CONTAINMENT ANNULUS CONCRETE
DESIGN, CONSTRUCTION
and
TESTING
for the
PERRY NUCLEAR POWER PLANT
North Perry, Ohio
Rev. 2

The Cleveland Electric Illuminating Company
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CONTAINMENT ANNULUS CONCRETE DESIGN, CONSTRUCTION AND TESTING

1:00 INTRODUCTION

The Perry Nuclear Power Plant (PNPP) is located in North Perry, Ohio, 35 miles northeast of Cleveland, on the south shore of Lake Erie. The plant consists of two identical units, each powered by a Boiling Water Reactor (BWR), nominally rated at 1200 Megawatts, electrical output.

Each of the reactors is housed in a separate Reactor Building and contained by a steel Containment Vessel. The containment vessels are free-standing right cylindrical steel shells with ellipsoidal steel domes, designed and fabricated by Newport News Industrial Corporation of Ohio. The cylindrical steel shell and steel dome comprise the pressure boundary for the sides and top, and were designed and built in accordance with Section, III, Division 1 of the ASME Code[1]; but, the bottom of the pressure boundary is formed by a reinforced concrete basemat. For this reason, the steel portion of the containment was not "N" stamped, even though it was built in accordance with the rules of ASME.

Originally, there was a five (5) foot wide annulus between the Containment Vessel and the Shield Building for the entire height. (See Figure 1.1). With the inclusion of safety relief valve (SRV) vibrations for the BWR Mark III, it was necessary to fill this annulus with concrete for a height of 23'-6" above the top of the basemat in order to dampen vibrations in the Containment Vessel due to the SRV actuations. Safety relief valve discharge response spectra are presented in Appendix A to this report for three locations on the containment vessel. Two sets of response spectra are provided for each location. The response spectra are shown for the containment vessel with and without the annulus concrete in order to provide an indication of the changes in response which are caused by the annulus concrete. Since the annulus concrete was only required to provide stiffness to the Containment Vessel and was initially not required for strength, the design philosophy was to design the annulus concrete to ACI 318-71[2]. This was the same design criteria used for the concrete Shield Building. However, since the original design, several conditions have developed as a result of increased loads, the methods of applying load calculations and construction problems. These conditions have dictated that the annulus concrete be used for strength and that ASME Code Case N-258 "Design of Interaction Zones for Concrete Containments Section III, Division 2"[3] be followed.

Accordingly, the annulus concrete has been evaluated against the ASME Code, Section III, Division 2, Subsection CC, 1980 edition with the Summer 1981 Addenda[4]. The design meets all Code provisions as interpreted by ASME Code Case N-258[3] which states that the steel containment vessel shall be designed to Section III, Division 1 and the annulus concrete shall be designed to Section III, Division 2. The annulus concrete also complies with NUREG-0800, SRP 3.8.1 Concrete Containment[6] with one exception. The exception pertains to the allowable tangential shear stress to be resisted by the concrete (v_c) which is limited to 40 psi and 60 psi, depending on the load category. in SRP 3.8.1. These allowable values for v_c are more stringent than the values in the ASME Code. Sections 3:05 through 3:08 herein provide the justification for using the higher values for the Perry concrete. Consideration is given to recent research results, strain limits for reinforcement and concrete, and the tangential shear transfer at the basemat. It is concluded that the present reinforced concrete design for the annulus concrete has sufficient strength and stiffness to resist the design tangential shear forces and that the acceptance criteria for concrete and reinforcement strains are met.

The following discussion is divided into four sections:

Modelling considerations

Design

Materials, Testing and Construction Considerations

Conclusion

2:00 MODELLING CONSIDERATIONS

2:01 INTRODUCTION

One of the first steps in the design process is to define the model to be used for analysis. The model, to be complete, must include the Containment Vessel, Shield Building, basemat foundation, as well as the annulus concrete being designed. Because the annulus concrete is to be placed after all surrounding structures are complete, some unique modelling problems concerning the interface between these structures and this new concrete are introduced.

The manner in which each of these interfaces was considered is discussed below.

