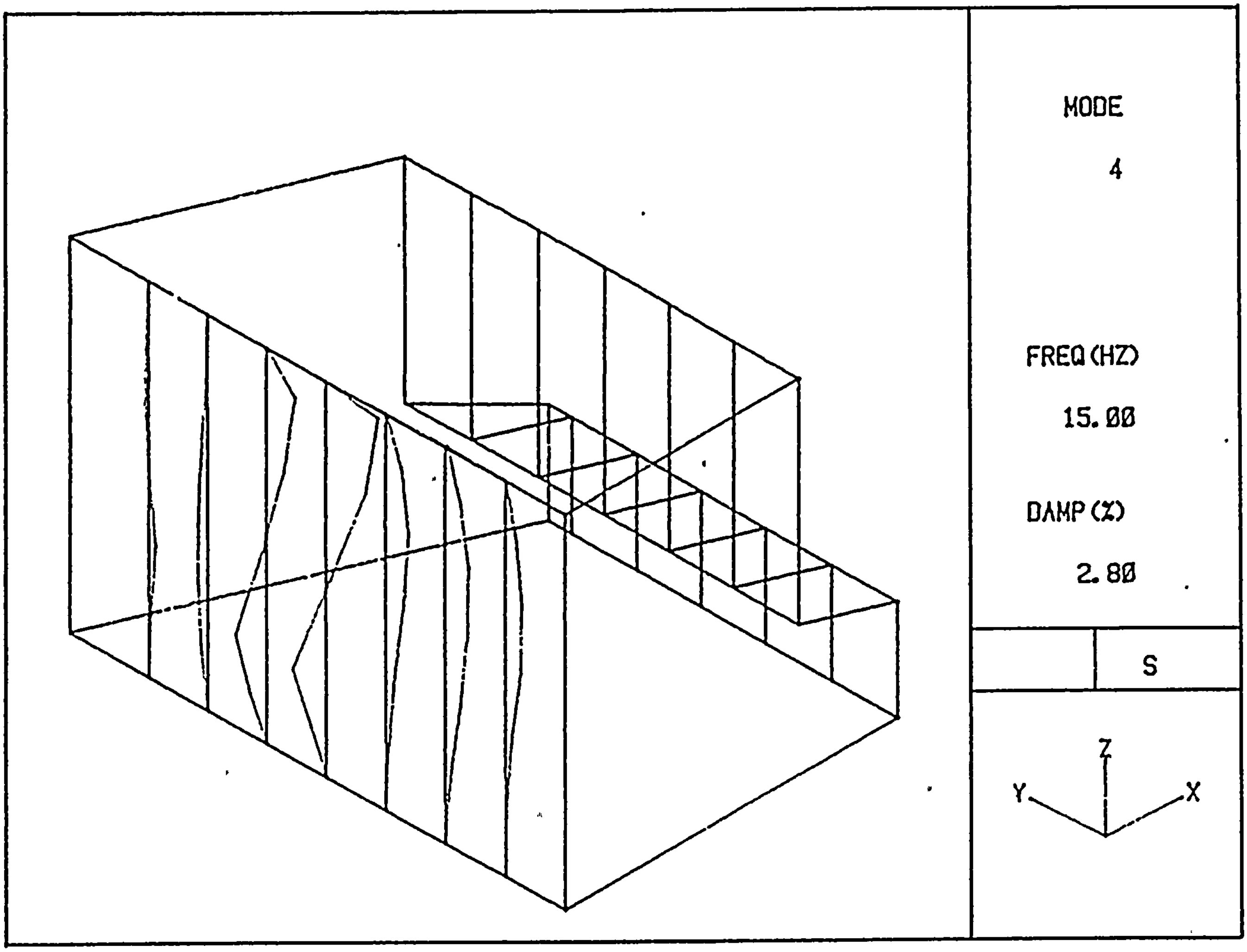


FIGURE A.16
MODE 4 LEFT REAR CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

