*Robert L. Cloud and Associates, Inc.*

RLCA

# Interim Technical Report

DIABLO CANYON UNIT 1
INDEPENDENT DESIGN VERIFICATION PROGRAM
- AUXILIARY BUILDING -

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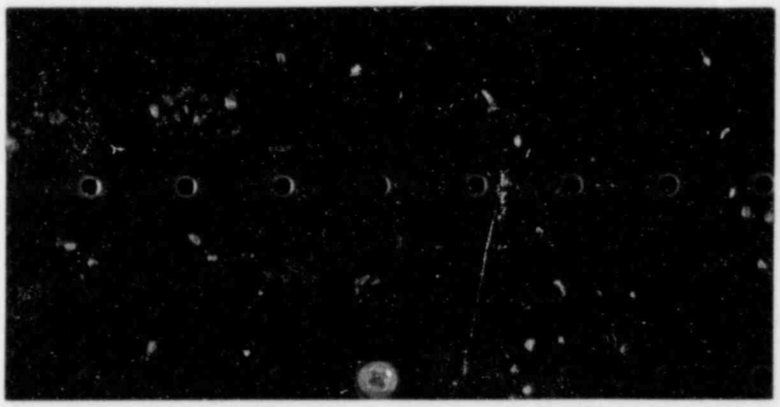

Auxiliary Building Report

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## AUXILIARY BUILDING

This is the sixth of a series of Interim Technical Reports prepared by the DCNPP-IDVP for the purpose of providing a conclusion of the program.

This report contains the methodology, analytical models, results, result comparisons, findings, recommendations and conclusions of the IDVP with respect to the auxiliary building which constitutes the initial building sample. The auxiliary building includes the fuel handling structure and the control room. The results and result comparisons to the design analysis are presented only for building properties, soil spring properties, structure frequencies and mode shapes. Based on the PG&E presentation in July, 1982 of their Internal Technical Program which includes a reanalysis of the auxiliary building and fuel handling structure, the IDVP determined that continued comparison of the RLCA analysis with the previous PG&E design analyses was no longer significant and hence, this effort was discontinued. The IDVP verification effort will instead concentrate on the PG&E corrective action on all civil structures including the auxiliary building and fuel handling structure.

As IDVP Program Manager, Teledyne Engineering Services has approved this ITR including the analytical results presented, conclusions and recommendations. The methodology followed by TES in performing this review and evaluation is described by Appendix C to this report.

ITR Reviewed and Approved
IDVP Program Manager
Teledyne Engineering Services

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R. Wray
Assistant Project Manager

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## 1.0 INTRODUCTION

### Purpose and Scope

This interim technical report summarizes the independent design verification program (IDVP) work performed for the auxiliary building of the Diablo Canyon Nuclear Power Plant (DCNPP). The report is intended to present the results of a portion of the verification effort for the sample of structures, list error and open items issued, and serve as a vehicle for Nuclear Regulatory Commission (NRC) review.

Interim technical reports were discussed at the June 10, 1982 NRC meeting in Waltham, Massachusetts. These reports will include: analytical references, results, sample definitions and descriptions, methodology, a listing of Error and Open Items, an examination of trends and concerns, and a conclusion.

In Section 9.1(6) of the Phase I Program Management Plan, Revision 0 (Reference 1), the following definition is given:

> Interim Technical Reports are prepared when a program participant has completed an aspect of their assigned effort in order to provide completed analysis and conclusions. These may be in support of an Error, Open Item or Program Resolution Report or in support of a portion of the work which verifies acceptability. Since such a report is a conclusion of the program, it is subject to the review of the Program Manager. The report will be transmitted simultaneously to PGandE and to NRC.

The auxiliary building was chosen as the initial structures sample for the following reasons:

- o The auxiliary building contains the largest amount of safe shutdown piping, equipment and components.

- o The building itself supports the fuel handling structure and the control room.

- o The building is structurally complex with both concrete shear walls and steel framing.

- o As discussed in the preliminary report on the review of the URS/Blume - PGandE interface, there is a controversy regarding masses in the seismic model of the building (EOI 985 & Reference 2).

The verification analysis of the auxiliary building was undertaken to determine whether deficiencies existed in the seismic analysis.

The phase I engineering plan specifies that comparisons of models, properties, frequencies, mode shapes, response spectra and member stresses be made. In July 1982, PGandE announced the structural review portion of their Internal Technical Program which includes a review and/or re-analysis of the auxiliary and fuel handling buildings. Since this re-analysis will provide new results, a comparison of the verification analysis with the original design analysis is no longer appropriate. This interim technical report presents work completed through July 1982 which include comparisons of models, properties, frequencies, and mode shapes. Response spectra and member stress comparisons are not included because the work was not finalized and comparison with the design analysis is now inappropriate.

## Background

On September 28, 1981, PGandE reported that a diagram error had been found in a portion of the seismic qualification of the Diablo Canyon Unit 1 Nuclear Power Plant. This error resulted in an incorrect application of the seismic floor response spe... for sections of the annulus of the Unit 1 containment building. The error originated when PGandE transmitted a sketch of Unit 2 to a consultant. This sketch contained geometry incorrectly identified as Unit 1 geometry.

As a result of this error, a seismic reverification program was established to determine if the seismic qualification of the plant was adequate for the postulated Hosgri 7.5 M earthquake. This program was presented orally to the NRC in a meeting at Bethesda, Maryland on October 9, 1981.

At an NRC meeting on November 3, 1981 RLCA orally presented preliminary results and described a revision to the review program based on independent calculations. Robert L. Cloud Associates presented a preliminary report for the Seismic Reverification Program to the NRC on November 12, 1981. The NRC Commissioners met during the week of November 16, 1981 to review the preliminary report and the overall situation. On November 19, 1981 an Order Suspending License CLI-81-30 was issued which suspended PGandE's license to load fuel and conduct low power tests up to 5% of rated power at DCNPP-1. This suspending order also specified that an Independent Design Verification Program be conducted to assure that the plant met the licensing criteria.

PGandE retained Robert L. Cloud Associates as program manager to develop and implement a program that would address the concerns cited in the Order Suspending License CLI-81-30. Phase I plan for this program was transmitted to the NRC on December 4, 1981 and discussed with the NRC staff on February 3, 1981. Phase I deals with seismic service related contracts prior to June 1978.

On March 19, 1982 the NRC approved Teledyne Engineering Services (TES) as program manager to replace Robert L. Cloud Associates. However, RLCA continued to perform the independent review of seismic, structural and mechanical aspects of Phase I.

The NRC approved the Independent Design Verification Program Phase I Engineering Program Plan on April 27, 1982. This plan dictates that a sample of piping, equipment, structures and components be selected for independent analysis. The results of these analyses are to be compared to the design analyses results. If the acceptance criteria is exceeded, an Open Item Report is to be filed. Interim technical reports are to be issued to explain the progress of different segments of the technical work.

## 2.0 DESCRIPTION OF THE STRUCTURE

A site plan of the Diablo Canyon Nuclear Power Plant is shown in Figure 1 with the auxiliary building shown in a crosshatched pattern. The auxiliary building is a reinforced concrete structure with maximum plan dimensions of 230 by 500 feet and an irregular shape. The concrete portion of the structure varies from 43 to 107 feet high and is designed as a shear wall building with a mat foundation on bedrock. Floor plans for elevations 85 feet, 100 feet, 115 feet, and 140 feet are shown in Figures 2-5, respectively. Three sections through the building are shown in Figures 6-8.

The building is essentially symmetric with respect to column line 18 which runs in the East-West (E-W) direction. It is on a sloping grade which varies from an elevation of 85 feet at the West side to an elevation of 115 feet at the East side. A portion of the structure is below grade between elevations 52 feet and 85 feet while the remaining parts of the structure foundation are at elevations 85 feet and 97 feet (Figure 8).

The fuel handling building located at elevation 140 feet is a steel structure with braced frames in the longitudinal N-S direction and moment-resisting frames in the transverse E-W direction. The building has plan dimensions of 58 by 366 feet and is 48 feet high. The auxiliary building is located adjacent to both the containment and turbine buildings. Structural gaps have been provided to isolate the auxiliary building from these adjacent structures.

## 3.0 DESIGN ANALYSIS HISTORY

Three separate sets of building loads and response spectra for the auxiliary building have been documented. These analyses were reported in 1971, 1977 and 1979. The 1971 report considered the design earthquake (DE) and double design earthquake (DDE). The 1977 and 1979 reports considered a different earthquake associated with the offshore Hosgri geologic fault. Model properties were reported to be identical for the 1971, 1977 and 1979 analyses, even though the physical configuration of the building had been changed. The following sections provide additional background regarding these analyses.

The initial analysis is contained in "Auxiliary Structure - Seismic Analysis, Diablo Canyon Nuclear Power Plant Unit No. 1," John A. Blume Associates, January 1971 (Reference 5). This analysis produced building loads and response spectra for the design earthquake and double design earthquake. A two-dimensional horizontal model, including soil/structure interaction effects at the base and soil springs, was reported for the analysis.

Model properties for the concrete portion of the building were calculated by PGandE using the computer program DYBOX. PGandE also calculated the model properties for typical bays in the fuel handling building. These properties were combined by John A. Blume & Associates to represent the stiffness of the entire structure. This model was analyzed by John A. Blume & Associates using the DE time histories shown in the Preliminary Safety Analysis Report (PSAR). Vertical response spectra for this analysis are defined as 2/3 of horizontal ground response spectra.

The second analysis of the auxiliary building is contained in the "Hosgri Report" (Reference 6). This analysis produced building loads and response spectra for the postulated Hosgri earthquake. Three models were reported for this analysis: N-S, E-W and vertical. These models employed a fixed base. With the exception of the soil springs, the N-S and E-W models used the same properties as those in the 1971 analysis. The E-W model employed soil springs identical to those in the 1971 analysis.

In the model for the N-S direction, the soil springs were inadvertantly omitted (Reference 17). This item was noted by RLCA in in an Open Item Report 920. These models were analyzed by URS/John A. Blume & Associates using the internally developed time histories corresponding to the Newmark and Blume design spectra constructed for the Hosgri event.

The third analysis of the auxiliary building is contained in "Auxiliary Building Dynamic Seismic Analysis for the 7.5M Hosgri Earthquake, Diablo Canyon Nuclear Power Plant, URS/Blume, October 1979" (Reference 4). This analysis produced building loads and response spectra for the postulated Hosgri earthquake. The E-W and vertical analyses reported are identical to those in the Hosgri Report. The N-S analysis contained in the Hosgri Report was corrected to include the inadvertantly omitted soil springs. All other aspects of this analysis were reported to be identical to the Hosgri Report analysis.

## 4.0 VERIFICATION SCOPE AND ACCEPTANCE CRITERIA

### 4.1 SCOPE

The scope of this interim technical report for the auxiliary building includes the following:

- Review the URS/Blume horizontal models for the seismic analyses of the auxiliary and fuel handling buildings.

- Calculate and compare the building properties for the horizontal models.

- Calculate and compare natural frequencies and modes of vibration for the horizontal models.

The results of verification analyses are compared with the design analyses. The design results used in the comparison were those reported in the URS/Blume report dated October 1979, "Auxiliary Building Dynamic Seismic Analyses for the 7.5M Hosgri Earthquake" (Reference 4).

### 4.2 ACCEPTANCE CRITERIA

The acceptance criteria for the seismic verification of the auxiliary building are contained in the Phase I Engineering Program Plan (DCNPP-IDVP-001 Revision 0). This plan received NRC approval on April 27, 1982 (Denton to Crane - Licensing Tab #195). Section 5.4.1.3 of the plan contains the following acceptance criteria:

> Additional verification will be required if the results vary by more than:
>
> - 15% for the building dimensions and properties.
>
> - For the building, 15% for the frequencies, provided the mode shapes agree.

The acceptance criteria for the seismic verification of the auxiliary building were established as a means to accomplish the program objectives of confirming the building analyses for Diablo Canyon and to provide a measure for the acceptance of final results used for qualification of structures and subcomponents. Criteria for the acceptance of seismic input and calculated building properties were established as a means of identifying and reconciling differences in engineering methodology. The establishment of these criteria assures that intermediate factors which fall on either side of the acceptance bound do not cancel each other with the net effect of the final results being lost.

## 5.0 VERIFICATION MODEL SPECIFICATIONS

The following are the major steps in the analysis procedure:

- o Develop mathematical models which approximate the actual response of the building to base excitations;
- o Construct subsidiary models of the fuel handling building to establish properties in the main models; and
- o Calculate mode shapes and frequencies.

The purpose of this section is to discuss the first two steps; Section 6.0 discusses the procedures and the computer code that was used to calculate mode shapes and frequencies.

### 5.1 SOIL STRUCTURE INTERACTION

The dynamic response of a massive structure may significantly alter the motions at the base of the structure from those in the free field. This interaction between a structure and the surrounding soil can be a significant aspect of seismic response - particularly if it introduces rocking of the structure about its base. Soil structure interaction affects the boundary conditions at the base of the mathematical models used for seismic analysis.

For structures supported on rock, with a shear wave velocity greater than 3500 ft/sec, a fixed-base boundary condition is appropriate (Reference 9). The shear wave velocity in the vicinity of the foundation slab provided in the Diablo Canyon FSAR exceeds 3500 ft/sec. In addition, the specification for seismic review (Reference 7) stipulates a fixed-base model. Therefore, a fixed-base boundary condition was selected at elevation 85 feet.

### 5.1.1 Soil Springs

The effects of variable base foundation elevation and the soil mass on the East side of the auxiliary building (which begins at elevation 115 feet and slopes down to elevation 85 feet on the West side) are accounted for by the inclusion of translational and torsional soil springs in the verification models.

Soil spring values were developed using the theoretical formulation for rigid plates embedded in semi-infinite elastic half spaces. The irregular dimensions of the auxiliary building were approximated by calculating the foundation contact area and using two methods to derive equivalent radii. Results from circular and rectangular foundation methods were considered in the selection of translational and rotational spring values acting at the 100 feet elevation. All soils data were taken from the Diablo Canyon FSAR and soil damping was neglected.

## 5.2 NORTH-SOUTH AND EAST-WEST MODELS

Discrete lumped mass and beam models were selected for the seismic analysis of the auxiliary building. Two sets of planar models designated North-South (N-S) and East-West (E-W) were developed to model the response of the auxiliary building to horizontal excitations (Figures 13 and 14).

The lumped masses were located at floor elevations and included the mass of the floor slab, equipment on the floor slab, and one half of the mass of columns and walls above and below the elevation. The remaining two coordinates of a lumped mass were defined with respect to N-S and E-W reference lines. The floor slabs were considered to be infinitely rigid in their own planes.

The lumped masses were connected in the mathematical models by two kinds of weightless beams: elastic beams and rigid beams. Elastic beams were used to represent the resistance of each story to displacements and rotations. These were located at the center of rigidity of the walls for the horizontal models. Rigid beams were used to model the floor slabs connecting lumped masses and elastic beams.

Figures 13 and 14 show N-S and E-W models. The direction of excitation for the two horizontal models is normal to the plane of the model. The eccentricities shown in the figures only correspond to geometric (actual) eccentricities.

There are five N-S models corresponding to 0%, 5%E, 5%W, 7%E, and 7%W accidental eccentricity, where the percentage refers to an increment or decrement to the geometric (actual) value in the amount of 5% or 7% of the maximum E-W dimension of the building. Similarily there are three E-W models corresponding to 0%, 5% and 7% N or S, accidental eccentricity. The symmetry of the building about column line 18 makes the North and South accidental eccentricities identical for the E-W model. As shown in Figure 14, the geometric (actual) eccentricity in the N-S direction was assumed to be zero for the E-W model.

A system of simultaneous second-order differential equations of motion governs the response of each model. The unknowns in these equations are the degrees of freedom permitted each node (each end point of a elastic or rigid beam) in a model. Degrees of freedom consist of two types, "static" and "dynamic". Dynamic degrees of freedom are variables with associated inertia that define second-order differential equations of motion. Static degrees of freedom are variables without associated inertia that define equations of statics. After solving for the time history of the dynamic degrees of freedom, the time histories of the static degrees of freedom are obtained at each point using the associated fixed relationships.

The horizontal models have the same number of mass points and degrees of freedom. The base node is fully restrained. Nodes 1, 2, 3, 4 and 6 correspond, respectively, to the control room roof at elevation 163 feet, the 140 foot, 115 foot and 100 foot concrete floor slabs, and fuel handling building roof at elevation 188 feet. These nodes are permitted two dynamic degrees of freedom corresponding to translation in the direction of excitation and torsion about the vertical axis. The procedures for calculating associated weights and torsional mass moments of inertia are discussed in Section 5.4. These nodes (except node 6 corresponding to the fuel handling building) are also given static degrees of freedom to rotate about the horizontal axis normal to the direction of excitation. These degrees of freedom are provided to model coupled shear and bending of the walls in line with the direction of excitation. All other nodes lie on some rigid beam associated with a mass node. Nodes within the same rigid system are completely coupled to the associated mass node.

In summary, the motions of the horizontal models are defined by ten second-order differential equations (two dynamic degrees of freedom per mass node) and a number of fixed linear relationships associated with static degrees of freedom and rigid floor diaphrams.

## 5.3 MODEL PROPERTIES

The models discussed in Section 5.2 are specified by: calculating the locations of the nodes; calculating the weights and rotary inertiae associated with dynamic degrees of freedom; calculating the elastic beam properties associated with the resistance to translations and rotations; and linearly coupling rigid beams with associated dynamic degees of freedom.

### 5.3.1 Mass Properties and Centers of Mass

As described in Section 5.2, there are five mass nodes in each model. Each of these is assigned a weight (a translational inertia) associated with each translational dynamic degree of freedom.

These weights were calculated by defining tributary regions associated with each mass node. To define these regions, imaginary horizontal planes were passed through the auxiliary building at elevations 92.5 feet, 107.5 feet, 127.5 feet, 152.5 feet, and 164 feet. Masses were assigned to the nodes as follows:

- o Masses below elevation 92.5 feet were assigned to the restrained node at elevation 85 feet and so did not contribute to dynamic response.
- o Masses between elevations 92.5 feet and 107.5 feet were assigned to the node representing the 100 foot slab.
- o Masses between elevations 107.5 feet and 127.5 feet were assigned to the 115 foot slab mass node.
- o Masses between 127.5 feet and 140 feet, together with the portion of the fuel handling building from 140 feet to 164 feet and the portion of the control room mass from 140 feet to 152.5 feet were assigned to the 140 foot slab node.
- o The portion of the control room mass above 152.5 feet was assigned to the control room mass node.
- o The portion of the fuel handling building mass above 164 feet was assigned to the fuel handling building node.

For each of these regions, significant masses were determined by review of design drawings and site verification of "as-built" configurations. Major masses included (as appropriate) floor slabs, the control room and fuel handling build-ding roofs, shear walls, columns and concrete block walls and pieces of equipment weighing more than five thousand pounds resting on, or hanging from, the slab of concern. The following were considered in the determination of masses:

- o Floor openings greater than two by two feet shown on design drawings
- o Door openings in walls shown on plan drawings
- o An allowance for minor equipment and live loads of 100 psf of floor slab area (Reference 10)
- o No roof live loads for the fuel handling building
- o Lift loads of 125 and 15 tons for the fuel handling crane
- o Mass of concrete items on a volume-density basis
- o Mass of steel structural members on a mass per linear foot basis
- o Masses of major equipment as specified on design and vendor drawings

Rotary inertiae (mass moments of inertia) were calculated for each mass item with respect to the vertical axis. The cumulative rotary inertia associated with a mass node was calculated as the summation of the rotary inertia of each mass item plus the summation of the product of each mass and the squared distance from the center of each mass to the axis of rotation for the mass node.

The center of mass of the items associated with each mass node was calculated as the sum of the products of each mass item and its distance to a reference point, divided by the sum of the mass items. The two coordinates were used (together with centers of rigidity, discussed in Section 5.3) to establish geometric eccentricities.

### 5.3.2 Stiffness Properties and Centers of Rigidity

The significant stiffness properties resisting the motion of the modeled degrees of freedom were represented with elastic beam members. Elastic beams permit modeling of axial deformation, shear and bending in two directions, and torsion.

For both horizontal models, shear areas, moments of inertia, torsional moments of inertia and centers of rigidity were calculated for each mass node and associated rigid system. The stiffness properties of all model beam members except those representing the fuel handling building are discussed below.

#### Shear Areas

Shear areas were conservatively taken as 5/6 of the gross area of shear walls in line with the direction of excitation, neglecting the effects of cracking on the gross section and the effects of cross walls and columns on the shear area.

#### Centers of Rigidity

The principal contribution to horizontal displacements of the auxiliary building is due to shear strain rather than flexural strain in the supporting walls. This is the result of the small ratio of story height to horizontal dimension of supporting walls (i.e. aspect ratio) of the building. Therefore, for the horizontal models, the centers of rigidity were taken as the sums of all individual shear wall stiffnesses multiplied by their distances to a reference line parallel to the direction of excitation, divided by the sums of all shear stiffnesses. For each floor, this calculation amounts to the cumulative moment of the shear area divided by the cumulative shear area.

Moments of Inertia

For both horizontal models, moments of inertia about the horizontal axis normal to the direction of excitation were calculated for each shear wall in line with the direction of excitation. The cumulative moment of inertia associated with a model beam element was calculated as the sum of the individual moments of walls in line with the excitation about their centroids, plus the sum of the products of each cross sectional area and the squared distance from the center of the area to an axis of rotation passing through the center of rigidity. Any capability of columns or shear walls normal to the direction of excitation was neglected as the dimensions of the structure are large and shear lag effects significantly limit their effective stiffness.

Torsional Rigidities

Torsional rigidities for the horizontal models were calculated by considering the in-plane shear rigidities of shear walls between floor slabs in both N-S and E-W directions. The torsional rigidities were taken as the sum of the shear rigidities multiplied by the squared perpendicular distance from the centroid of the area to an axis of rotation passing through the center of rigidity. The torsional constant of the modeled beam, representing the walls between floor levels, is the torsional rigidity multiplied by the (nominal) story height and divided by the shear modulus.

Material Properties

The actual properties of the auxiliary building concrete were used, namely a Young's Modulus of 616,320 kip/ft  and a Poisson's ratio of 0.25 (Reference 6).

## 5.4 FUEL HANDLING BUILDING STIFFNESSES

For both horizontal models, the fuel handling building mass node was permitted two dynamic degrees of freedom corresponding to translation in the direction of base excitation and torsion about the vertical axis. Procedures for calculating the translational and rotary inertiae are the same as described in Section 5.3.1.

The structural configuration of the fuel handling building was modified as a result of the Hosgri seismic re-evaluation. The modifications included: the addition of slotted joints at structural connections at E-W end bays and the N-S wall along column line V; and the addition of bracing to both the N-S frames.

Stiffness values for the fuel handling building were calculated by finite element methods using simplifying assumptions to simulate the structural action of this complex building. ANSYS models (Reference 16) were made for the two N-S walls, a typical transverse frame, and an end frame. The structural characteristics of member connections modified with slots and the behavior of slender bracing elements under compression were approximated by assuming that these members carry no load. The assumption results in a conservative decrease in stiffness. However, the assumption has not been verified for the actual member forces developed. Influence loads were applied to one end of the roof truss in these finite element models. The average deflection along the truss was used to determine force-deflection relationships.

The analyses indicated an eccentric stiffness in the N-S direction with the West wall resisting only one quarter of the load resisted by the East wall. The geometric (actual) eccentricity used in the dynamic model was calculated by using the procedure described in Section 5.3 for centers of rigidity and centers of mass.

The N-S stiffness was taken as the sum of the stiffness calculated for the East and West walls; the E-W stiffness was taken as the sum of the stiffnesses calculated for the interior and end frames. Torsional constants and moments of inertia of equivalent beams were obtained by equating elastic beam stiffnesses with the calculated stiffnesses.

However, specific details of the fuel handling building framing and joints configurations remain in question and their effectiveness remains uncertain. The slotted joint details shown on design drawings differ from those in the Hosgri report. In addition, field verification of these details differs from both design documents.

As a consequence, the validity of all reported fuel handling building results should be considered unsubstantiated for the purpose of fuel handling building structural evaluation. However, due to the small mass of the fuel handling building, the analyses of the building were deemed adequate for determining the effect of interaction of the fuel handling building with the remainder of the auxiliary building.

## 6.0 VERIFICATION ANALYSIS PROCEDURES

Free vibration analyses were performed for each of the eight models (five N-S and three E-W). This section discusses the analytic steps as carried out using the STARDYNE computer code (Reference 15).

The system of linear differential equations, described in Section 5.0, has associated eigenvalues and eigenvectors that correspond physically to natural frequencies and natural modes of vibration, respectively. Mode shapes and natural frequencies are found by solving, q and $\omega$ in the matrix equation:

$$[K]\{q\} = \omega^2 [M]\{q\}$$

where,

q = an n-dimensional vector,

$\omega$ = a scalar;

[M]= is the mass matrix,

[K]= the stiffness matrix.

In STARDYNE, mode shapes are calculated using the Householder - QR (HQR) modal extraction procedure and are normalized by dividing all entries in q by the largest magnitude entry.

In addition to the natural frequencies and mode shapes, other quantities are calculated that provide some insight into the structural characteristics. Principal among these are the translational participation factors. Translational participation factors reported by the STARDYNE program are calculated as follows:

$$Q_i = \frac{\sum_{j=1}^{\text{No. of Nodes}} \begin{pmatrix}\text{Component of } \{q\} \text{ for } j^{th} \\ \text{node in } i^{th} \text{ dimension}\end{pmatrix} \begin{pmatrix}\text{Associated} \\ \text{Weight}\end{pmatrix}}{\text{Generalized Weight}}$$

$$\text{Generalized weight} = \{q\}^T [W] \{q\}$$

In summary, the STARDYNE - HQR program calculates frequencies and mode shapes. The mode shapes reported for the design analysis and shown in Figures 16-36 have been re-normalized to the same basis as the verification mode shapes. Hence, comparisons of frequencies and mode shapes between analyses are valid. Direct comparisons of the participation factors reported in the verification and design analyses are not valid because they are normalized differently.

