UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In The Matter of CONSUMERS POWER COMPANY (Midland Plant, Units 1 and 2)	Docket Nos. 50-329 OM 50-330 OM Docket Nos. 50-329 OL 50-330 OL

TESTIMONY OF ROBERT P. KENNEDY

My name is Robert P. Kennedy. I am President of Structural Mechanics Associates, Inc. (SMA), located at 5160 Birch Street, Newport Beach, California. Under my direct supervision, SMA has been retained by Consumers Power Company to act as general consultants to assist in reviewing specialized aspects of the seismic design of the Midland Plant since September, 1980. I have over 19 years of consulting experience in the design and analysis of structures and components subjected to dynamic loadings (seismic, wind, impact, and blast) and the officers and senior employees of SMA have over 150 years of combined experience in this field. A statement of my qualifications is attached to my testimony as Attachment A.

My testimony concerns the dynamic mathematical models being used by Bechtel to perform the seismic evaluation of structures in conjunction with foundation remedial work. My testimony summarizes the dynamic mathematical models developed for 1) the Auxiliary Building - Control Tower - Electrical Penetration Area (Aux. Building) which are all supported on an interconnected foundation system; 2) the Service Water Pump Structure (SWPS) and 3) the Borated Water Storage Tank (BWST). The Aux. Building and SWPS models have been developed by staff members of the Bechtel Corporation, Ann Arbor, Michigan, and important features of the models have been reviewed by myself and SMA staff members under my direct supervision. The BWST model was developed by myself and SMA staff members. I consider these models to be adequate for the purpose of

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1) defining seismic design forces to be used in the design of foundation remedial work on these structures, 2) conservatively estimating the seismic-induced forces in these structures, and 3) defining the seismic input to equipment, systems, and components mounted on these structures.

It is intended that the models used in this testimony will also be used by SMA to perform a seismic margin review of the Midland Plant for earthquake ground motion defined by the Site Specific Response Spectra (SSRS) presented in the testimony of Richard Holt of Weston Geophysical Corporation and Jeffrey Kimball of the NRC staff. The criteria to be used by SMA with these models in conducting the seismic margin review will be different from the criteria described in this testimony and used by Bechtel Corporation in the design of the founation remedial work and in their subsequent analyses of the structures and equipment affected by the foundation remedial work.

The dynamic models discussed in the testimony are those submitted in a letter dated September 30, 1981, from D. W. Cook to H. R. Denton. It is not unusual for models to change somewhat during the course of finalizing the analyses. Any significant changes would, of course, be reported to the NRC.

1. Introduction to Dynamic Mathematical Models

Dynamic mathematical models are used by a dynamic analyst to define the dynamic response characteristics of a structure when subjected to a dynamic forcing function. The level of complexity of these models depends upon the purpose for which they are being used and the characteristics of the dynamic forcing function being applied.

For the seismic evaluation of complex buildings such as the Aux. Building or the SWPS, the most common procedure (and the procedure which I recommend) is called a two-step procedure. In the first step, an overall dynamic response model of the complete structure is developed.

This model must be adequate to determine the seismic-induced forces, shears, moments, displacements and accelerations at all important locations throughout the structure. This model is also used to determine the seismic input to equipment mounted on the structure. In the second step, detailed static models are developed for local regions of the complex structure to convert the maximum overall seismic-induced dynamic responses from step one to local forces and stresses for use in the seismic evaluation or design of individual structural elements. This two-step procedure avoids having to place excessive and extraneous details into the overall dynamic model. The dynamic analyst can concentrate on those factors which influence overall structural response rather than on those features which influence local stresses. In my judgment, the two-step procedure leads to more reliable estimates of the overall response of complex structures to seismic input than can be obtained from more complex one-step models which must also be capable of computing local stresses and local responses in the same analysis. The dynamic mathematical models presented in my testimony are good overall dynamic models for use in a two-step procedure. They can be used to adequately and conservatively determine the overall seismic-induced forces, shears, moments, displacements, and accelerations throughout the Aux. Building, SWPS, and BWST structures including their foundations, and to determine the seismic input to equipment mounted on these structures. However, these models are not adequate for determining local stresses and local forces; i.e., the second-step of the two-step procedure. My testimony is limited to the overall dynamic mathematical models.

The overall dynamic response of a complex structural system to seismic input is heavily influenced by:

a. The distribution of mass (weight divided by gravity) throughout the structural system.

b. The distribution of stiffness (the forces required to produce a unit deformation of the structural system).

c. How the structure is founded on the supporting soil (soil-structure interaction).

d. How major separate structural systems are intertied together (for instance, how is the main auxiliary building interconnected to the control tower and the electrical penetration wings in the case of the Aux. Building).

and

e. The amount of energy dissipation capability (damping within the structural system and the radiation of energy away from the structure through the supporting soil).

A proper dynamic mathematical model must either reasonably accurately or conservatively incorporate the important aspects of each of the above listed factors. For the Aux. Building and SWPS dynamic models, the actual building and major equipment masses can be concentrated (lumped) at the individual floor levels within these buildings. However, it is necessary that these lumped masses be located at their proper elevations. In a plan view, they must be placed with reasonable accuracy at the center of gravity of the actual distributed masses. The building stiffnesses can be modeled as concentrated stiffness elements which account for the actual distributed shear, flexural, and axial stiffnesses of the seismic-resistant structural systems. These stiffness elements link together vertically and horizontally each of the concentrated mass points in the dynamic model. These stiffness values are determined by more detailed local static analyses using local structural models. It is important that these concentrated stiffness elements be located in a plan view at the approximate center of rigidity of the actual distributed seismic-resistant structural system. Individual vertical and horizontal mass-stiffness models are developed for each major separate structural system (for instance, the main auxiliary building, control tower, and electrical penetration wings of the Aux. Building). These individual models are linked together horizontally at each mass point by horizontal concentrated stiffness elements which define the stiffnesses (inverse of flexibilities) associated with the interties between the individual structural systems.

The seismic input is fed into the overall building dynamic model through a soil-structure interaction model which connects the building foundation elements to the supporting soil. These soil-structure interaction models must:

a. feed the seismic input into the building models at the appropriate elevations and plan view locations (center of rigidity of the supporting soil),

b. account for the reduced stiffness of the overall building system due to the flexibility (inverse of stiffness) of the supporting soil,

and

c. conservatively account for the radiation of energy (associated with building response relative to the soil) from the building model into the surrounding soil.

Important aspects of how these soil-structure interaction factors are incorporated into the Aux. Building, SWPS, and BWST dynamic models are described in the next section.

Energy dissipation within the structural system is approximated in the dynamic models as viscous (velocity proportional) damping. Viscous dashpots are also used to model the radiation of energy from the structure back into the supporting soil. Damping aspects of the models will be discussed in a subsequent section.

The resultant mathematical models for overall dynamic response of the actual structural systems have the appearance of a series of interconnected "lollypops" with the "ball" of each "lollypop" representing a concentrated mass point and the "stick" of each "lollypop" representing a concentrated stiffness element. The responses (accelerations, and displacements) of each "ball", which are obtained from a dynamic analysis using this model, define the responses of specific locations within the actual structure. Similarly, the responses (forces, shears, and moments) computed within each "stick" of the mathematical model define these same responses within the seismic-resistant structural system represented by

the "stick" in the actual structural system. It is totally irrelevant that the mathematical model does not look like the actual structure. It is only relevant that the mathematical model reproduces the important response parameters of the actual structure.

Mathematical dynamic models of the type described are not always necessary. For a very simple building, such as a one-story structure, an analyst can determine the natural frequency of vibration and thus the structural responses (forces, moments, shears, accelerations, and displacements) without constructing such a model. Similarly, for simple belowground structures such as valve pits and retaining walls, the seismic-induced soil forces on the walls can be determined without constructing such mathematical models. Mathematical models are an aid to the dynamic analyst in predicting structural response and have no inherent value unto themselves except as an aid to the analyst when needed.

Secondly, the mathematical representation of complex structures is a combination of science and art. Given the same set of drawings or the same actual structure, a group of qualified engineers would describe the dynamic characteristics of that structure by a wide variety of differing dynamic models simply because of the degree of engineering judgment that goes into the modeling process. So long as each model describes the important structural characteristics which I listed earlier as heavily influencing dynamic response, no one can say that one model is necessarily better than another. In my experience, competent structural analysts can obtain reasonably similar results even though each may use totally different models which have no resemblance to each other so long as each model incorporates the important structural response characteristics. Also, within my experience of actual structures subjected to actual earthquake and nuclear weapon test generated ground motion, competent structural analysts will nearly always overpredict the actual measured structural responses because of conservatism which they embed into their modeling practice to cover uncertainty.

Based upon my review, it is my opinion that the models described in my testimony incorporate all of the important structural response parameters and are adequate for conservatively predicting the overall seismic response of these structures to a defined ground motion.

2. Soil-Structure Interaction Modeling

The soil-structure interaction effect on complex buildings such as the Aux. Building is a complex and controversial subject in which there is a lack of unanimity among experts. Experts agree that a complete interaction analysis would have to 1) account for the variation of soil properties with depth, 2) give appropriate consideration to the material nonlinear behavior of soil, 3) consider the three-dimensional nature of the problem, 4) consider the complex nature of wave propagation which produced the ground motions, 5) consider possible interactions with neighboring structures, and 6) consider the overall three-dimensional response characteristics of the structure. Unfortunately, such an analysis is beyond the current state-of-the-art and cannot be performed for complex buildings. Therefore, all soil-structure interaction modeling involves approximations and assumptions. Despite this situation, conservative seismic evaluations can be performed and safe structures can be designed by: a) conservatively approximating soil-structure interaction effects, and b) parameter variation.

The soil-structure interaction models incorporated into the Aux. Building, SWPS, and BWST models for the foundation remedial design work are very simple models. They do not represent the most advanced state-of-art models. If I were engaged in a project to provide the "best possible" prediction of soil-structure interaction effects, I would use more sophisticated models. The models described in this testimony either do not incorporate or very greatly simplify some of the complete interaction effects described above. However, considerable judgment and expertise was exercised in developing these models so as to provide high confidence that these simple models will conservatively overpredict the

seismic response of the structures because of the factors that the models do not explicitly consider. These models are adequate for the purposes of conservatively designing the structures (including foundation remedial work) and evaluating the seismic safety of equipment. More sophisticated models could be used to eliminate some of the conservatism. However, I would judge that the added effort involved is not likely to be warranted.

For the foundation remedial work on the Aux. Building, SWPS, BWST, soil-structure interaction effects are modeled using frequency independent impedance functions based upon the soil beneath the foundations being treated as an elastic half-space. These impedance functions consist of real terms which can be modeled as stiffness linkages ("sticks") between the structure foundation and soil, and imaginary terms which can be modeled as dashpots (viscous dampers) which radiate energy out from the structure to the surrounding soil. All six degrees of freedom of response (two horizontal translations, one vertical translation, two rocking rotations, and one torsional rotation) are included.

Best estimate soil properties have been used to establish the impedance function "stiffness" and "dashpot" values. Strain degradation of the soil stiffness properties (approximate nonlinear behavior of the soil) was included in establishing these properties. For instance, the majority of the Aux. Building after the foundation remedial work is completed will be founded at elevations between 562 feet and 571 feet, i.e., on undisturbed glacial till. Below these elevations, the best estimated effective modulus of elasticity of the soil, including strain degradation due to seismic strains in the range of $3x10^{-3}\%$ to $3x10^{-2}\%$ is $22x10^{6}$ psf. Similarly, best estimate soil properties were established under other foundations such as the Railroad Bay of the Aux. Building model. The additional stiffening effects of the soil on the side walls due to embedment of the foundation below the ground surface level was incorporated.

In my judgment, this soil-structure interaction approach provides a good "best estimate" of the "softening" of the overall structural system stiffnesses due to soil-structure interaction effects (i.e., the effect of soil-structure interaction on the natural frequencies and mode shapes of vibration of the structure is reasonably approximated by this modeling of soil-structure interaction). However, because of uncertainties in soil properties, and uncertainties in the mathematical modeling of soil-structure interaction, there is significant uncertainty in the "softening" effect of soil-structure interaction. In order to cover this uncertainty, the soil-structure interaction stiffnesses are varied within the range from 0.5 to 1.5 times the "best estimate" soil-structure interaction stiffnesses. The seismic evaluation used for the foundation remedial work or for any future seismic margin evaluations will be based on the envelope of seismic responses obtained for soil-structure interaction stiffnesses which vary throughout this range of possible stiffnesses. I am convinced that this parameter variation in soil-structure interaction stiffnesses fully covers any uncertainty which exists concerning the "softening" effect on structural response due to soil-structure interaction, and avoids the need for more sophisticated modeling from this standpoint.