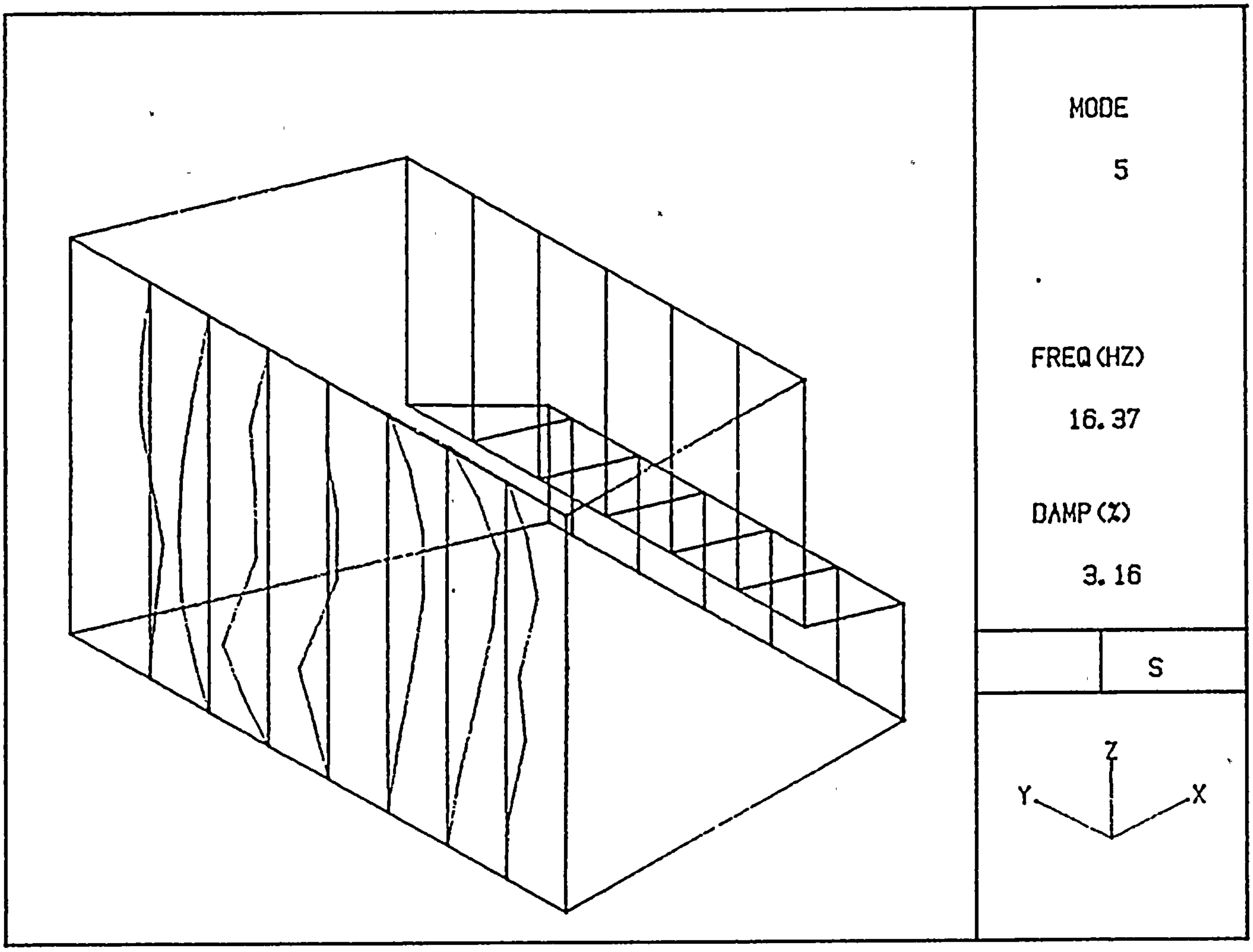


FIGURE A.17

MODE 5 LEFT REAR CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

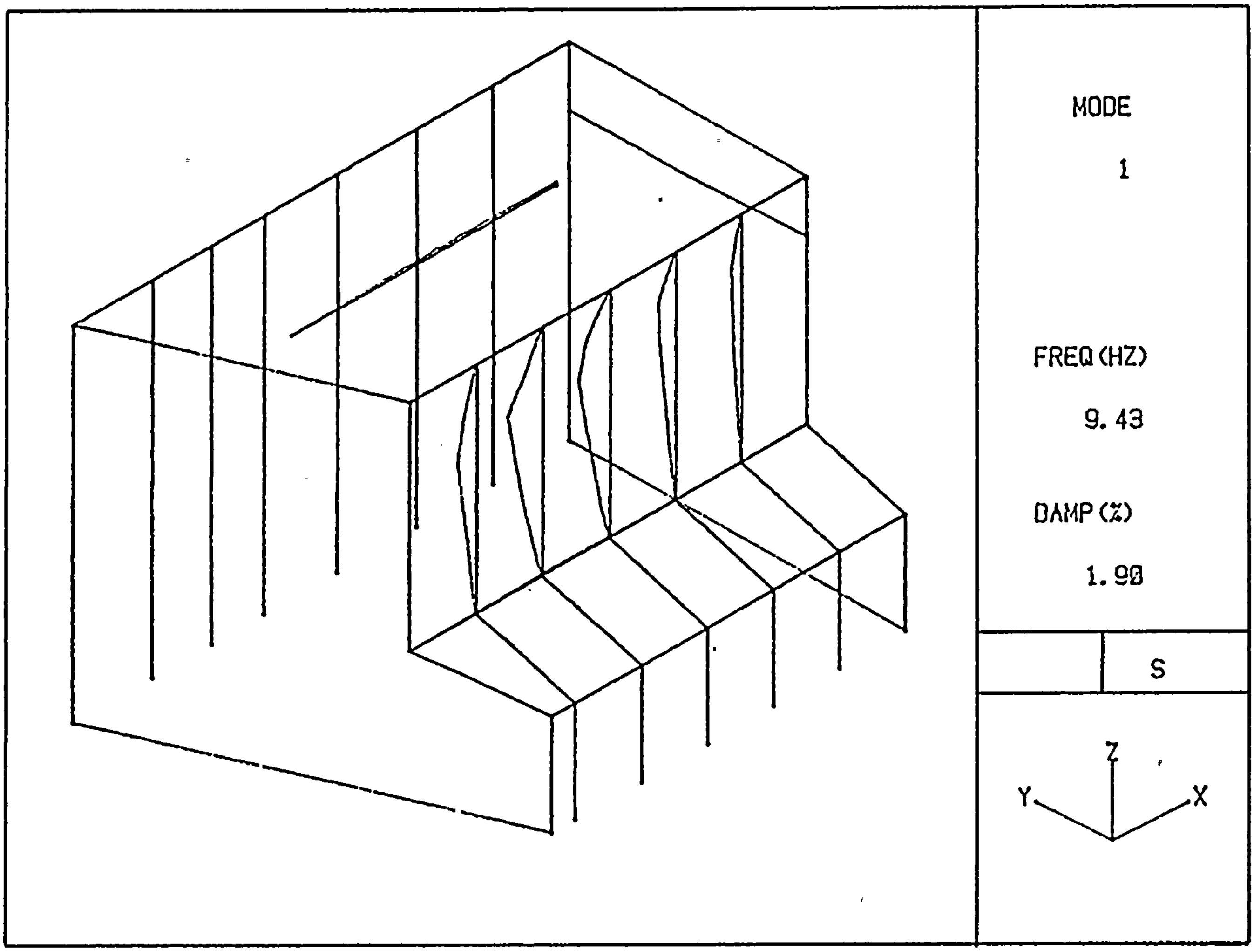


FIGURE A.18

MODE 1 RIGHT FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

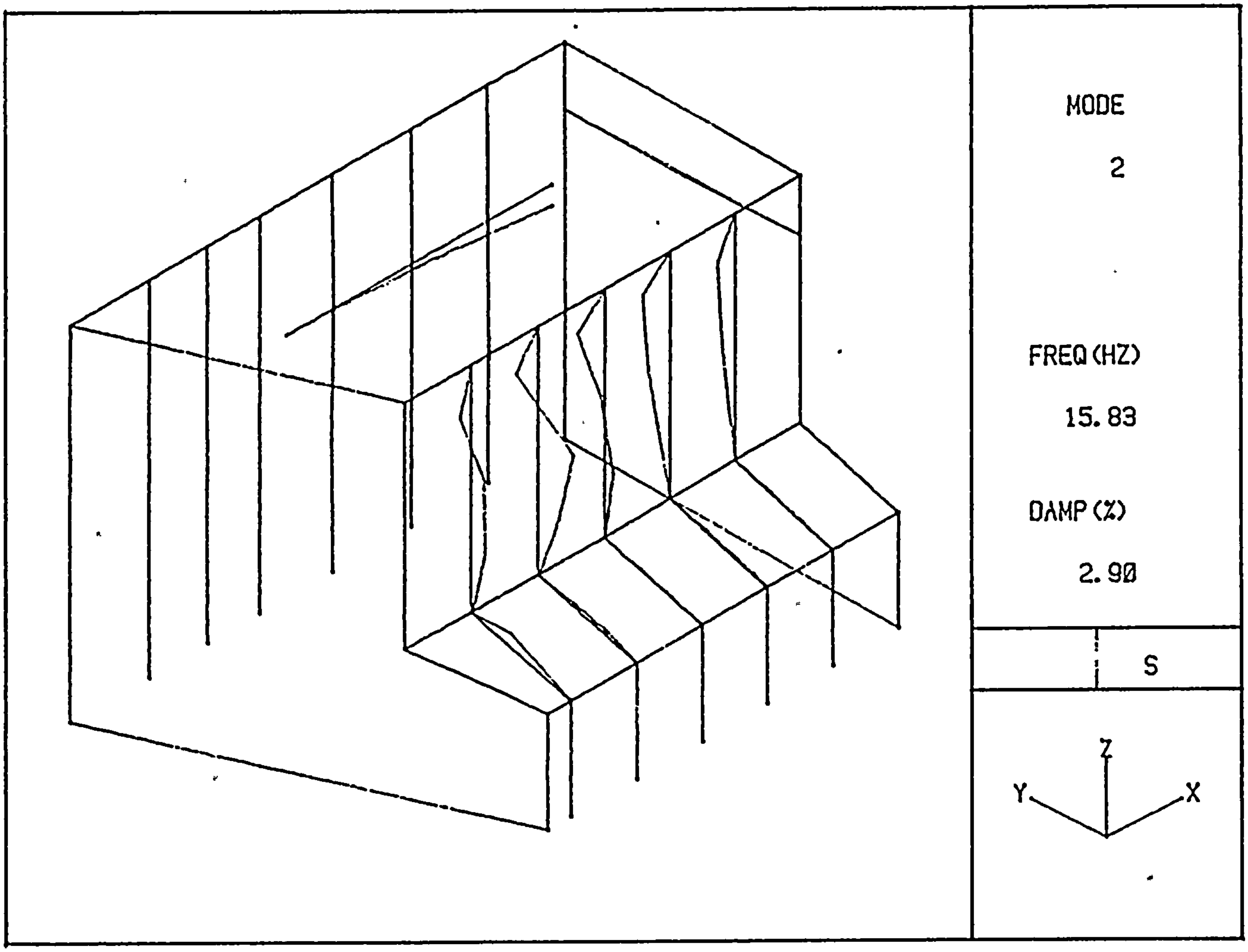