## 7.0 COMPARISON OF DESIGN AND VERIFICATION MODELS, PROPERTIES AND PARAMETERS

Section 5.0 describes the verification procedures for the determination of soil springs, mass properties, stiffness properties, fuel handling building stiffnesses, and centers of mass and rigidity. Comparisons of the verification calculated values with those reported in design analysis are presented in Tables 1 through 4 and differences greater than 15% are discussed in the following section.

### 7.1 SOIL SPRINGS

Soil springs stiffnesses for the auxiliary building were calculated in the design and verification analyses. Comparisons of soil spring stiffnesses (presented in Table 1) show significant differences in the calculated translational values. The difference between design and verifition calculated soil spring values, reported in EOI 1070, results from differences in calculative procedures and soils data used in the two analyses.

Sensitivity studies were performed as part of the IDVP, using the design N-S model data, to determine the maximum effects of variation of the soil spring values from zero to the value used in the design analysis. These studies are discussed in Section 9.0.

### 7.2 MASS PROPERTIES

Mass properties were calculated for both horizontal models as discussed in Section 5.3. For the horizontal models, masses and mass moments of inertia about the vertical axis are compared with design values (see Table 2). The mass properties calculated for the horizontal models are all within the 15% acceptable criteria.

## 7.3 STIFFNESS PROPERTIES

Stiffness properties were calculated for both horizontal models as discussed in Section 5.3. Verification values for shear areas and moments of inertia are compared with design values in Table 3A. Shear areas for both the N-S and E-W models are within the 15% acceptance criteria.

Comparisons of moments of inertia in Table 3A show large differences. The differences in values for the concrete structure as calculated in the design and verification efforts appear to be the result of a difference in methodology in the calculation of the moment of inertia for individual wall elements.

The verification methodology includes terms for both the moment of inertia of each element about its own centroid and the transformed moment term about the centroid of the group of walls. The design methodology calculated only the moment term of each wall about its own centroid. The use of the local moment of inertia would tend to make the structure more flexible in bending. The effect of this difference is discussed in Section 9.0.

The differences in structural stiffness of the fuel handling building shown in Table 3A are believed to be the result of modifications made to the building since the design computation of steel frame stiffness in 1970 and 1971. The modifications included: addition of end concrete wall supports to the steel frame in both the N-S E-W directions; and additions and modifications to cross-bracing between steel and concrete walls. In addition, a discrepancy exists between design drawings and IDVP field verification for a structural member between column lines 17 and 18. These differences were reported in EOIs 1027, 1079, 1091 and 1092.

Comparisons of torsional rigidity in Table 3B show differences greater than 15% for the fuel handling building and member 2 between elevations 115 feet and 140 feet. These differences were reported in EOI 1029 and remain partially unresolved.

The differences in the fuel handling building torsional rigidities are a result of the structural modifications described above and analytical procedures used in the design analysis.

The difference in calculated torsional rigidity for member 2 is 37%. A comparison of the torsional rigidities between member 2 and 3 for the design analysis indicates a stiffness ratio of 2.8. When story heights for members 2 and 3 are taken into consideration, the comparison of the torsional moments of inertia of members 2 and 3 (torsional rigidity multiplied by the story height and divided by the shear modulus) for the design analysis indicates that a significant discontinuity in torsional moment of inertia (ratio 1.6) exists between these levels. This discontinuity is not reflected in the current as-built drawings.

A comparison of the general arrangement drawings shown in the 1971 Blume "Auxiliary Building Seismic Analysis Report" with drawings used in the verification effort indicates that additional shear walls are shown on the verification drawings (Figures 9-12). Torsional stiffness effects of these additional walls were estimated to be approximately 70% of the difference between the reported design and verification values. The reason for the difference between the values is therefore only partially understood.

## 7.4 CENTERS OF MASS AND RIGIDITY

Centers of mass and rigidity were calculated for both horizontal models as discussed in Section 5.3. Table 4 compares the centers of mass and rigidity for the N-S horizontal model with the design values. The table shows differences of greater than 15% for four calculated values. Three of these values are centers of mass at elevations 100, 115 and 140 feet in the auxiliary building.

The values presented in Table 4 are all measured relative to column line R, which was chosen for convenience, being the closest column line to all centers. Comparison of the calculated centers using the appropriate dimension of the building edge to center shows the values differ by less than 5%. While this difference is less than the 15% acceptance criteria, it implies an increase in the total induced torsion of the same magnitude as the accidental eccentricity.

## 8.0 COMPARISON OF DESIGN AND VERIFICATION DYNAMIC CHARACTERISTICS

### 8.1 HORIZONTAL MODAL PERIODS FOR ACCIDENTAL ECCENTRICITY MODELS

Comparisons of modal periods between design and verification analyses for the N-S and E-W accidental eccentricity cases are presented in Tables 5A & 5B, respectively (only the 5% case results are presented for comparative purposes).

Computed periods compare within 15% for all modes except modes 1 in the N-S model and modes 1 and 2 in the E-W model. It should be noted that the apparently favorable comparison of mode 2 in the N-S model between analyses is coincidental. Mode 2 in the design analysis represents torsion of the fuel handling building whereas in the verification analysis it predicts translation. The lack of agreement in periods for the fuel handling building results from the differences in the modeled structural configuration used to calculate moments of inertia, torsional rigidity and center of rigidity (discussed in Section 5.3.3).

## 8.2. NORTH-SOUTH MODE SHAPES FOR ACCIDENTAL ECCENTRICITY MODELS

Mode shape comparisons for 5% West and 5% East accidental eccentricity between design and verification analyses are shown in Figures 16-22 and Figures 23-29 respectively. Mass nodes for the 5% West and 5% East models are shown to the West and East, respectively, of the corresponding nodes shown in Figure 13. The response characteristics of the N-S building models for modes 1 through 5, which comprise the majority of the seismic response, appear to be relatively insensitive to the shift in center of mass from 5% East to 5% West. Since these modes in both design and verification models retain similar displacement relationships and amplitudes, they are consequently described together.

The percentage participation factors for the design and verification models are referred to in the following discussion of mode shapes. These factors are included for the comparison of relative importance of modes within each analysis. The percentage participation factors from the verification analysis cases are shown on Figures 16-29; the factors from the design analysis, the E-W and N-S 5% East eccentricity cases, are shown on Figures 23-36. The factors for the N-S 5% West eccentricity were not available and are not presented. These design and verification participation factors should not be compared between analyses due to the difference in the methods of computation mentioned in Section 6.0.

Modes 1 and 2 for the N-S 5% accidental eccentricity cases are fuel handling building translation and torsion. The verification analyses predominantly predict torsion with a small amount of translation for mode 1 whereas the design analyses predominantly predict translation. Mode 2 is translation in the verification analysis and torsion in the design analysis. Both results are consistent with the calculated torsional and translational stiffnesses for the fuel handling building. Both design and verification analyses differ in stiffness values but show close agreement in calculated mass and mass moment values.

The verification analyses show a torsional stiffness value 3.5 times lower than the design analyses, and a translational stiffness value of 1.6 times higher; this difference results in the observed transposition of translational and torsional modes between analyses.

The design model for mode 2 (torsion of the fuel handling building) showed a small amount of coupling with the auxiliary building floor slab at elevation 140 feet and with the control room. Similar coupling was not observed in the corresponding verification analysis torsional mode.

Modes 3 and 4 in both analyses were observed to be coupled translation and torsion for the concrete portions of the auxiliary building with large participation factors. Although the mode shapes are similar they differ primarily in the amount of torsion at elevation 140 feet and above. This difference in torsional response is believed to result from the differences in calculated torsional rigidity between design and verification models from elevations 115 to 140 feet. In addition, the modal comparisons indicate that for the N-S modes little structural interaction occurs between the auxiliary building and fuel handling buildings.

Mode 5 response for both models is control room translation coupled with a small amount of translation at elevation 140 feet and lower. Mode 5 shows significant participation factors in both models and little coupling with the fuel handling building.

Modes 6 and 7 are coupled translation and torsion with small participation factors. The modes differ significantly in shape; however, the effects of these differences on gross building behavior is small due to their small participation factors.

In summary, modes 1 and 2 are largely uncoupled fuel handling building response and show poor agreement between analyses. Modes 3, 4 and 5 are the dominant response of the auxiliary building in translation and torsion and show reasonable agreement between analyses. Modes 6 and 7 in the N-S direction show poor agreement due to the differences in calculated properties but have a small effect on overall building response.

## 8.3 EAST-WEST MODE SHA' . FOR ACCIDENTAL ECCENTRICITY MODEL.

Mode shape comparisons for 5% accidental eccentricity between desi and verification analyses are shown in Figures 30-36. The E-W 5% accidental eccentricity case indicates that modes 1 and 2 are fuel handling building translation and torsion, respectively. Both analyses show Mode 2 torsion to be lightly coupled with torsion at elevations below 140 feet.

Modes 3 and 4 in both analyses were observed to be cantilever translation of all floors with a small amount of coupled torsion and little coupling with the fuel handling building.

Mode 5 response for both models is dominated by control room translation, coupled with some translation and torsion of floor levels at 140 feet and below. Modes 6 and 7 are higher-order translational modes in both models with small participation factors. The comparison between models shows general agreement in shape and magnitude.

## 9.0 PARAMETER STUDIES

Comparisons of design and verification model properties are presented in Section 7.0. As discussed, several property comparisons did not satisfy the established acceptance criteria. These differences in calculated values have not been resolved.

Sensitivity studies were performed to evaluate the effects of varying two of the parameters for which major differences exist between design and verification analyses. These studies were undertaken to determine effects of these parameters on building periods.

Sensitivity studies were conducted for the effect of soil spring stiffnesses and moments of inertia. These studies utilized the design analysis N-S 5% West eccentricity model that was reconstructed from URS/Blume data. The reconstructed model utilized identical values for all design analysis parameters (mass, shear stiffnesses, etc.), except for the two parameters studied. Three study cases were run using values for the soil spring stiffnesses from zero up to the design value, and moments of inertiae from the design value to the verification value for the concrete portions of the structure. These cases included: design soil spring and verification moment of inertia; no soil spring and the verification moments of inertia; and no soil spring and the design moments of inertia.

The results of these analyses are presented along with the design analysis and the verification analysis results in Table 6. The columns in Table 6 represent the following: column one presents results for study cases with no soil spring and design bending moments of inertia; column two presents results for study cases with no soil spring and verification bending moments of inertia; column three presents design analysis results; column four presents the study case using the design soil spring value and the verification moment of inertia; column five is the verification analysis results.

Comparison of the study cases and design verification analyses shows the following trends:

- o Periods for modes 1 and 2 for all analyses are associated with the fuel handling building and are based on properties as calculated in either the design or verification analyses. Modes 1 and 2 are not affected by the parameter variations.

- o Periods for the study case using zero soil spring values and the design moments of inertia correspond to the periods presented in the 1977 Hosgri analysis for North-South results, as reported in EOI 920.

- o Variation of the soil spring, from the design values to zero resulted in a maximum effect on mode 4 of approximately 12%. The effect on mode 3 was approximately 6%. This trend was observed for two sets of results: cases using the design moments of inertia and cases using the verification moments of inertia.

- o Variation of the moments of inertia from the design values to the verification values resulted in a maximum effect on mode 4 of approximately 15%. The effect on mode 3 and 5 was approximately 6%. This trend was observed for two sets of results: cases using the design soil spring values and cases using zero soil spring values.

## 10.0 ERROR AND OPEN ITEM REPORTS ISSUED

RLCA issued EOI reports for the auxiliary building. Appendix A shows the EOI file number, revision, date and status.

EOI 920 reports spectra differences between the Hosgri Report and the 1979 URS/Blume Report. This item was combined with EOI 1097 as an Error Class A or B.

EOI 985 reports weight differences noted in a transmittal between PGandE and URS/Blume. This item was closed because the Phase I Program provided for the independent calculation of weights.

EOI 986 notes a large difference between the preliminary and final URS/Blume generated control room vertical spectra. This item was combined with EOI 1097 as an Error Class A or B.

EOI 987 cites a need to review the member qualification analyses for the auxiliary building. This Open Item was later closed because the Phase I Plan specified that members be independently evaluated.

EOI 990 reports a need to check the applicablity of design information transmitted from PGandE to URS/Blume. This item was combined with EOI 1092 as an Error Class A.

EOI 991 calls for a check of the fuel handling building crane modifications. This Open Item was combined with EOI 1092 as an Error Class A.

EOI 1027 reports differences between the PGandE drawings and the field configuration of steel joints in the fuel handling building. This item was combined with EOI 1092 as an Error Class A.

EOI 1028 notes differences between the Hosgri Report and the URS/Blume Report with respect to the response combination criteria. This item remains unresolved.

EOI 1029 reports model property differences between the verification and design analyses that exceed 15%. This Open Item was combined with EOI 1097 as an Error Class A or B.

EOI 1070 notes soil spring differences and was later combined with EOI 1097 as an Error Class A or B.

EOI 1079 addresses a difference between the Hosgri Report and the design drawing regarding the fuel handling building bracing. EOI 1092 incorporates this item as an Error Class A.

EOI 1091 notes a difference between the design analysis and field configuration regarding fuel handling building bracing. This item was combined with EOI 1092 as an Error Class A.

EOI 1092 notes inconsistencies within the Hosgri Report involving the fuel handling building bracing. This item was combined with EOIs 990, 991, 1027, 1091, and 1079 as an Error Class A.

EOI 1093 notes two areas of the auxiliary building for which spectra were not provided. This item was combined with EOI 1097 as an Error Class A or B.

EOI 1095 notes concern with the input time history. The floor response spectra obtained from the input may not be conservative at all frequencies. This item remains unresolved.

EOI 1097 notes an area of the auxiliary building for which spectra were not generated. This item was combined with EOIs 920, 986, 1029, 1070 and 1093 as an Error Class A or B.

# 11.0 EVALUATION OF BUILDING ANALYSIS

## 11.1 INTERPRETATION

Two generic concerns for structures were identified after examining the comparisons of independent and design analyses described in Sections 7.0, 8.0, 9.0 and the EOI reports in Section 10.0:

1) The methodology used to calculate the bending moments of inertia in the design analysis was different than that used in the independent analysis. The resulting bending moments of inertia differ by more than 15%. The effect of this difference on important building periods is from 6% to 15%.

2) Differences in the key properties calculations (fuel handling building stiffness, torsional rigidity of member 2, and centers of mass) and discrepancies between field and analyzed conditions suggest that design control measures were inadequate.

In addition to the two generic concerns, one specific concern related to the qualification of the auxiliary building was identified; this concern was considered specific because no other building analysis reports the use of soil springs.

1) Differences in the calculated values for soil springs were reported which have not been reconciled. Sensitivity studies indicate that the effects of variation of this parameter on important building periods is from 6% to 12%.

## 11.2 RECOMMENDATIONS

The following recommendations for additional verification are made to address the two generic concerns listed in Section 11.1.

1) Examine the analyses of the remaining buildings to determine the procedures used to calculate moments of inertia and reconcile differences in methodology.

2) Review design and field changes that affect models used in the seismic qualification of the remaining safety-related structures. Field verify selected changes to ensure that "as built" configurations conform to design drawings.

As a result of the PGandE re-analysis of the auxiliary building, the following recommendation for additional verification is made to address the one specific concern listed in Section 11.1.

1) Review the newly initated Diablo Canyon auxiliary building analysis. Verify corrective action for the specific concern listed in Section 11.1 and other changes made in the re-analysis.

All of the above review efforts associated with the identified concerns will be part of the overall IDVP verification effort of reviewing PGandE corrective action on structures.

## 12.0 CONCLUSION

The independent analysis of the auxiliary building has resulted in a number of open items which were combined with two error reports. EOI files 1028 and 1095 remain open with work ongoing; they will be resolved during the verification of the PGandE corrective action. These EOI reports lead to three concerns. Recommendations for the review of corrective actions have been made to address: two generic concerns for other structures, and the one specific concern for the auxiliary building re-analysis.

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Section 13.0

Tables

(8 Pages)

| Elevation | Translational Spring (kip/ft x $10^6$) | | | | Torsional Spring (kip-ft/rad x $10^9$) | |
|---|---|---|---|---|---|---|
| | N-S Direction | | E-W Direction | | | |
| | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 100.0 | 12.41 [a] | 26.26 | 12.41 [a] | 26.26 | 289.09 | 279.42 |

[a] Differ by more than 15%

Table 1

Comparison of Soil Springs

| Mass Point[a] | Elevation (ft) | Weight (kips) | | Mass Moment of Inertia (kip-ft-sec$^2$ x $10^3$) | |
|---|---|---|---|---|---|
| | | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 6 | 188.0 | 2664.0 | 2637.0 | 873.0 | 954.0 |
| 1 | 163.0 | 10378.0 | 11595.0 | 773.0 | 864.0 |
| 2 | 140.0 | 63075.0 | 58079.0 | 39513.0 | 38094.0 |
| 3 | 115.0 | 65552.0 | 64292.0 | 33748.0 | 38059.0 |
| 4 | 100.0 | 57744.0 | 58892.0 | 34859.0 | 35444.0 |

[a]Refer to Figures 13, 14 and 15

Table 2

Comparison of Weights and Mass Moments of Inertia
Horizontal Models

| Member[a] | Shear Area ($ft^2$) | | | | Moment of Inertia ($ft^4 \times 10^3$) | | | |
|---|---|---|---|---|---|---|---|---|
| | N-S Direction | | E-W Direction | | N-S Direction | | E-W Direction | |
| | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 1 | 782.0 | 846.0 | 678.0 | 682.0 | 1092.0[b] | 233.0 | 914.0[b] | 196.0 |
| 2 | 4595.0 | 5287.0 | 4390.0 | 4897.0 | 84793.0[b] | 3276.0 | 16450.0[b] | 1900.0 |
| 3 | 5510.0 | 6106.0 | 4781.0 | 5307.0 | 91801.0[b] | 2773.0 | 21125.0[b] | 3334.0 |
| 4 | 4065.0 | 4018.0 | 3234.0 | 3306.0 | 61742.0[b] | 3422.0 | 13314.0[b] | 2114.0 |

| Member[a] | $k_{N-S}$ Steel Structure (k/ft) | | $k_{E-W}$ Steel Structure (k/ft) | |
|---|---|---|---|---|
| | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 5 | 24977.0[b] | 15693.0 | 6110.0[b] | 12177.0 |

[a] Refers to Figures 13, 14 and 15

[b] Differ by more than 15%

Table 3A

Comparison of Shear Areas and Moments of Inertia

Horizontal Models

| Members[a] | Torsional Rigidity (kip-ft/rad x $10^6$) | |
|---|---|---|
| | Verification Analysis | Design Analysis |
| 5 | 86.8[b] | 298.0 |
| 1 | 29682.0 | 29785.0 |
| 2 | 968700.0[b] | 614800.0 |
| 3 | 1529900.0 | 1721200.0 |
| 4 | 787331.0 | 787250.0 |

[a]Refers to Figures 13, 14 and 15

[b]Differs by more than 15%

Table 3B

Comparison of Torsional Rigidities
Horizontal Models

| Mass Point [a] | Elevation (ft) | Center of Mass (ft) [b] | | Center of Rigidity (ft) [b] | | Difference Between (ft) Center of Mass & Rigidity | |
|---|---|---|---|---|---|---|---|
| | | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 6 | 188.0 | -81.5 | -81.5 | -97.50 | -81.5 | 16.0 | -0- |
| 1 | 163.0 | 67.4 | 67.7 | 72.2 | +66.9 | 4.8 | 0.8 |
| 2 | 140.0 | -20.6 | -24.9 | -32.8 | -33.6 | 12.3 | 8.7 |
| 3 | 115.0 | -23.0 | -27.6 | -33.9 | -34.1 | 10.9 | 6.5 |
| 4 | 100.0 | -30.3 | -35.7 | -14.2 | -14.1 | 16.1 | 21.6 |

[a] Refers to Figures 13, 14 and 15

[b] Reference to Column Line R (- East of R)

Table 4

Comparisons of Centers of Mass and Rigidity

North-South Model

| | 5% West Accidental Eccentricity Period (sec) | | 5% East Accidental Eccentricity Period (sec) | |
|---|---|---|---|---|
| Mode Number | Verification Analysis | Design Analysis | Verification Analysis | Design Analysis |
| 1 | 0.641[a] | 0.454 | 0.641[a] | 0.454 |
| 2 | 0.356 | 0.356 | 0.356 | 0.356 |
| 3 | 0.088 | 0.095 | 0.082 | 0.089 |
| 4 | 0.071 | 0.074 | 0.077 | 0.080 |
| 5 | 0.039 | 0.042 | 0.040 | 0.040 |
| 6 | 0.032 | 0.034 | 0.032 | 0.036 |
| 7 | 0.030 | 0.033 | 0.030 | 0.032 |

[a] Differs by more than 15%

Table 5A

Comparison of Periods of Vibration
North-South Model

| Mode Number | 5% Accidental Eccentricity Period (Sec) | |
|---|---|---|
| | Verification Analysis | Design Analysis |
| 1 | 0.731[a] | 0.500 |
| 2 | 0.630[a] | 0.352 |
| 3 | 0.091 | 0.097 |
| 4 | 0.076 | 0.081 |
| 5 | 0.043 | 0.045 |
| 6 | 0.033 | 0.037 |
| 7 | 0.032 | 0.033 |

[a] Differs by more than 15%

Table 5B

Comparison of Periods of Vibration
East-West Model

| Parameter \ Model | Design Model | | | | Verification Model |
|---|---|---|---|---|---|
| Soil Spring* | None | None | Design URS/Blume | Design URS/Blume | IDVP |
| Moment of Inertia** | Design | IDVP | Design | IDVP | IDVP |
| MODE NUMBER | PERIOD (Sec) | PERIOD (Sec) | PERIOD (Sec) | PERIOD (Sec) | PERIOD (Sec) |
| 1 | 0.456 | 0.455 | 0.454 | 0.455 | 0.641 |
| 2 | 0.356 | 0.356 | 0.356 | 0.356 | 0.356 |
| 3 | 0.099 | 0.095 | 0.095 | 0.088 | 0.088 |
| 4 | 0.084 | 0.072 | 0.074 | 0.064 | 0.071 |
| 5 | 0.043 | 0.041 | 0.042 | 0.039 | 0.039 |
| 6 | 0.036 | 0.034 | 0.034 | 0.033 | 0.032 |
| 7 | 0.033 | 0.033 | 0.033 | 0.032 | 0.030 |

* - Refers to Table 1

** - Refers to Table 3A

Table 6
Comparison of Periods of Vibration
North-South Model
5% West Accidental Eccentricity
Parameter Studies

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Section 14.0

Figures

(36 Pages)