The design ground resonse spectra is fed directly into the soil-structure interaction impedance function elements ("stiffnesses" and "dashpots") and through them into the structure foundations. This approach ignores the spatial variation (both vertically and horizontally) of the ground motion. This spatial variation of ground motion occurs because the ground motion arrives at the site as a result of a series of propagating waves. These waves consist of a mixture of surface waves, and body waves (compressive and shear) propagating to the site at various incident angles relative to the ground surface. All theoretical wave propagation analysis models indicate a reduction in ground motion with depth below the ground surface within certain frequency bands. Field data are limited, but also shows this reduction. In other words, there is a theoretical basis and some field data which indicates a vertical spatial variation of the ground motion. This vertical spatial variation

of the ground motion significantly reduces the translational ground motion input to embedded structure foundations at least within certain frequency bands. However, vertical spatial variation also creates a rocking ground motion input to embedded structures. The simple soil-structure interaction model used for the Aux. Building and SWPS ignores the vertical spatial variation of the ground motion. The BWST is founded at the ground surface and vertical spatial variation of the ground motion has no impact. Ignoring vertical spatial variation of the ground motion leads to a significant overprediction of the translational response of the structure and a slight underprediction of the rocking response. Within my experience, the net effect is to produce considerable conservatism in the predicted response of the structure at lower elevations at least within certain frequency bands and approximately correct or sometimes considerably conservative predicted responses at higher elevations in the structure. The subject of vertical spatial variation of the ground motion is uncertain and the prudent engineer should either ignore the effect for design as is being done on Midland or must do parameter studies with more sophisticated soil-structure interaction models if he wished to take credit for this effect. So long as no credit is taken for the vertical spatial variation of the ground motion, the simple soil-structure interaction models of the type used on Midland will produce conservative response results.

Non-vertically propagating waves also produce horizontal spatial variation of the ground motion. Such horizontal variation of the ground motion also reduces the translational response of the structure but increases the torsional response at least within certain frequency ranges. The simple soil-structure interaction models used on Midland ignore this effect. Again, within my experience, the net effect is to produce considerable conservatism in the response of the central region of the structure at least in certain frequency ranges but approximately correct responses of the extremities of the structure. Again, the simplified soil-structure interaction models are adequate for conservatively computing responses so long as the benefits of this effect are ignored.

The assumption that the soil beneath the foundations is an elastic half-space can lead to an overprediction of the radiation damping (i.e., the radiation of energy from the structure into the ground). This situation occurs because an elastic half-space assumption does not account for the variation of soil properties with depth. Overprediction of the radiation damping results in too much energy dissipation being incorporated into the overall dynamic model which can lead to underprediction of the structural responses from this model. For the foundation remedial work and Bechtel's subsequent analyses of the structures and equipment, this potential problem is compensated for in the following, conservative fashion:

1. For modes of structural vibrations which are a combination of soil-structure interaction and flexible structural response, the composite modal damping is computed. If this composite modal damping (made up of structural damping, soil material damping, and radiation damping) exceeds 10% of critical, then the composite modal damping is arbitrarily and conservatively limited to 10% of critical.

2. For modes of structural vibrations which are nearly exclusively soil-structure interaction modes (i.e., rigid body structural response modes), the radiation damping used will be limited to 75% of the theoretical radiation damping levels.

3. For modes of structural vibration which are nearly exclusively structural modes, the composite modal damping value is not influenced by radiation damping into the soil and this discussion is irrelevant.

In my judgment, the layering effects (variation of soil properties with depth) beneath the Aux. Building, BWST, and SWPS are relatively minor and the radiation damping levels will be at least 75% of the theoretical elastic half-space values. Furthermore, the limitation of composite modal damping levels to 10% of critical for combined vibration modes has been shown by many studies to be an extremely conservative criteria which leads to overprediction of structural responses. Imposing this criteria more than compensates for any unconservatism which might result from the use of elastic half-space theory to estimate the radiation damping levels.

My conclusions are that the soil-structure interaction models are very simplified. However, these models have been carefully developed so as to provide high confidence that the simplifications result in conservatism in the prediction of structural response when these models are used. Thus, these models are adequate for the purpose of establishing conservative seismic forces to be used in the design of the foundation remedial fixes and in Bechtel's analysis of the structures and equipment.

3. Damping Levels Used in Models

The dynamic seismic models must also incorporate estimates of the energy dissipation capability of the structure. Viscous damping is used as a measure of the rate of energy dissipation for structures and equipment. Viscous damping is defined as a percentage of critical damping where critical damping is the minimum level of damping at which a structure will not oscillate in free vibration. Earthquake ground motion feeds a limited amount of energy into the structures and equipment over the duration of the ground motion. The higher the damping (rate of energy dissipation) the lower the maximum structural response resulting from earthquake ground motion. The importance of damping in reducing maximum structural response can be observed from Figure 1 in which the Housner response spectra (References 1 and 2) are presented for various damping levels. At the time the Midland Plant design was initiated, very low damping levels were generally used. The FSAR SSE damping levels are summarized in Table 1. More recently, considerably higher damping levels have been able to be justified. For instance, current design practice is to use U.S. NRC Regulatory Guide 1.61 (Reference 3) damping levels. These damping levels are considerably higher than those given in the Midland FSAR Plant design (see Table 1 for comparison), and result in a reduction of computed response. Use of the low damping levels in the Midland Plant design represents a source of conservatism.

Actually, the Regulatory Guide 1.61 damping levels are generally considered to be highly conservative. Although appropriate for design, they are too conservative for use in reviewing the seismic safety of

existing facilities. N. M. Newmark and W. J. Hall, acting as consultants to the U.S. NRC, have recommended ranges of damping levels appropriate for reviewing the seismic adequacy of existing nuclear plants (Reference 4). These damping levels are also tabulated in Table 1. Reference 4 states that the lower values of each damping range are nearly lower bounds while the upper values are essentially average (median) values. The NRC Staff formed a Senior Seismic Review Team (N. M. Newmark, W. J. Hall, J. D. Stevenson, F. J. Tokarz, and myself) to provide recommendations of criteria for the seismic review of existing nuclear plants in the Systematic Evaluation Program (SEP). This Senior Seismic Review Team (SSRT) has recommended the use of the upper damping values when reviewing the seismic safety of existing plants and such has been done in published SEP reviews (for instance, Reference 5).

It is my belief that the highest damping range values from NUREG/CR-0098 are the most appropriate values to use to make a best estimate of the seismic response. For new designs, the damping values from Regulatory Guide 1.61 should be used to introduce intentional conservatism into the seismic design. However, for the foundation remedial work, the Midland FSAR (Reference 1) damping levels are used in the mathematical models. This added conservatism is introduced to ensure that the remedial work will be shown to be seismically safe in a subsequent seismic margin review which incorporates Site Specific Response Spectra.

4. Auxiliary Building Dynamic Model

The model described herein will be used to evaluate overall building response to seismic loadings as well as to generate in-structure response spectra. The responses developed from this model will provide input to other static analyses to develop forces in the individual structural elements. The building is represented by a three-dimensional, lumped-mass stick model (with additional detail in the electrical penetration areas) which preserves the physical geometry of the various building components. This model has been developed accounting for the factors described in Sections 1 through 3 of my testimony.

Figure 2 presents a schematic plan of the Aux. Building. This building can be subdivided into a main auxiliary building, a control tower, and two electrical penetration wings which are interconnected. The foundation remedial work is being performed under the control tower and the electrical penetration wings under which underpinning is being extended down to undistrubed glacial till. The overall dynamic response of the Aux. Building can be modeled using a series of vertical sticks (stiffness elements) with each representing a major portion of the building (Figures 3 and 4). The stiffness and mass characteristics of the main auxiliary building (north of column line G) are modeled as one vertical stick. The control tower (south of column line H) is modeled as a second vertical stick. Flexible beam elements at each floor elevation between column lines G and H are used to model the flexible interconnectivity between these two portions of the Aux. Building. The stiffness for these connecting beam elements reflects both the floor properties and any interconnecting shear walls between column lines G and H. Rigid elements are used as connection members between column line H and the center line of the control tower stick, and between column line G and the center line of the main auxiliary stick to reflect the actual geometry between the two sticks.

The wing areas are made up of a major vertical wall (south side) with several intermediate cross walls; therefore, a series of plate elements have been used to represent the south wall along with three sticks per wing to reflect the intermediate cross walls. The plate elements are connected to the wing sticks by a series of rigid elements. The individual wing sticks are connected by horizontal beam elements whose stiffness represents the existing floor properties. The wing boundary nearest the control tower is connected to the control stick by rigid elements, representing the geometric distance to the control tower stick.

Figures 5 and 6 present the overall Aux. Building dynamic model. Solid balls (●) on this model represent locations where masses are lumped in this model. The mass associated with the main auxiliary and control tower has beem lumped at the major floor elevations. The mass includes concrete, steel, blockwalls, major equipment and 25% of the floor design live loading. The center of mass was established for each floor level and these mass nodes were placed at these centers of mass. For the wing areas, the mass associated with each plate element have been lumped in accordance with the plate thicknesses and the remaining mass associated with each wing lumped at the floor elevations along the six sticks. Stick (beam) elements in Figure 5 define the stiffness characteristics of the structural systems being represented. These stick elements have been located at the calculated centers of rigidity and are thus horizontally offset from the mass points and from each other. These offsets (eccentricities) are included to properly account for torsional vibrations. These offsets at floor levels are accomplished by the use of static node points (O) which do not have mass associated with them. The plate elements (Figure 6) are used to model the stiffnesses of the south wall of the electrical penetration wings and of the underpinning under these wings. These plate stiffness elements are interconnected to the stick elements at the node points shown.

The proposed underpinning design underneath the control tower has been accounted for in the section properties of the control tower stick below Elevation 614 feet. The underpinning wall layout is connected to the existing column line H wall to make up the extension of the control tower stick to Elevation 562 feet. This portion of the control tower stick is also connected to the main auxiliary stick by beam elements representing the floor properties and interconnecting shear walls in the same manner as the higher elevations. The mass associated with this portion of the control tower stick includes both the concrete and any effective entrapped soil.

The wing area underpinning is represented by a series of plate elements having section properties equivalent to the underpinning concrete sections. The wing underpinning plates are connected to the

structural wing sticks and plates by rigid beams to maintain the geometric location and continuity of stiffness between the underpinning and existing structure. The underpinning plates are connected to the control tower stick by rigid elements to reflect the geometry. The mass associated with the wing underpinning has been lumped to the nodes connecting the plate elements. This mass includes the actual concrete volume and the effective entrapped soil.

Soil-structure interaction impedance functions (represented by stiffnesses and dashpots) are attached to the structural model at the foundation mass points (■) shown. A single set of soil-structure interaction impedance functions are used for the main auxiliary and control tower portions of the foundation and are attached at the center of resistance for this foundation system. Individual impedance functions are placed at distributed node points at the base of the underpinning system for the electrical penetration area.

The soil-structure interaction impedance functions were developed in accordance with the approach discussed in Section 2. In order to cover uncertainty in soil-structure interaction stiffnesses, the stiffness properties will be varied over the range from 0.5 to 1.5 times the best estimate properties. However, for the Aux. Building model, there is also uncertainty in the relative stiffnesses between the soil-structure interaction stiffnesses beneath the electrical penetration wings and the stiffnesses beneath the main auxiliary building and control tower complex. The relative stiffnesses are likely to influence the electrical penetration wing responses in the N-S direction. Thus, for the N-S direction, very conservative upper and lower bounds on the electrical penetration wing soil-structure interaction stiffnesses relative to the main auxiliary building soil-structure interaction stiffnesses were estimated with the upper bound being a factor of 5 greater than the lower bound. The relative soil-structure interaction stiffnesses of the electrical penetration wings will be selected within this factor of 5 range of possible uncertainty so as to maximize the computed

responses of the electrical penetration wings if it is found that this relative stiffness ratio significantly influences the N-S responses of the electrical penetration wings.

The Aux. Building dynamic model is a complete three-dimensional model as schematically illustrated in Figure 7. This model is capable of conservatively computing the overall responses in all three orthogonal directions including all torsional effects.

5. Service Water Pump Structure Dynamic Model

The model described herein will be used to evaluate the overall building response to seismic loadings as well as to generate in-structure response spectra. The responses developed from this model will provide input to other static analyses to develop forces in the individual structural elements. The building is represented by a three-dimensional lumped-mass stick model using beam elements. This model has been developed in a manner very similar to that used for the Aux. Building model.

Figures 8 and 9 present a schematic view and plan of the SWPS. The foundation remedial work consists of placing an underpinning wall under the northern portion of this building so as to bring its foundation down to Elevation 587 feet which is beneath the backfill. The overall dynamic response of the SWPS can be modeled using a single vertical stick as shown in Figures 10 and 11. Figure 12 presents the resultant SWPS model. The mass of the structure is lumped at the major floor elevations. The mass includes concrete, steel, major equipment, water within building, entrapped soil, and 25% of the floor design live loading. The center of mass was established for each floor level. The horizontal inertial effects of the water mass within the SWPS are lumped at Elevations 589.5, 605, and 620 feet. The vertical water mass inertial effects are lumped at the base (Elevation 589.5 feet) because the water transmits its own dynamic responses down to this level rather than having these effects transmitted through the wall. Similarly, the soil entrapped within the

underpinning walls respond horizontally with these walls and horizontal inertial mass of this entrapped soil is also lumped at Elevations 589.5, 605, and 620 feet. The vertical inertial entrapped soil mass effects are carried by the soil and do not load the building so this inertial mass is not included. Because the vertical and horizontal inertial (mass) effects of the water and entrapped soil are treated differently, the vertical and horizontal centers of mass at which the masses are lumped differ from each other at Elevations 589.5, 605, and 620 feet. These differences are incorporated into the model as shown in Figure 12.