FIGURE A.19

MODE 2 RIGHT FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

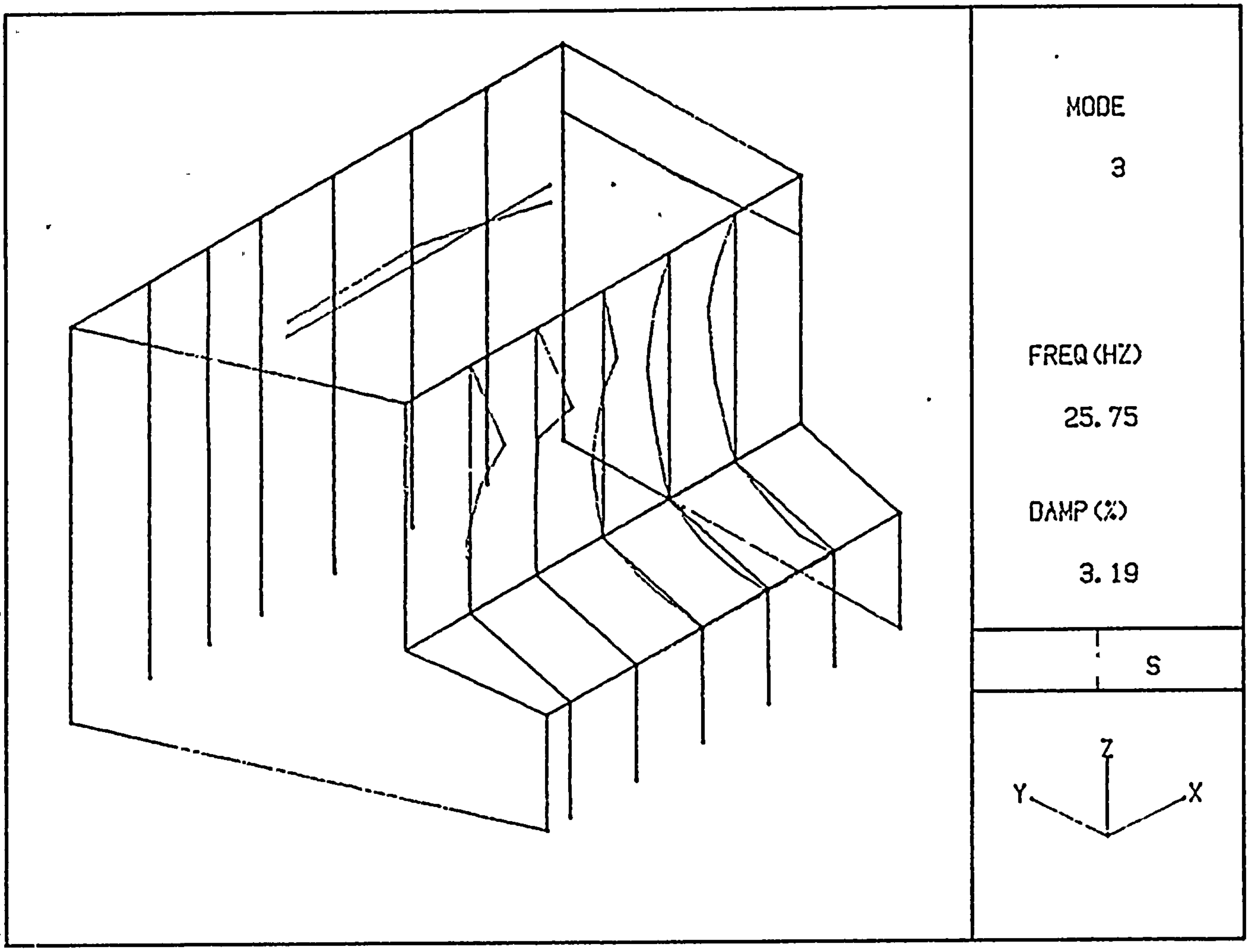


FIGURE A.20

MODE 3 RIGHT FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

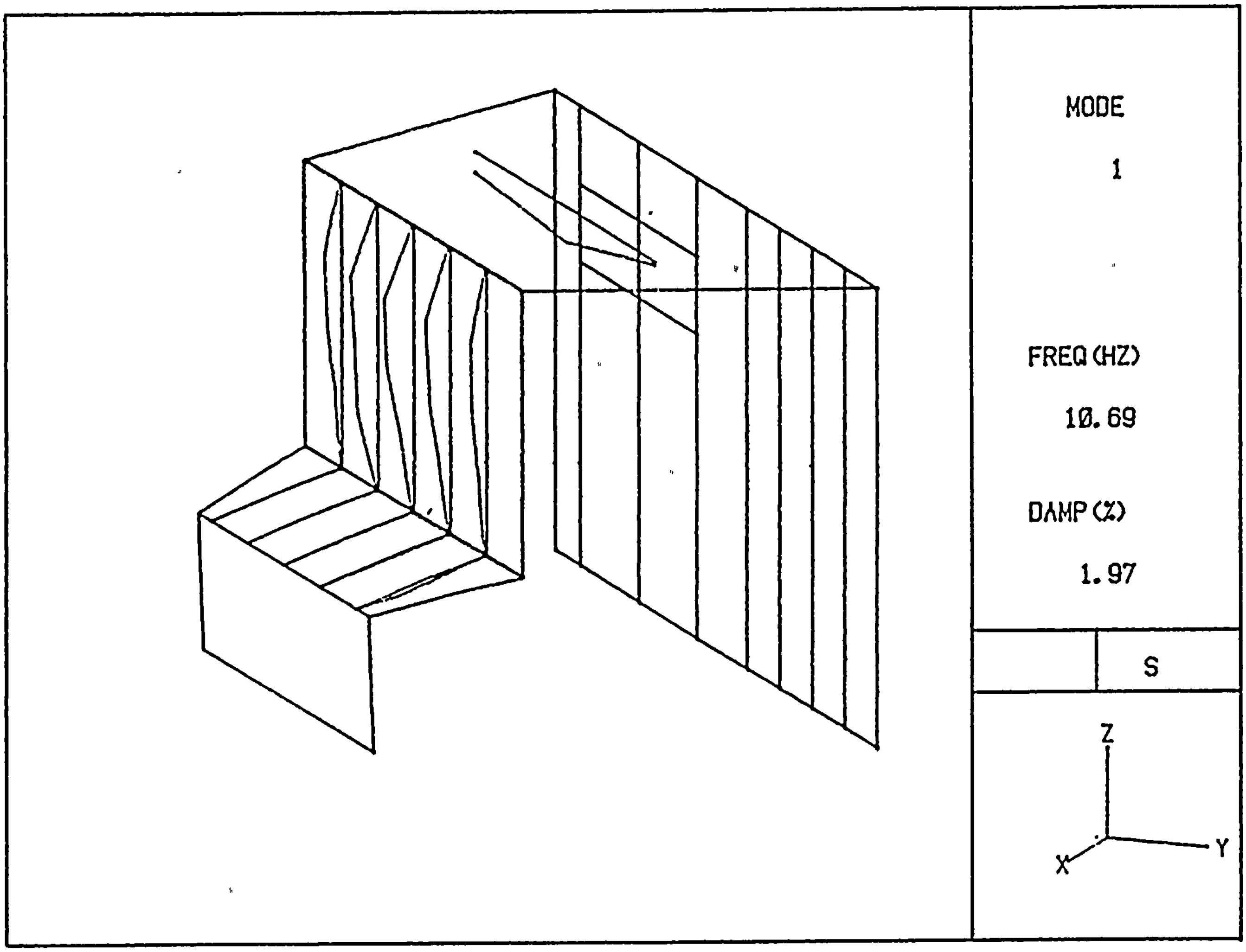


FIGURE A.21