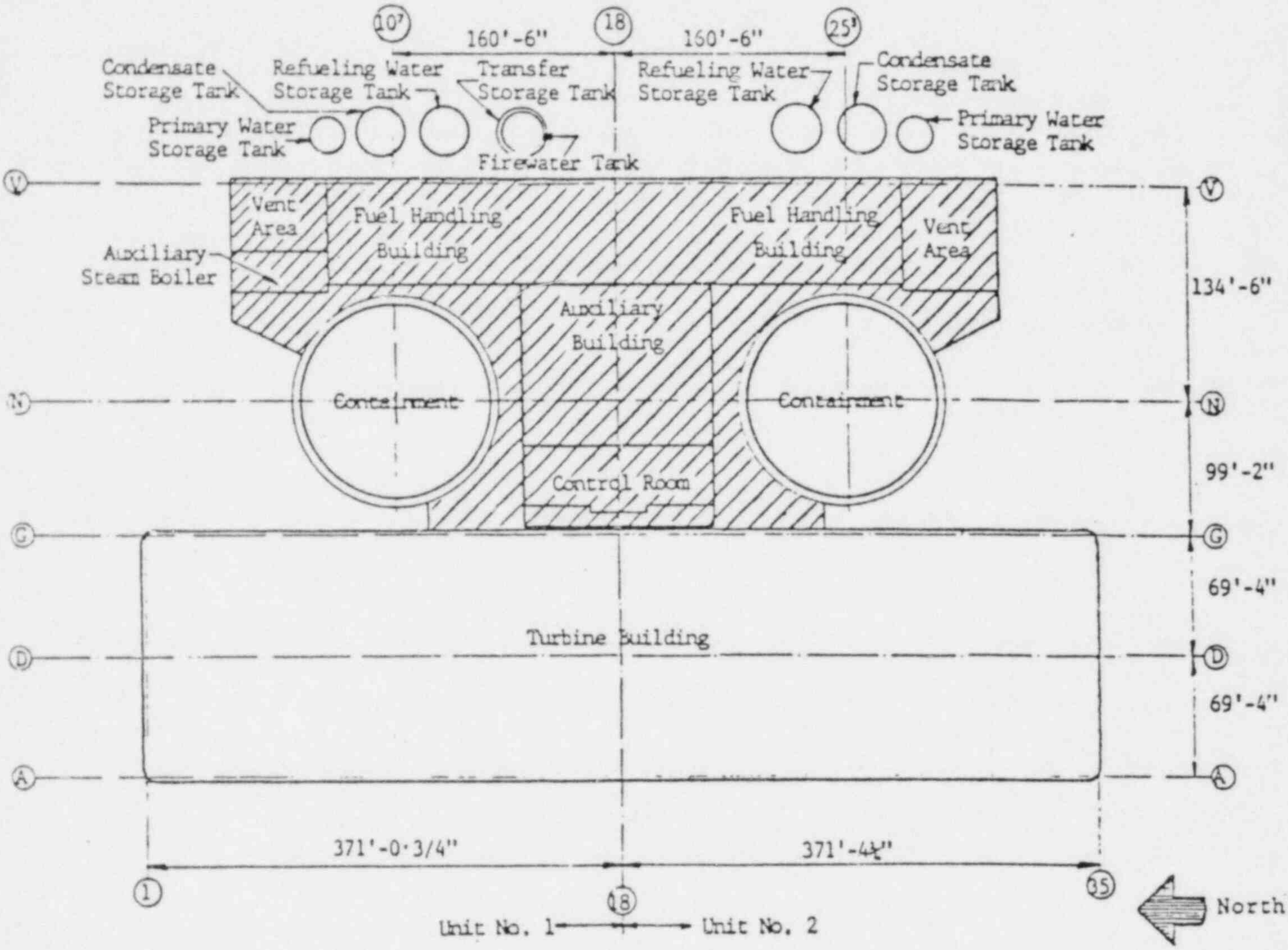


Figure 1
Site Plan for Units 1 and 2
Diablo Canyon Nuclear Power Plant

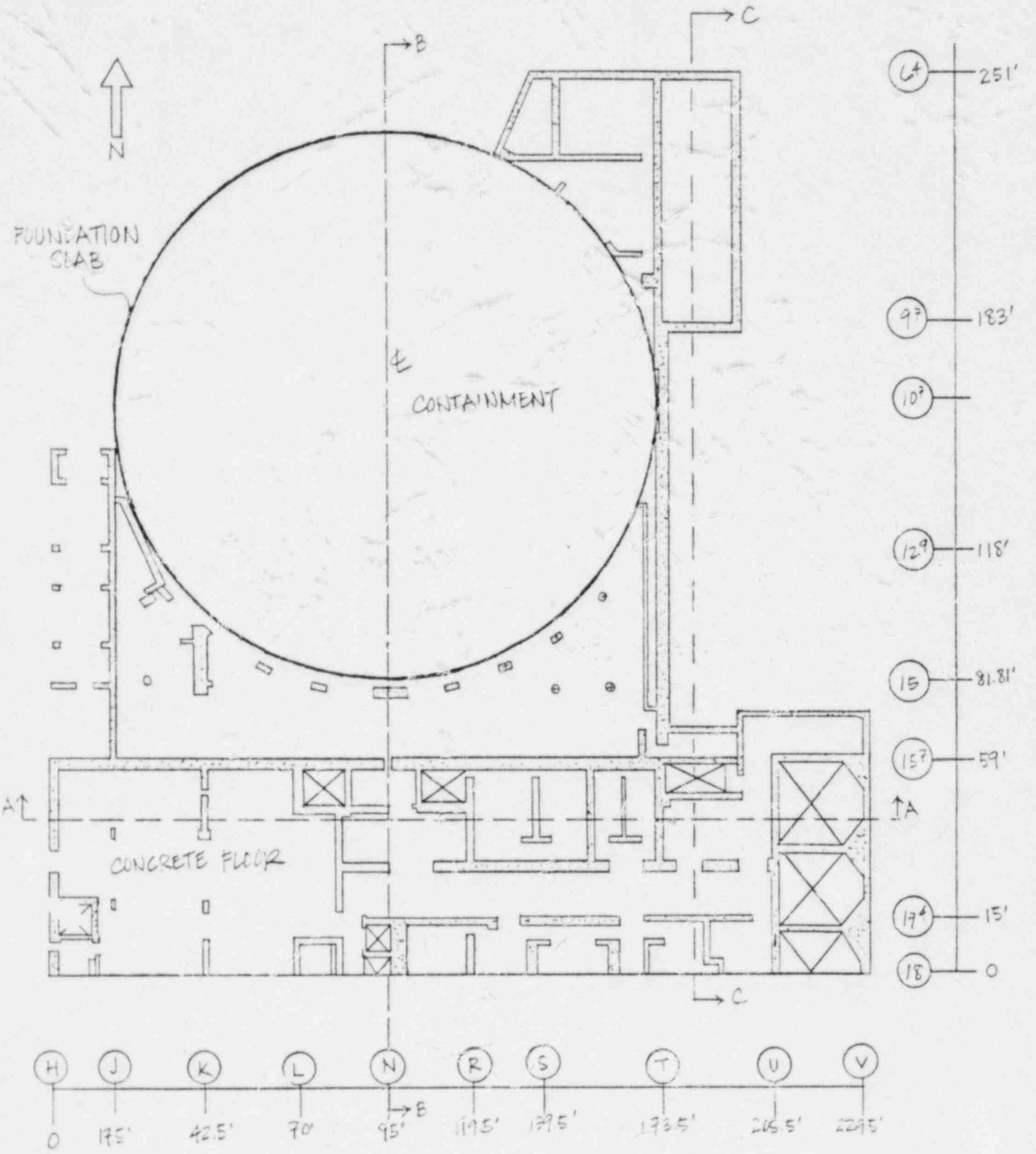


Figure 2
Unit 1 Floor Plan At Elevation 85'-0"

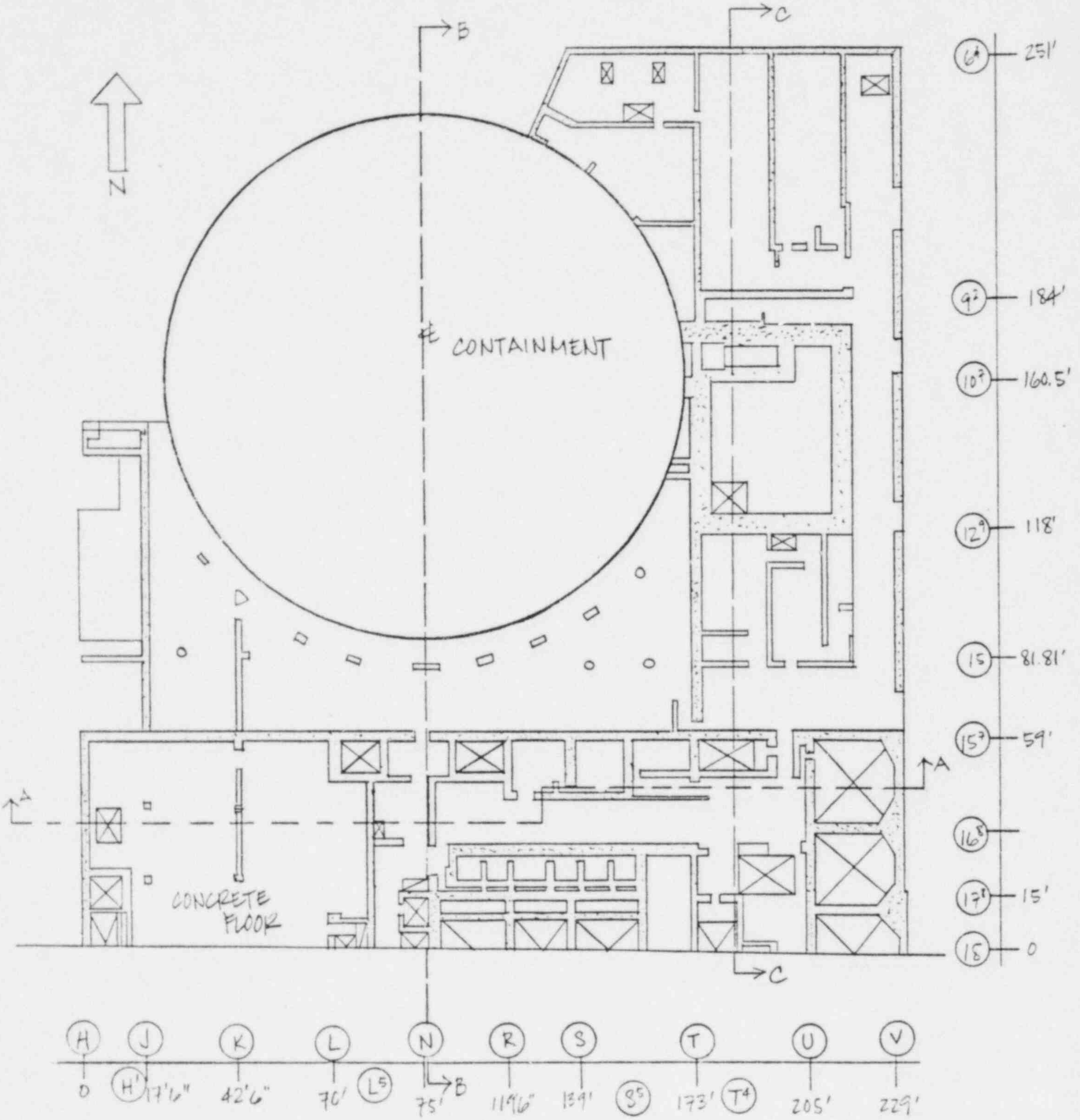


Figure 3
Unit 1 Floor Plan at Elevation 100'-0"

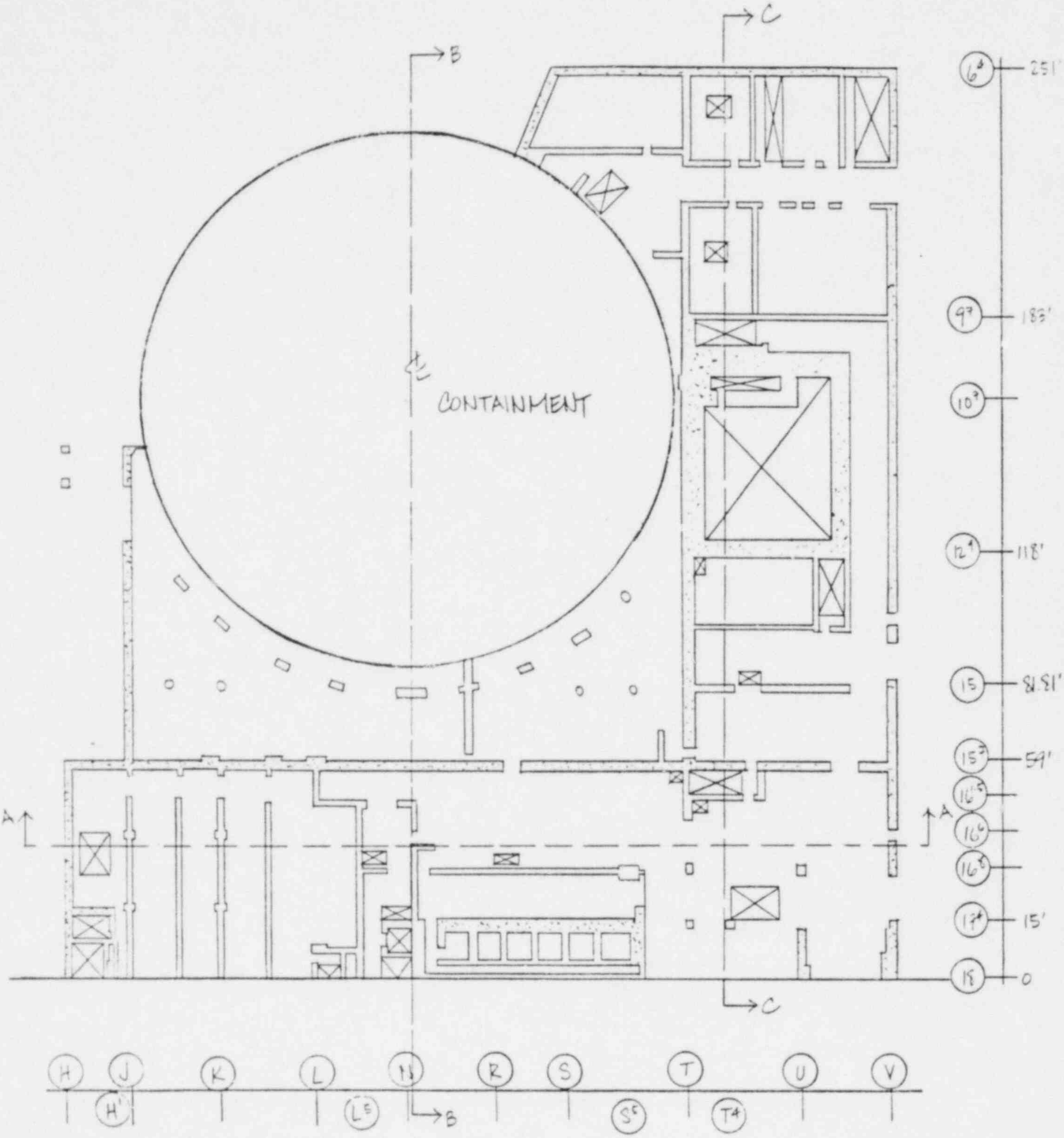


Figure 4

Unit 1 Floor Plan at Elevation 115'-0"

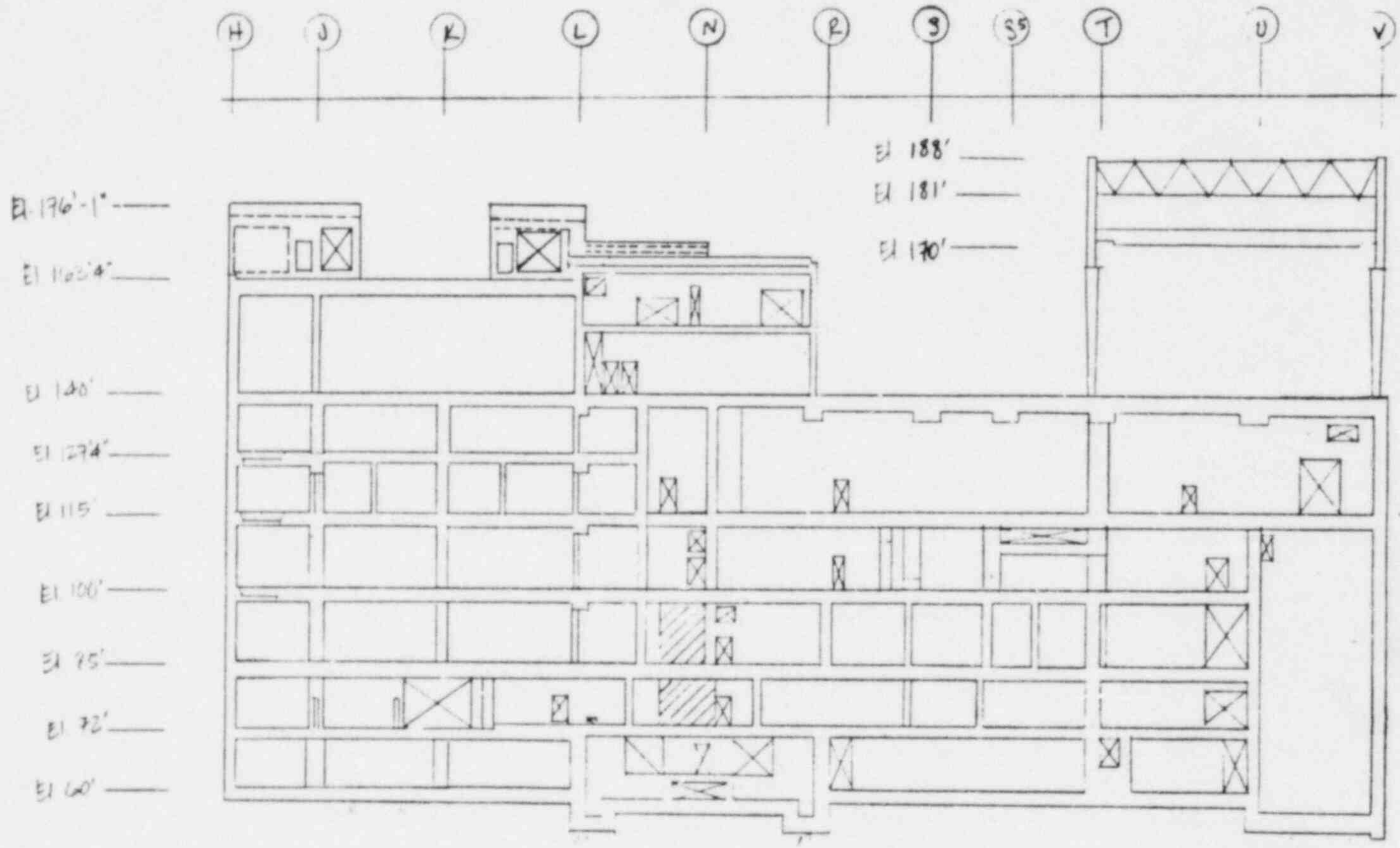


Figure 6

Section A-A Unit 1

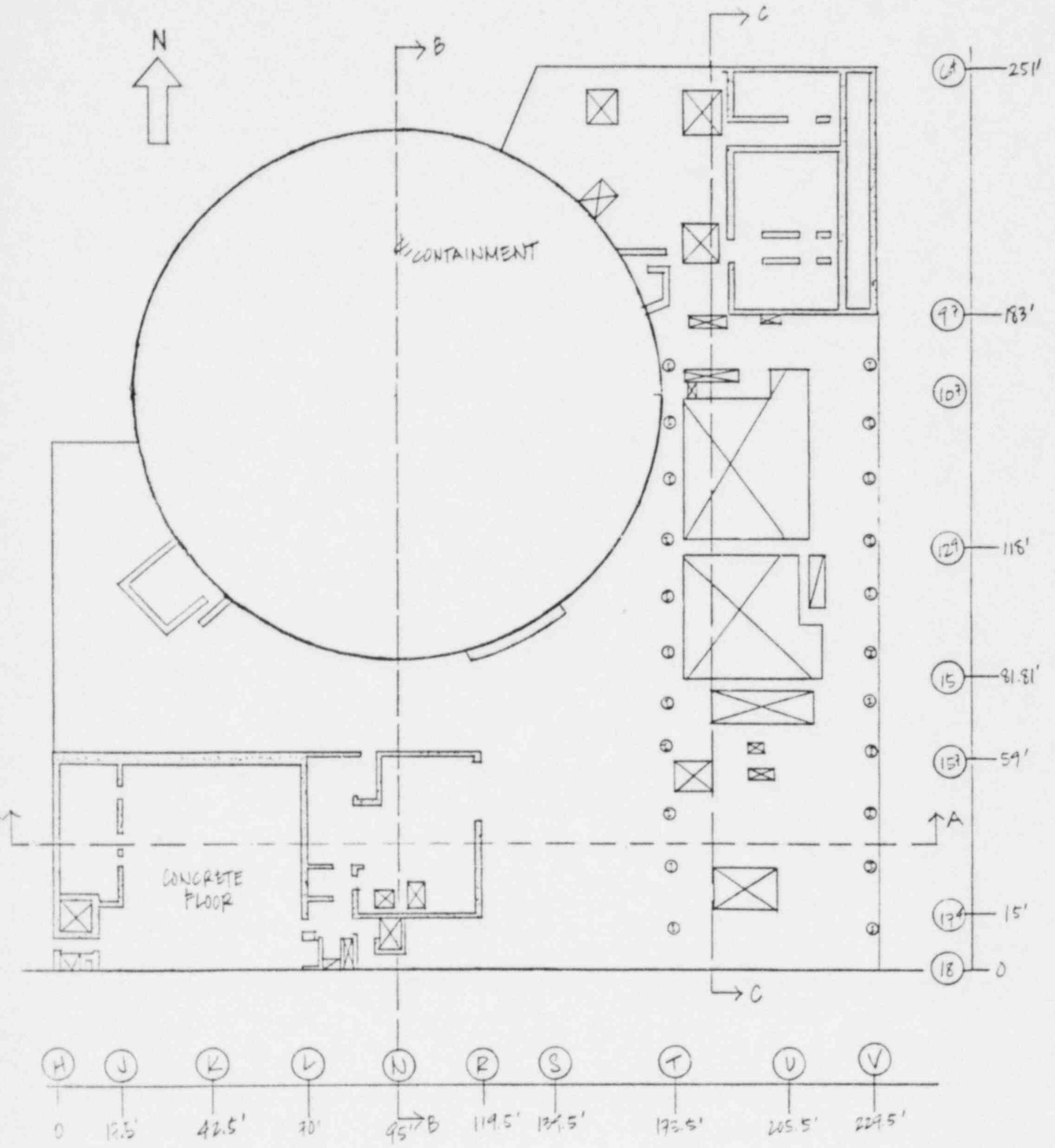


Figure 5

Unit 1 Floor Plan at Elevation 140'-0"

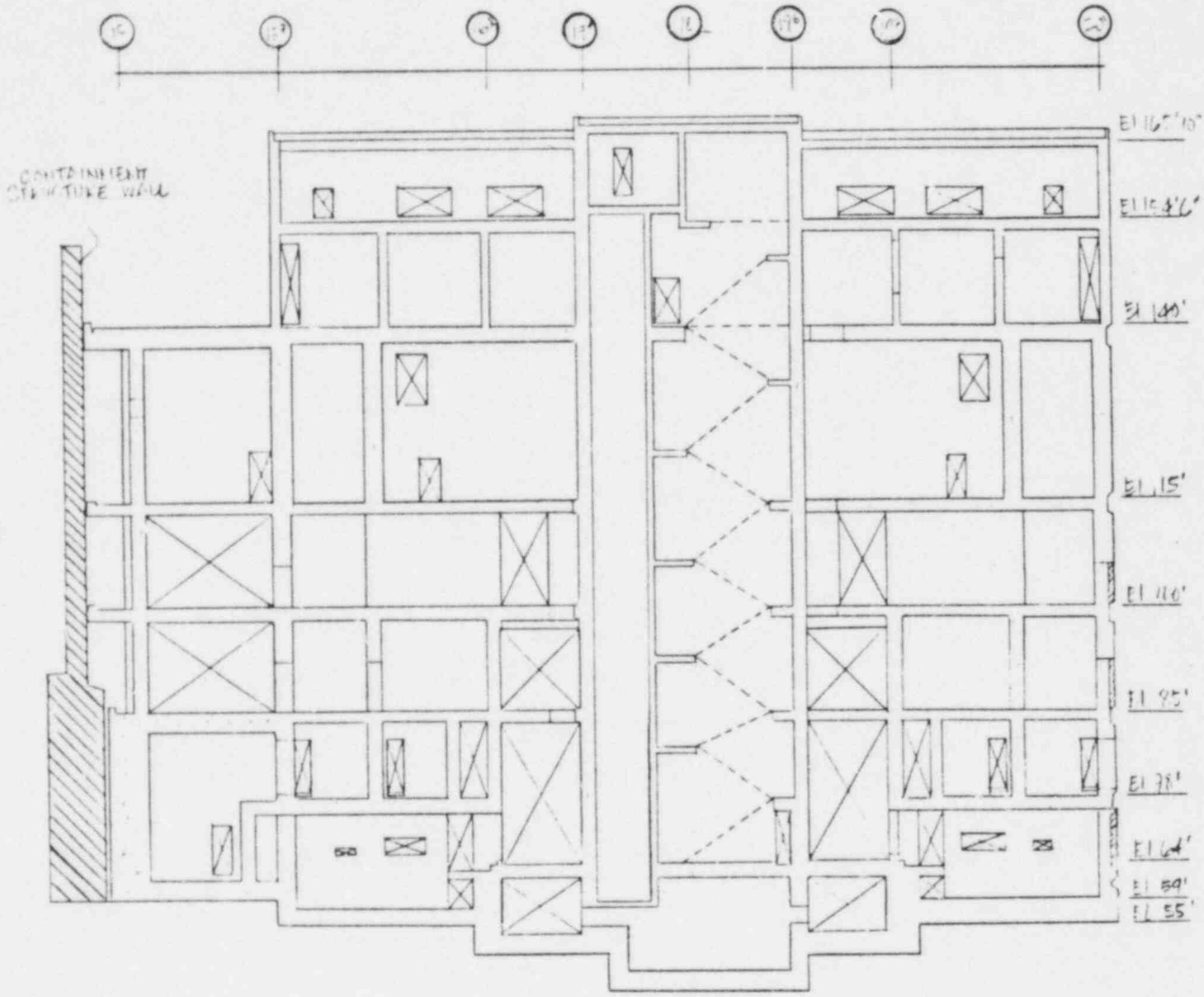


Figure '