Stick (beam elements) in Figure 12 define the stiffness characteristics of the structural systems between floor levels. These stick elements have been located at the calculated centers of rigidity between each level and are thus horizontally offset from the mass points and from each other. These offsets (eccentricities) are included to properly account for torsional vibrations. Rigid elements are used to connect the center of stiffness and center of mass.

The proposed underpinning design underneath the northern portion of the building has been accounted for in the section properties below Elevation 620 feet. The underpinning wall layout is connected to the existing wall to make up the extension of the stick to Elevation 587 feet.

The soil-structure interaction impedance functions (developed in accordance with the approach discussed in Section 2) are attached to the structure portion of the SWPS model at Elevation 589.5 feet. The design ground motion is fed into this structure through these soil-structure interaction impedance functions.

The SWPS dynamic model is a complete three-dimensional model. This model is capable of conservatively computing the overall responses in all three orthogonal directions including all torsional effects.

6. Borated Water Storage Tank Model

The BWST represents a special structure in which the dynamic analysis model differs significantly from that for buildings. A professional (voluntary) committee of the American Society of Civil Engineers is currently engaged in writing a standard for the Seismic Analysis of Safety Class Structures. I am Chairman of this committee. One of the sections of this standard deals with Aboveground Vertical Tanks. I have written the current draft of this section. This draft has been approved by the other committee members. This section of the draft seismic analysis standard which deals with Aboveground Vertical Tanks is attached as Attachment B. This standard section and its commentary is directly applicable to the BWST.

The BWST is a vertical cylindrical tank with a diameter of 52 feet and cylindrical wall height of 32 feet and a domed head (Figure 13). Borated water is stored up to a height of 32 feet. The tank shell is supported on a ring foundation which must withstand the seismic-induced forces in the tank shell. These forces are nearly totally due to the water in the tank since the tank shell weight is negligible compared to the weight of this water. Thus, the primary seismic modelling concern is to properly or conservatively model the seismic forces induced by this water on the tank shell and thus on the foundation. One must model the impulsive mode, the sloshing mode, and the vertical mode of fluid-stucture interaction. Each of these modes of response is best modeled with its own individual model. The seismic forces imposed upon the tank shell and ring foundation from each of these three models are added by the square-root-sum-of-squares (SRSS) method.

6.1 Impulsive Mode

Figure 14 presents the dynamic model of the BWST which I would recommend be used for determining the seismic forces on the ring foundation from the horizontal impulsive fluid mode. The tank shell stiffness (shear, flexure, and ovalling) is modeled by vertical stick elements between mass points distributed up the tank shell. The roof

weight, W_R, is lumped at the roof level. The shell wall weights, W_S, are lumped at discrete points on the tank shell. Impulsive fluid effective weights, W_I, are added to the tank shell weights at each of these node points at and below the top of the fluid.

For a rigid mode of horizontal tank vibration, it has been shown by Housner (Reference 2) that the total effective horizontal impulsive weight of the fluid, W_I, is given by Equation 3633.1a of the commentary of Attachment B. This total effective impulsive weight is distributed parabolically over the fluid height as shown in Figure 14. With a flexible tank, the impulsive fluid effects should more precisely be considered as an impulsive pressure rather than effective impulsive weights. However, it has been shown by Veletsos (Reference 6) that the effective impulsive weight distribution developed by Housner for rigid tanks can be used to conservatively predict impulsive mode base shears and overturning moments at the bottom of flexible tanks (i.e., the forces on the ring foundation). For the BWST, this approximation leads to base shears which are between a factor of 1.1 and 1.2 times greater than would be obtained using flexible tank impulsive pressures. The overturning moments obtained assuming a Housner effective weight distribution are within 2% of those obtained using a flexible tank impulsive pressure distribution. This slight improvement in accuracy does not warrant the substantial added effort of treating the tank shell as flexible when determining the impulsive fluid effects. The effective impulsive fluid weight distribution shown in Figure 14 is adequate.

The horizontal impulsive mode tank model is attached to the ground at its base by soil-structure interaction impedance functions (defined in terms of a translational and rocking stiffnesses and dashpots) derived as per Section 2 of this testimony. The ground motion is fed into this tank model through these impedance functions. The resultant overturning moment and base shear at the base of this model represent the forces imposed on the ring foundation by the horizontal impulsive mode.

6.2 Sloshing Mode

The horizontal fluid sloshing mode is a long period (low-frequency) mode of vibration. Because of its low frequency, this mode of vibration does not interact with the effects of tank flexibility or soil-structure interaction. A dynamic model is not required in order to evaluate the forces imposed on the tank shell and ring foundation by this mode. The natural frequency of vibration, ω_2, of this mode is accurately computed from Equation 3634.3 of the commentary of Attachment B, while the fluid effective sloshing weight, W_2, and height of application, X_2, above the tank base are given by Equations 3634.1 and 3634.2. The base overturning moment on the ring foundation due to this sloshing mode is given by Equation 3634.4.

6.3 Vertical Mode

In the vertical mode, the water in the tank is supported directly on the soil and the tank itself is very stiff. Therefore, both the tank and the fluid can be modeled as rigid in this mode. The only source of flexibility comes about because of soil-structure interaction effects. A dynamic model is not required for such a simple problem. The natural frequency of vibration is given by:

$$\omega_V = \sqrt{\frac{K_V\, g}{W_V}}$$

where W_V is the sum of the tank shell weight, W_S, and the total fluid weight, W_W, and K_V is the vertical soil-structure interaction impedance function stiffness. This is a rigid structure mode of vibration for which the fraction of critical damping, β_V, is given by:

$$\beta_V = \frac{C_V}{2\sqrt{K_V W_V / g}} + \beta_S$$

where C_V is the vertical dashpot coefficient from the soil-structure interaction impedance function for the foundation, g is gravity units (386.4 inches/second^2), and β_S is the appropriate soil material damping (3% of critical).

The ring foundation only supports the vertical seismic forces from the shell. The vertical fluid forces are supported directly on the soil. Thus, the vertical seismic forces on the ring foundation are given by:

$$F_V = W_S \cdot S_{A_V}(f_V, \beta_V)$$

where S_{A_V} represents the design seismic vertical spectral acceleration at damping level β_V and cyclic natural frequency f_V where $f_V = \omega_V/2\pi$. Thus, the vertical seismic forces on the ring foundation can be determined without developing a mathematical model.

7. Conclusions

I have reviewed the seismic models presented in this testimony for the Aux. Building, SWPS, and BWST. I consider these models to be adequate for the purpose of 1) defining seismic design forces to be used in the design of foundation remedial work on these structures, 2) conservatively estimating the seismic-induced forces in these structures, and 3) defining the seismic input to equipment, systems, and components mounted on these structures.

REFERENCES

1. "Final Safety Analysis Report, Midland Plant - Units 1 and 2", Volumes 7 and 9, Revision 32, Consumers Power Company, 1981.

2. TID-7024, "Nuclear Reactors and Earthquakes", Lockheed Aircraft Corporation and Holmes and Narver, Inc., August, 1963.

3. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants", U.S. Nuclear Regulatory Commission, October, 1973.

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

5. Newmark, N. M., W. J. Hall, R. P. Kennedy, J. D. Stevenson, and F. J. Tokarz, "Seismic Review of Dresden Nuclear Power Station - Unit 2 for the Systematic Evaluation Program", NUREG/CR-0891, U.S. Nuclear Regulatory Commission, April, 1980.

6. Veletsos, A. S., and Yang, J. Y., "Dynamics of Fixed-Base Liquid-Storage Tanks", Presented at U.S.-Japan Seminar for Earthquake Engineering Research with Emphasis on Lifeline Systems, Tokyo, Japan, November, 1976.

TABLE 1

COMPARISON OF DAMPING VALUES

Type and Condition of Structures	Midland SSE Analysis (Reference 1) (% of Critical)	U.S. NRC Reg. Guide 1.61 SSE Levels for Current Design (Ref. 3) (% of Critical)	U.S. NRC NUREG/CR-0098 SSE Levels for Seismic Review of Existing Facilities (Ref. 4) (% of Critical)
a. Vital Piping	0.5	2 to 3	2 to 3
b. Welded Steel Plate Assemblies	1	3 to 4	5 to 7
c. Welded Frame Structures	2	4	5 to 7
d. Bolted Steel Structures	2.5	7	10 to 15
e. Prestressed Concrete (w/o loss of prestress)	5	5	5 to 7
f. Prestressed Concrete (w/no prestress left)	—	—	7 to 10
g. Reinforced Concrete Equipment Supports	3	7	7 to 10
h. Reinforced Concrete Buildings	5	7	7 to 10

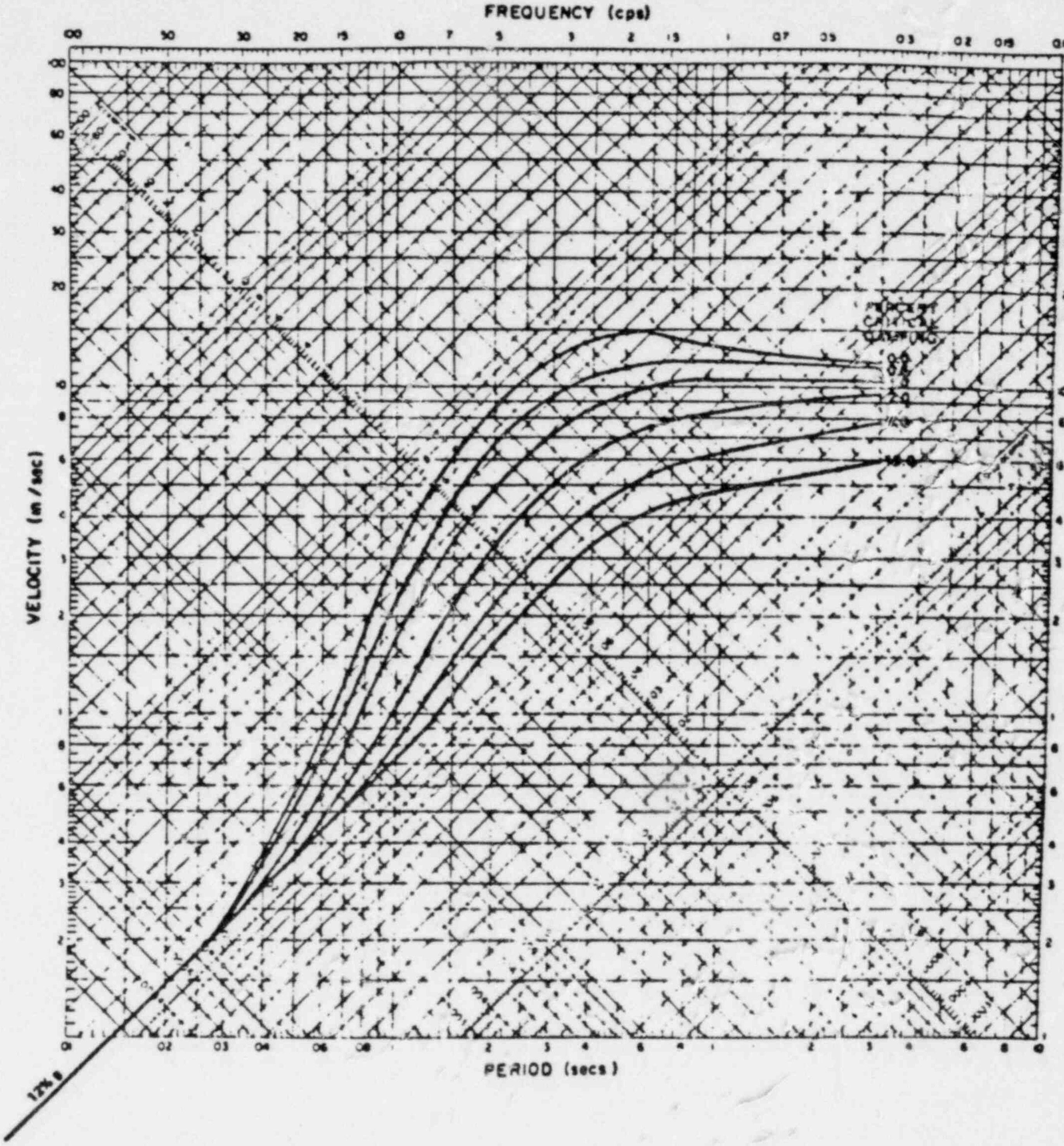
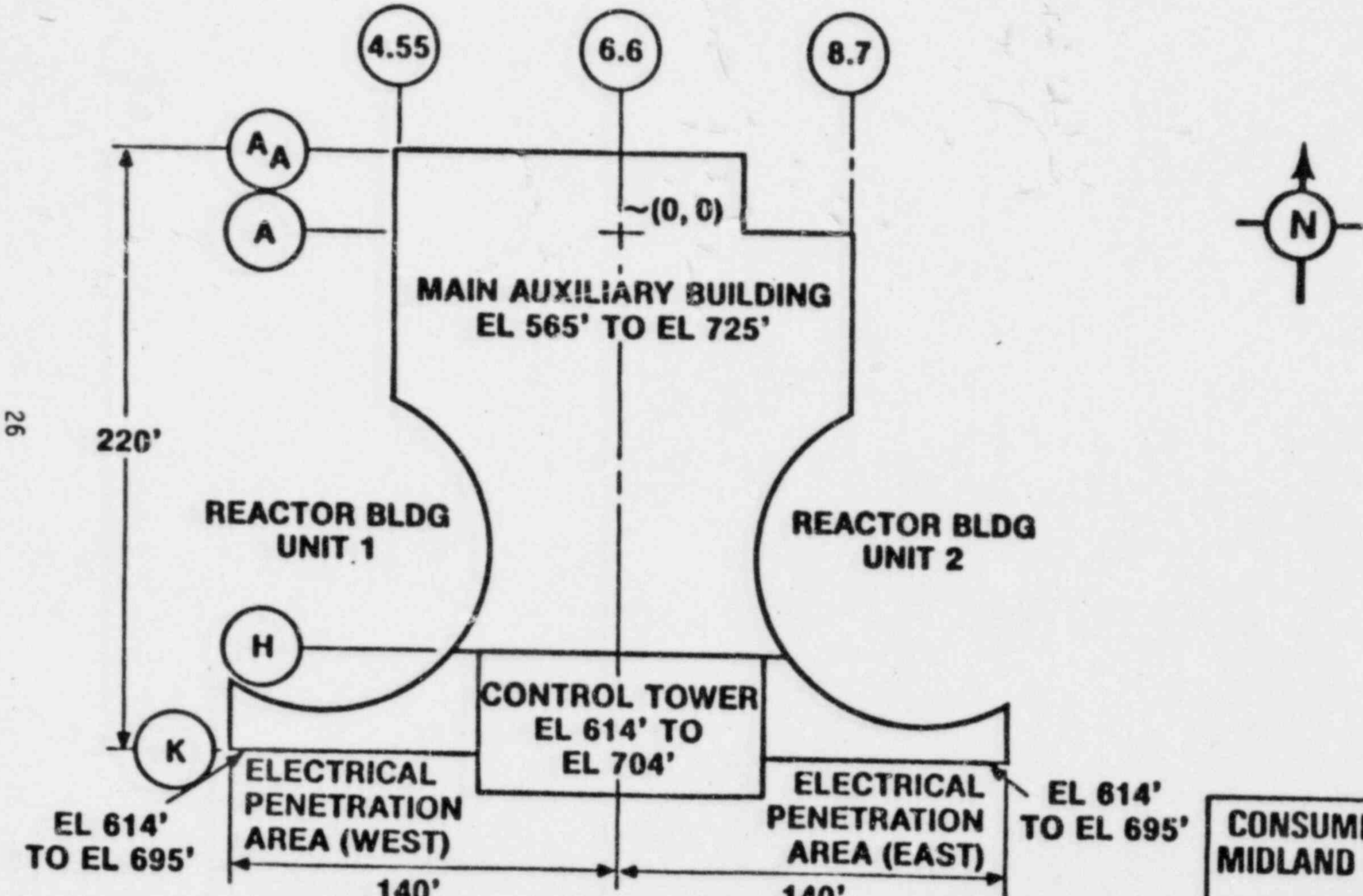