MODE 1 CENTER FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

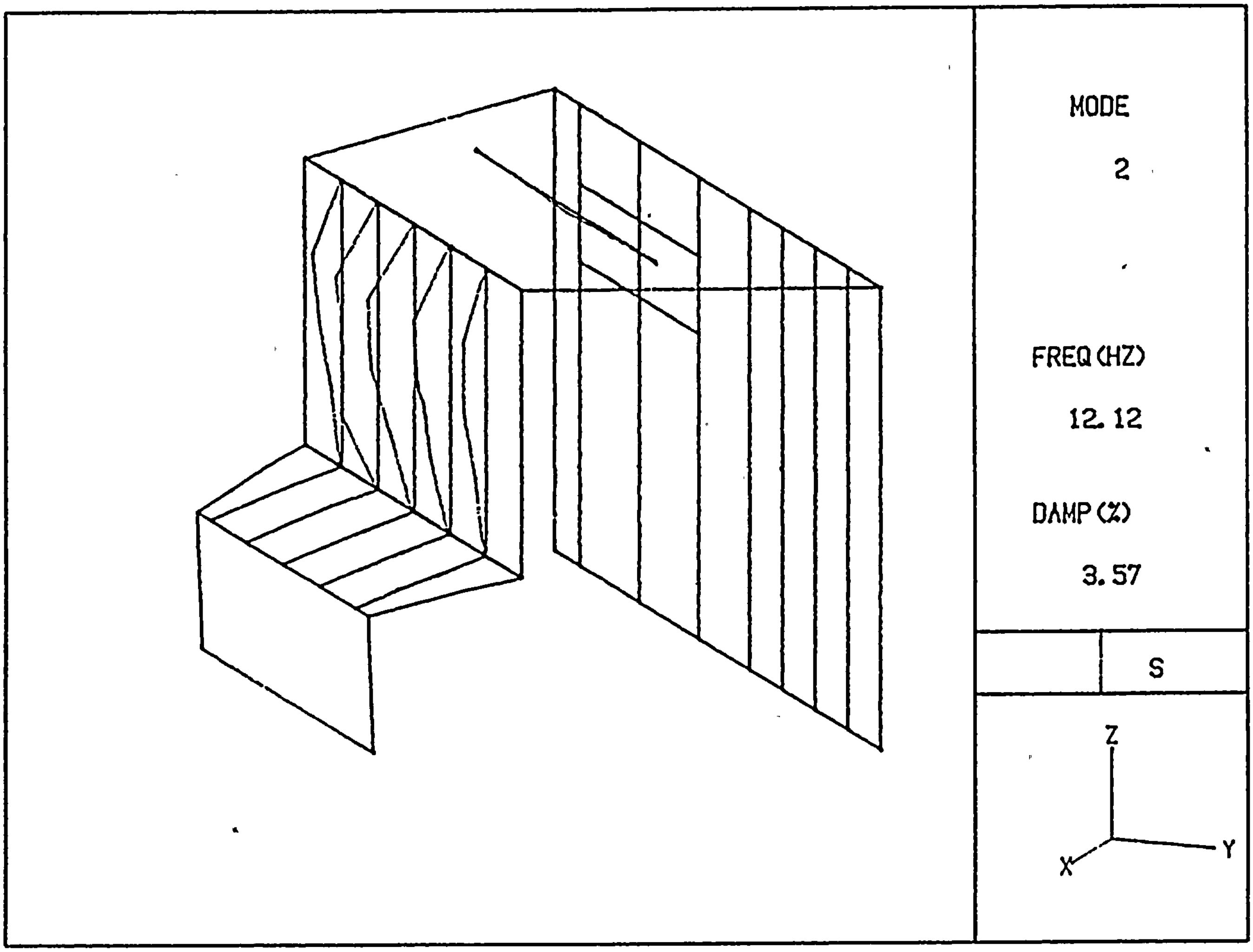


FIGURE A.22
MODE 2 CENTER FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

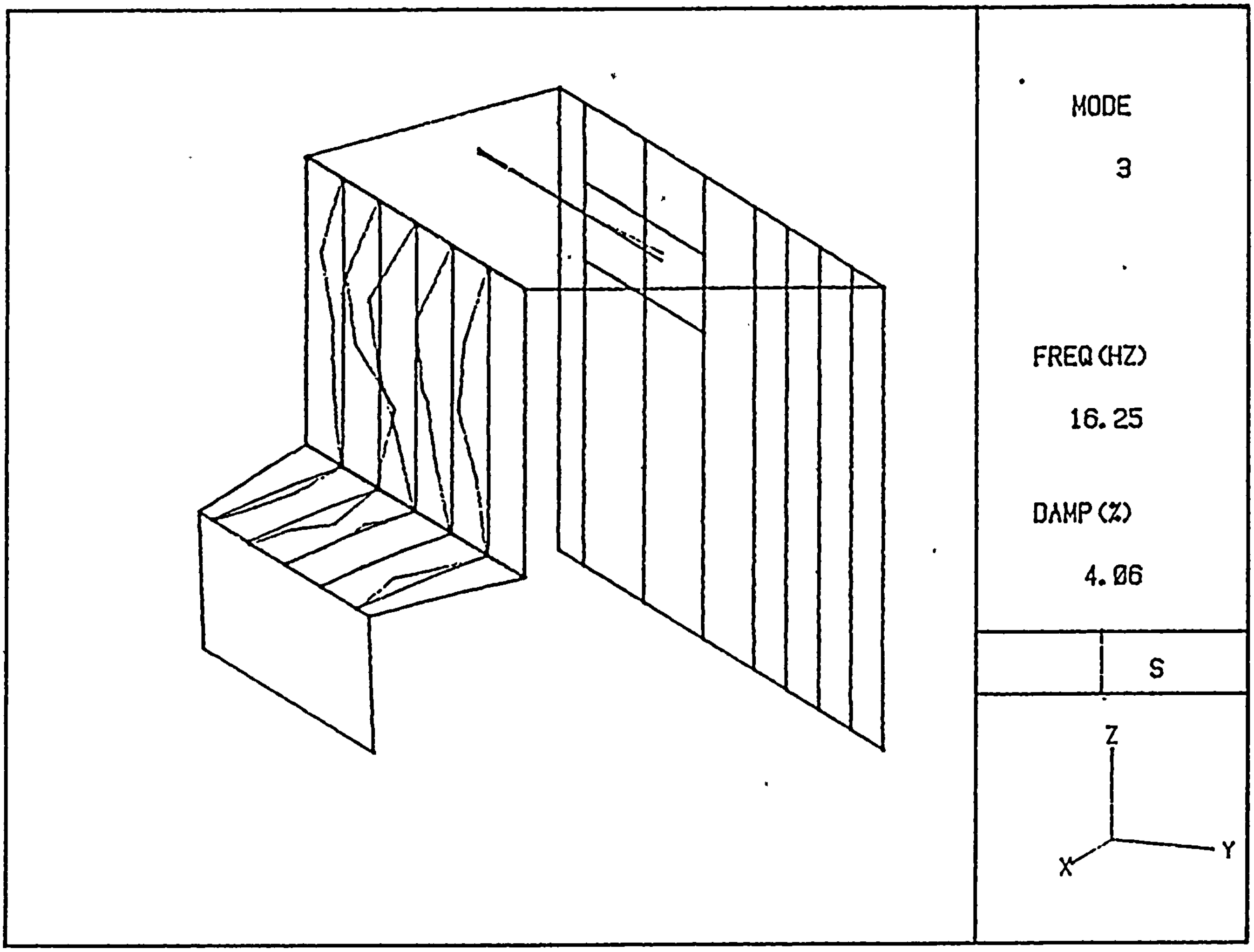


FIGURE A.23
MODE 3 CENTER FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