Section B-B Unit 1

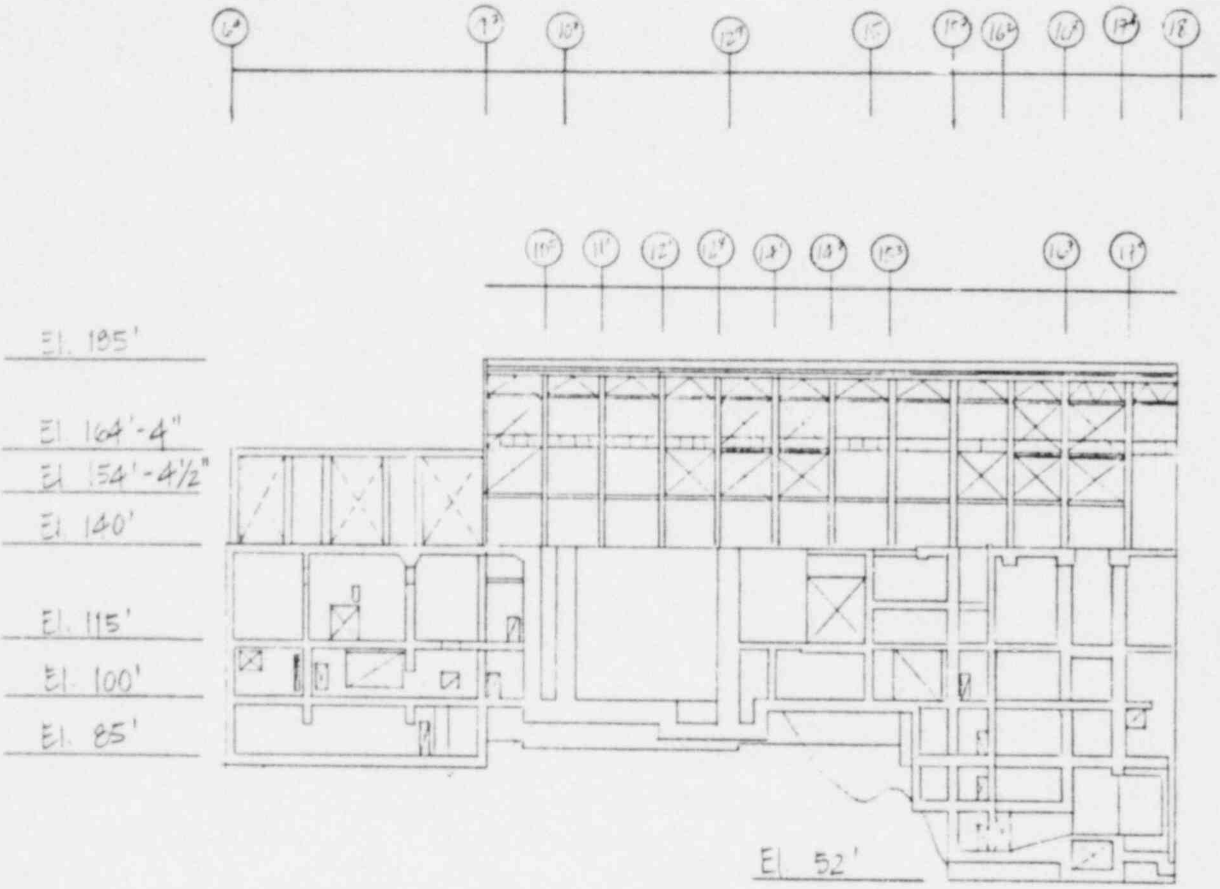


Figure 8

Section C-C Unit 1

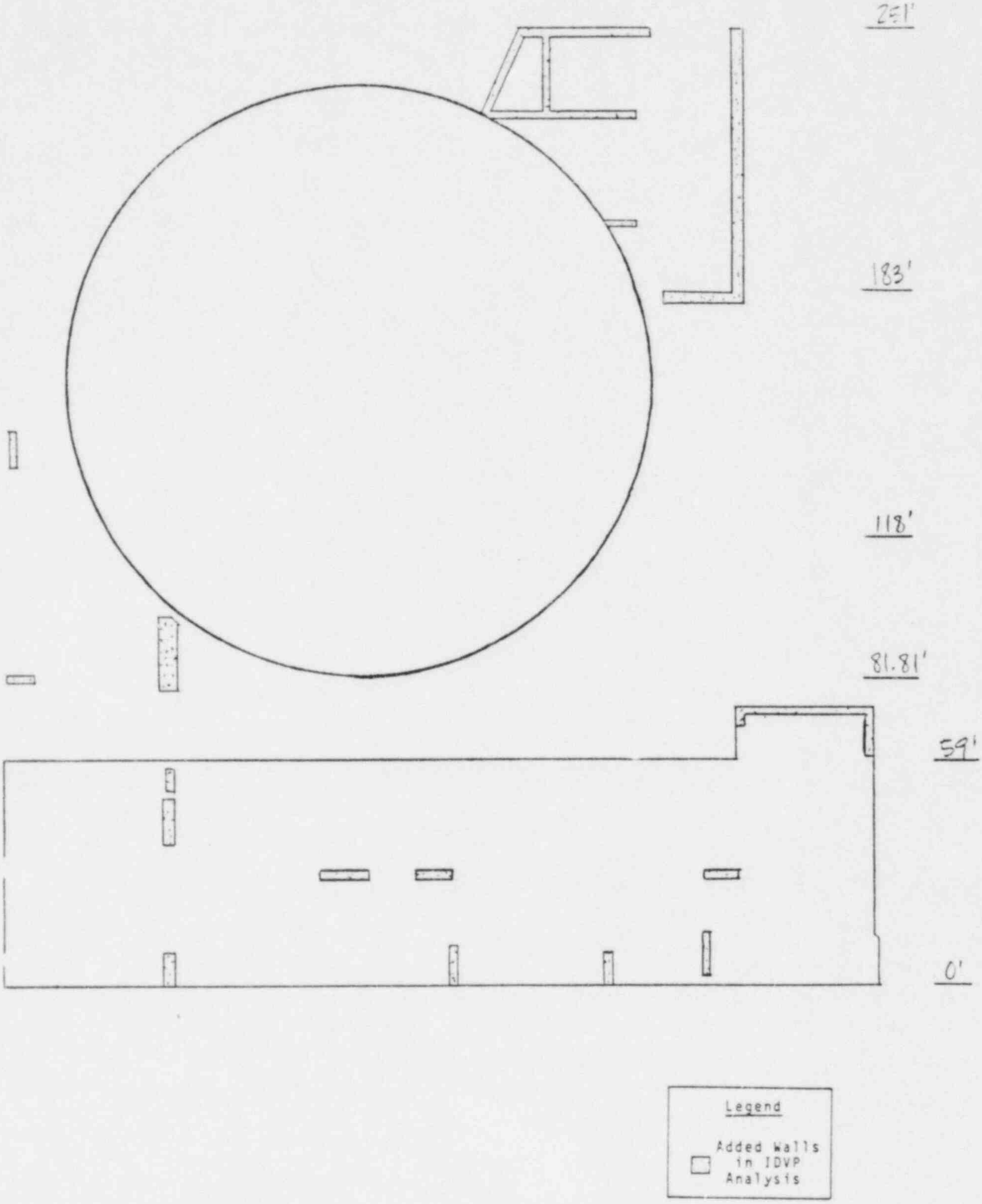


Figure 9

Differences Between Walls Considered in IDVP Analysis and Those Shown in 1971 Blume Report

Elevation 85'-0"

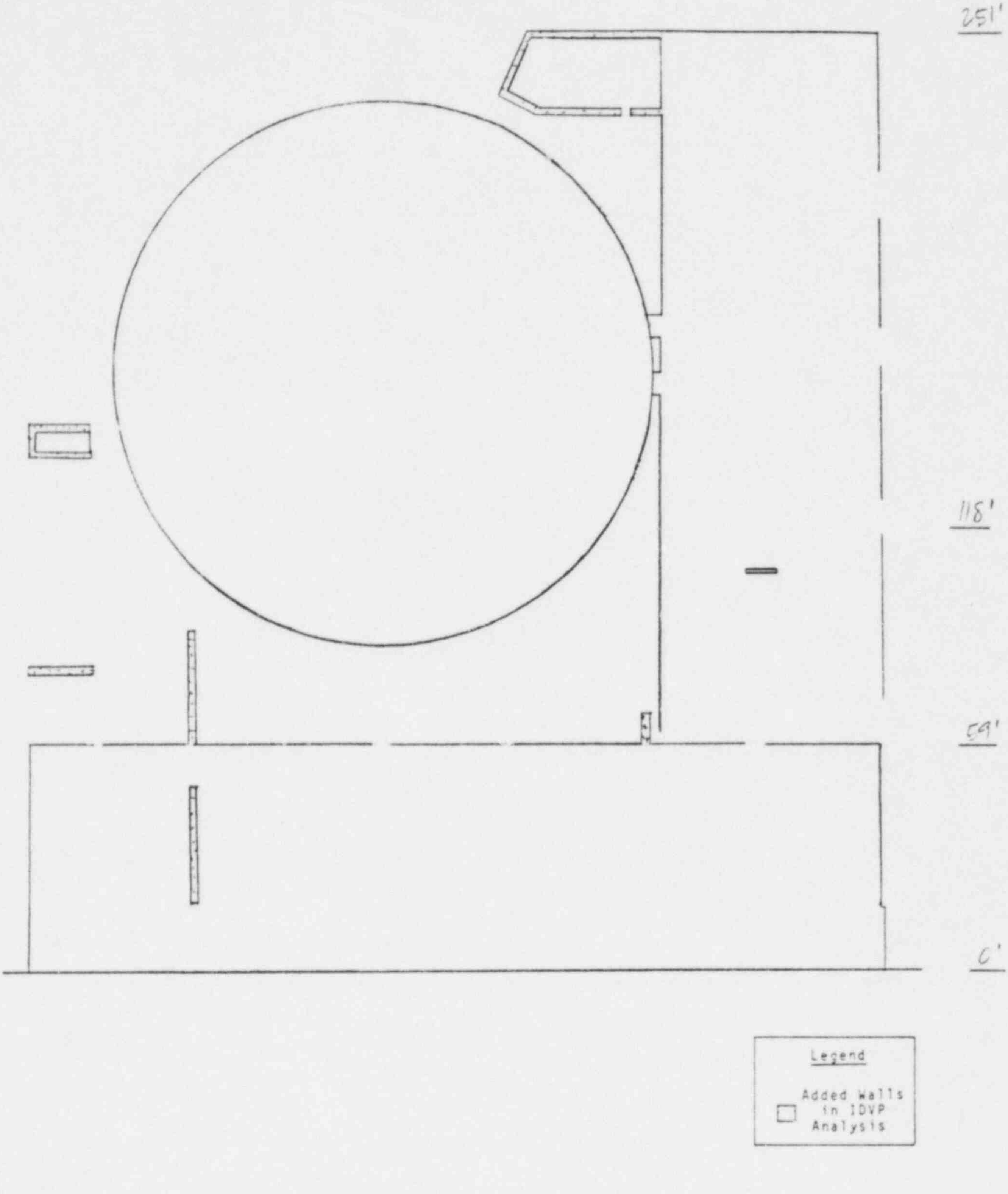


Figure 10

Differences Between Walls Considered in IDVP Analysis and Those Shown in 1971 Blume Report
Elevation 100'-0"

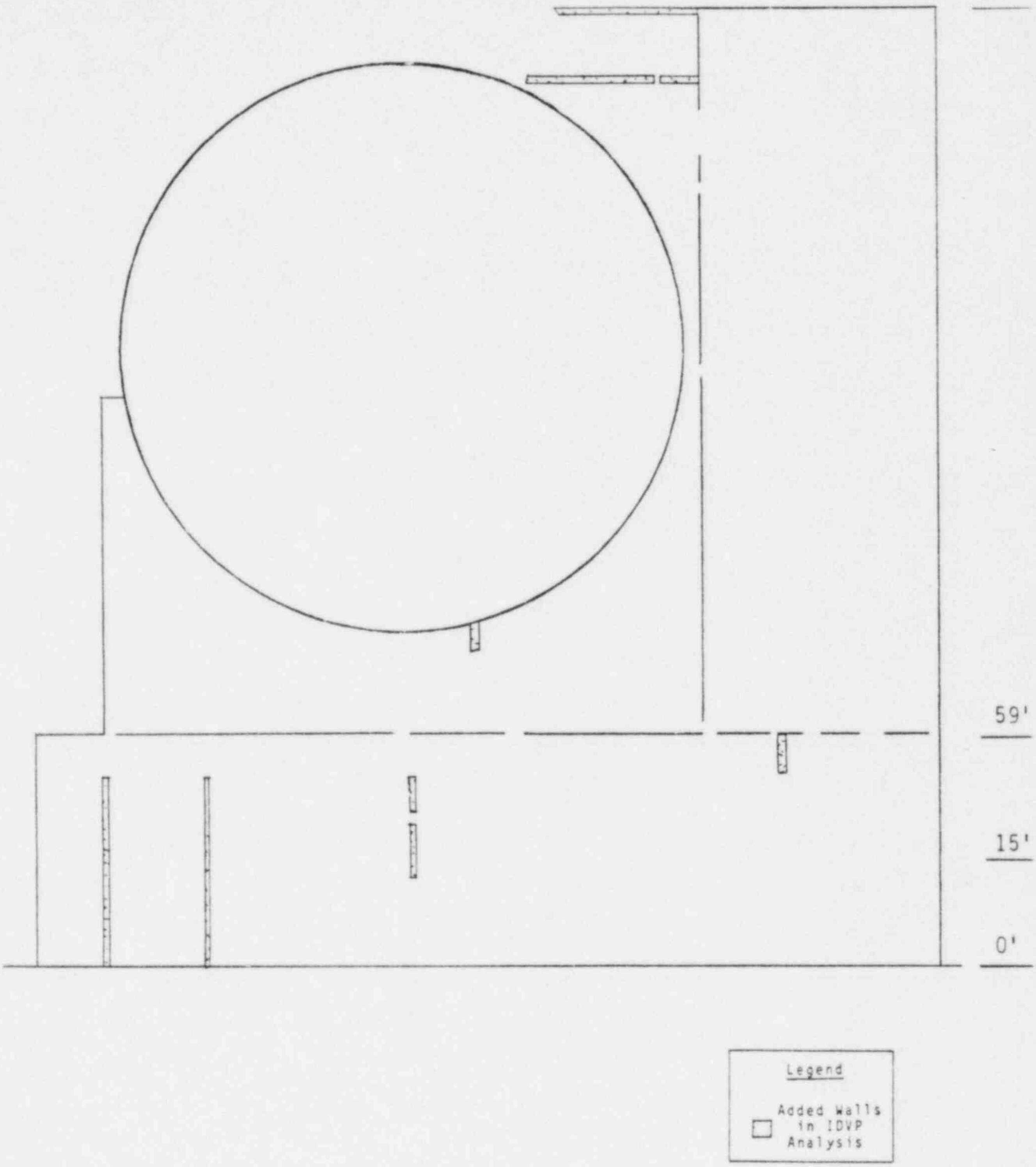


Figure 11

Differences Between Walls Considered in IDVP Analvsis and Those Shown in 1971 Blume Report Elevat'on 115'-0"

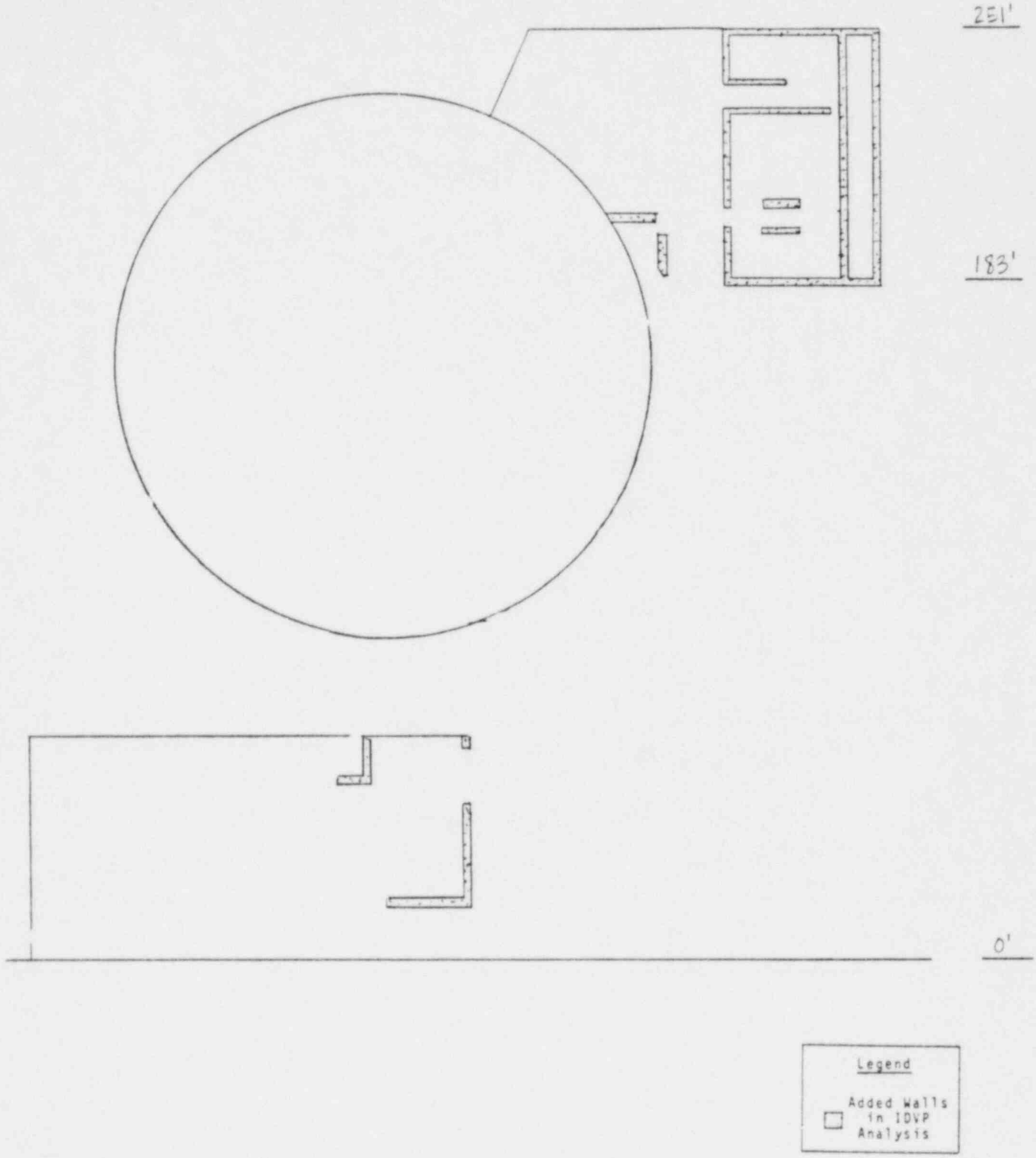


Figure 12

Differences Between Walls Considered in IDVP Analysis and Those Shown in 1971 Blume Report
Elevation 140'-0"

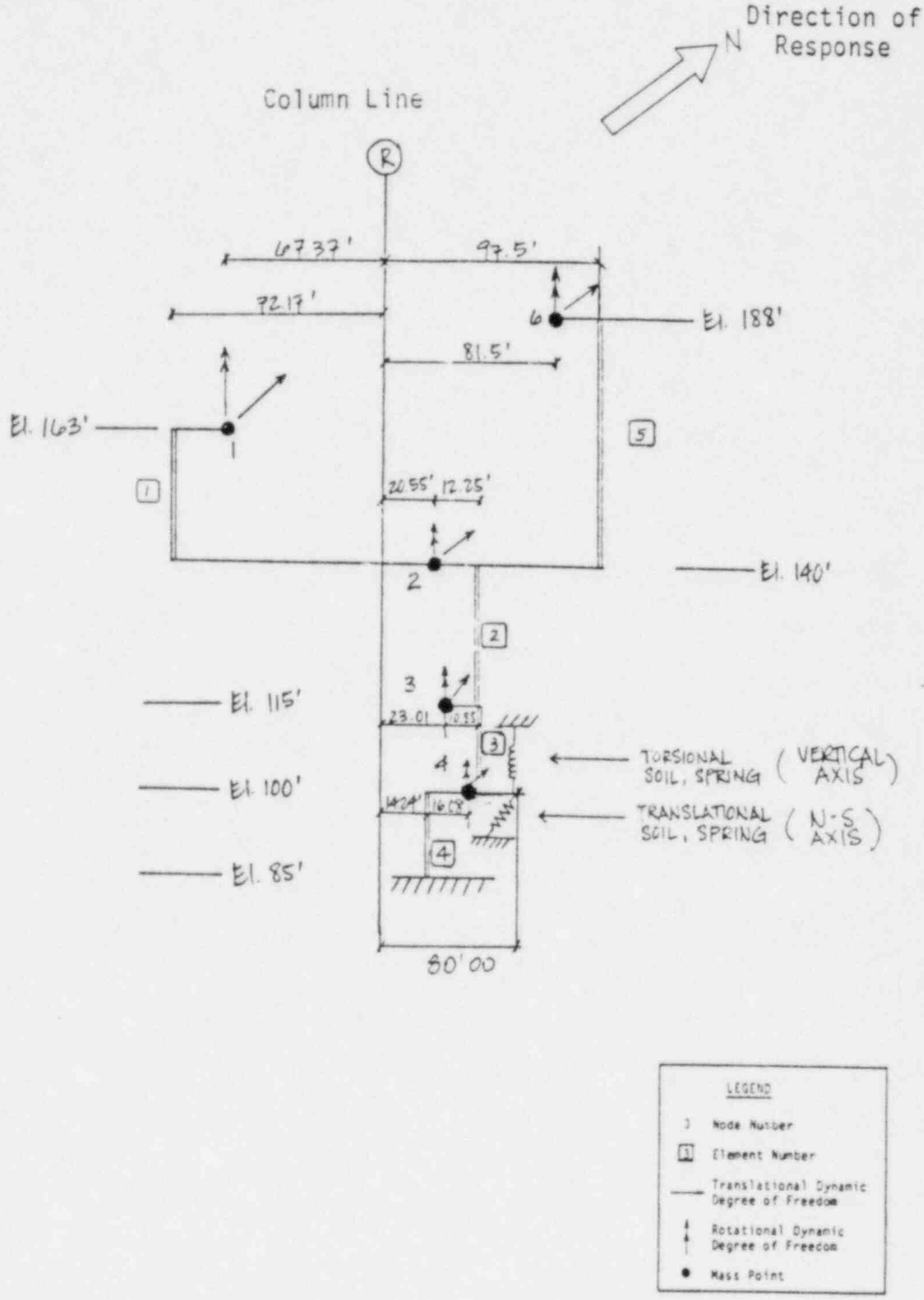


Figure 13
Verification Mathematical Model for North-South Direction

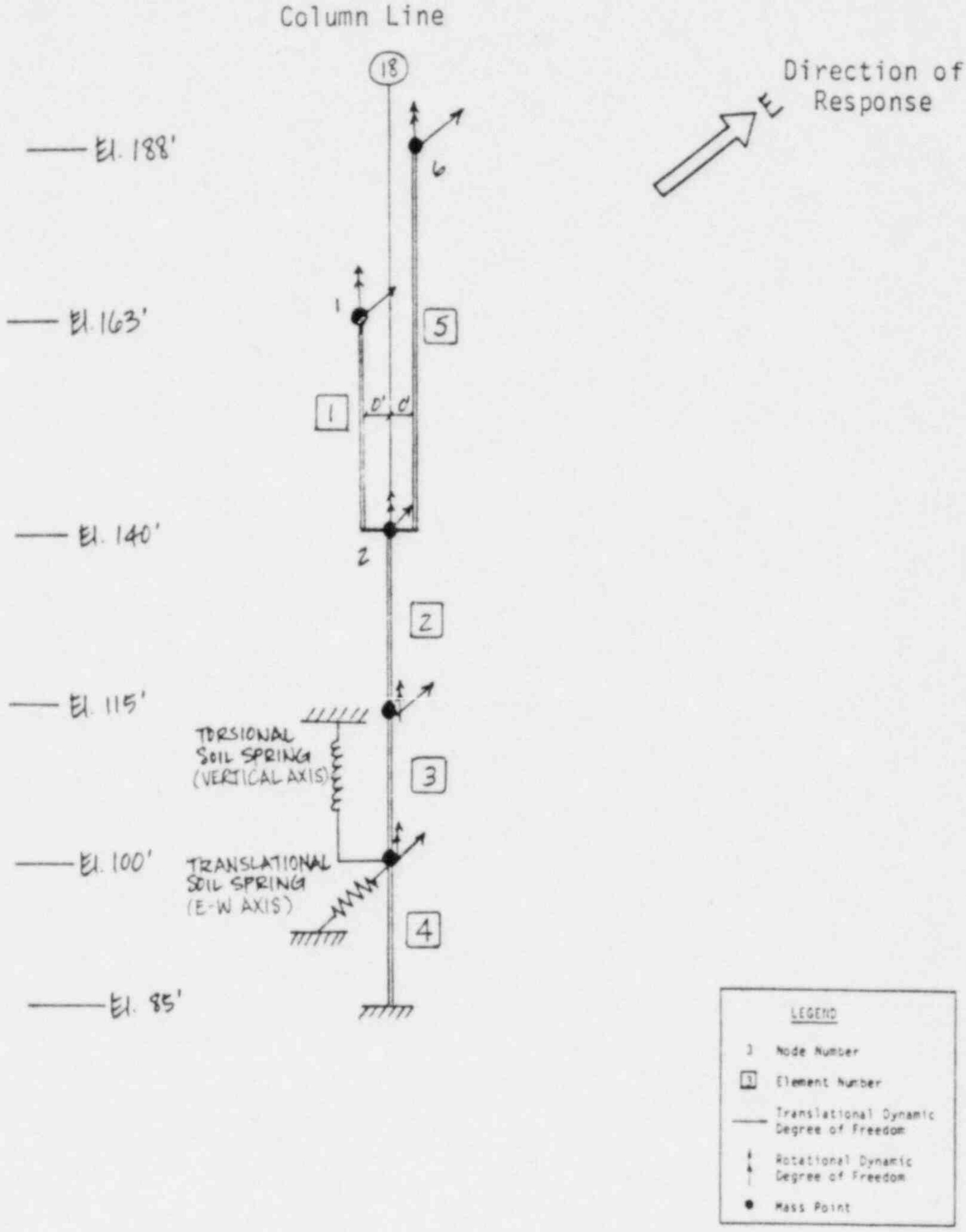


Figure 14

Verification Mathematical Model for East-West Direction

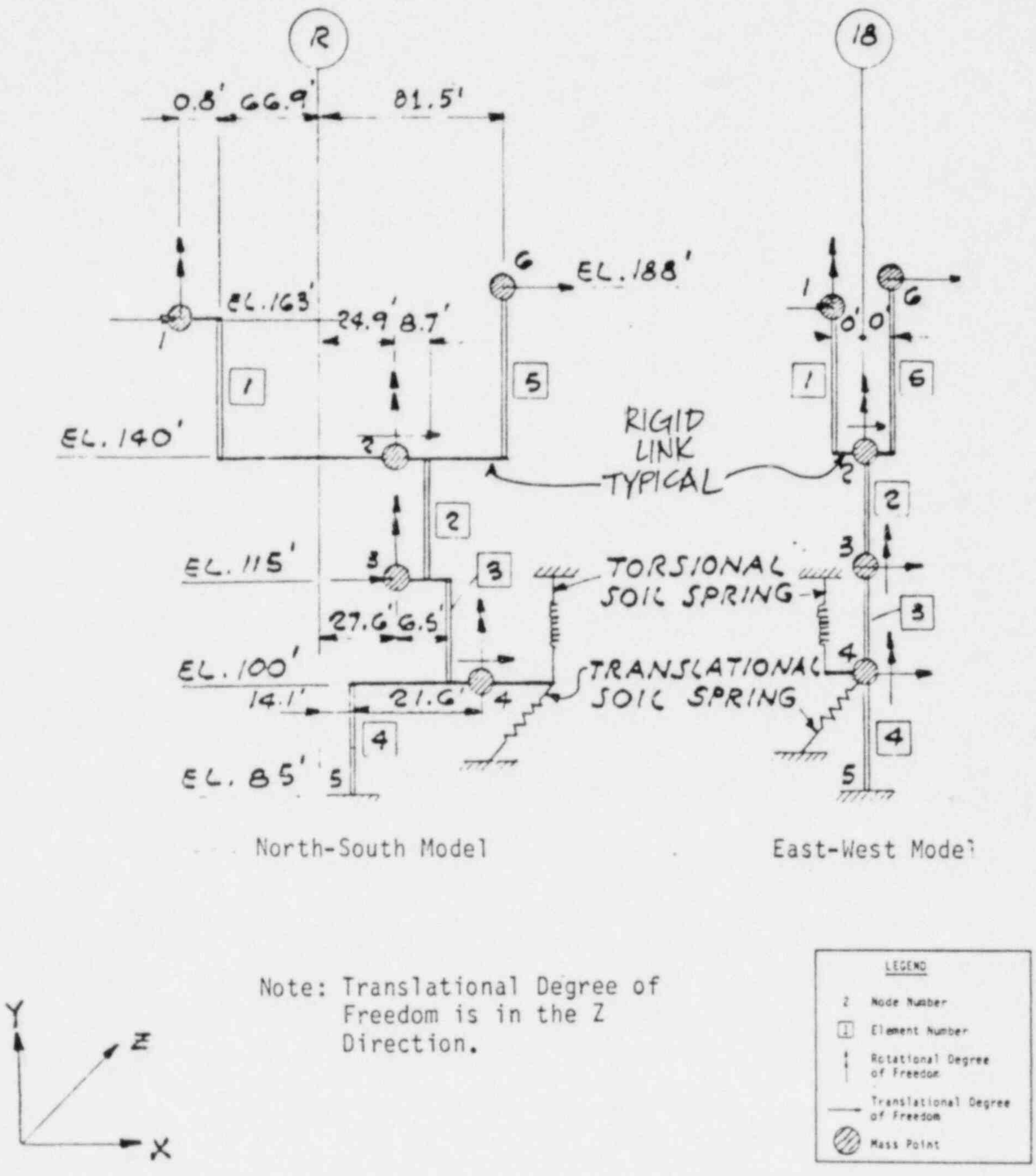


Figure 15
Design Mathematical Models for
North-South and East-West Directions
(Figure taken directly from Reference 4)