FIGURE 1. HOUSNER RESPONSE SPECTRUM ANCHORED TO 0.12g FOR SSE (REFERENCE 1)*

* FSAR DESIGN SPECTRUM IS MODIFIED FROM THE HOUSNER SPECTRUM IN THE PERIOD RANGE OF 0.2 TO 0.6 HZ BY A AMPLIFICATION FACTOR OF 1.5

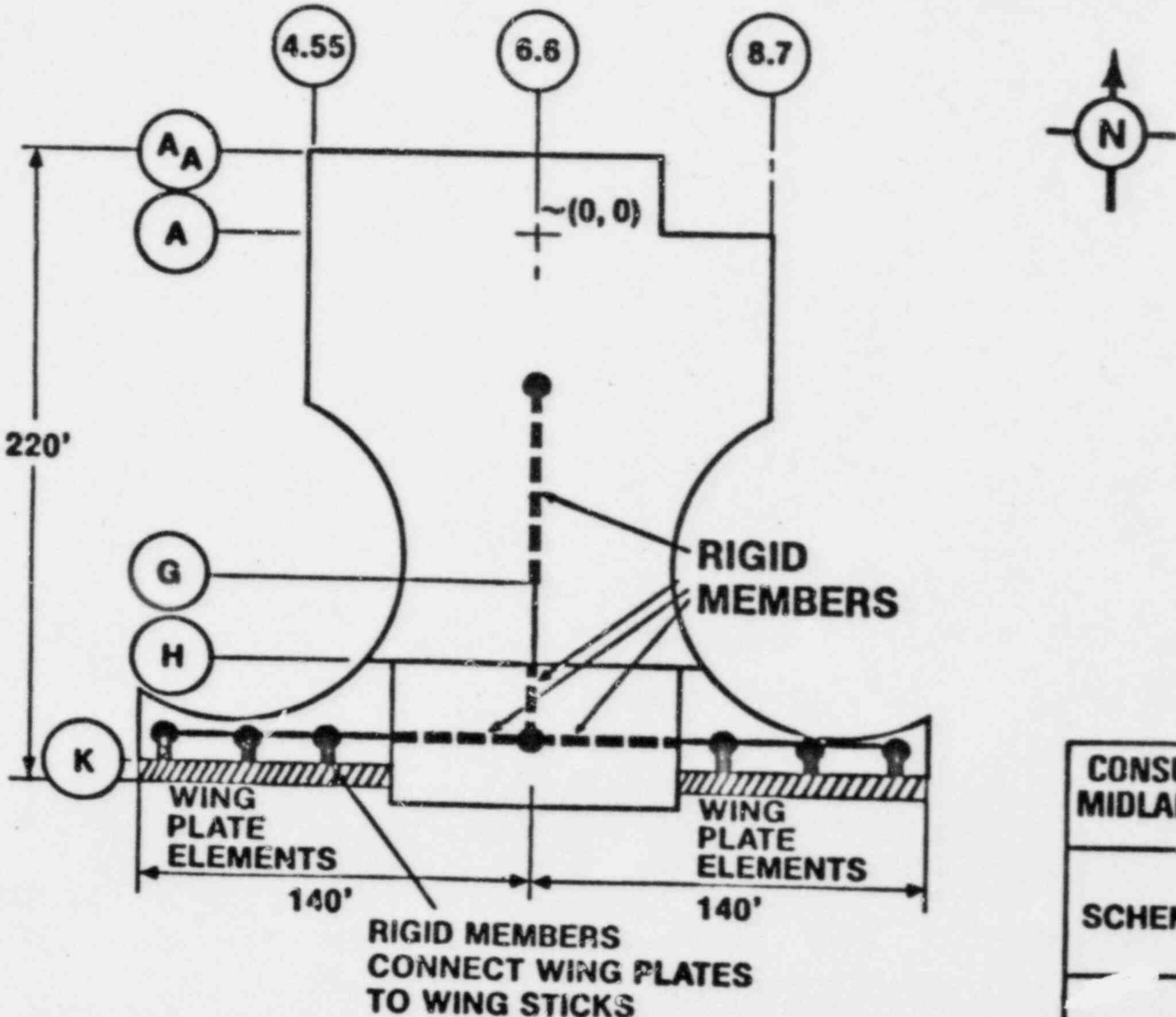
AUXILIARY BUILDING
SCHEMATIC PLAN



CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2
SCHEMATIC PLAN
FIGURE 2

G-1529-136

AUXILIARY BUILDING
SCHEMATIC PLAN
(With Conceptual Seismic Model)



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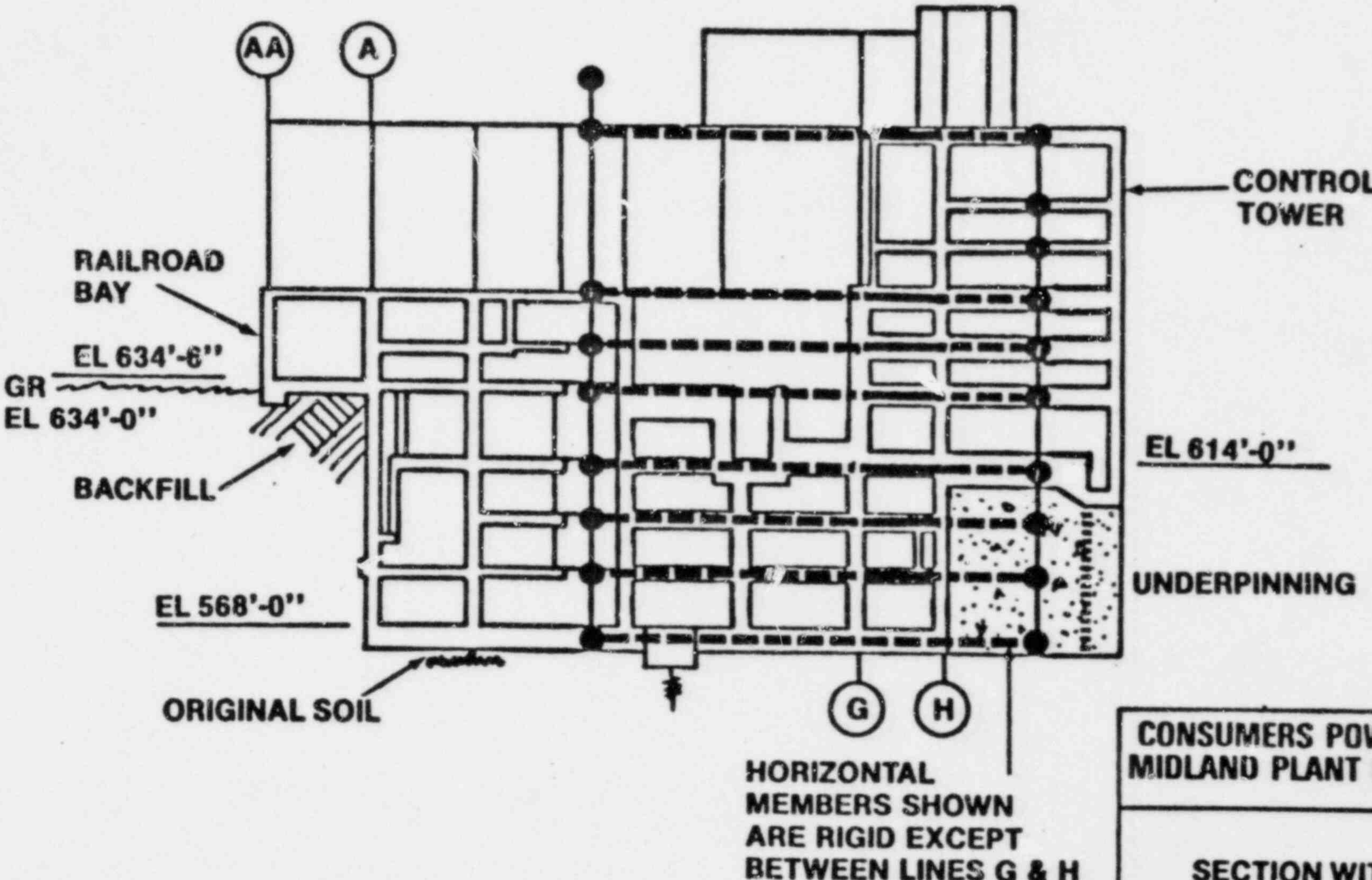
CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2
SCHEMATIC PLAN WITH MODEL
FIGURE 3

AUXILIARY BUILDING
TYPICAL SECTION LOOKING EAST
(With Conceptual Seismic Model)



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

SECTION WITH MODEL

FIGURE 4

MIDLAND UNITS 1 AND 2
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G-1529-134