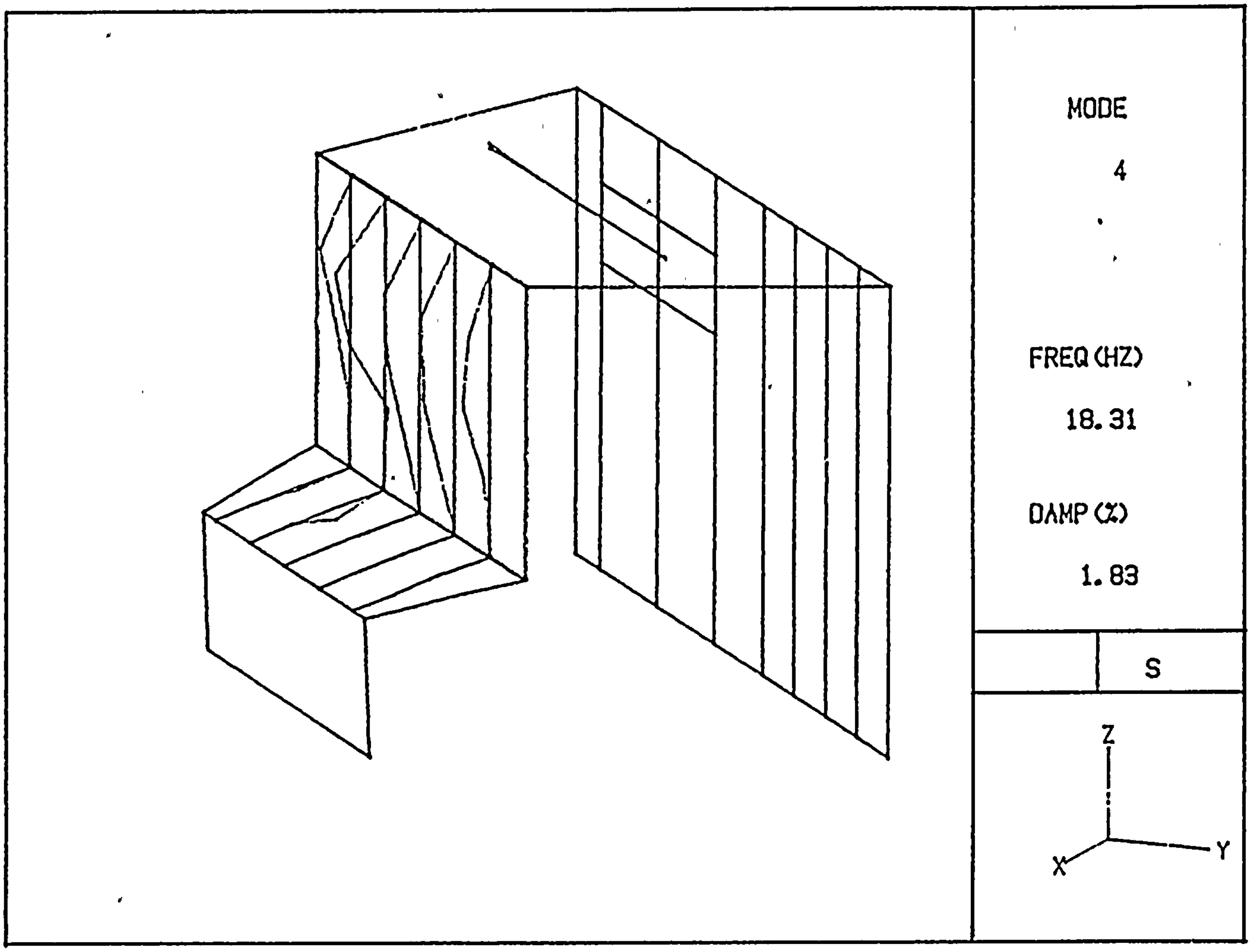


FIGURE A.24

MODE 4 CENTER FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

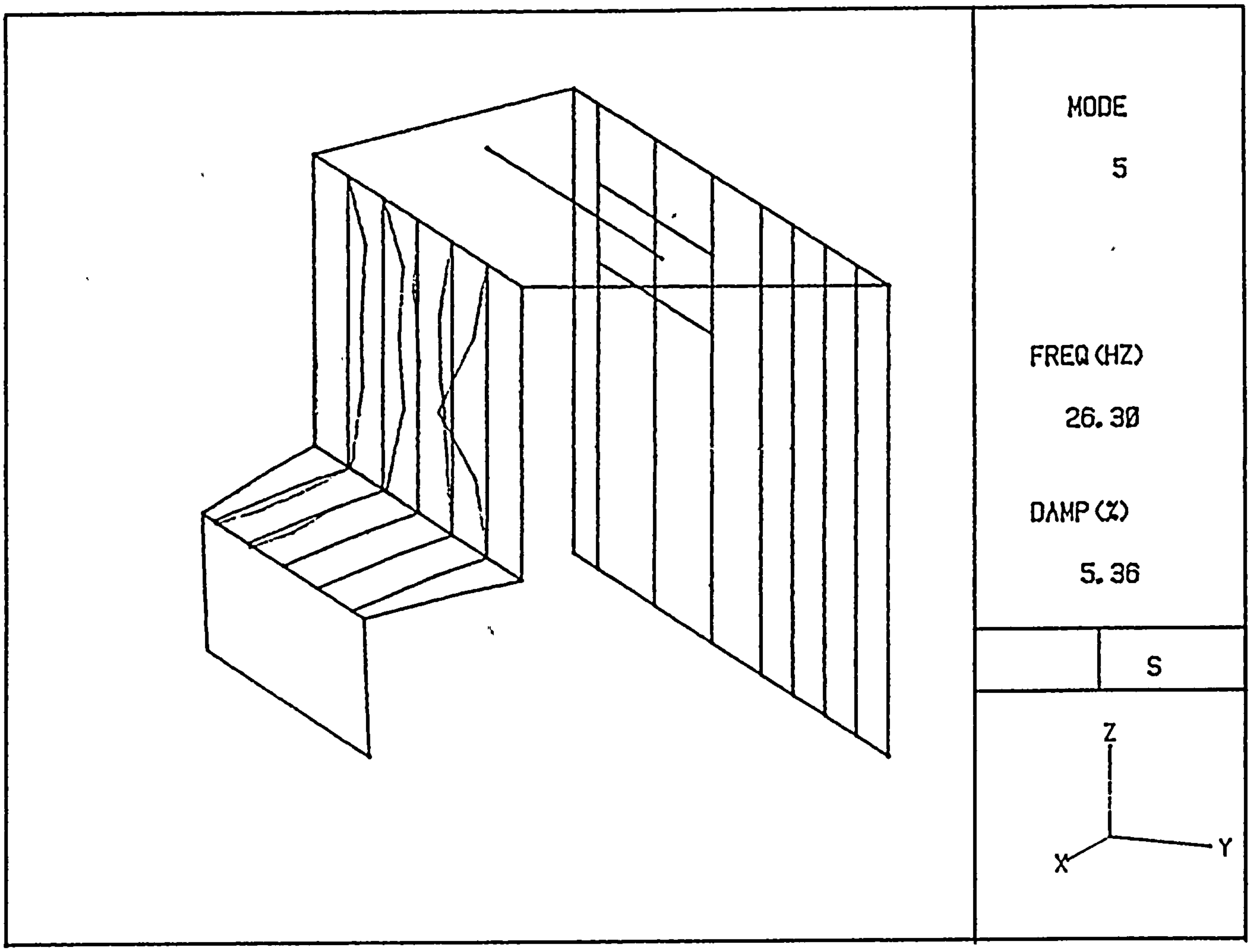


FIGURE A.25
MODE 5 CENTER FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