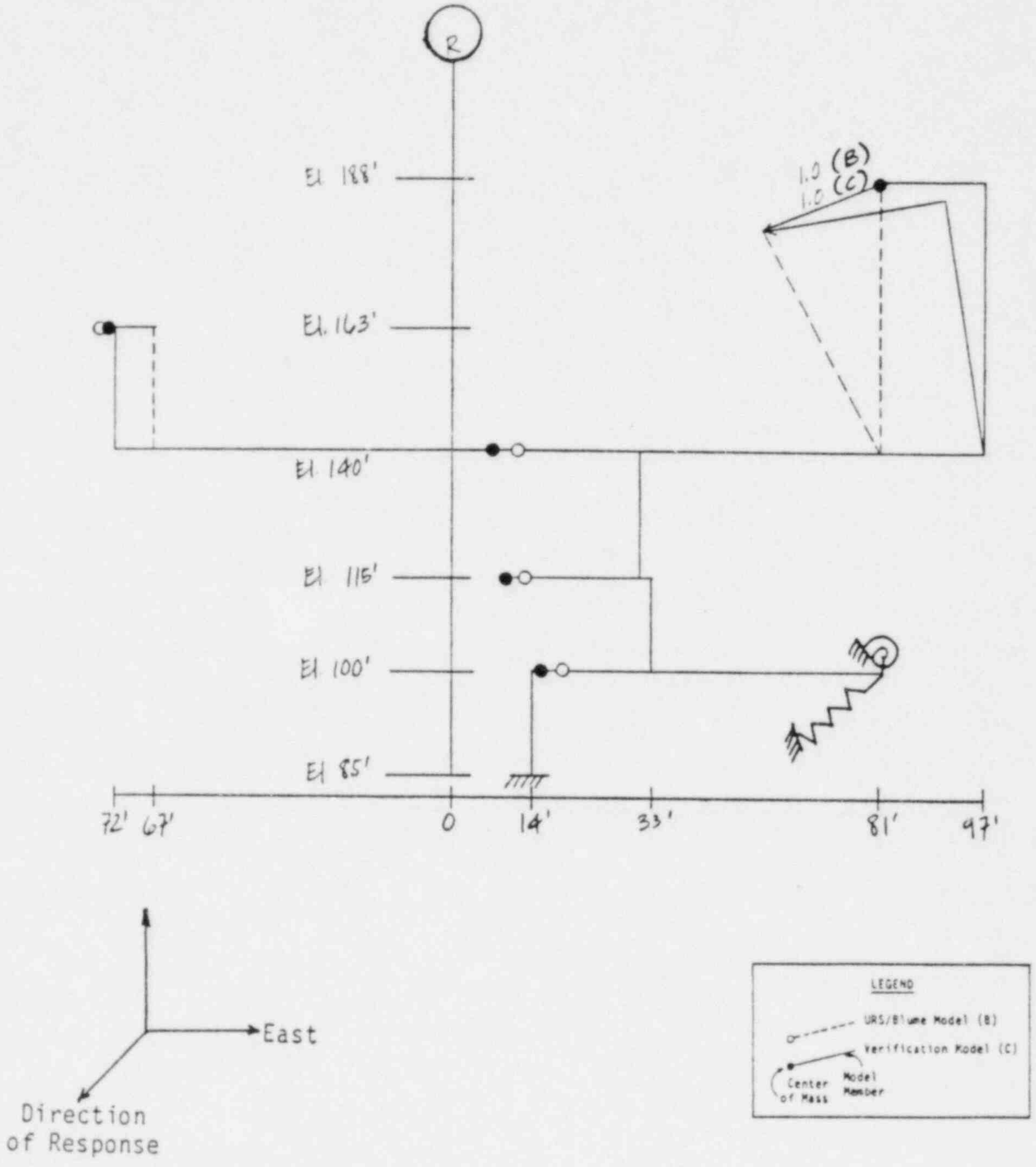


Figure 16

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 1.2%, Design = Not Available
Mode 1

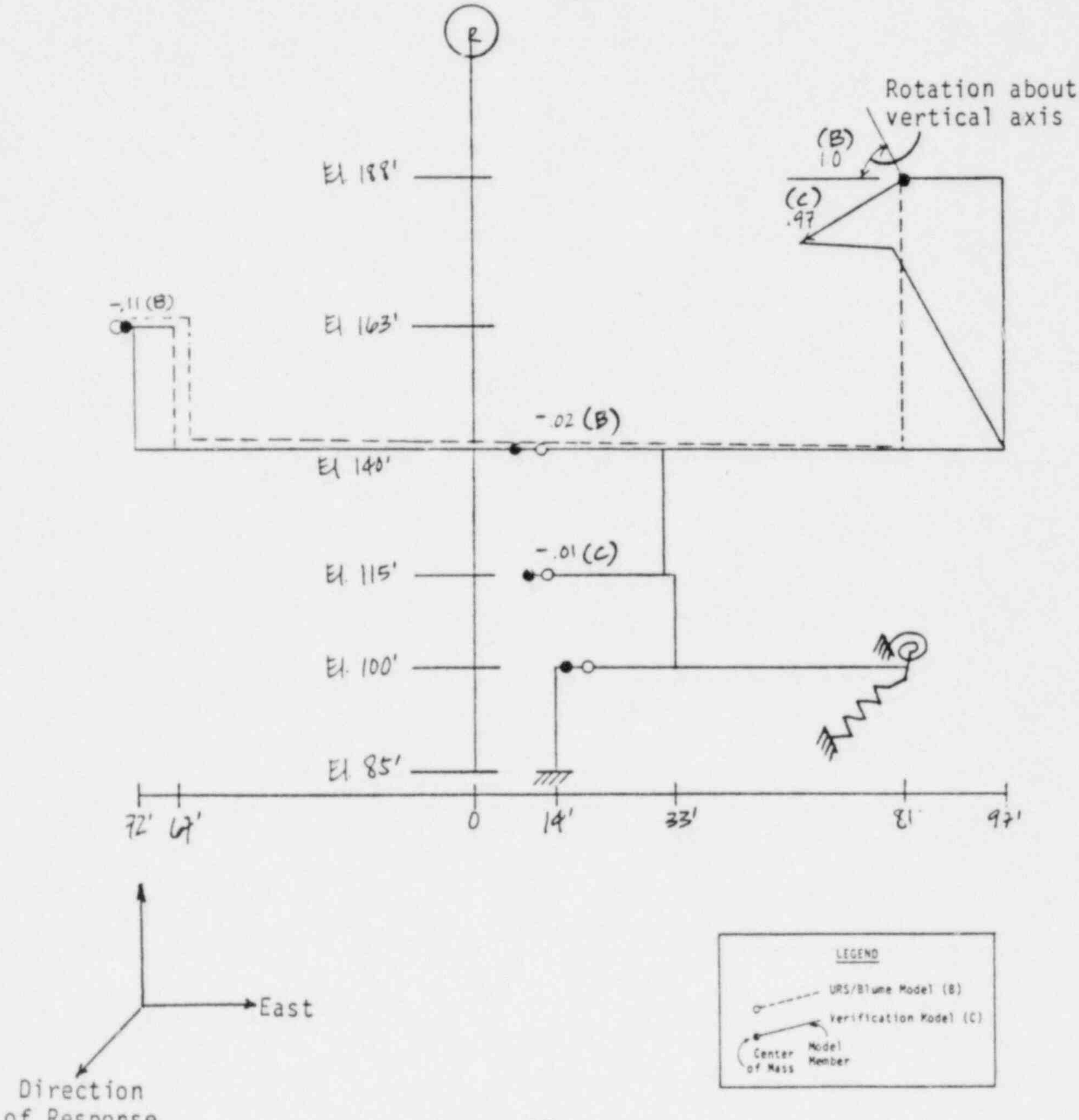


Figure 17

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 23.5%, Design = Not Available
Mode 2

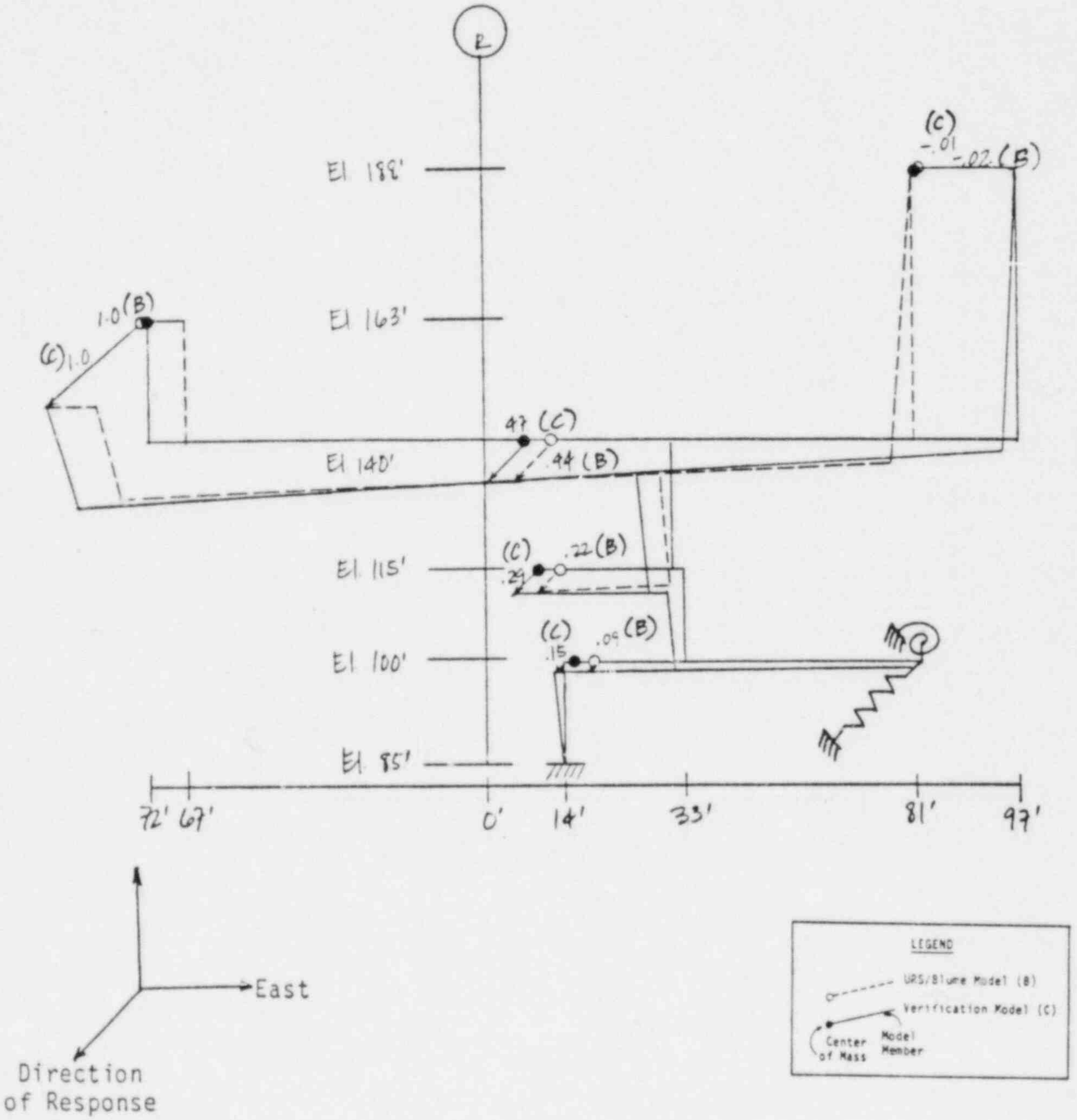


Figure 18

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 25.3%, Design = Not Available
Mode 3

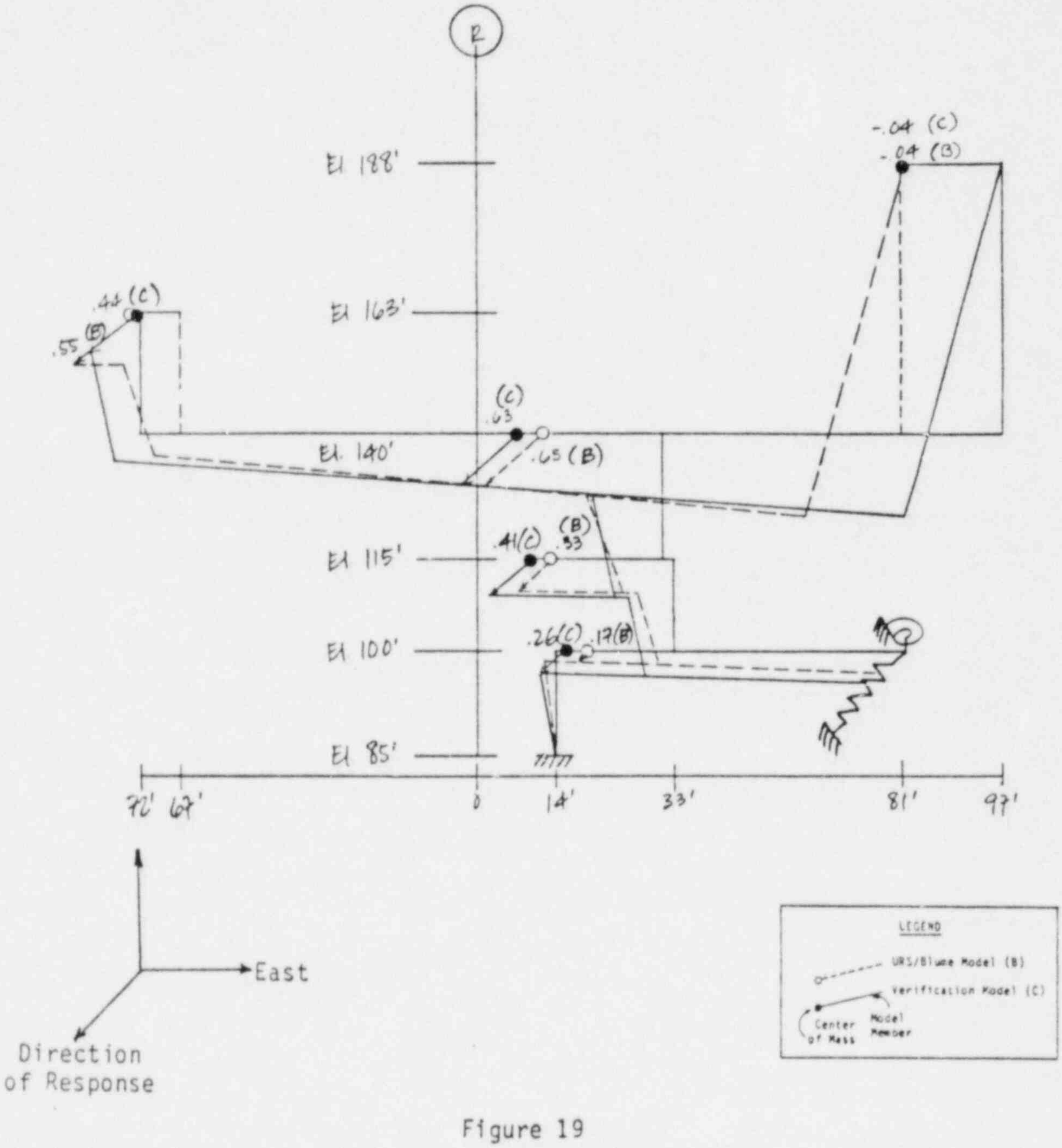


Figure 19

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 24.4%, Design = Not Available
Mode 4

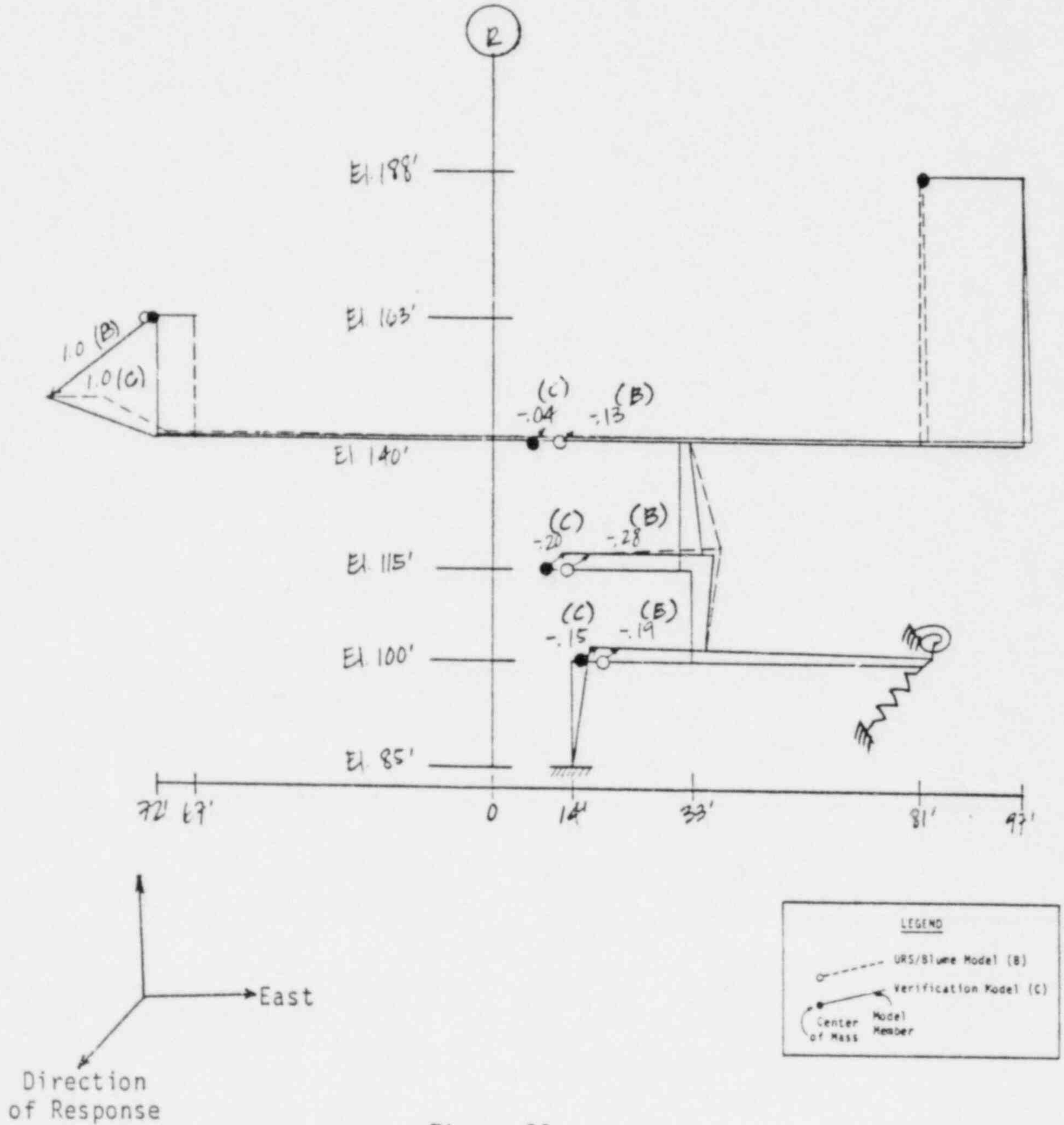


Figure 20

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 20.1%, Design = Not Available
Mode 5

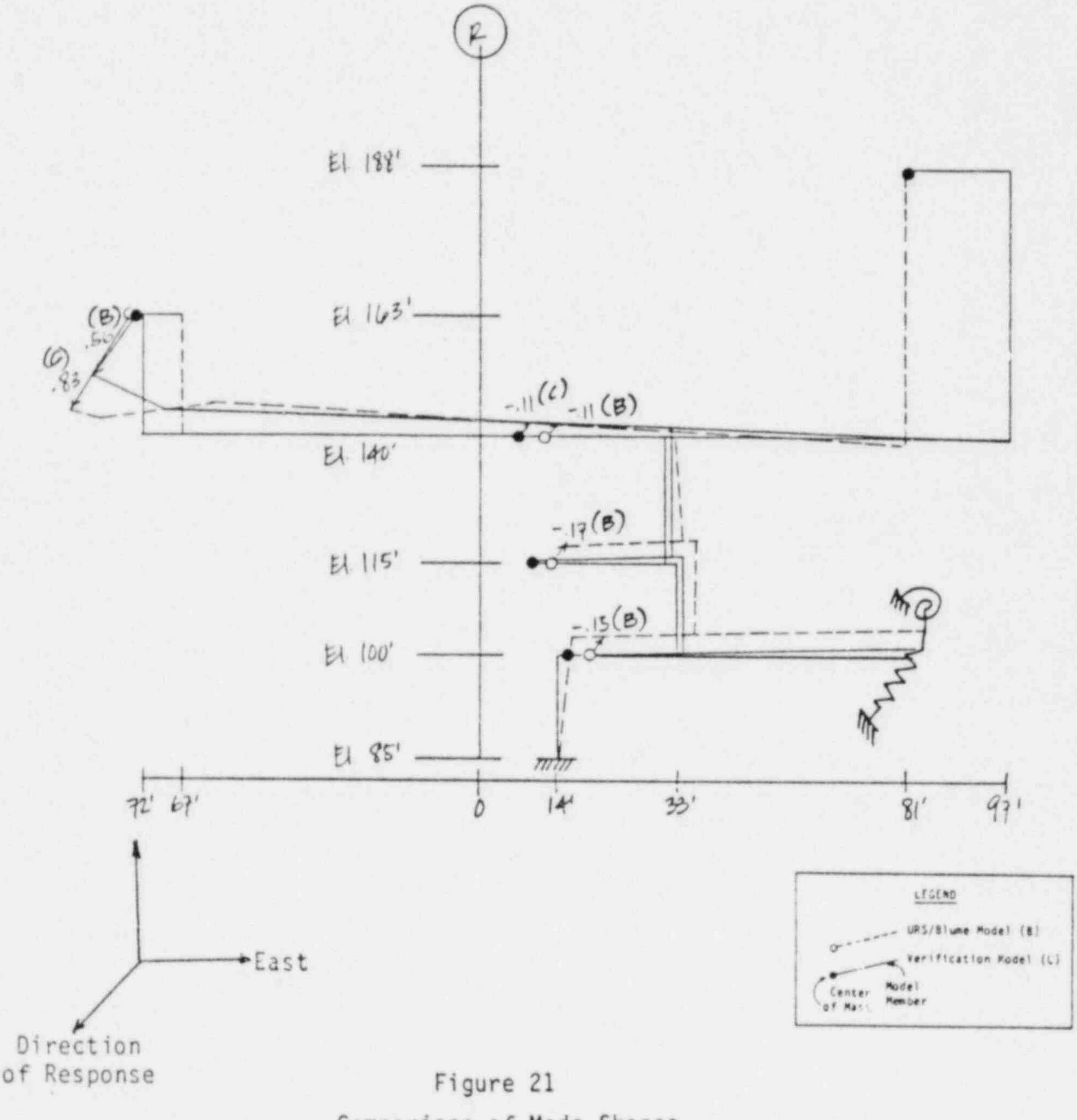


Figure 21

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = .2%, Design = Not Available
Mode 6