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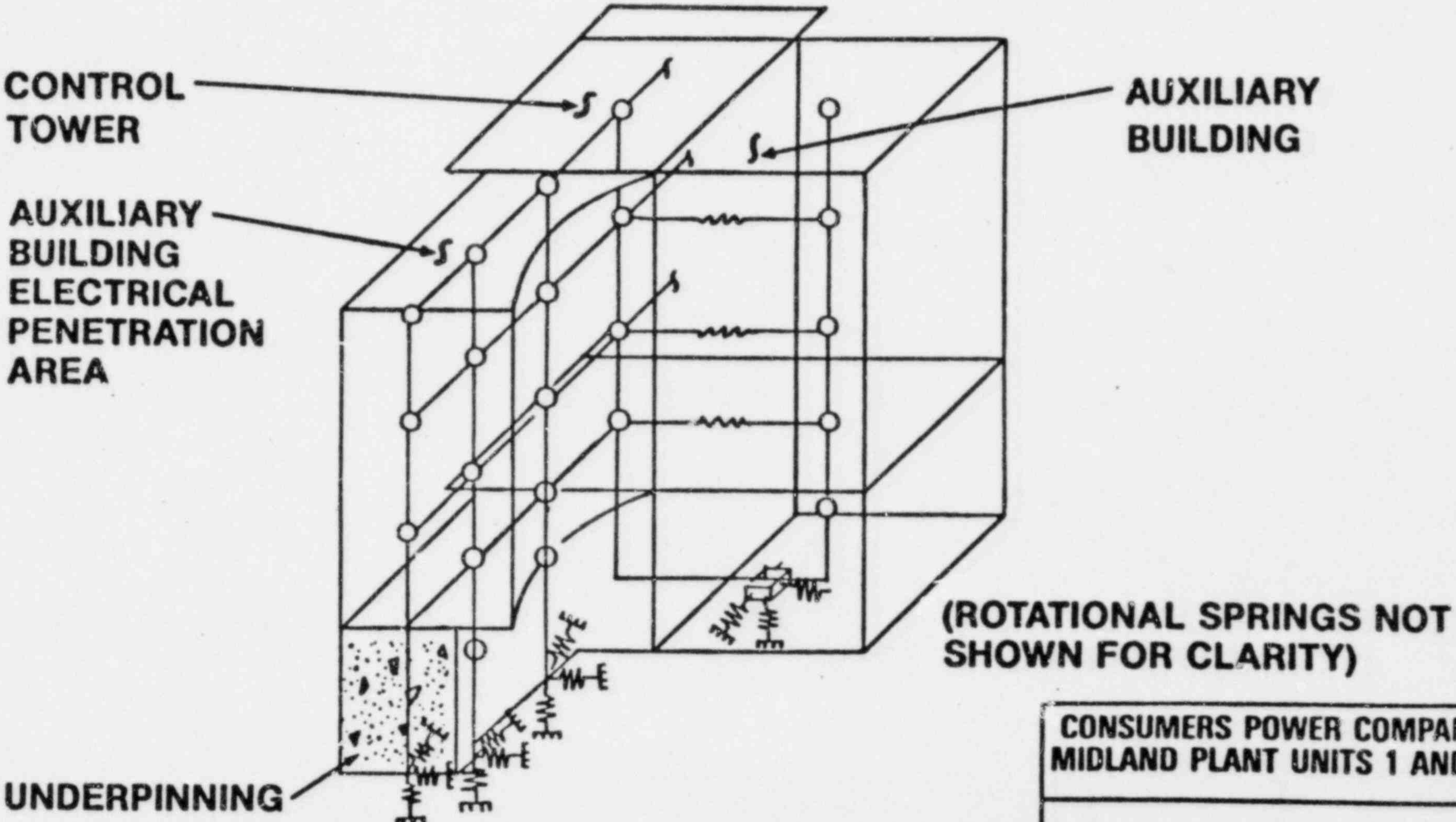
Figure 5

Figure 6

AUXILIARY BUILDING

(With Conceptual Seismic Model)



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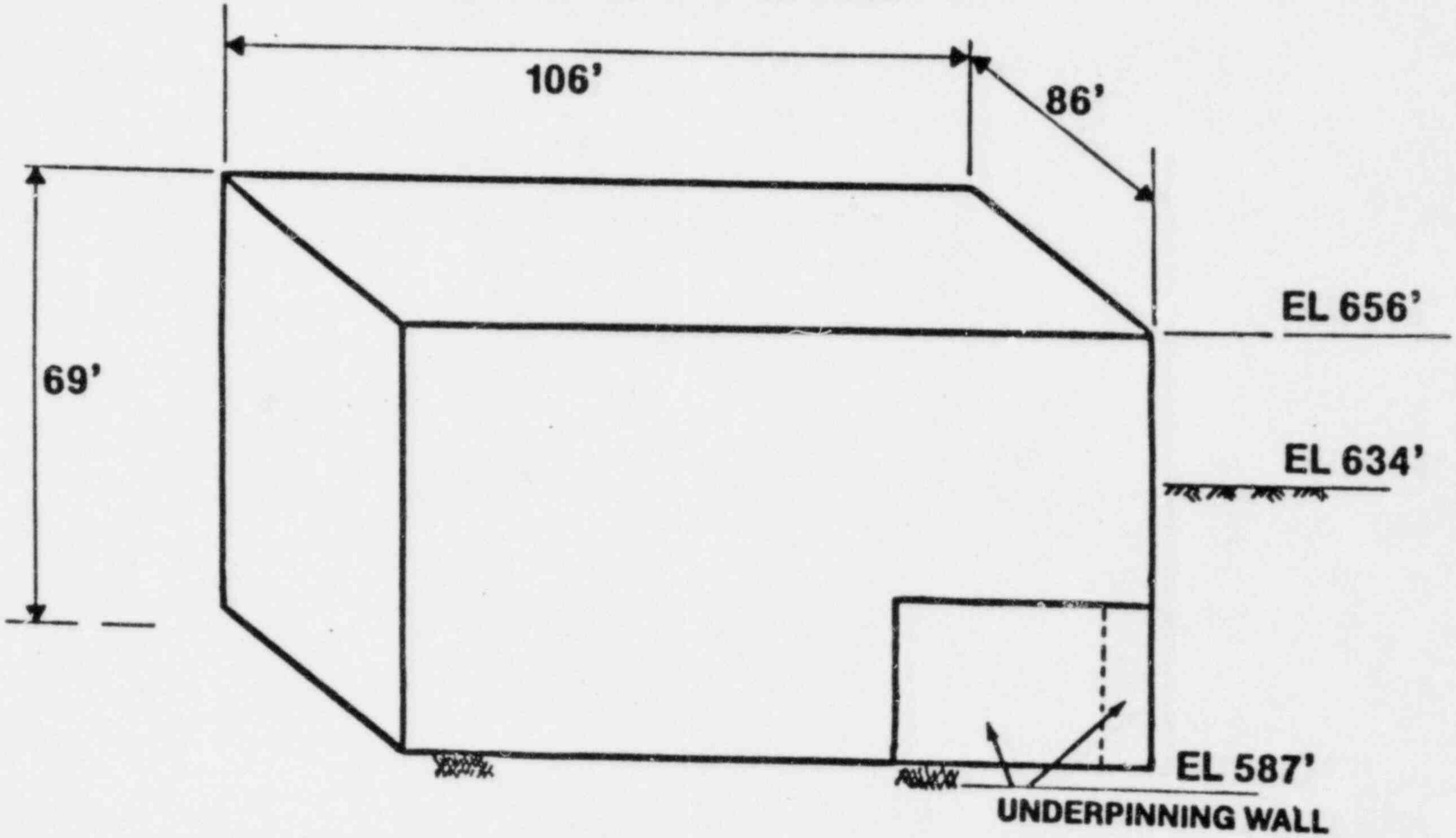
CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2
ROUGH 3-D MODEL
FIGURE 7

MIDLAND UNITS 1 AND 2
BLC 9/8/81

G-15

SERVICE WATER PUMP STRUCTURE



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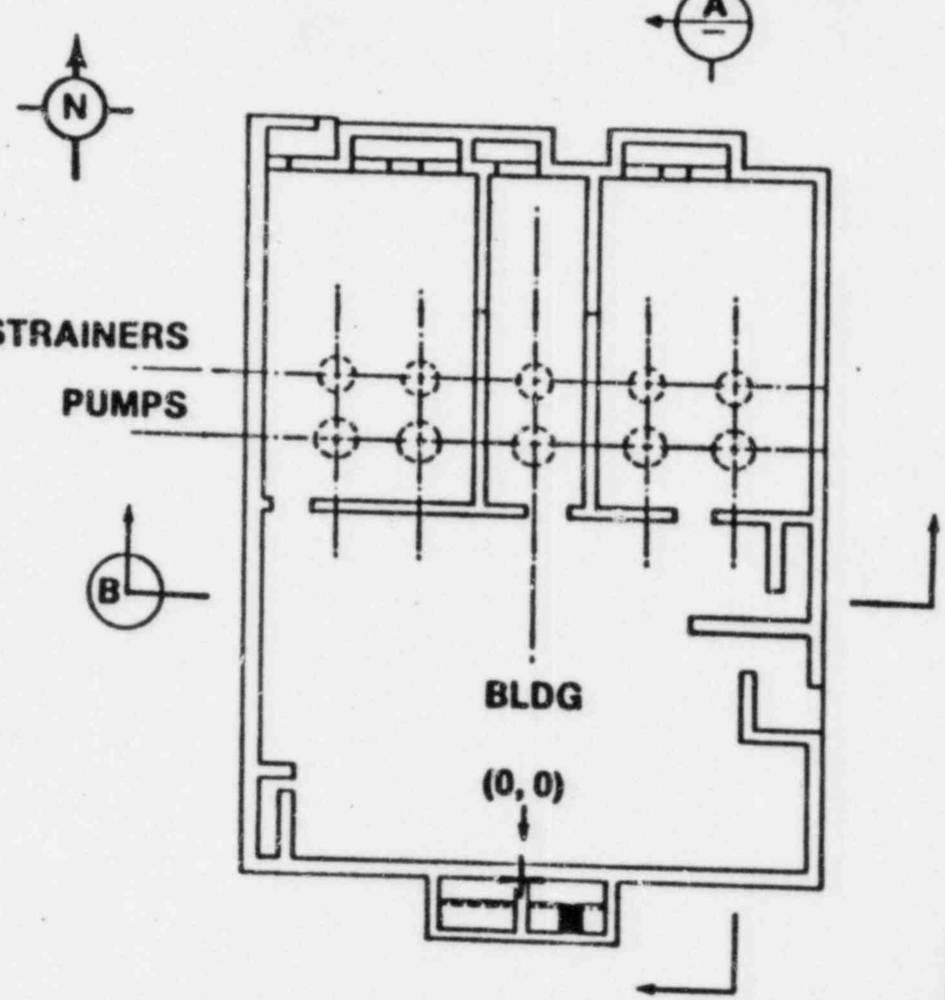
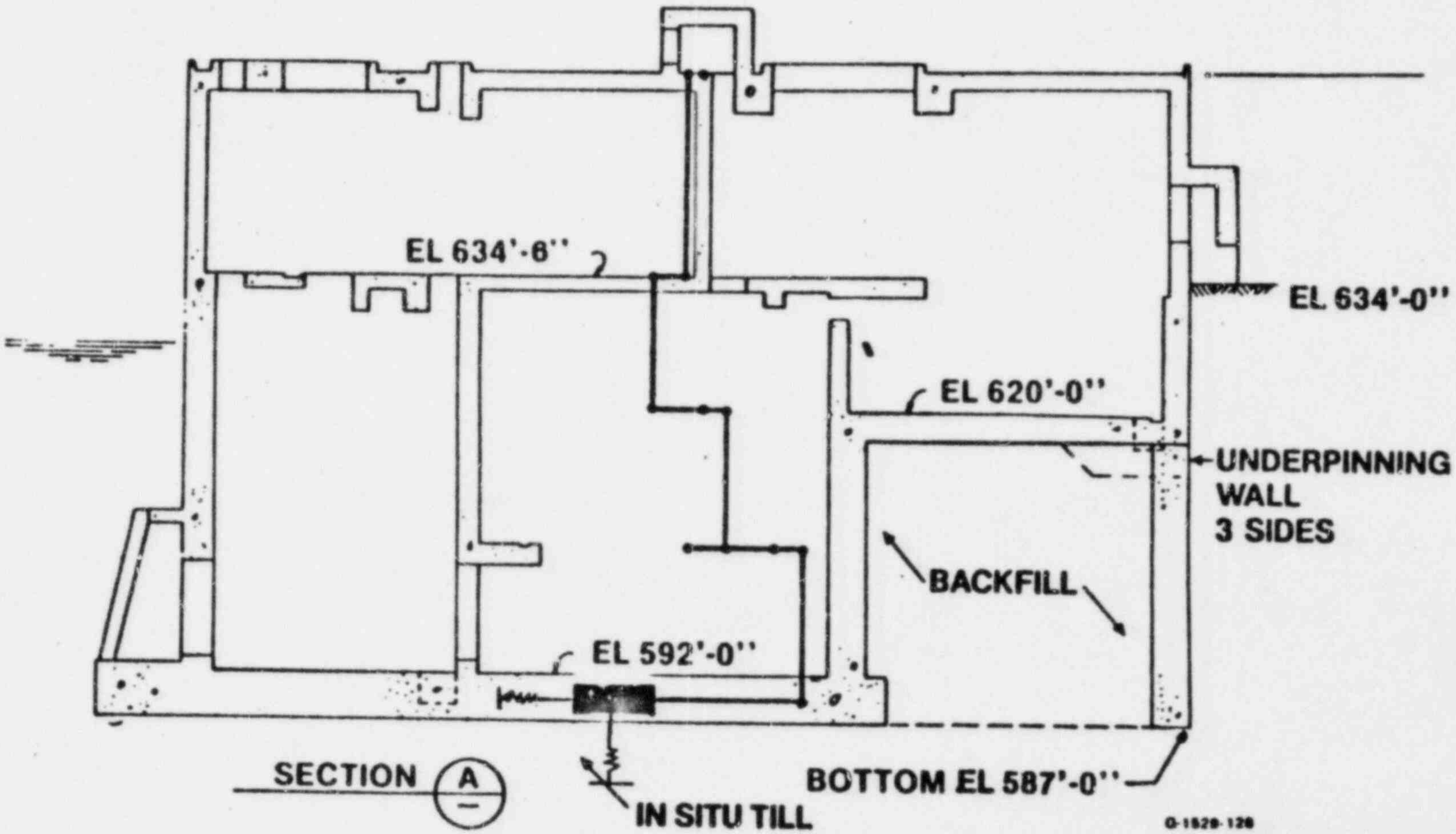
CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2
SERVICE WATER PUMP STRUCTURE SCHEMATIC VIEW
FIGURE 8