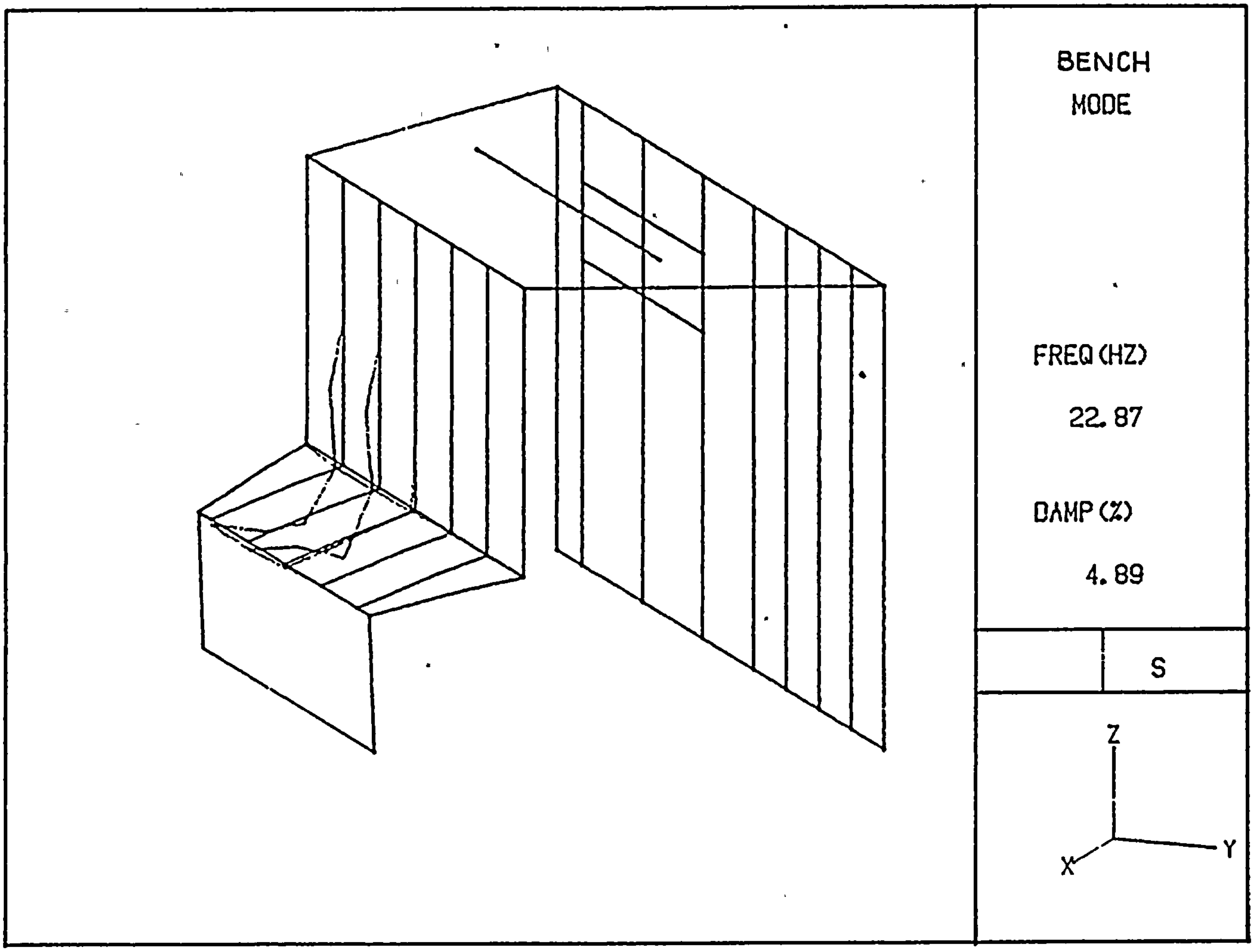


FIGURE A.26

BENCH MODE CENTER FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

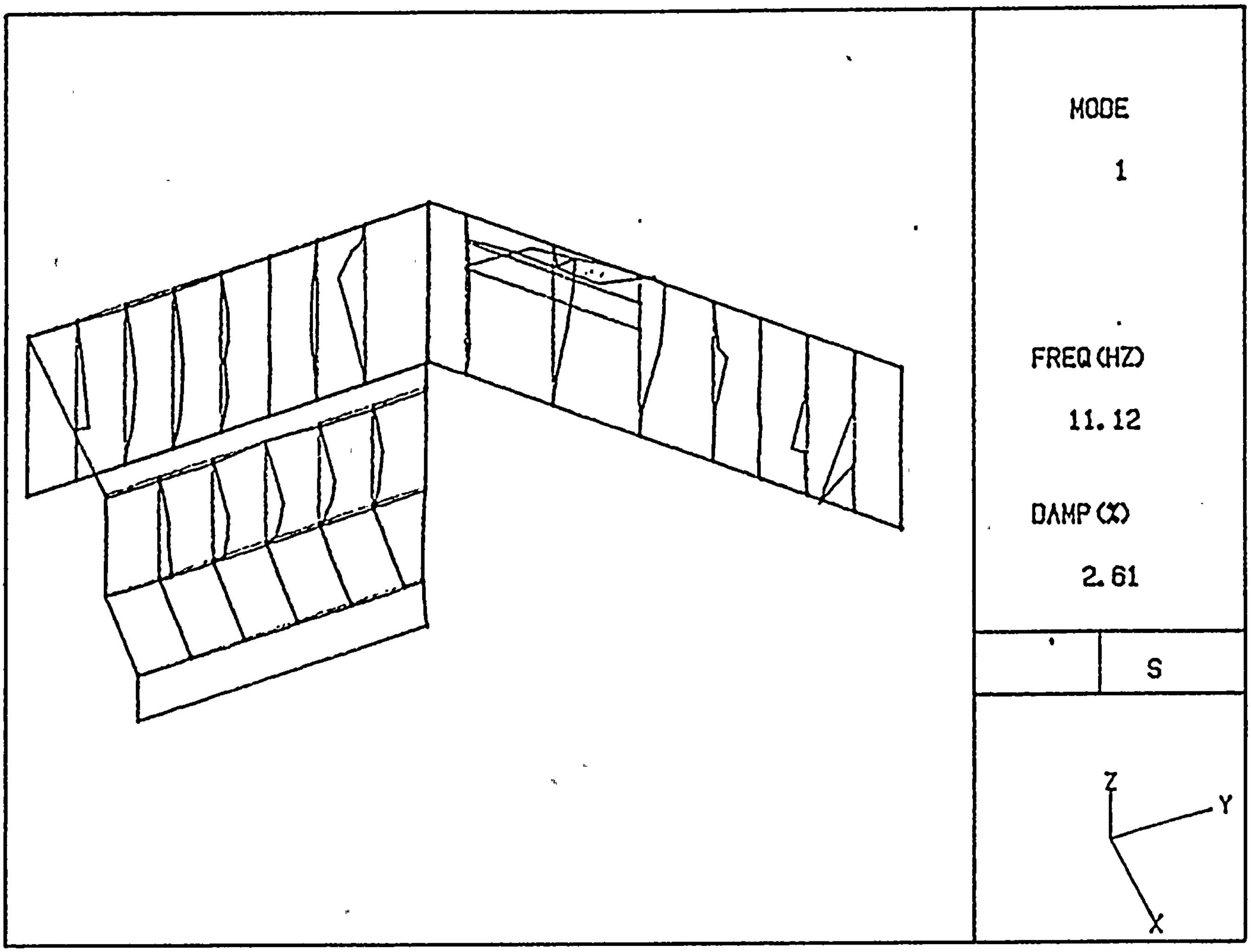


FIGURE A.27

MODE 1 LEFT FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

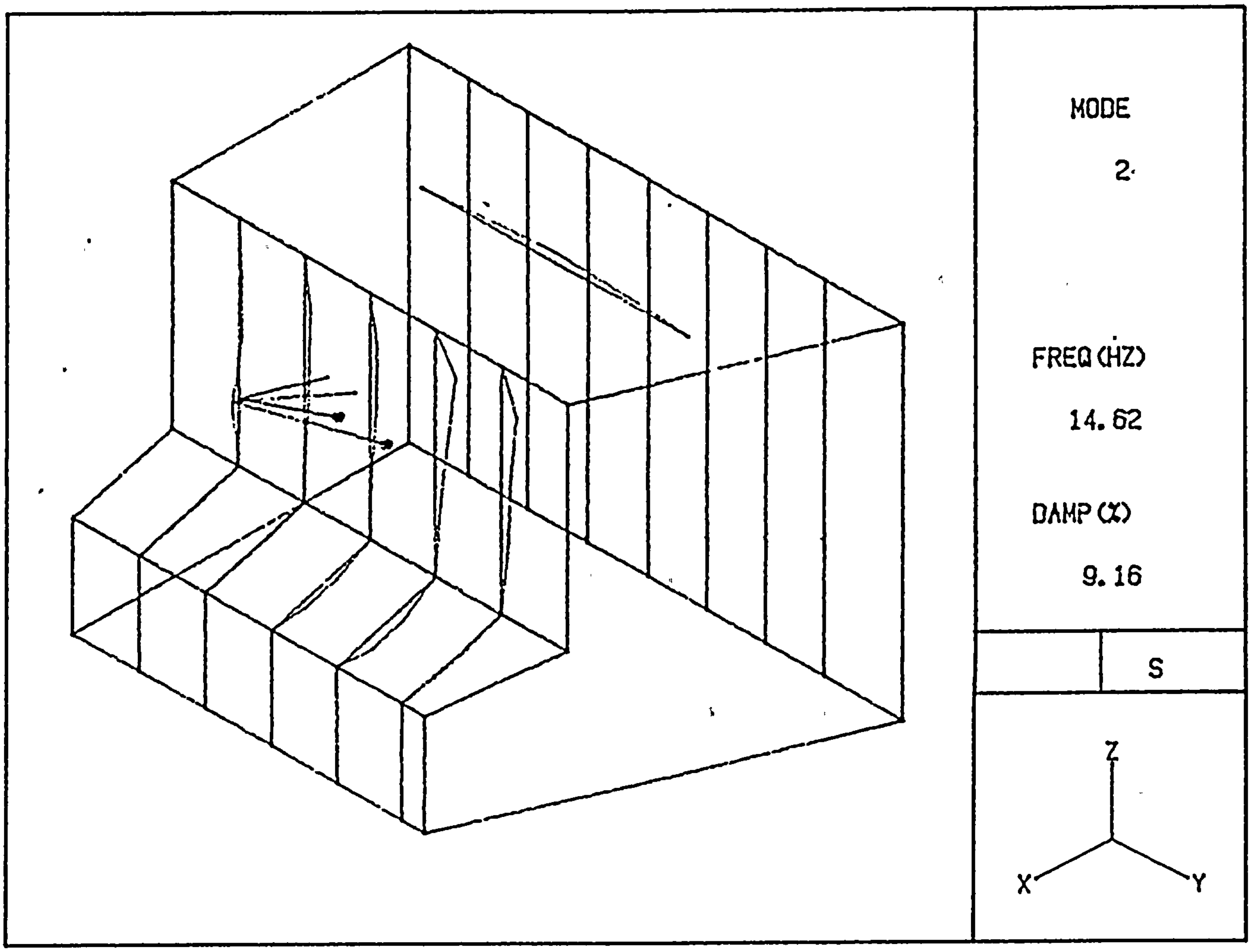