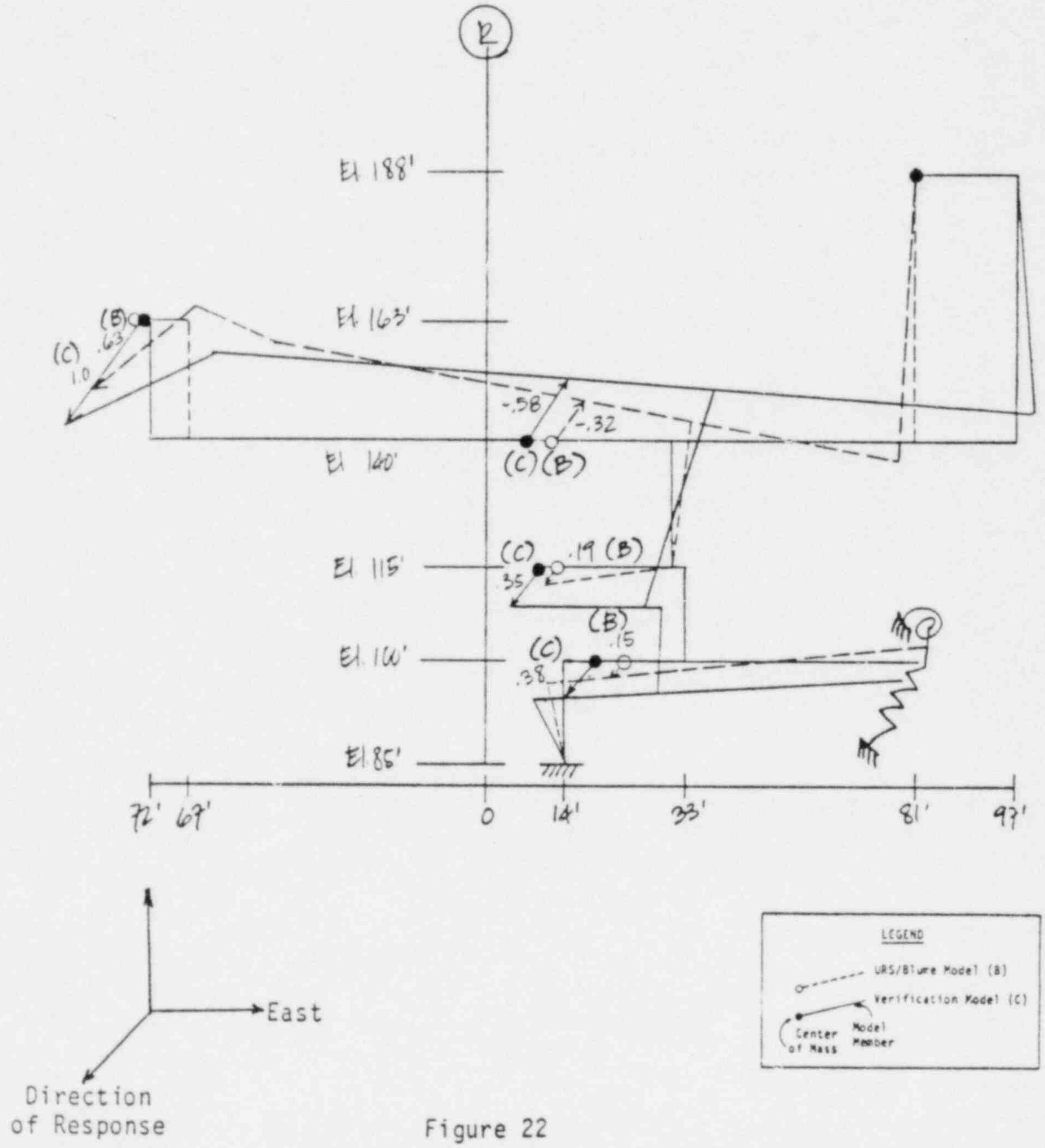


Figure 22

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to West
Participation Factor: IDVP = 5.4%, Design = Not Available
Mode 7

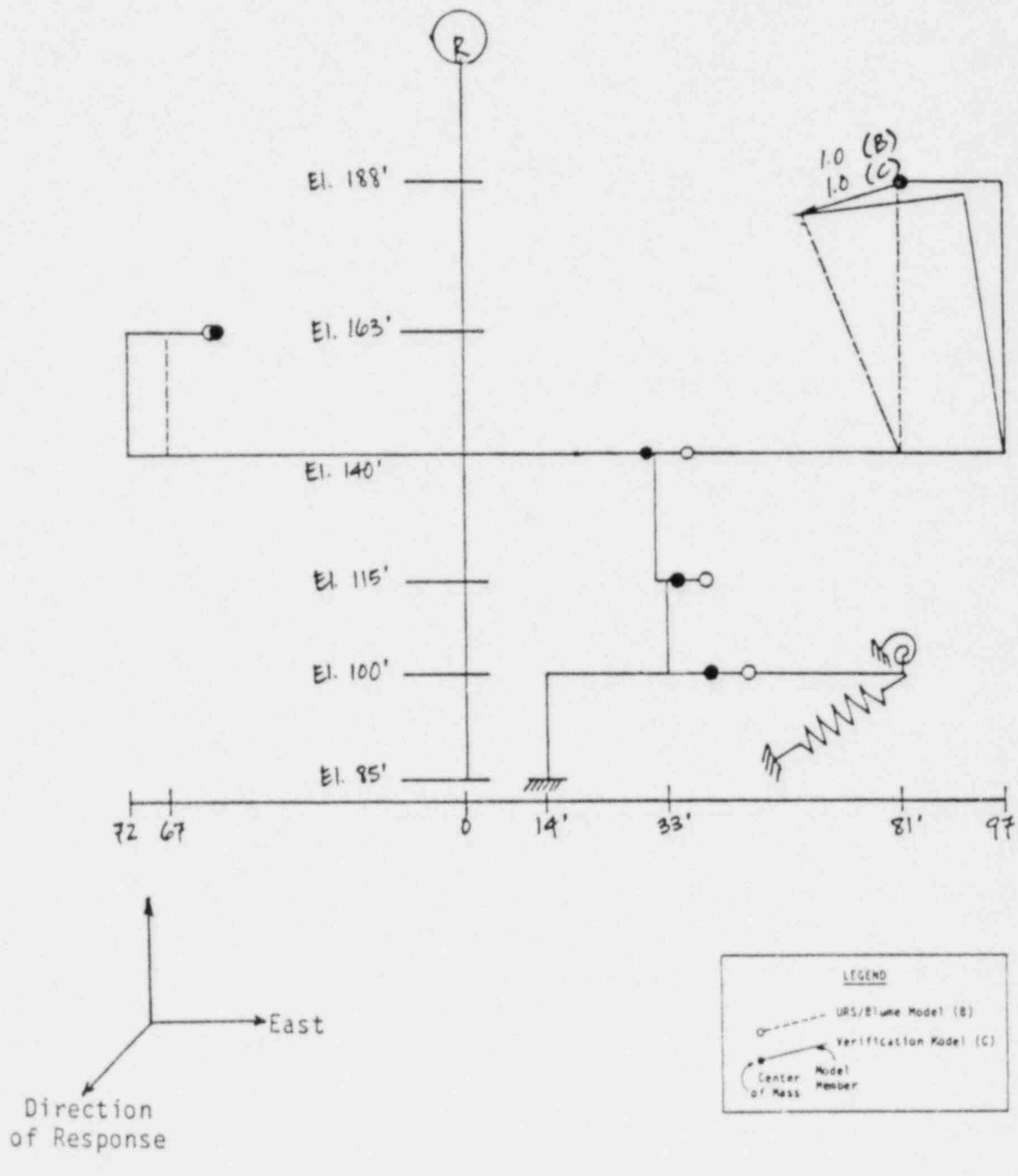


Figure 23

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 1.2%, Design = 6.3%
Mode 1

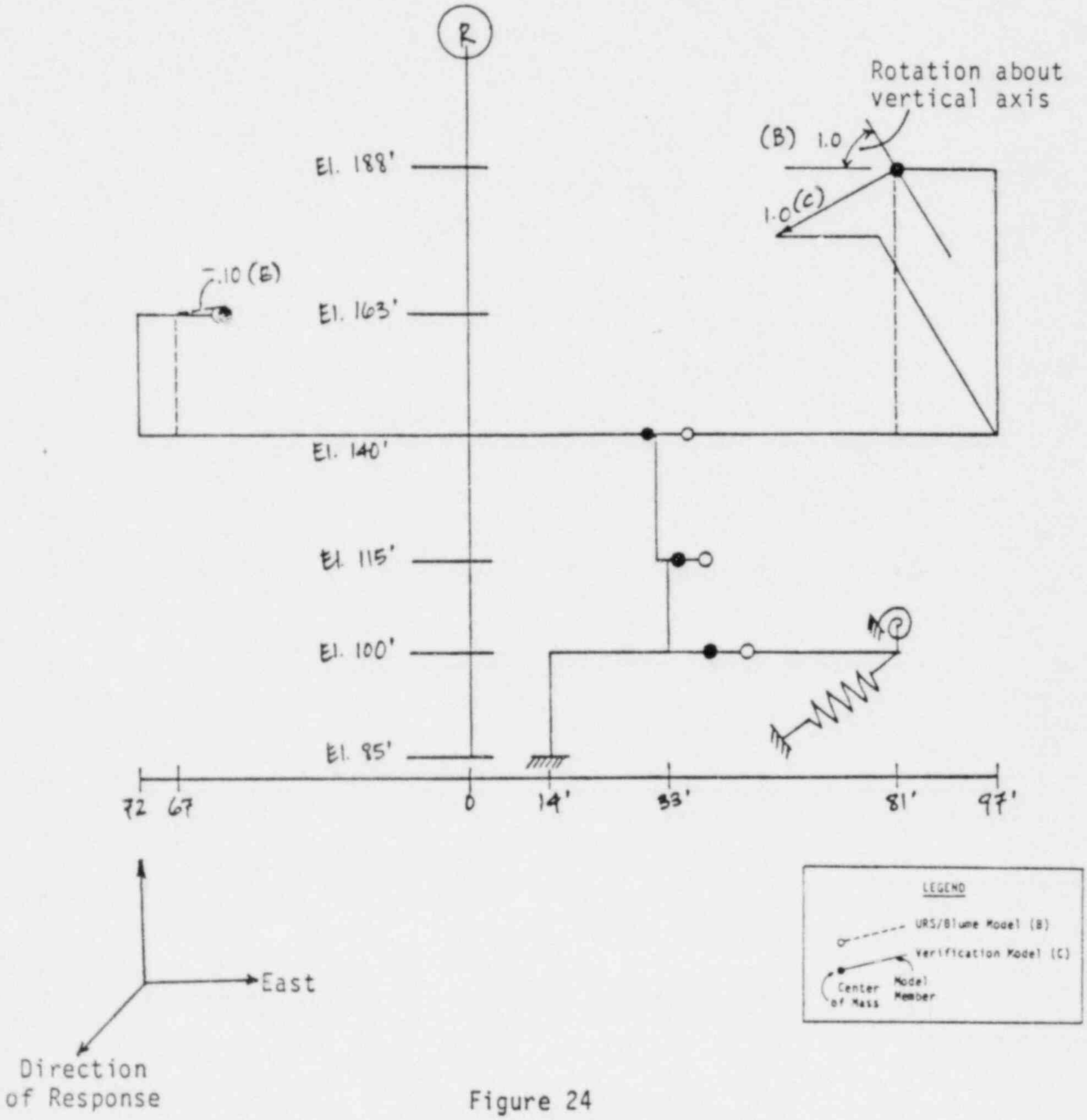


Figure 24

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 25.1% Design = 0%
Mode 2

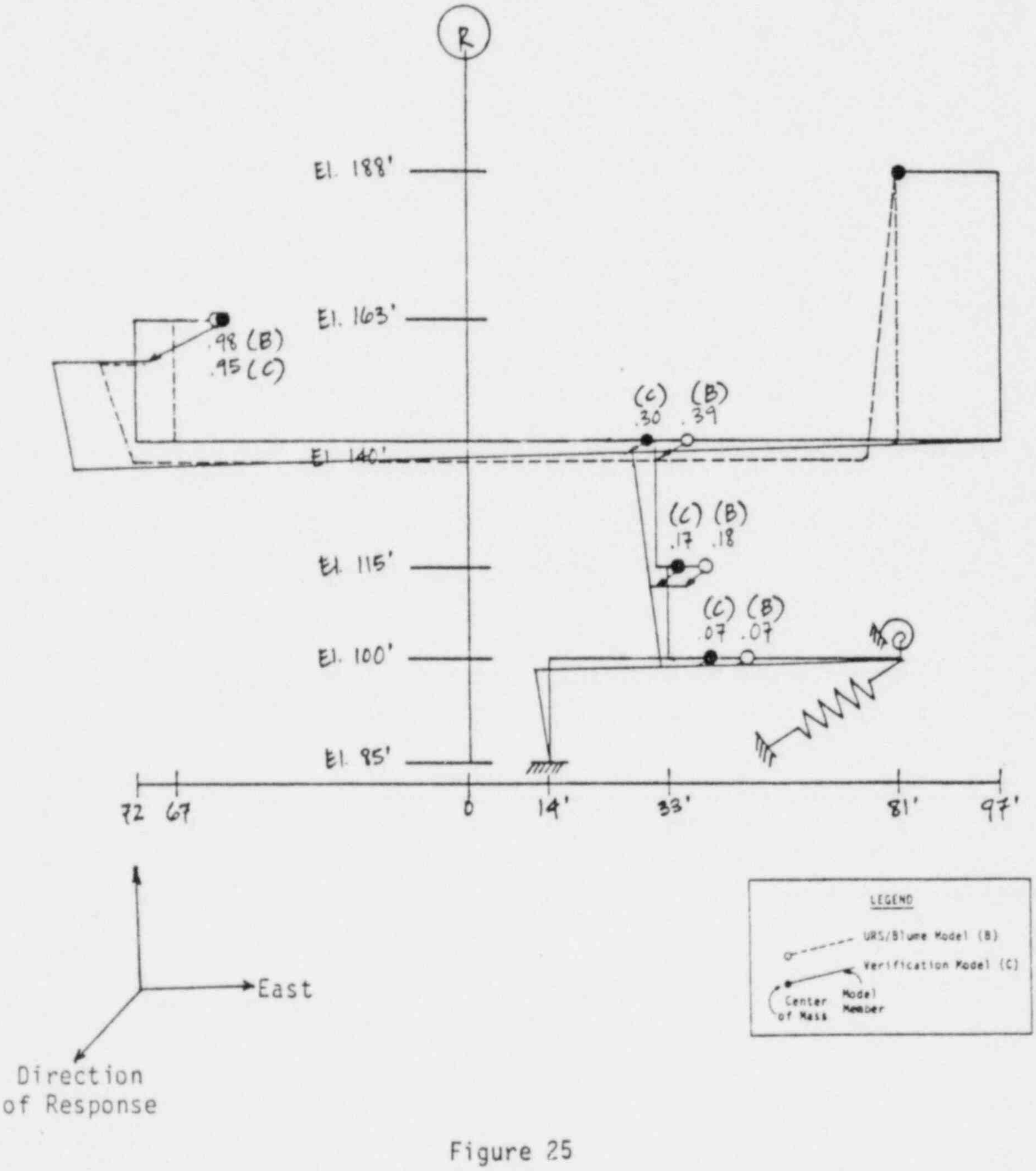


Figure 25

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 16.8%, Design = 30.3%
Mode 3

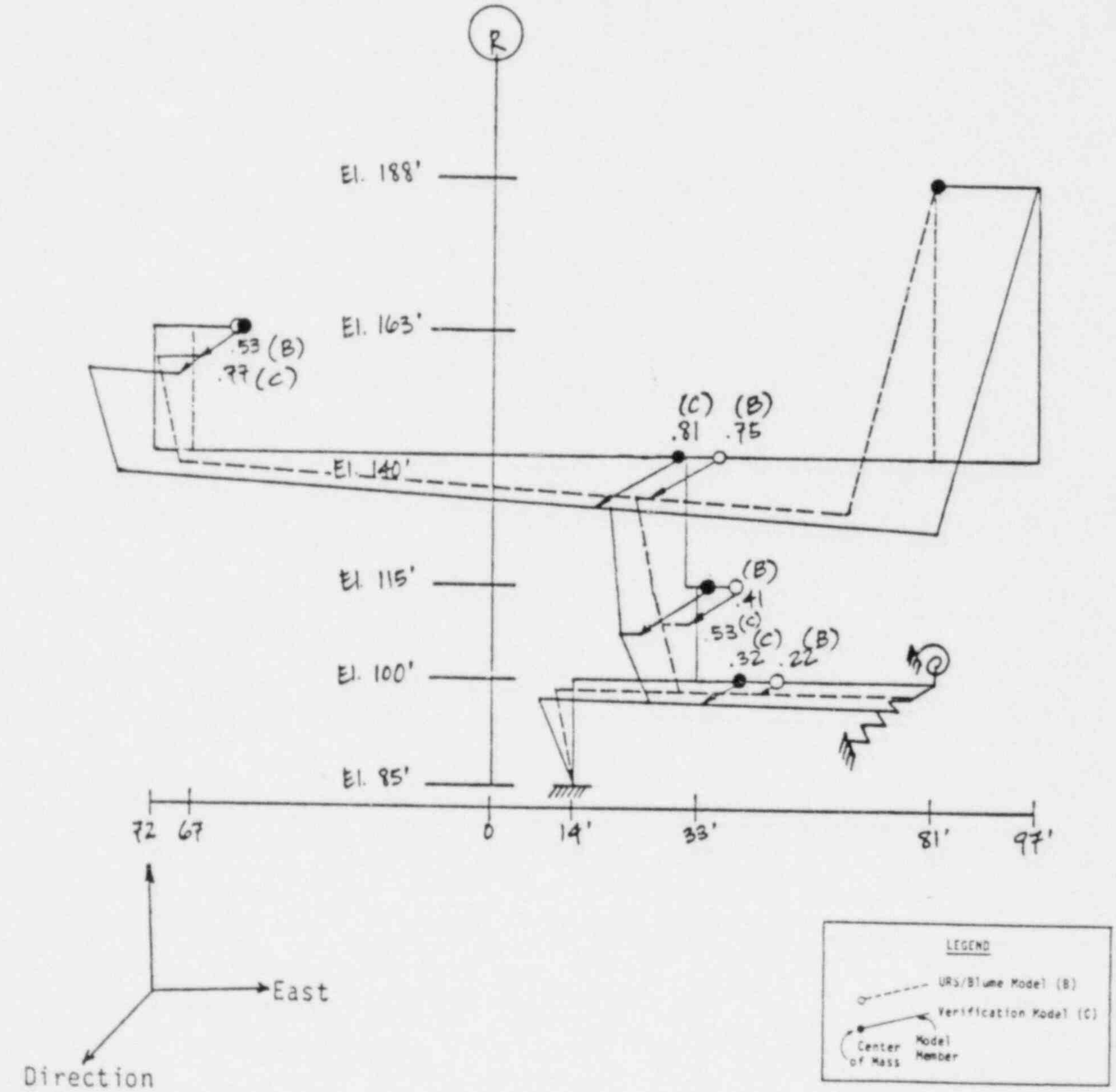


Figure 26

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 29.5%, Design = 31.2%
Mode 4

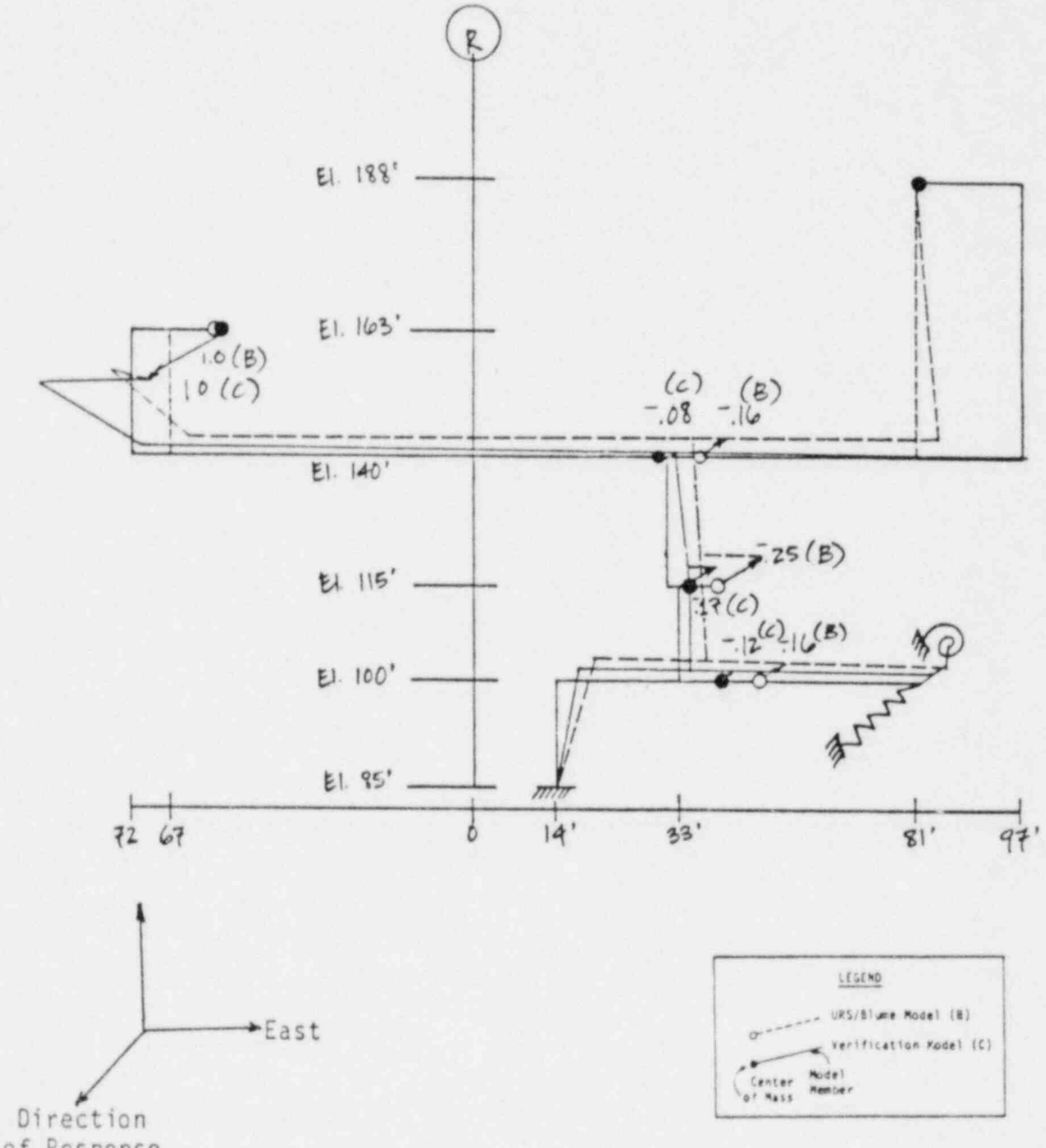


Figure 27

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 19.0%, Design = 17.8%
Mode 5