SERVICE WATER PUMP STRUCTURE
PLAN AT EL 634'-6"



CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2
SERVICE WATER PUMP STRUCTURE PLAN
FIGURE 9

SERVICE WATER PUMP STRUCTURE
SECTION A

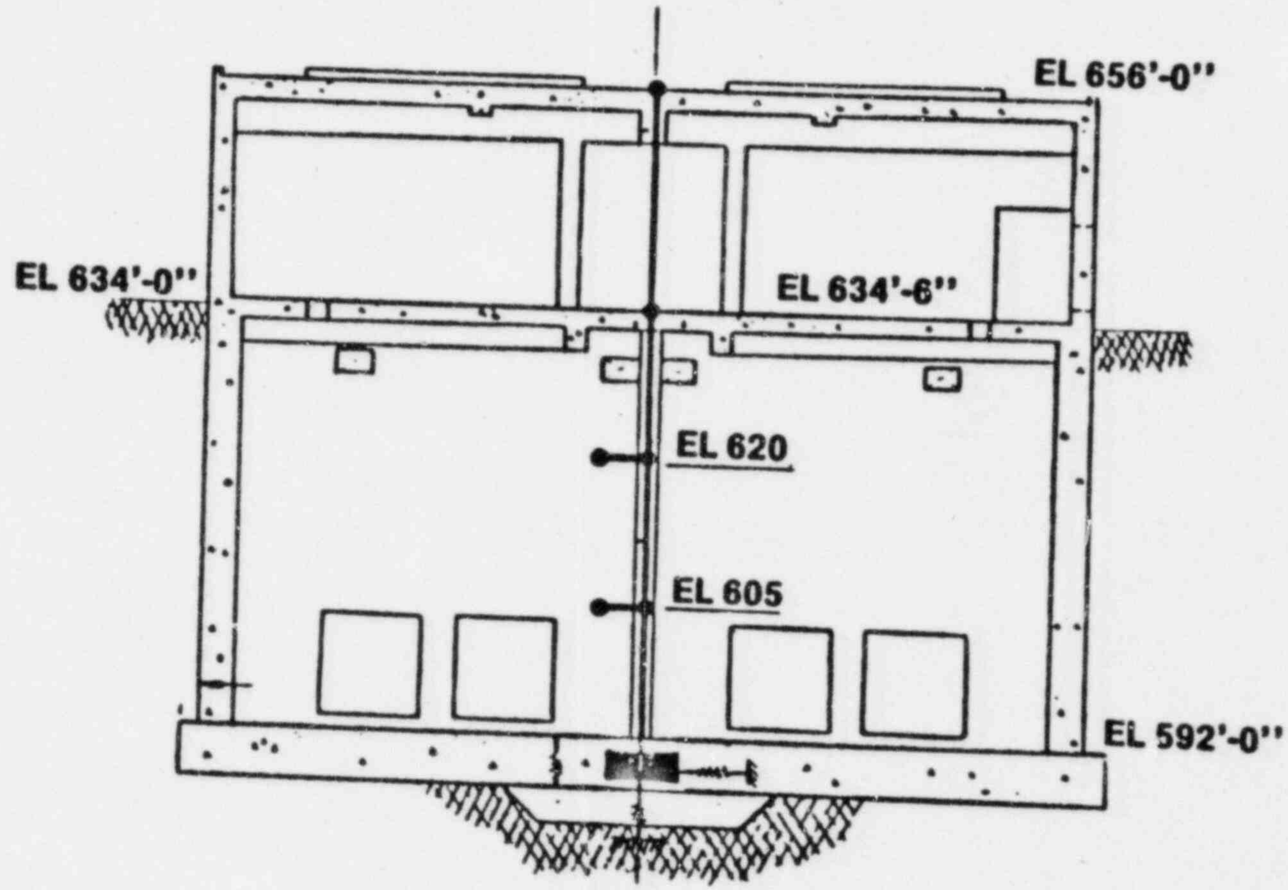


CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

SERVICE WATER
PUMP STRUCTURE
NORTH-SOUTH SECTION

FIGURE 10

SERVICE WATER PUMP STRUCTURE
SECTION B



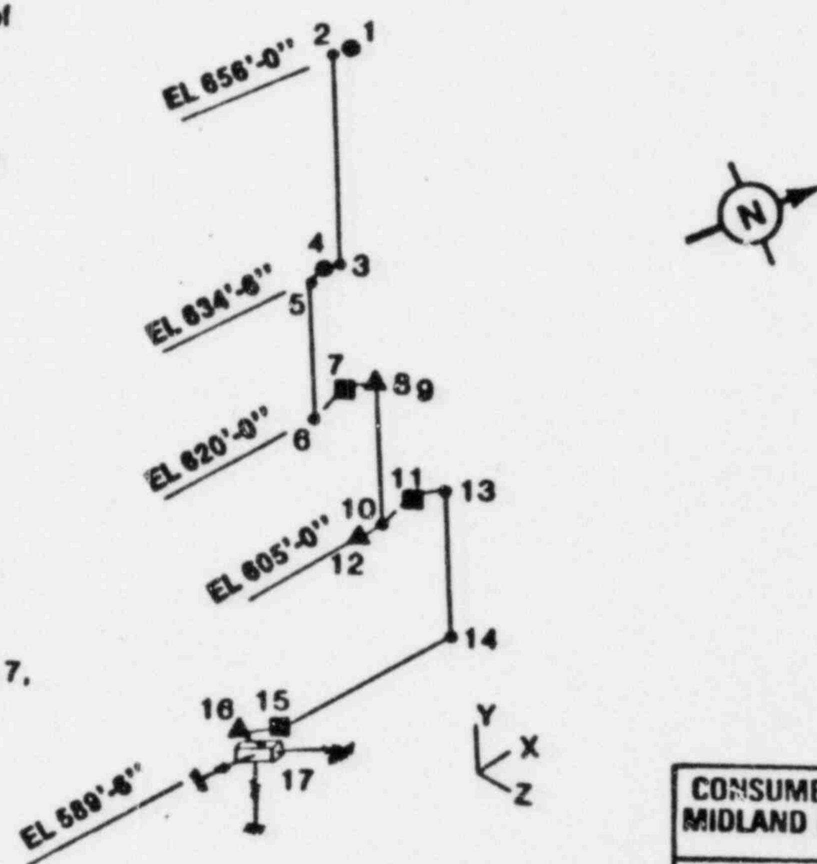
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

SERVICE WATER
PUMP STURCTURE
EAST-WEST VIEW

FIGURE 11

LEGEND

- Node locations
- Mass for all 3 degrees of freedom
- Mass for two horizontal degrees of freedom
- Mass for vertical degree of freedom
- Base location. Damper rotational springs not shown for clarity



NOTES:

1. The mass of the water is lumped at mass points 7, 11, and 15 horizontally and at mass point 16 vertically.
2. The mass of the fill entrapped within the underpinning walls is lumped at mass points 7, 11, and 15 for the two horizontal degrees of freedom only.

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

SERVICE WATER
PUMP STRUCTURE
NODE LAYOUT

FIGURE 12

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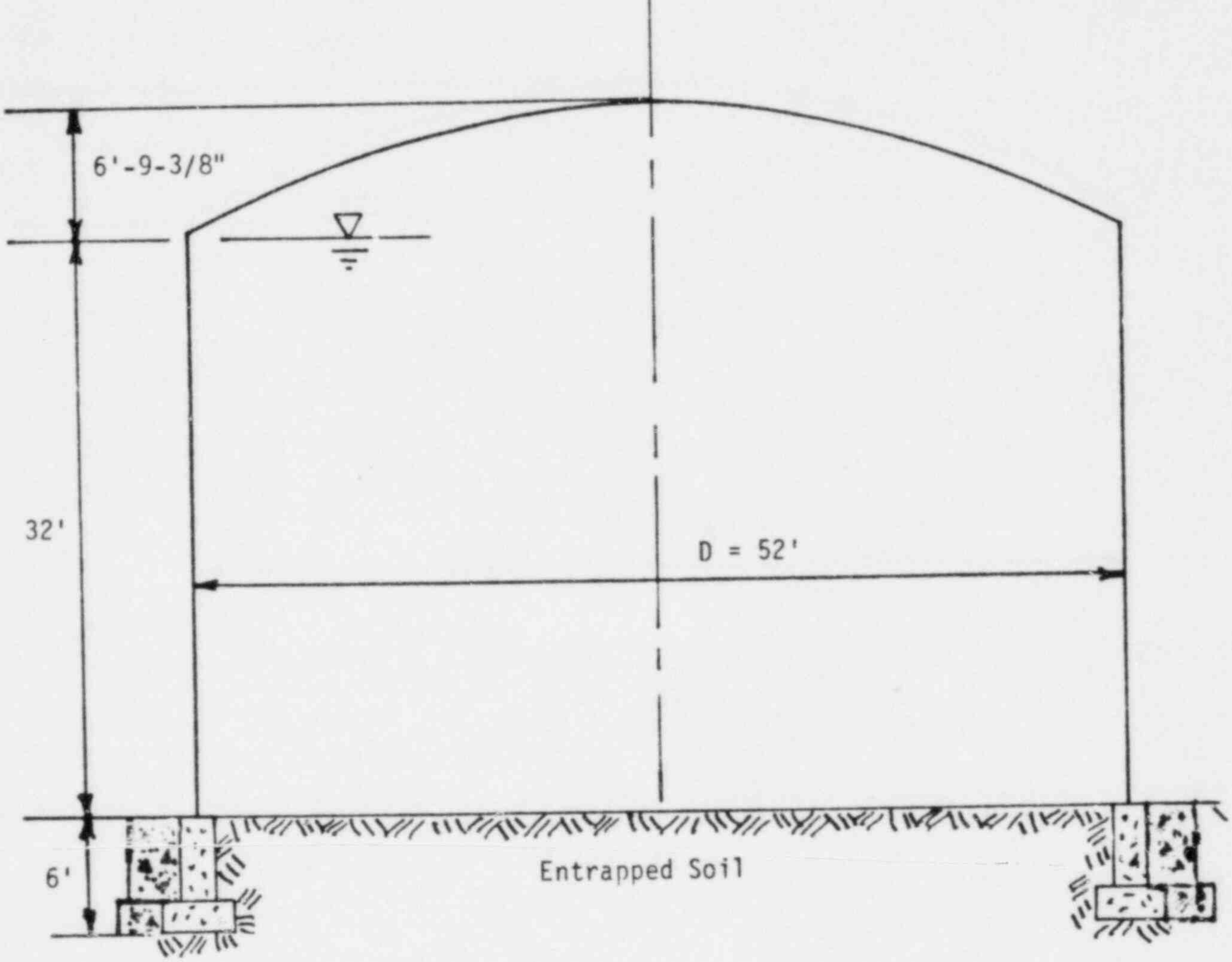