FIGURE A.28

MODE 2 LEFT FRONT CONTROL PANEL

R. E. GINNA PLANT

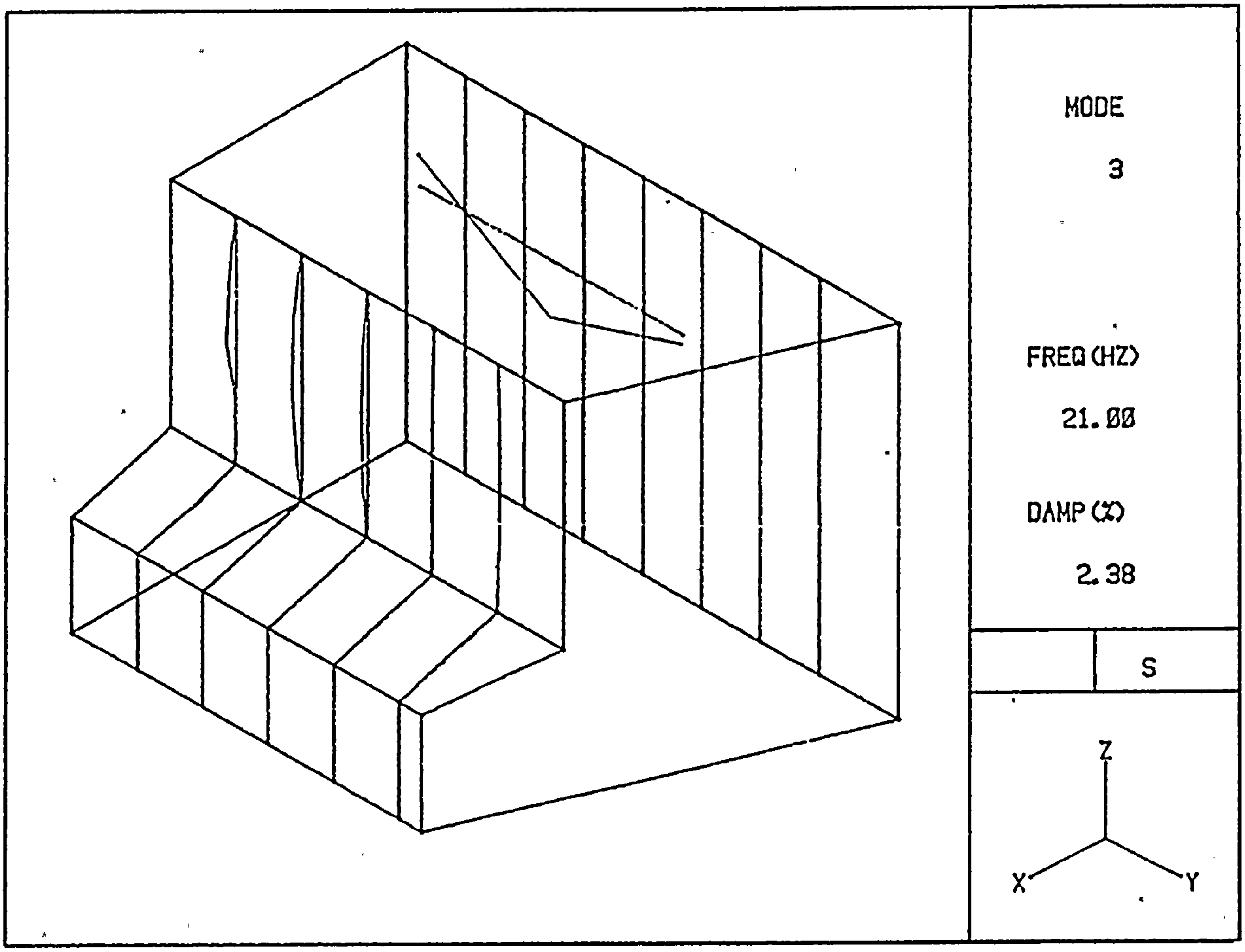


FIGURE A.29
MODE 3 LEFT FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

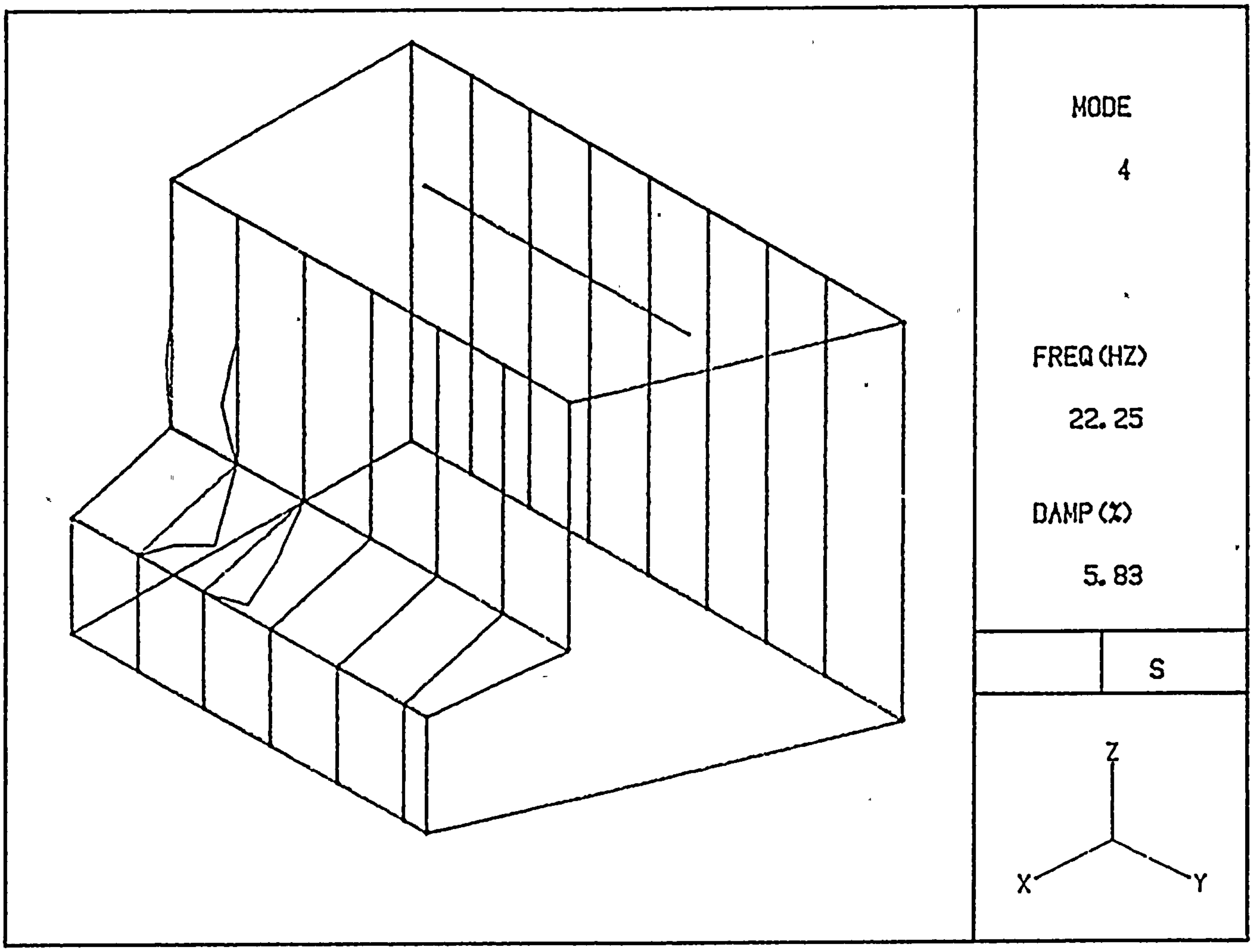


FIGURE A.30

MODE 4 LEFT FRONT CONTROL PANEL