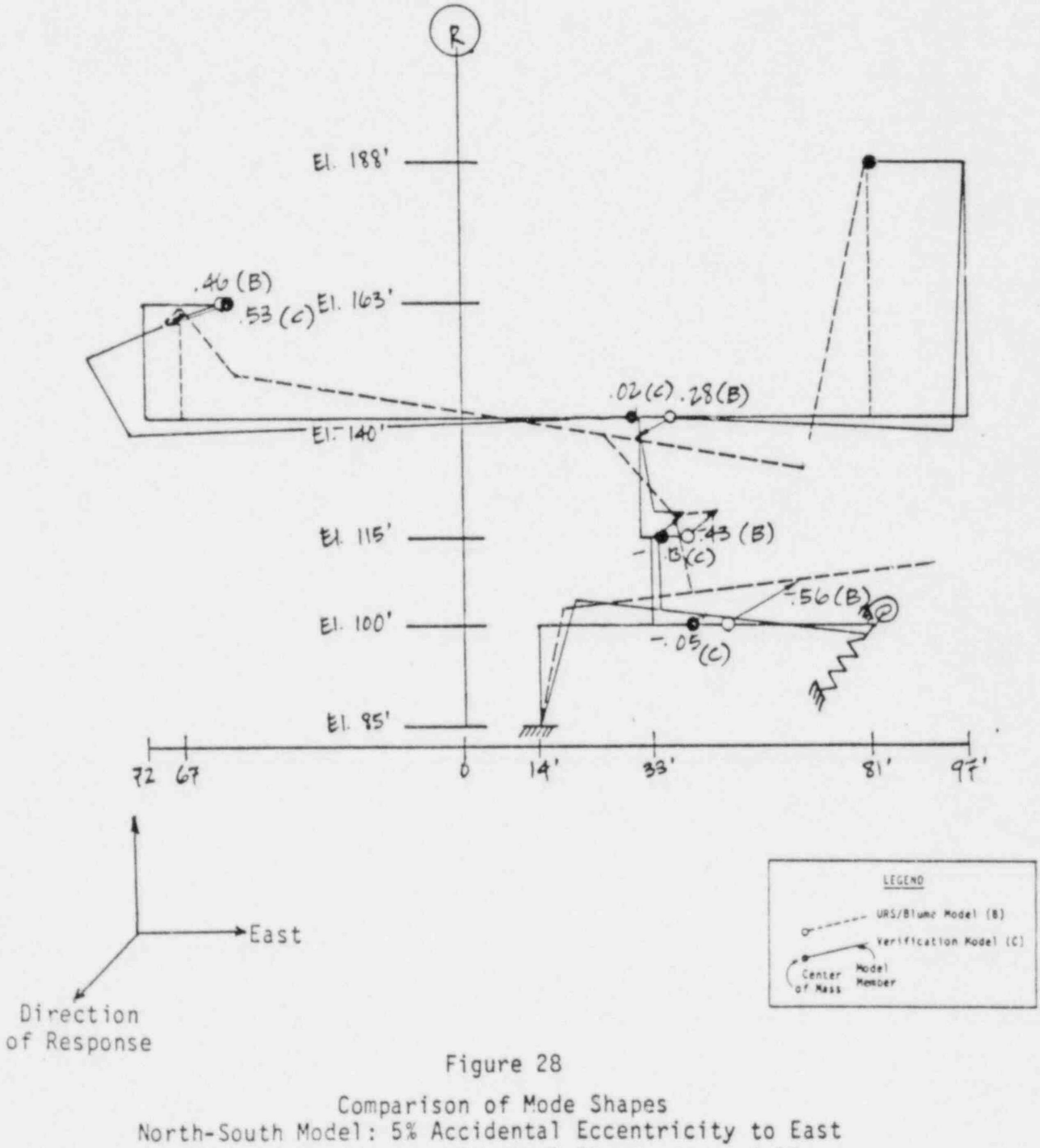


Figure 28

Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Particication Factor: IDVP = 2.2%, Design = 4.7%
Mode 6

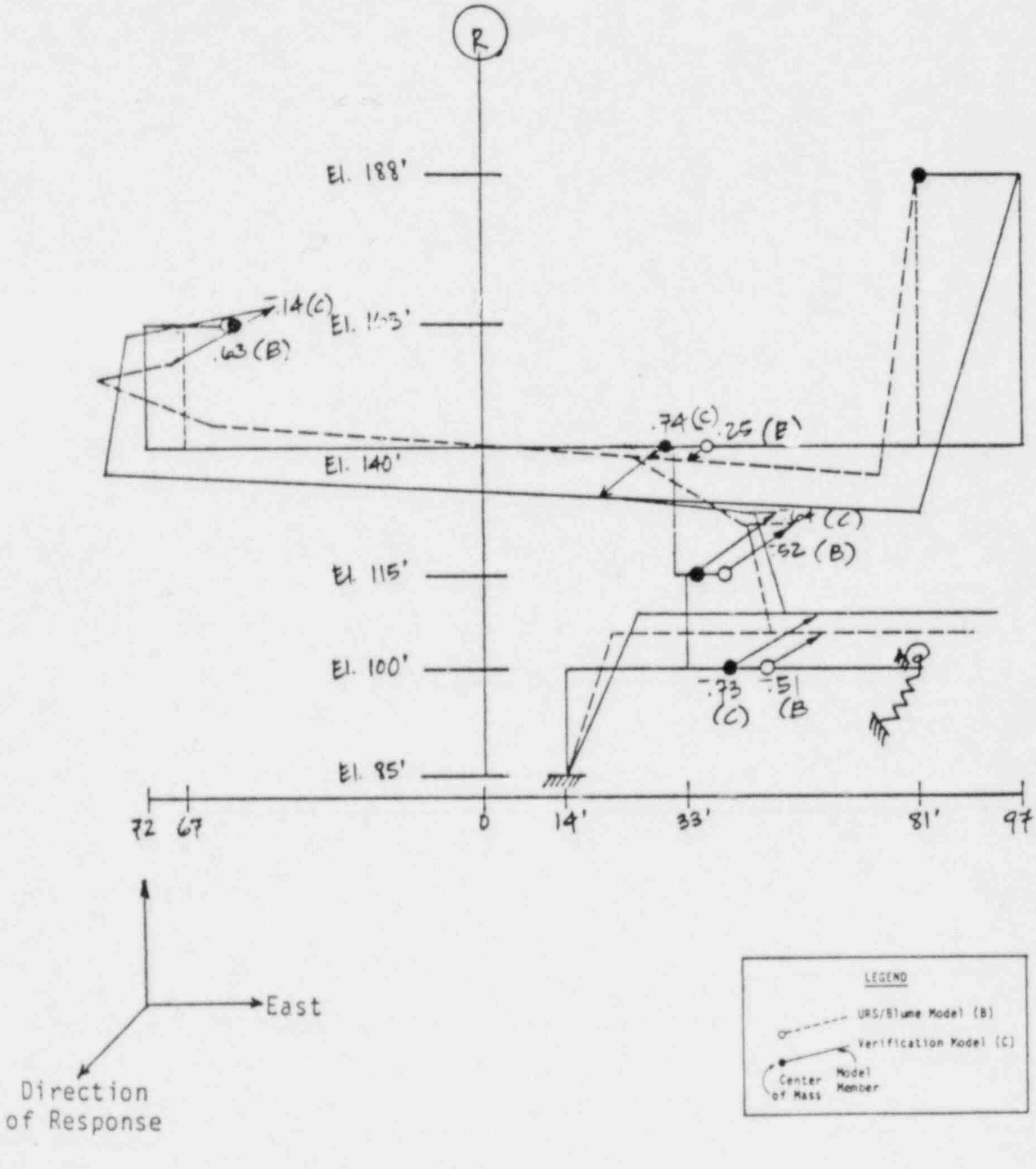


Figure 29
Comparison of Mode Shapes
North-South Model: 5% Accidental Eccentricity to East
Participation Factor: IDVP = 6.2%, Design = 9.7%
Mode 7

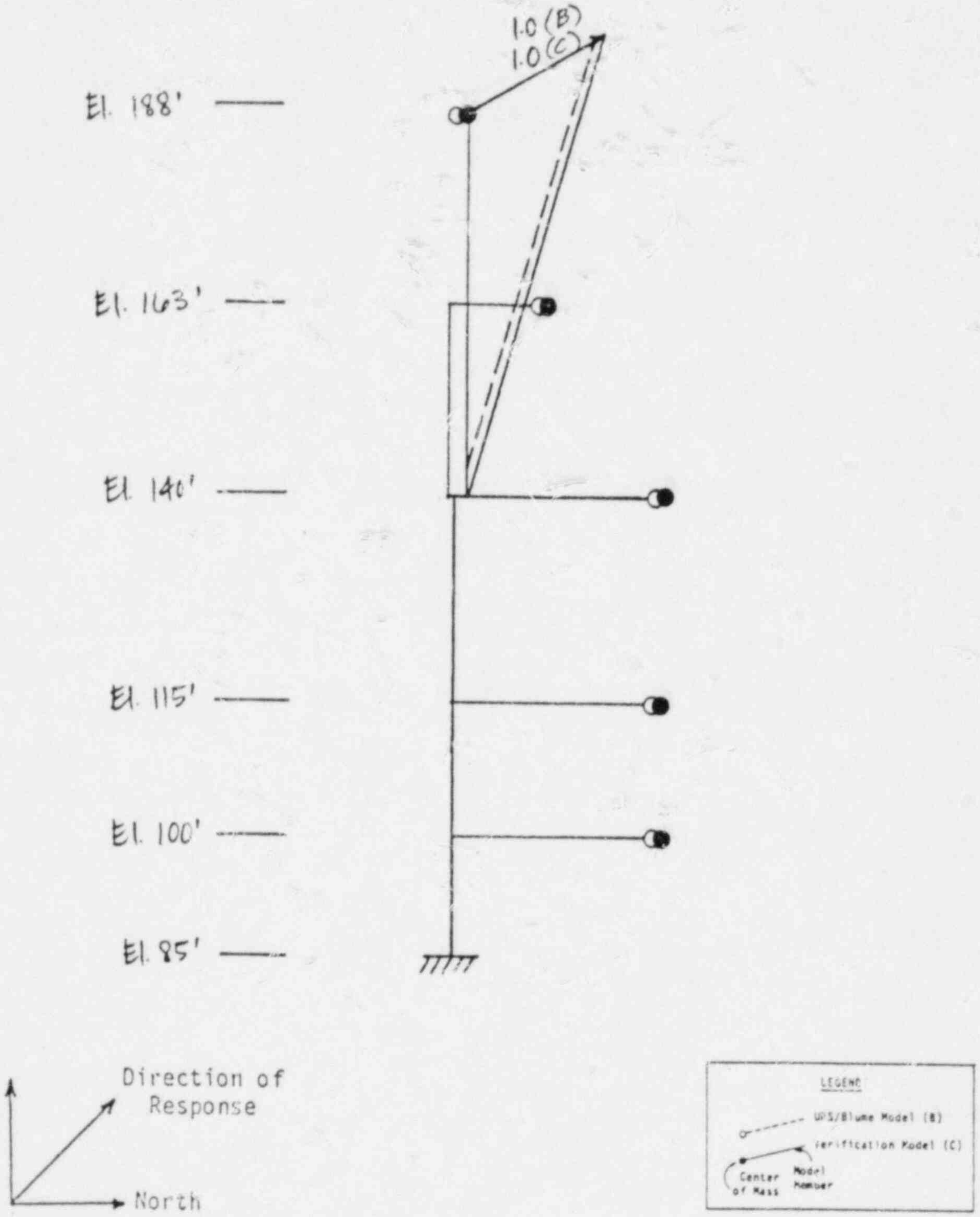


Figure 30

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 25.6%, Design = 6.9%
Mode 1

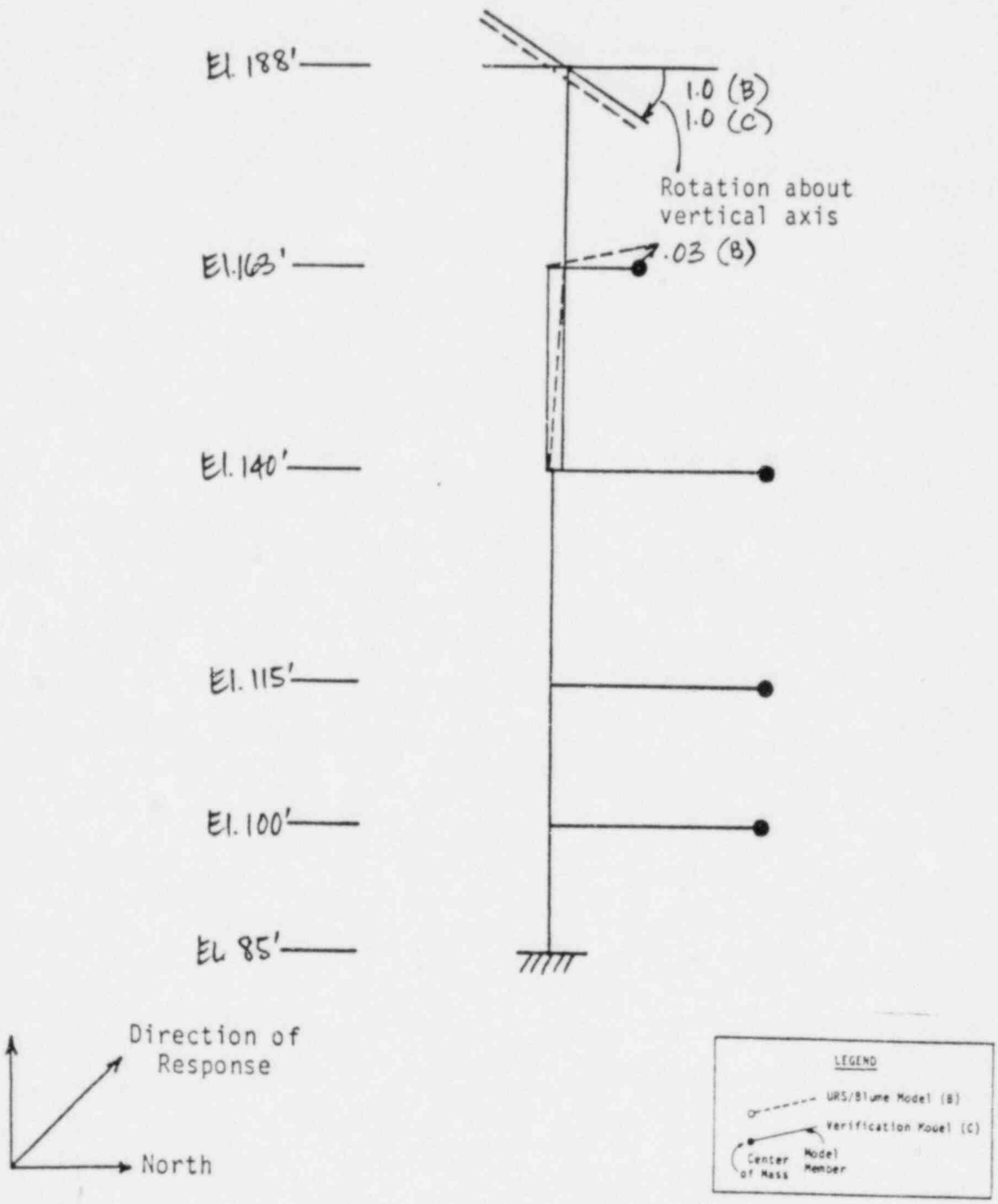


Figure 31

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 0%, Design = 0%
Mode 2

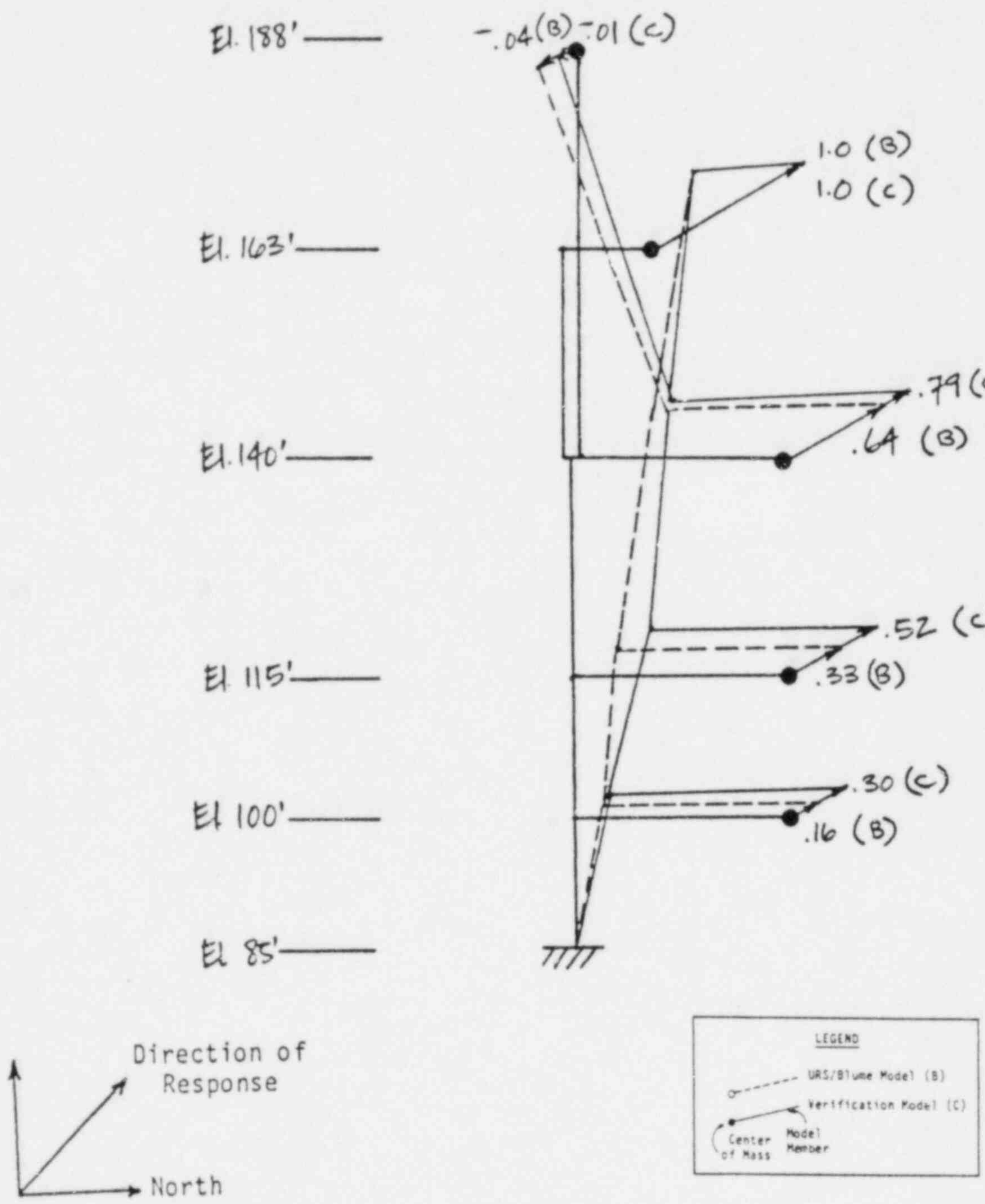


Figure 32

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 29.1%, Design = 44.0%
Mode 3

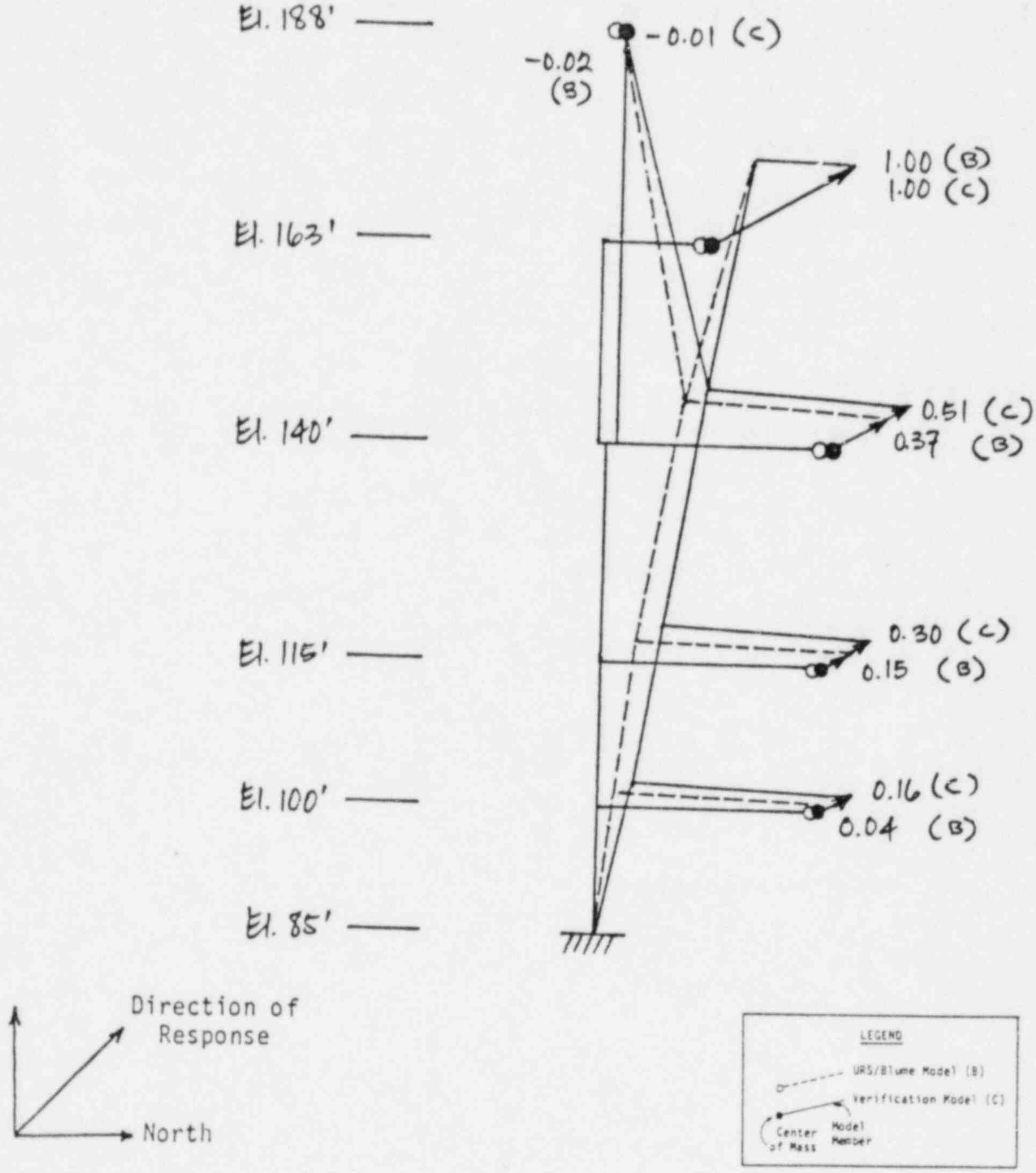


Figure 33

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 13.9%, Design = 16.6%
Mode 4

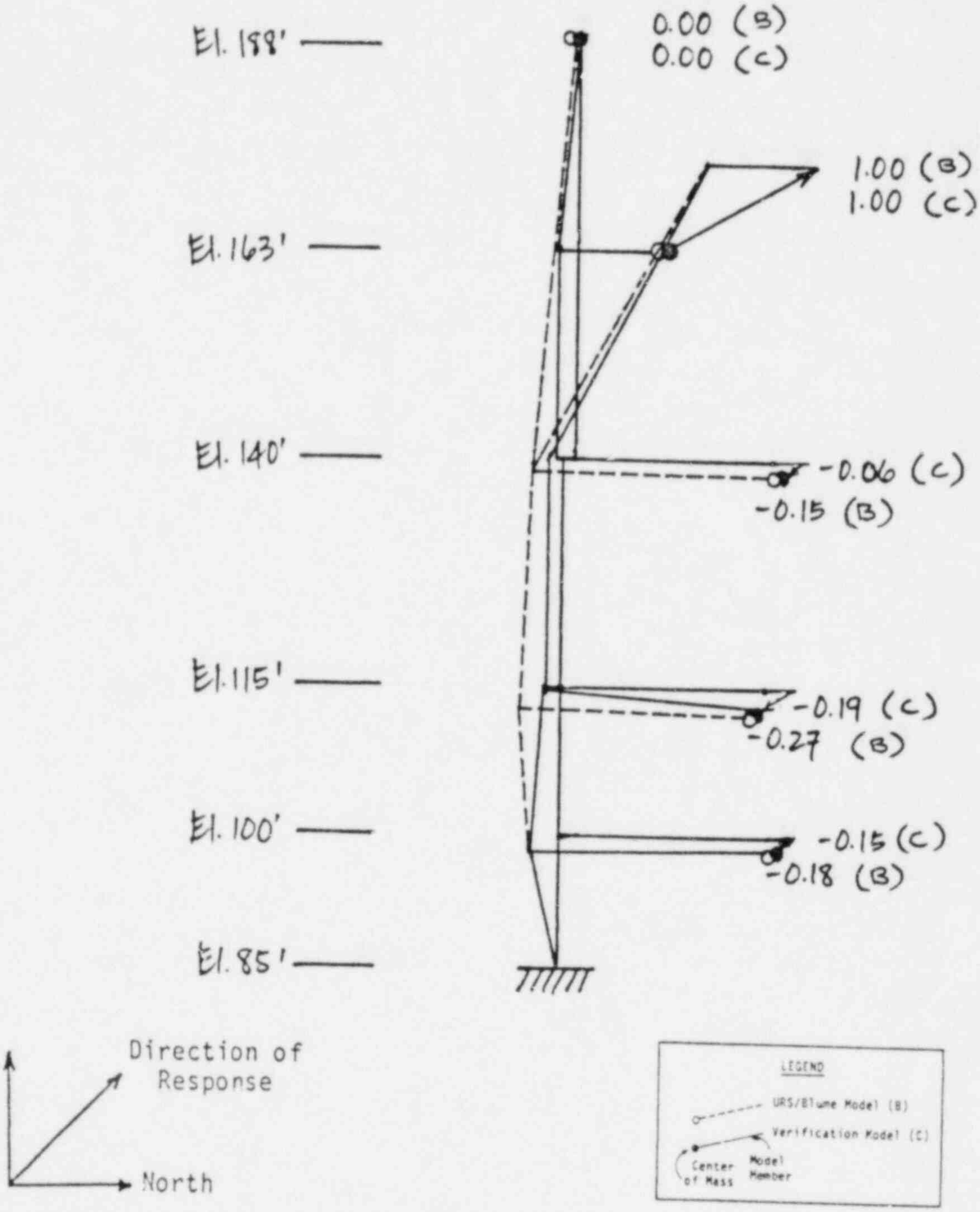


Figure 34

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 24.6%, Design = 22.3%
Mode 5