FIGURE 13: BORATED WATER STORAGE TANK CONFIGURATION

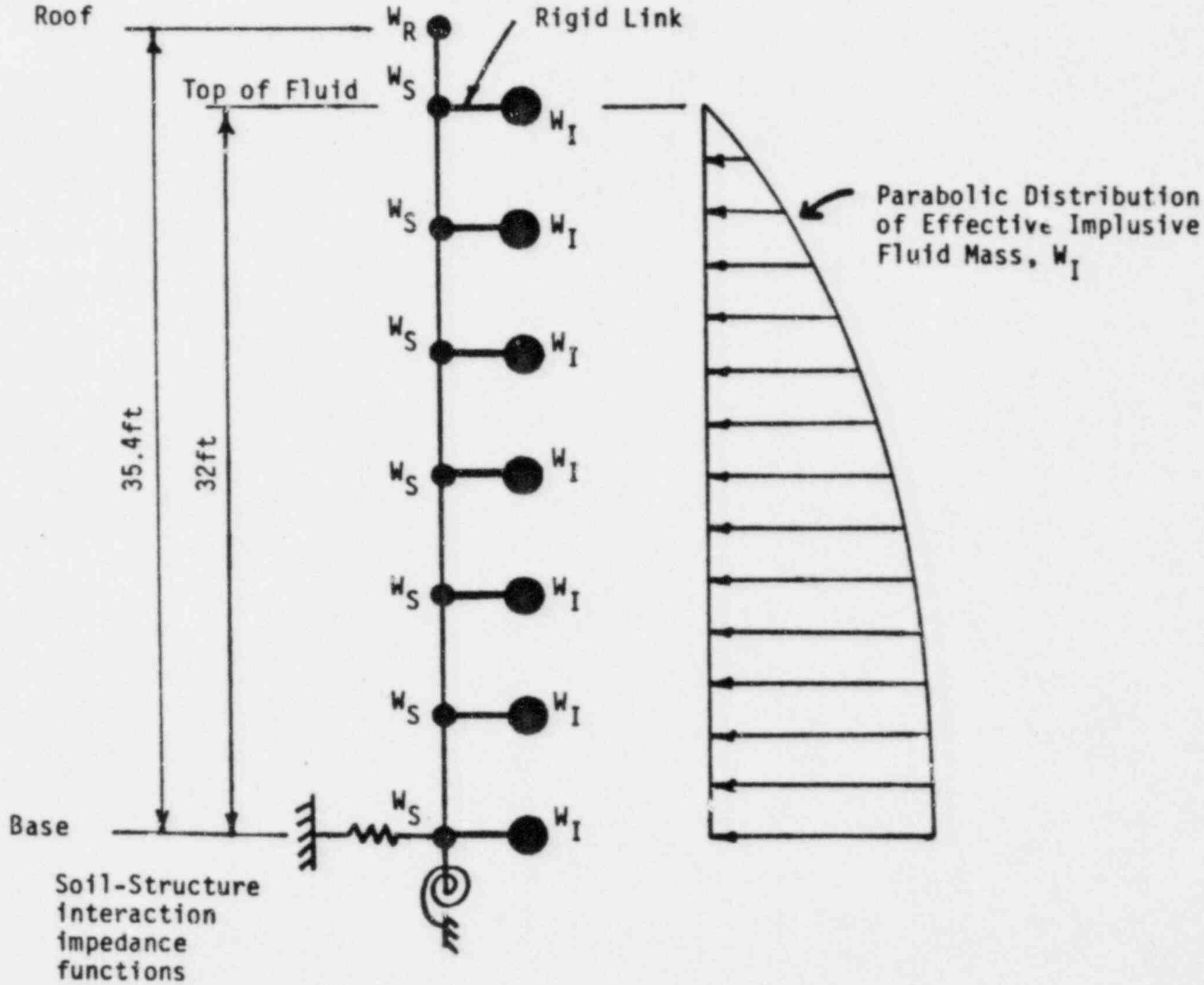

FIGURE 14: BWST HORIZONTAL IMPULSIVE MODE MODEL

ATTACHMENT A

RESUME

ROBERT P. KENNEDY - Principal

EDUCATION

B.S. - Civil Engineering, Stanford University
M.S. - Structural Engineering, Stanford University
Ph.D. - Structural Engineering, Stanford University

REGISTRATION

Civil Engineer, States of California, Tennessee, Texas and Alabama

SUMMARY

Nineteen years experience in static and dynamic analysis plus design of special purpose civil and mechanical type structures, particularly for the nuclear, petroleum, and defense industries: design of structures to resist extreme loadings including seismic, missile impact, extreme wind, impulsive loads, and nuclear environmental effects; development of computerized structural analysis methods; administrative and program management; and teaching.

PROFESSIONAL EXPERIENCE

Dynamic Loads - Nuclear Facilties

Extensive experience in the analysis of nuclear facilities subjected to extreme dynamic loads including effects of external missile and aircraft impact, and impulsive loading resulting from loss-of-coolant accident and SRV discharge. Prime developer of the method currently in extensive use by the nuclear industry in the United States for evaluating the local effects of missile impact on concrete. Consultant on the effects of aircraft impact for several nuclear plants. Consultant to General Electric on effects of pool swell loads resulting from LOCA, and on the increased dynamic reserve margin available in structures subjected to pulsive loads. Consultant to G.E. and Mark I, Mark II, and Mark III Owner's Group on combination of responses from multiple dynamic loadings. Consultant on Mark II and Mark III evaluations to address the conservatism and uncertainty associated with standard structural analyses for SRV loadings. Consultant on methods of response combination and expert witness at Black Fox hearings. Consultant to Mark I and Mark III groups on conservatism, uncertainty, structural modeling, and load definition for new dynamic loads. Consultant on three Mark III BWR plants with free-standing steel containment, Leibstadt, Allens Creek, and River Bend, in

order to evaluate realistic containment response to SRV loadings as current approaches are overconservative and lead to serious design problems. Developed floor response spectra for final design of attached piping for Leibstadt plant by coupled analysis such that beneficial effects of energy feedback are included. Developed method to account for the coupling of equipment and piping to the main structure and to account for energy feedback from the subsystem to the structure. Developed method to account for random phasing of multiple harmonics of condensation oscillation loading in order to compute responses more compatible with measured results. Member ASCE committee on impact and impulse analysis of nuclear facilities, and ACI committee which developed code for the design of nuclear safety-related concrete structures subjected to impact and impulse loads.

Seismic Design - Nuclear Facilities

Consultant on seismic evaluation or design for more than 20 nuclear facilities. Consultant to the NRC and member of Senior Seismic Review Team on seismic reevaluation criteria for 9 of the oldest SEP nuclear power facilities in U.S. Consultant to NRC on seismic evaluation criteria. Directed seismic analysis of many nuclear power plant buildings and components. Directed many nonlinear seismic response analyses investigations. Evaluated effects of differential earth movement (faulting) on nuclear facility. Performed a number of dynamic soil-structure interaction analyses of nuclear reactor containment building accounting for the nonlinear effects of base slab uplift. Directed nonlinear seismic evaluation of nuclear facility to demonstrate increased seismic capacity. Developed a methodology for defining the conditional probability of failure of structures, piping, equipment, controls, and components for varying ground acceleration levels to demonstrate seismic reserve margins and provide input to a study of probability of natural hazard induced radioactive release for existing plant. Consultant on seismic probabilistic risk assessment studies conducted on 9 nuclear power plants in the U.S. Evaluated concepts for seismic response mitigation and increased energy absorption. Chairman, ASCE committee on seismic analysis of nuclear facilities. Chairman, ASCE standards committee developing an industry-wide standard on the seismic analysis of nuclear facilities.

Seismic Design - Pipeline, Tankage, and Industrial Facilities

Co-author of the seismic design criteria for the Alaska segment of the ANGTS Pipeline project (Alaskan natural gas pipeline). Consultant on the seismic design of compressor stations for the Yukon segment of ANGTS. In responsible charge of the development of all seismic design criteria for SOHIO WEST Coast/Mid-Continent Pipeline including belowground and aboveground large diameter pipe, pump stations, equipment, and tank farms plus large oil storange tanks. Developed design criteria and designs for pipeline crossings of active faults for several projects. In charge of the development of seismic design criteria for berthing structures at the Port of Long Beach. Conducted seismic analyses of trestle supported pipe-

line at the Port of Long Beach. Conducted evaluation of past earthquake records to determine basis for selecting design ground acceleration for the Northwest Alaska gas pipeline project. Consultant on seismic analysis and design to Alyeska on the Alaskan Pipeline. Responsible for seismic and structural audit of offshore berthing structures for Alaskan Pipeline. Responsible for seismic evaluation of gathering lines at Prudhoe Bay. Performed seismic analyses for several pipeline systems including aboveground, buried and seabed configurations. Conducted seismic evaluations of pipeline river crossings, both aboveground and buried. Developed design criteria and seismic design calculations for large diameter water storage tanks in the Antelope Valley. Consultant on LNG storage tanks for seismic considerations including soil-structure interaction. Responsible for major seismic audit program of plutonium storage and handling facilities throughout the United States. Responsible for developing seismic design criteria including the design basis earthquake ground motion and structural response spectra for several large industrial and test facilities. Directed evaluation of site seismicity and establishment of seismic design criteria for major industrial and nuclear facilities as well as for several large diameter gas transmission pipelines, including the influence of traveling seismic waves. Former chairman, ASCE committee on seismic design of gas and liquid fuel lifelines. Member AWAA subgroup for seismic design of water storage tanks.

Hardened Structures

Member, Defense Nuclear Agency Hardened Structures Experiments Review Panel. Consultant to DNA since 1971 on the design of structures and mechanical equipment subjected to high pressure, missile debris impact, temperature, and ground shock from underground nuclear detonations. Responsible since 1971 for the design of containment structures for DNA to contain the effects of underground nuclear explosions. Responsible for hardened structures portion of TRW preliminary design of SANGUINE system for U.S. Navy. Participated in conceptual design of hardened missile shelters. Developed the structural design basis for several COMSAT and Pacific T&T hardened communications facilities. Designed numerous shock mounting systems to protect equipment from effects of ground shock. Responsible for the design of many defense, nuclear facility, and heavy industrial facility structures to withstand the effects of explosive generated missiles and debris. Participated in design of hardened aircraft shelters to withstand effects of conventional explosions. Directed analytical and experimental program to develop methodology for predicting occurrence and magnitude of ground shock induced fault movement (block motion) in rock. Directed program to document the performance of minimally hardened tunnels during the passage of ground shock. Directed feasibility and cost study for the mining and support of large underground cavities up to 300 feet in span. Responsible for development of methods to predict peak dynamic response of structures (both with and without shock mounting) close to surface zero above underground nuclear detonations.

Special Assignments

Chairman, Seismic Analysis, Nuclear Structure and Materials Committee, Structures Division, ASCE.

Chairman, Seismic Analysis of Safety Class Structures Standard Committee, Technical Council on Codes and Standards, ASCE.

Former Chairman, Gas and Liquid Fuel Lifelines Committee, Technical Council on Lifeline Earthquake Engineering, ASCE.

Member, Nuclear Structures and Materials Technical and Administrative Committee, Structures Division, ASCE.

Member, Impact and Impulse Analysis, Nuclear Structures and Materials Committee Structures Division, ASCE.

Member, Editing Board, ASCE Report entitled "Structural Analysis and Design of Nuclear Plant Facilities."

Member, Ad Hoc Group of Soil-Structure Intersection, Nuclear Structures and Materials Committee, Structures Division, ASCE.

Member, ACI 349, "Subcommittee on Standard Requirements for Nuclear Safety-Related Concrete Structures", Design Committee and Working Group 5 - "Impactive and Impulsive Loads."

Member, AWWA D100 Revision Task Force, charged with revising the AWWA Standard for Welded Steel Tanks for Water Storage.

ATTACHMENT B

ANALYSIS STANDARD FOR ABOVEGROUND VERTICAL TANKS

R. P. Kennedy
October, 1981

SECTION 3630

ABOVEGROUND VERTICAL TANKS

3631 - SCOPE

This section deals with seismic analysis requirements and special seismic design requirements for aboveground vertical fluid containing tanks in which the upper fluid surface is essentially unconstrained (free). These requirements represent minimum requirements and are not intended to preclude the use of more sophisticated analytical precedures which account for each of the minimum requirements contained herein.

3632 - MODES OF VIBRATION

A minimum acceptable analysis must incorporate at least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration. The horizontal response analysis must include at least one impulsive mode in which the response of the tank shell and roof are coupled together with the portion of the fluid contents which moves in unison with the shell. Furthermore, at least the fundemental sloshing (convective) mode of the fluid must be included in the horizontal analysis.

3633 - HORIZONTAL IMPULSIVE MODE

3633.1 - Effective Weight of Fluid - Impulsive Mode

In the fundamental horizontal impulsive mode, the effective fluid weight (only a portion of the total weight) should be determined and used for analyses in lieu of the total fluid weight. When determining the effective fluid weight, the tank can be assumed to be rigid.

3633.2 - Spectral Acceleration - Impulsive Mode

Damping values to be used to determine the spectral acceleration in the impulsive mode shall be based upon the appropriate values for the tank shell material as specified in Section 2200.

It is necessary to estimate the fundamental frequency of vibration of the tank including the impulsive contained fluid weight. It is unacceptable to assume a rigid tank unless such an assumption can be analytically justified. The horizontal impulsive mode spectral acceleration, S_{a_I}, is then determined using this impulsive mode frequency and tank shell damping. In lieu of determining the impulsive mode fundamental frequency, it is permissible to use the maximum horizontal spectral acceleration associated with the tank support at the tank shell damping level.