R. E. GINNA PLANT

URS/Blume

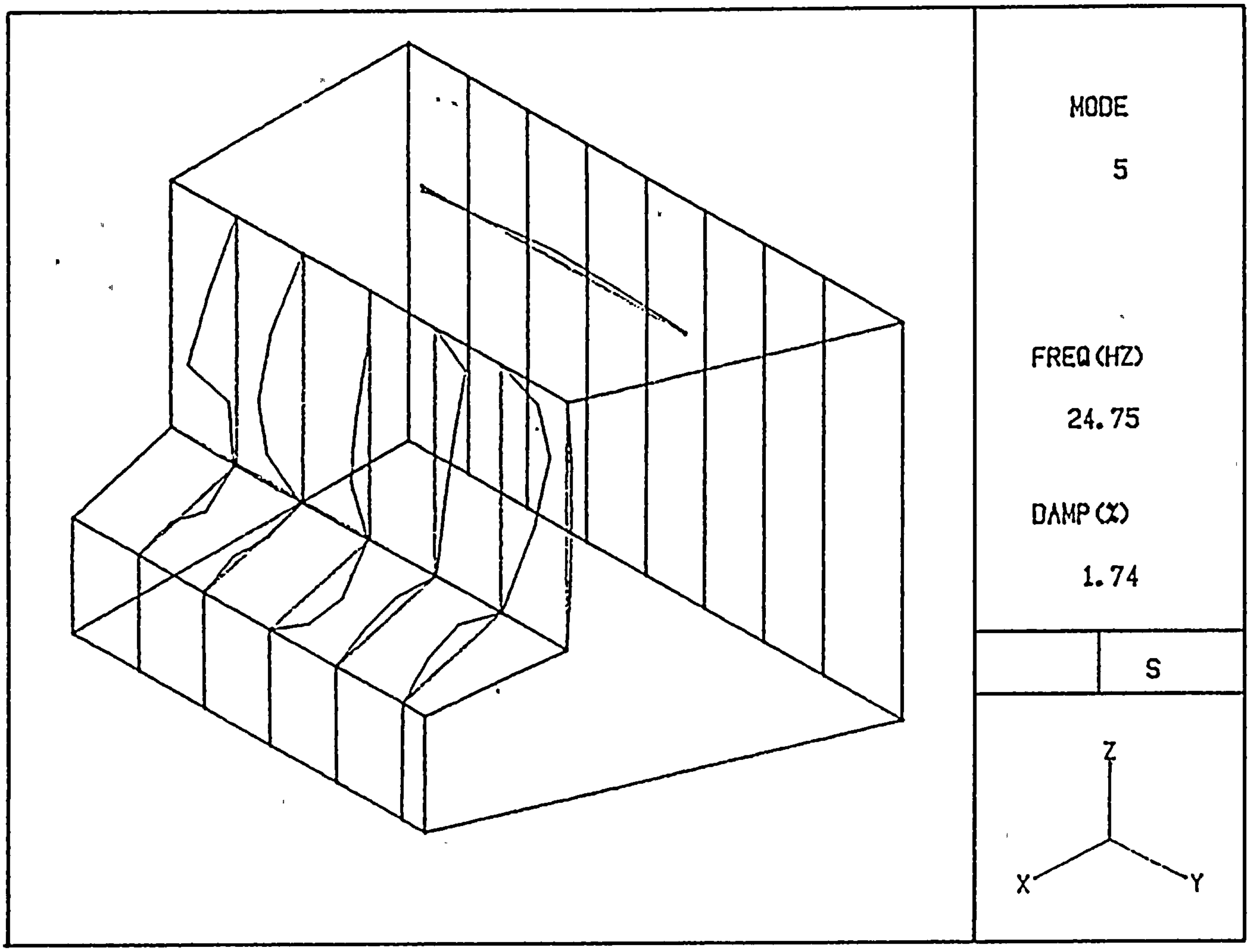


FIGURE A.31
MODE 5 LEFT FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume

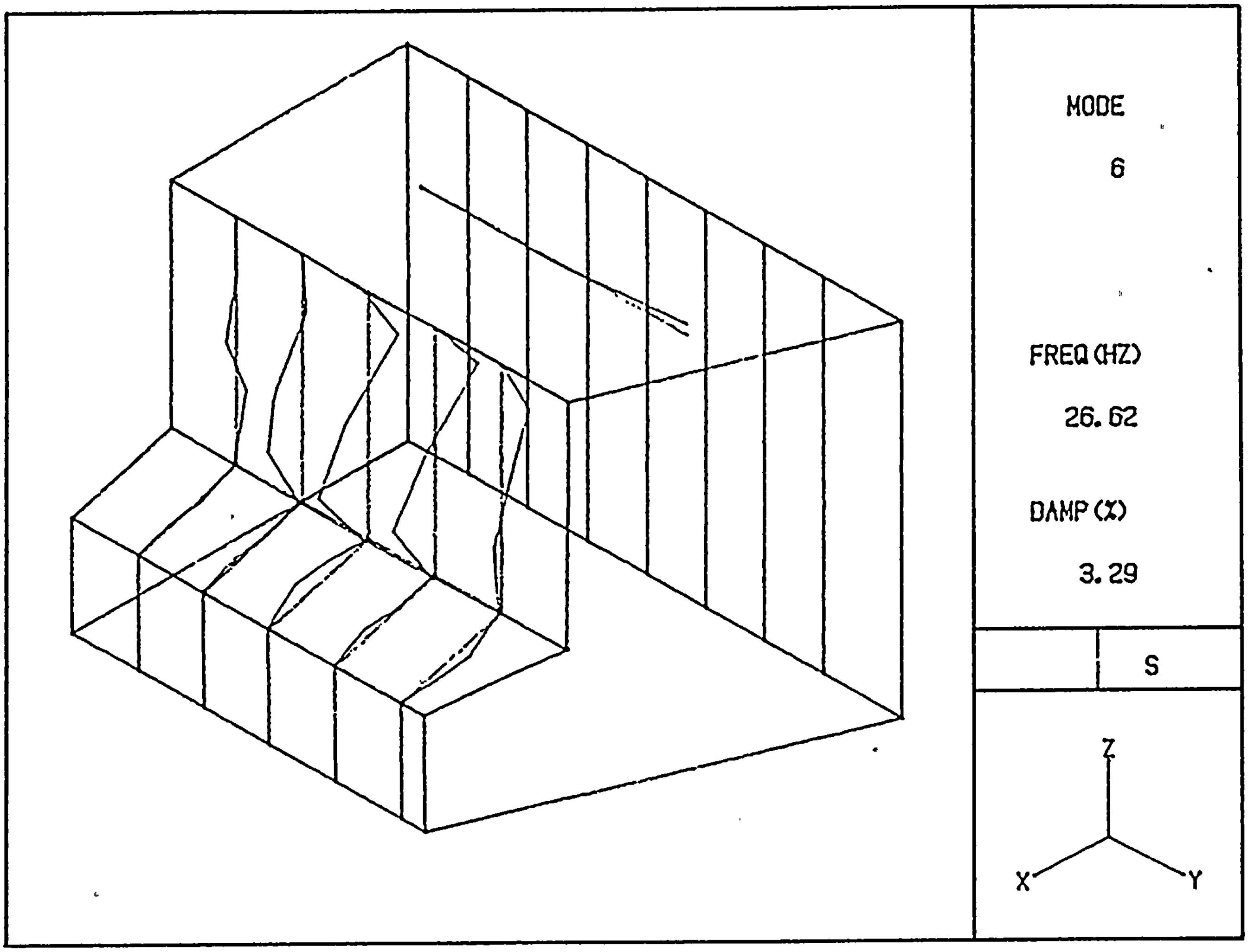


FIGURE A.32
MODE 6 LEFT FRONT CONTROL PANEL
R. E. GINNA PLANT

URS/Blume