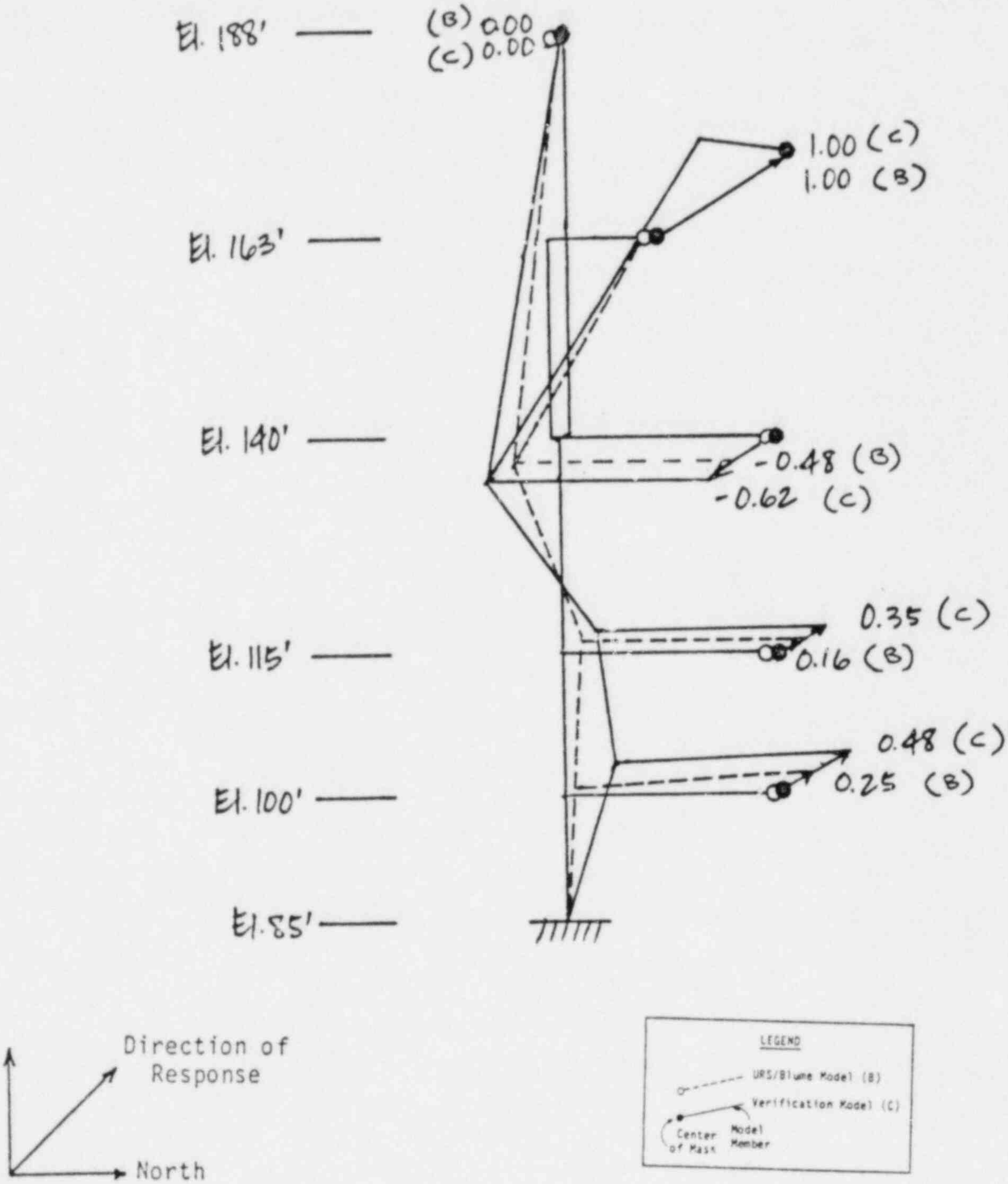


Figure 35

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 5.2%, Design = 2.6%
Mode 6

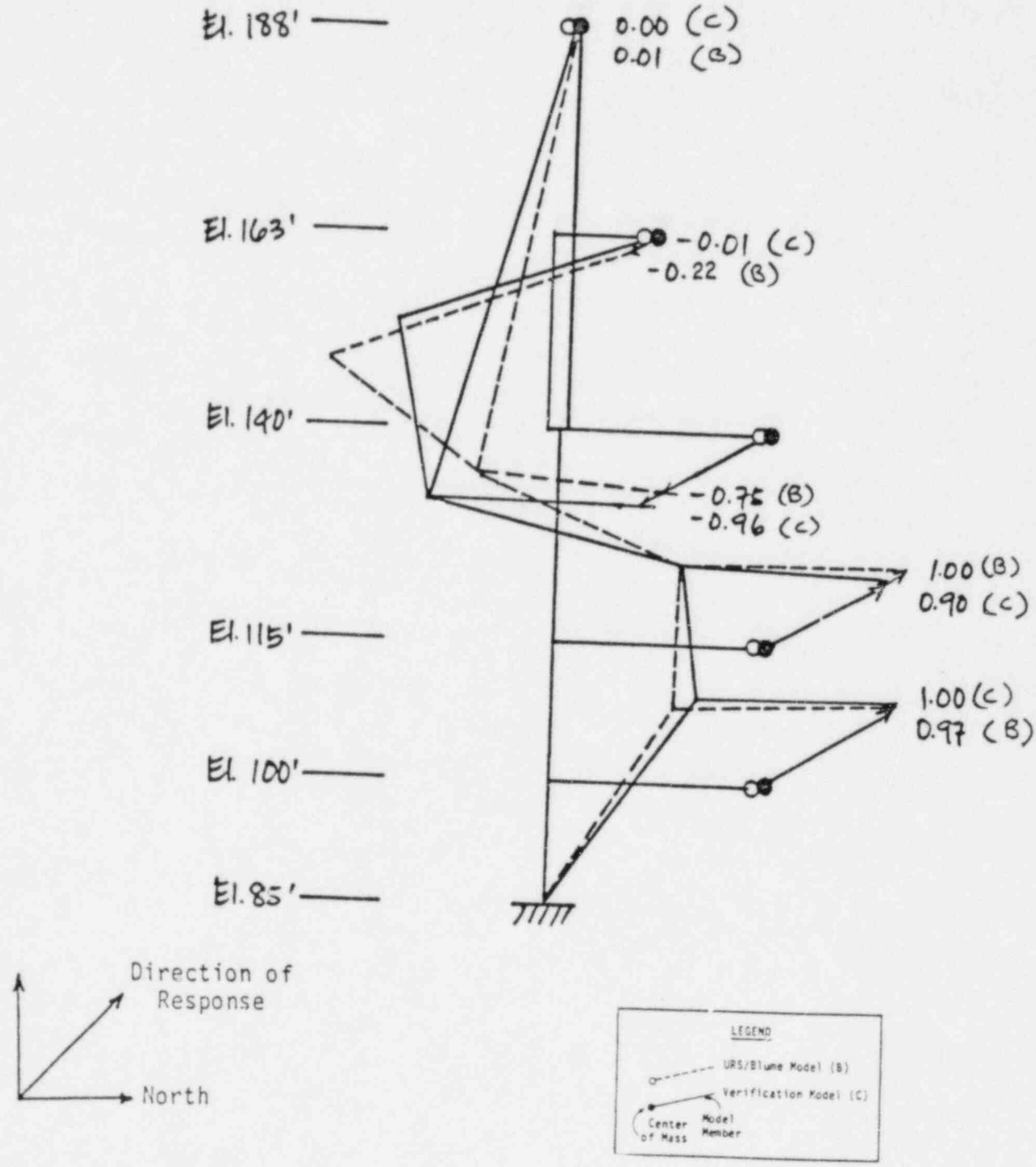


Figure 36

Comparison of Mode Shapes
East-West Model: 5% Accidental Eccentricity
Participation Factor: IDVP = 1.6%, Design = 7.5%
Mode 7

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Section 15.0

References

## 15.0 REFERENCES

### 15.1 IDVP REFERENCES


1. "Independent Design Verification Program, Phase I Program Management Plan, Revision 1", July 6, 1982.

2. "Seismic Reverification Program, Preliminary Report", Report of Work Performed for PGandE by RLCA, November 12, 1982

3. Calculations and Computer Runs

"Property Calculations, Mathematical Models of Auxiliary Building, Diablo Canyon Nuclear Power Plant", P105-4-510-002 Revision 2.

"Soil Spring Calculations" P105-4-521-006, Revision 0".


| Title | Computer Run I.D. | RLCA File No. |
|---|---|---|
| **North-South** | | |
| 0% Eccentricity | STAGG23 | P 105-4-510-013 |
| 5% W Eccentricity | STAGGXG | P 105-4-510-014 |
| 5% E Eccentricity | STAGGN2 | P 105-4-510-015 |
| 7% E Eccentricity | STAGG8D | P 105-4-510-016 |
| 7% W Eccentricity | STAGGAM | P 105-4-510-017 |
| **East-West** | | |
| 0% Eccentricity | STAGGJX | P 105-4-510-018 |
| 5% Eccentricity | STAGG43 | P 105-4-510-019 |
| 7% Eccentricity | STAGG3Z | P 105-4-510-020 |
| **N-S 5% W Eccentricity Parameters Studies** | | |
| RLCA Repro of 5% W Design Model No Soil Spring | VMSGGLG | P 105-4-510-021 |
| RLCA Repro of 5% W Design Model | VMSGG1M | P 105-4-510-022 |
| RLCA Repro of 5% W w/RLCA Moment of Inertia | VMSGG14 | P 105-4-510-023 |
| RLCA Repro of 5% W w/RLCA Moment of Inertia and No Soil Spring | VMSGGNZ | P 105-4-510-024 |

| Title | Computer Run I.D. | RLCA File No. |
|---|---|---|
| Fuel Handling Building | K150PZF | P 105-4-510-008 |
| Fuel Handling Building | K152Q8F | P 105-4-510-009 |
| Fuel Handling Building | K152G4R | P 105-4-510-010 |
| Fuel Handling Building | K152LK7 | P 105-4-510-011 |
| Fuel Handling Building | K15TQ3B | P 105-4-510-012 |

15.2 PGandE REFERENCES


4. "Auxiliary Building Dynamic Seismic Analysis for the 7.5 M Hosgri Earthquake, Diablo Canyon Nuclear Power Plant", URS/Blume, October 1979.

5. "Auxiliary Structure - Revised Dynamic Seismic Analysis, Diablo Canyon Nuclear Power Plant, Unit No. 1", John A. Blume Associates, January 1971.

6. "Seismic Evaluation for Postulated 7.5 M Hosgri Earthquake," USNRC Docket Nos. 50-275 and 50-323.

7. "Diablo Canyon Specification for Seismic Review of Major Structures for 7.5 M Hosgri Earthquake", February 8,1977.

8. PGandE Drawings:


| PGandE Drawing No. & Rev. No. | RLCA File No. |
|---|---|
| 438428-15 | P 105-4-457-003 |
| 438429-20 | P 105-4-457-004 |
| 438449-7 | P 105-4-457-024 |
| 438430-15 | P 105-4-457-005 |
| 438431-12 | P 105-4-457-006 |
| 438432-8 | P 105-4-457-007 |
| 438433-10 | P 105-4-457-008 |
| 438436-6 | P 105-4-457-011 |
| 438435-9 | P 105-4-457-010 |
| 438450-9 | P 105-4-457-025 |
| 443223-6 | P 105-4-457-057 |
| 438440-9 | P 105-4-457-015 |
| 443220-6 | P 105-4-457-056 |
| 443470-2 | P 105-4-457-074 |
| 468128-6 | P 105-4-457-073 |
| 438445-10 | P 105-4-457-020 |
| 438442-12 | P 105-4-457-017 |
| 438443-15 | P 105-4-457-018 |
| 438444-11 | P 105-4-457-019 |
| 438446-16 | P 105-4-457-021 |
| 57724-4 | P 105-4-452-007 |

| PGandE Drawing No. & Rev. No. | RLCA File No. |
|---|---|
| 57725-6 | P 105-4-452-008 |
| 57726-5 | P 105-4-452-009 |
| 55727-4 | P 105-4-452-010 |
| 57735-5 | P 105-4-452-018 |
| 59550-11 | P 105-4-457-102 |
| 59563-12 | P 105-4-457-103 |
| 59567-8 | P 105-4-457-104 |
| 59581-10 | P 105-4-457-106 |
| 443199-7 | P 105-4-457-041 |
| 438465-3 | P 105-4-457-027 |
| 438434-7 | P 105-4-457-009 |
| 439509-1 | P 105-4-457-032 |
| 439507-5 | P 105-4-457-030 |
| 439505-4 | P 105-4-457-028 |
| 439508-5 | P 105-4-457-031 |
| 451597-3 | P 105-4-457-080 |

## 15.3 PUBLIC DOMAIN REFERENCES


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10. "Variability of Dynamic Characteristics of Nuclear Power Plant Structures", Seismic Safety Margins Research Program (SSMRP) Lawrence Livermore Laboratory NUREG/CR-1661.

11. "Structural Analysis and Design of Nuclear Power Facilities", Ed. Committee on Nuclear Structures and Materials of the Structural Division of the ASCE, 1980.

12. "Design of Multi-story Reinforced Concrete Buildings for Earthquake Motions", Blume, J. A., Newmark, N. M., and Cornign, L. H., Portland Cement Association, 1961.

13. "Dynamics of Bases and Foundations", Barkan, D. D., McGraw-Hill, 1962.

14. "Vibrations of Soils and Foundations", Richard, F. E., Hall, J. R. and Woods, R. D., Prentice-Hall, Inc. 1970.

15. "STARDYNE - A Structural Analysis System Computer Code, Level H," January 1982, developed by System Development Corporation.

16. "ANSYS - Swanson Analysis Systems Computer Code, Revision 3." Version 67L, June 1, 1979.

17. Memorandum of February 3, 1982 from K.S. Herring to Frank Miraglia, "Trip-Report Review of URS/Blume Hosgri Analyses of the Diablo Canyon Auxiliary Building, Intake Structure, Containment Polar Crane, and Containment Annulus."

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Appendix A

EOI Reports

(3 Pages)

APPENDIX A
ERROR AND OPEN ITEM REPORTS
- AUXILIARY BUILDING -

| EOI File No. | Subject | Rev. | Date | By | Type | Action Required | Physical Modifications |
|---|---|---|---|---|---|---|---|
| 920 | Auxiliary Building Floor Response Spectra Differences Between the Hosgri and Blume Reports | 0 | 1/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 3/22/82 | RLCA | PPPR/OIP | TES | |
| | | 2 | 4/17/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/19/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/22/82 | TES | PRR/CI | TES | |
| | | 6 | 7/22/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1097 |
| 985 | Auxiliary Building Weight Differences | 0 | 2/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 2/27/82 | RLCA | CI | TES | |
| | | 2 | 4/17/82 | TES | CR | NONE | NO |
| 986 | Control Room Spectra | 0 | 2/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 3/22/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 5/11/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/19/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/22/82 | TES | PRR/CI | TES | |
| | | 6 | 7/22/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1097 |
| 987 | Auxiliary Building Qualification Review | 0 | 2/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 3/09/82 | RLCA | CI | TES | |
| | | 2 | 4/17/82 | TES | CR | NONE | NO |
| 990 | Fuel Handling Building Crane Design Information | 0 | 2/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/18/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 7/01/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/20/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/23/82 | TES | PRR/CI | TES | |
| | | 6 | 7/23/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1092 |
| 991 | Fuel Handling Building Crane Modifications | 0 | 2/06/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/18/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 7/01/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/20/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/23/82 | TES | PRR/CI | TES | |
| | | 6 | 7/23/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1092 |

APPENDIX A
ERROR AND OPEN ITEM REPORTS
- AUXILIARY BUILDING -

| EOI File No. | Subject | Rev. | Date | By | Type | Action Required | Physical Modifications |
|---|---|---|---|---|---|---|---|
| 1027 | Fuel Handling Crane Support Slotted Joints | 0 | 2/23/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/07/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 6/30/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/20/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/23/82 | TES | PRR/CI | TES | |
| | | 6 | 7/23/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1092 |
| 1028 | Auxiliary Building Response Combination Criteria | 0 | 2/23/82 | RLCA | OIR | RLCA | |
| | | 1 | 3/22/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 4/17/82 | TES | PRR/OIP | PGE | |
| | | 3 | 5/24/82 | TES | OIR | RLCA | |
| | | 4 | 7/02/82 | RLCA | PPRR/OIP | TES | |
| | | 5 | 7/13/82 | TES | PRR/OIP | PGE | |
| 1029 | Auxiliary Building Model Property Discrepancies | 0 | 2/25/82 | RLCA | OIR | RLCA | |
| | | 1 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 2 | 7/22/82 | TES | PRR/CI | TES | |
| | | 3 | 7/22/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1097 |
| 1070 | Soil Spring Differences | 0 | 3/15/82 | RLCA | OIR | RLCA | |
| | | 1 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 2 | 7/22/82 | TES | PRR/CI | TES | |
| | | 3 | 7/22/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1097 |
| 1079 | Fuel Handling Building Cross Bracing | 0 | 4/19/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/11/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 6/19/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/21/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/23/82 | TES | PRR/CI | TES | |
| | | 6 | 7/23/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1092 |
| 1091 | Fuel Handling Building Cross Bracing | 0 | 5/21/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/11/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 6/19/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/09/82 | TES | OIR | RLCA | |
| | | 4 | 7/29/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 8/10/82 | TES | PRR/CI | TES | |
| | | 6 | 8/10/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1092 |

APPENDIX A

ERROR AND OPEN ITEM REPORTS
- AUXILIARY BUILDING -

| EOI File No. | Subject | Rev. | Date | By | Type | Action Required | Physical Modifications |
|---|---|---|---|---|---|---|---|
| 1092 | Fuel Handling Building - Hosgri Diagrams | 0 | 6/11/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/11/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 6/21/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/20/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PER/A | TES | |
| | | 5 | 7/23/82 | TES | ER/A | PGE | |
| | | 6 | 8/10/82 | TES | ER/A | PGE | |
| 1093 | Auxiliary Building - Spectra not Available for Two Areas | 0 | 6/18/82 | RLCA | OIR | RLCA | |
| | | 1 | 6/18/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 6/29/82 | TES | PRR/OIP | PGE | |
| | | 3 | 7/20/82 | TES | OIR | RLCA | |
| | | 4 | 7/21/82 | RLCA | PPRR/CI | TES | |
| | | 5 | 7/22/82 | TES | PRR/CI | TES | |
| | | 6 | 7/22/82 | TES | CR | NONE | PGE RE-ANALYSIS REF. 1097 |
| 1095 | Input Time History | 0 | 7/09/82 | RLCA | OIR | RLCA | |
| 1097 | Auxiliary Building - Spectra not Available for One Area | 0 | 7/13/82 | RLCA | OIR | RLCA | |
| | | 1 | 7/14/82 | RLCA | PPRR/OIP | TES | |
| | | 2 | 7/20/82 | TES | OIR | RLCA | |
| | | 3 | 7/21/82 | RLCA | PER/ A or B | TES | |
| | | 4 | 7/22/82 | TES | ER/A or B | PGE | |

STATUS: Status is indicated by the type of classification of latest report received by PGandE:

| | | | | |
|---|---|---|---|---|
| OIR | - Open Item Report | ER | - Error Report | A - Class A Error |
| PPRR | - Potential Program Resolution Report | CR | - Completion Report | B - Class B Error |
| PRR | - Program Resolution Report | CI | - Closed Item | C - Class C Error |
| PER | - Potential Error Report | DEV | - Deviation | D - Class D Error |
| OIP | - Open Item with future action by PGandE | | | |

PHY MODS: Physical modifications required to resolve the issue. Blank entry indicates that modification has not been determined

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Appendix B

Key Term Definitions

(5 Pages)

## KEY TERMS AND DEFINITIONS USED IN THE AUXILIARY/FUEL HANDLING BUILDING REPORT

The definitions in this glossary establish the meanings of words in the context of their use in this document. These meanings in no way replace the specific legal and licensing definitions.

Closed Item

- A form of program resolution of an Open Item which indicates that the reported aspect is neither an Error nor a Deviation. No further IDVP action is required.

Completion Report

- Used to indicate that the IDVP effort related to the Open Item identified by the File Number is complete. It references either a Program Resolution Report which has recategorized the item as a Closed Item or a PGandE document which states that no physical modification is to be applied in the case of a Deviation or a Class C or Class D Error.

DCNPP

- Diablo Canyon Nuclear Power Plant

Damping

- The measure of energy dissipation in a system.

Design Analysis

- Work performed by or for PGandE.

EOI

- Error and Open Item Report

Error Report

- An Error is a form of program resolution of an Open Item indicating an incorrect result that has been verified as such. It may be due to mathematical mistake, use of wrong analytical method, omission of data, or use of inapplicable data.

- Each Error shall be classified as one of the following:

  - o Class A: An Error is considered Class A if design criteria or operating limits of safety related equipment are exceeded as a result, and physical modifications or changes in operating procedures are required. Any PGandE corrective action is subject to verification by the IDVP.

  - o Class B: An Error is considered Class B if design criteria or operating limits of safety related equipment are exceeded, but are resolvable by means or more realistic calculations or retesting. Any PGandE corrective action is subject to verification by the IDVP.

  - o Class C: An Error is considered Class C if incorrect engineering or installation of safety related equipment is found, but no design criteria or operating limits are exceeded. No physical modifications are required, but if any are applied they are subject to verification by the IDVP.

  - o Class D: An Error is considered Class D if safety related equipment is not affected. No physical modifications are required, but if any are applied, they are subject to verification by the IDVP.

FSAR

- PGandE's Final Safety Analysis Report

Finite Element Model

- A computer method used to construct a mathematical representation of a loading on a structure.

Hosgri 7.5M Earthquake

- Maximum intensity earthquake for which the plant is designed to remain functional. Same as Safe Shutdown Earthquake (SSE).

Hosgri Report

- An amendment to the Diablo Canyon licensing application that summarizes the evaluation of the plant for the postulated Hosgri event. This amendment is a seismic report developed by PGandE that gives allowable criteria (licensing criteria) and cites plant qualifications.

Hosgri Fault

- Geological fault off the coast of California.

Hosgri Event

- Postulated earthquake along the Hosgri Fault.

IDVP

- Independent Design Verification Program undertaken by R. L. Cloud Associates, R. F. Reedy, Teledyne Engineering and Stone & Webster Engineering to evaluate Diablo Canyon Nuclear Power Plant for compliance with the licensing criteria.

Interim Technical Report

- Interim Technical Reports are prepared when a program participant has completed an aspect of their assigned effort in order to provide the completed analysis and conclusions. These may be in support of an Error, Open Item or Program Resolution Report or in support of a portion of the work which verifies acceptability. Since such a report is a conclusion of the program, it is subject to the review and approval of the Program Manager. The report will be transmitted simultaneously to PGandE and to NRC.

Licensing Criteria

- Contained in PGandE Licensing Documents, includes allowable criteria. (See Hosgri Report definition.)

NRC

- Nuclear Regulatory Commission

NRC Order Suspending License CLI-81-30

- The order dated November 19, 1981 that suspended the license to load fuel and operate DCNPP-1 at power levels up to 5% of full power and specified the programs that must be completed prior to lifting of the suspension.

Open Item

- A concern that has not been verified, fully understood nor its significance assessed. The forms of program resolution of an Open Item are recategorization as an Error, as a Deviation, or as a Closed Item.

PGandE

- Pacific Gas and Electric

PGandE Technical Program

- Verification program undertaken by PGandE to evaluate DCNPP for compliance with licensing criteria.

Phase I Program

- Work performed by RLCA, RFR, TES restricted to Hosgri-related efforts of PGandE and their service contractors prior to June 1978.

Potential Program Resolution
Report and Potential Error Report

- Forms used only for communication within the IDVP.

Response Spectra

- A graph of the maximum response, as a function of a frequency that represents the response of a single degree of freedom system.

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Sample

- Initial sample as stipulated in Phase I Program, of buildings, equipment, and components to be design-verified by independent calculations.

Seismic

- Refers to earthquake data.

Smoothing

- The process of representing a graph with a jagged line as a smooth line.

TES

- Teledyne Engineering Services

Torsion Effects

- Torsion effects are added to the translational response to account for the twisting of the building about the vertical axis.

Verification Analysis

- Work performed by RLCA as part of the IDVP.

Verification Program

- Undertaken by the IDVP to evaluate Diablo Canyon Nuclear Power Plant for compliance with the licensing criteria.

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Appendix C

Program Manager's Assessment

# APPENDIX C

## PROGRAM MANAGER'S ASSESMENT

As program manager of the Independent Design Verification Program, TES has supervised RLCA personnel involved in writing this sixth Interim Technical Report. The Phase I Program Management Plan, Section 9.2 gives the requirements for ITR's; review by TES show that these requirements are met.

The program management function was performed by TES in accordance with the Plan and TES Engineering Procedure EP-1-014. The task of verification of the auxiliary building analysis which is part of the management function was carried out through several steps.

1. Meetings were held at TES and RLCA offices to review technical content and editorial comments.

2. The calculation package (Reference 2) was reviewed for overall methodology.

3. Spot checks of detailed calculations were made to ensure completeness and accuracy of mass and member property calculations.

4. TES personnel visited the auxiliary building at the Diablo Canyon Nuclear Power Plant to determine if RLCA had modeled properly certain portions of the building, including the Fuel Handling Building. Spot checks of shear walls and control room structural elements were made.

5. Professors M. J. Holley and J. M. Biggs were retained to review the dynamic analysis and the reported comparison of RLCA and the 1979 URS/Blume results.

From the above, it was determined that the RLCA calculations are accurate and this sixth ITR is therefore approved.

As a result of the PG and E internal program, TES and RLCA will review further reanalysis of the auxiliary building as well as reanalysis of other safety related structures in the near future. Error and Open Item concerns listed herein will be reconsidered in the review of that work. The structural integrity of the buildings will be reviewed in addition to analysis results leading to pipe and equipment support seismic response spectra.

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