3633.3 - Overturning Moment at Base of Tank - Impulsive Mode

The overturning moment at the base of the tank due to the fundamental implulsive mode must include the effects of the impulsive mode effective fluid weight and the effects of the weight of the tank shell acting in phase.

3633.4 - Hydrodynamic Pressure on Tank Shell - Impulsive Mode

The effect of tank shell flexibility must be included when determining the hydrodynamic pressure, P_1, on the tank shell for the impulsive mode.

3634 - HORIZONTAL SLOSHING (CONVECTIVE) MODE

3634.1 - Effective Weight of Fluid - Sloshing Mode

The effective fluid weight acting in the horizontal sloshing mode can be determined based upon an assumed rigid tank.

3634.2 - Spectral Acceleration - Sloshing Mode

In determining the spectral acceleration in the horizontal sloshing mode, the fluid damping ratio shall be taken as 0.5 percent of critical damping unless a higher value can be substantiated by properly documented experimental results.

The fundamental circular natural frequency in the sloshing mode can be determined based upon the assumption of a rigid tank shell. The horizontal sloshing mode spectral acceleration, S_{a2}, should be determined using the sloshing mode fundamental frequency and damping ratio.

3634.3 - Overturning Moment at Base of Tank - Sloshing Mode

The overturning moment at the base of the tank due to the fundamental sloshing mode must be determined.

3634.4 - Hydrodynamic Pressure on Tank Shell - Sloshing Mode

The hydrodynamic pressure, P_2, on the tank shell resulting from the horizontal sloshing fluid mode can be determined based upon the assumption of a rigid tank shell.

3634.5 - Fluid Slosh Height - Fundamental Sloshing Mode

The fluid slosh height, d, can be estimated based upon the assumption of a rigid tank shell.

3635 - VERTICAL RESPONSE MODE

3635.1 - Hydrodynamic Pressure on Tank Shell - Vertical Mode

The hydrodynamic pressure on the tank shell at depth y from the top of the fluid due to fluid response in the vertical mode can be obtained from;

$$P_V = (ZPA_V)\,\rho y \qquad (3635.1)$$

where ρ is the fluid mass density, and (ZPA_V) is the vertical zero period acceleration of the tank base.

3636 - DESIGN CONSIDERATIONS

3636.1 - Overturning Moment At Base of Tank

The maximum overturning moment, M_B, at the base of the tank should be obtained by the square-root-sum-of-squares (SRSS) combination of the impulsive and sloshing horizontal overturning moments. The uplift tension resulting from this base moment must be resisted either by tying the tank to the foundation with anchor bolts, etc., or by mobilizing sufficient fluid weight on a thickened base sketch plate.

The seismic induced longitudinal compressive force should be obtained by the SRSS combination of vertical and horizontal tank response modes. When combined with the dead load compressive force in the tank shell, this compressive force must be held below the applicable code allowable force levels to prevent buckling in the tank shell.

For tanks which experience uplift, the seismic induced longitudinal compressive force will be increased as a result of this uplift. In this case, an appropriate analysis accounting for the effects of uplift must be performed to determine the maximum seismic induced longitudinal compression force in the tank shell.

3636.2 - Hoop Tension in Tank Shell

The seismic induced hydrodynamic pressures on the tank shell at any level can be determined by the square-root-sum-of-squares (SRSS) combination of the impulsive (P_1), sloshing (P_2), and vertical (P_V) hydrodynamic pressures. The hydrodynamic pressure at any level must be added to the hydrostatic pressure at that level to determine the hoop tension in the tank shell. This hoop tension must be treated as a primary stress.

3636.3 - Freeboard Requirements

Either that tank top head must be located at greater than the slosh height, d, above the top of the fluid or else must be design for pressures resulting from fluid sloshing against this head.

3636.4 - Special Provisions for Full Tanks

If the top head is less than 50 percent of the slosh height above the top of the fluid, the tank must be treated as being full. For a full tank, 100 percent of the fluid weight must be incorporated into the horizontal impulsive mode in lieu of the requirements of Section 3633.1. In this case, a horizontal sloshing mode (Section 3634) does not have to be considered.

3636.5 - Attached Piping

At the point of attachment, the tank shell must be designed to withstand the seismic forces imposed by the attached piping. An appropriate analysis must be performed to verify this design.

3636.6 - Tank Foundation

The tank foundation must be designed to accommodate the seismic forces imposed by the base of the tank. These forces include the hydrodynamic fluid pressures imposed on the base of the tank as well as the tank shell longitudinal compressive and tensile forces resulting from the base moment, M_B, defined in Section 3636.1.

COMMENTARY

R. P. Kennedy
October, 1981

SECTION 3630

ABOVEGROUND VERTICAL TANKS

3631 - SCOPE

The majority of aboveground fluid containing vertical tanks do not warrant sophisticated finite element or finite difference hydrodynamic fluid-structure interaction analyses for seismic loading. However, the commonly used alternative of analyzing such tanks by the "Housner-method" contained in TID-7024 (Reference 4) may , in some cases, be significantly unconservative. The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, this assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. This assumption is unconservative for tanks mounted on the ground or low in structures when the spectral acceleration does not return to the zero period base acceleration at frequencies below about 20 Hz, or greater. More recent evaluation techniques (References 2 and 3) have shown that for typical tank designs, the modal frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically significantly greater than the zero period acceleration.

This section of the standard lays out the minimum requirements for the seismic analysis of aboveground vertical tanks. In particular, portions of the analysis where the assumption of a rigid wall tank is unacceptable are defined. For clarification, a procedure for meeting the requirements of this

standard for vertical cylindrical tanks is presented in this commentary. Similar equations are applicable to rectangular tanks.

3633 - HORIZONTAL IMPULSIVE MODE

3633.1 - Effective Weight of Fluid-Impulsive Mode

When determining the effective fluid weight for the horizontal impulsive mode, it is acceptable to assume a rigid tank shell. Thus, for a vertical cylindrical tank, the effective fluid weight, W_1, and height, X_1, from the bottom of the cylindrical shell to the centroid of this fluid weight can be obtained from the total fluid weight, W_T, tank diameter, D, and total fluid height, H, as follows (Reference 4):

<u>$D/H \geq 1.333$</u>

$$\frac{W_1}{W_T} = \frac{\tanh 0.866 \frac{D}{H}}{0.866 \frac{D}{H}} \qquad (3633.1a)$$

$$\frac{X_1}{H} = 0.375 \qquad (3633.2a)$$

<u>$D/H < 1.333$</u>

$$\frac{W_1}{W_T} = 1.0 - 0.218 \frac{D}{H} \qquad (3633.1b)$$

$$\frac{X_1}{H} = 0.500 - 0.094 \frac{D}{H} \qquad (3633.2b)$$

Alternately, the tank shell may be considered flexible and the methods in References 2, 3, or others may be used where applicable.

3633.2 - Spectral Acceleration - Impulsive Mode

The fundamental horizontal frequency of vibration of the tank shell including the impulsive contained fluid weight can be determined using the methods in References 2 or 3, or other methods which account for the flexibility of the tank shell and the effective impulsive fluid mass. The major influence

of a flexible tank shell is the lowering of the fundamental frequency so that this mode is often excited at a spectral acceleration, S_{a1}, substantially higher than the zero-period acceleration, ZPA. This influence cannot be ignored. Other than for this effect, the assumption of a rigid tank shell does not introduce major errors.

3633.3 - Overturning Moment at Base of Tank - Impulsive Mode

The overturning moment at the base of the tank due to the fundamental impulsive mode can be obtained from:

$$M_1 = \left[W_1 X_1 + W_s X_s\right] S_{a_1} \qquad (3633.3)$$

where W_s and X_s are the weight and height to the centroid of the tank shell.

3633.4 - Hydrodynamic Pressure on Tank Shell - Impulsive Mode

The hydrodynamic pressure, P_1, on the tank shell resulting from the horizontal impulsive fluid mode at depths y from the top of the fluid greater than 0.15 H can be obtained from:

$$\underline{y/H \geq 0.15} \qquad P_1 = \frac{W_1 \cdot X_1 \cdot S_{a_1}}{0.68DH^2} \qquad (3633.4)$$

with the pressure increasing linearly from the top of fluid (y=0) to the value from Equation 3633.4 at y=0.15 H. Equation 3633.4 provides an adequate description of the pressure distribution with respect to height on the tank shell for a flexible tank. Results from this equation are in reasonable agreement with results presented in References 2 and 3 for flexible tanks. The hydrodynamic pressure distribution derived for a rigid wall tank (Reference 4) is unconservative for the upper portion of the fluid, and overly conservative near the base of the tank so that it should generally not be used (Reference 2 & 3).

3634 - HORIZONTAL SLOSHING (CONVECTIVE) MODE

For the horizontal sloshing mode, the assumption of a rigid shell wall does not introduce unacceptable error. Thus, the rigid tank procedure presented in Reference 4 is acceptable and is reproduced herein for vertical cylindrical tanks.

3634.1 - Effective Weight of Fluid - Sloshing Mode

In the fundamental horizontal convective mode for a vertical cylindrical tank, the effective fluid weight, W_2, and height, X_2, from the bottom of the cylindrical shell to the centroid of the sloshing weight can be obtained from:

$$\frac{W_2}{W_T} = 0.230 \frac{D}{H} \tanh\left(\frac{3.67}{D/H}\right) \qquad (3634.1)$$

$$\frac{X_2}{H} = 1.0 - \frac{\cosh\left(\frac{3.67}{D/H}\right) - 1.0}{\frac{3.67}{D/H}\sinh\left(\frac{3.67}{D/H}\right)} \qquad (3634.2)$$

3634.2 - Spectral Acceleration - Sloshing Mode

The fundamental circular natural frequency, ω_2, in the sloshing mode can be determined from (Reference 4):

$$\omega_2^2 = \frac{3.67g}{D} \tanh\left(\frac{3.67H}{D}\right) \qquad (3634.3)$$

where g is gravity acceleration (32.17 feet/second^2).

3634.3 - Overturning Moment at Base of Tank - Sloshing Mode

The overturning moment at the base of the tank due to the fundamental sloshing mode can be obtained from:

$$M_2 = W_2 X_2 S_{a_2} \tag{3634.4}$$

3634.4 - Hydrodynamic Pressure on Tank Shell - Sloshing Mode

The hydrodynamic pressure, P_2, on the tank shell resulting from the horizontal sloshing fluid mode at depth y from the top of the fluid can be obtained from:

$$P_2 = \frac{0.533\ W_T\ S_{a_2}}{DH} \quad \frac{\cosh\left(3.68\ \frac{H-y}{D}\right)}{\cosh\left(3.68\ \frac{H}{D}\right)} \tag{3634.5}$$

3634.5 - Fluid Slosh Height - Fundamental Sloshing Mode

The fluid slosh height, d, can be estimated from:

$$d = 0.42D\left(\frac{S_{a_2}}{g}\right) \tag{3634.6}$$

where g is gravity acceleration (32.17 feet/second^2).

3635 - VERTICAL RESPONSE MODE

3635.1 - Hydrodynamic Pressure on Tank Shell - Vertical Mode

The vertical response mode of the fluid is sufficiently high that the fluid may be treated as a rigid body when computing its vertical response. Equation 3635.1 is based upon this assumption.

3636 - DESIGN CONSIDERATION

3636.1 - Overturning Moment at Base of Tank

When sufficiently anchored to prevent uplift, the seismic induced longitudinal compressive force per unit length, C, in the tank shell is given by:

$$C = \sqrt{(F_V)^2 + \left(\frac{1.273\ M_B}{D^2}\right)^2} \qquad (3636.1)$$

where F_V represents the maximum vertical response of the empty tank shell.

For tanks which experience uplift, the seismic induced longitudinal compressive force may be computed by the procedure defined in Reference 1, or by other procedures which account for the influence of uplift.

SECTION 3630

REFERENCES

1) Wozniak, R.S., and Mitchell, W.W., "Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks", Presented at Session on Advances in Storage Tank Design, API Refining 43rd Midyear Meeting, Toronto, Canada, May, 1978.

2) Veletsos, A.S., and Yang, J.Y., "Dynamics of Fixed-Base Liquid-Storage Tanks", Presented at U.S.-Japan Seminar for Earthquake Engineering Research with Emphasis on Lifeline Systems, Tokyo, Japan, November, 1976.

3) Veletsos, A.S., "Seismic Effects in Flexible Liquid Storage Tanks", Proceedings of Fifth World Conference on Earthquake Engineering, Rome, 1974.

4) "Nuclear Reactors and Earthquakes", TID-7024, Prepared by Lockheed Aircraft Corporation and Holmes & Narver, Inc., for the Division of Reactor Development, U.S. Atomic Energy Commission, Washington, D.C., August, 1963.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

CERTIFICATE OF SERVICE

I, Philip P. Steptoe, hereby certify that copies of the Testimony of Robert P. Kennedy, with attachments, were served upon all parties shown in the attached service list by hand-delivery on December 1, 1981, except those indicated by an asterisk which were served by deposit in the United States mail, first class postage prepaid, on the 30th day of November, 1981.

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