The annulus concrete was analyzed using two computer programs - ASHSD2 and ANSYS. The ASHSD2 program was used to analyze the Containment Vessel, annulus concrete, and Shield Building for static loads, suppression pool dynamic loads and seismic loads. The finite element model used for these analyses is shown in Figure 2.1. Because the ASHSD2 program does not have thermal load capability, a second finite element model was required to analyze the response to thermal loads. The ANSYS thermal analysis model is shown in Figure 2.2

2:02 CONTAINMENT VESSEL - ANNULUS CONCRETE INTERFACE

The interface between the Containment Vessel and the annulus concrete is represented in the ASHSD2 finite element model with common nodes for the axisymmetric solid elements and the axisymmetric shell elements. This representation is selected for the mechanical loads because these loads do not produce a tendency for significant slip at the interface, compared to the thermal loads discussed below. Some of these loads also are non-axisymmetric or dynamic and ASHSD2 does allow these types of loads.

Because ASHSD2 did not have thermal load capability, an ANSYS model was developed for the thermal loads.

The interface between the Containment Vessel and the annulus concrete is represented in the ANSYS finite element model by modelling the vessel and adjacent annulus concrete with separate nodes which are connected by "gap" elements. The vessel is anchored in the annulus concrete at the embedded circumferential stiffeners. The gap elements are used because under the accident temperature condition, the vessel experiences a temperature increase while the concrete through most of its thickness does not. This discontinuous temperature distribution creates thermal forces and moments in the vessel and in the annulus concrete which depend on the degree of bond at the interface between the two structures. The Containment Vessel and annulus concrete are analyzed for this condition by using a feature of ANSYS which considers the vertical shear stress between the vessel and between the annulus concrete to be a function of the normal stress between the two structures at the interface (Gap Element). If the vertical shear stress is less than or equal to a constant multiplied by the normal stress, no slip occurs between the two structures. If the vertical shear stress is greater than a constant multiplied by the normal stress, the surfaces can slip and a sustained value of shear stress equal to the constant times the normal stress is developed. This constant is similar to the static coefficient of friction between concrete and steel. Two different values of the constant, 0.7 and 0.0, were used for the design. A parametric study indicated that for values of the constant as large as 2.0 the forces and moments in the annulus concrete did not change significantly from those corresponding to a 0.7 value for the constant. This approach conservatively bounds the actual degree of bond at the interface since a bond breaker is applied to the Containment Vessel on the vertical surface to be covered by concrete. Above the fourth ring stiffener and below the first, 3 inches of compressible material is placed between the concrete and vessel to reduce thermal compressive stresses. The compressible material was included in both computer models. The analysis using each value of this constant produced different critical stress values; thus creating an envelope of maximum values for design.

As discussed above the design uses ANSYS model results with the non-linear "gap" element for the thermal loads and combines them with the linear ASHSD2 model results for the mechanical loads. To determine the acceptability of this approach, a study was made to evaluate the effect of combining the results from the two different finite element models used in the design. A finite element analysis was performed using the ANSYS model with gap elements and the dominant loads from the controlling load combination: pressure, seismic, and thermal. Since the model is limited to axisymmetric loads, an equivalent seismic load was used for this analysis. The results from the above approach were compared to a second approach which combine results from two ANSYS models. The first model did not include the gap elements and analyzed the pressure and equivalent seismic loads. The results from this model were combined with the thermal results from a second model with gap elements. This is the same approach used for the annulus concrete design.

Comparing the two approaches, reinforcing steel stresses at each section were calculated from element stresses generated by each approach. The maximum or design reinforcing steel stresses from each approach are within 11%. Observation of Table 3.1 indicates that these small differences will not effect the final design.

2:03 BASEMAT FOUNDATION - ANNULUS CONCRETE INTERFACE

The basemat had been placed without considering the annulus filled with concrete; therefore, there is no mechanical connection (dowels) between the basemat and the annulus concrete. The original ASHSD2 analysis for mechanical loads conservatively modelled this condition with the base of the annulus concrete being independent of the basemat with no restraint against either upward or downward vertical movement. However, the Shield Building and vessel were fixed at the basemat. This model required the vessel and Shield Building to carry all the transverse shear forces. The results of this analysis indicated that the Shield Building was overstressed. The next logical step was to more realistically model this interface area; therefore, the basemat stiffness was added to the model removing the fixed conditions of the vessel and Shield Building. The results of this analysis indicated that the Shield Building was marginally within allowables for the shear forces. Although the shear stresses were within allowables, the decision was made to mechanically protect the Shield Building. To achieve this, the basemat was prepared for the new concrete by cutting a shear key to resist some of the radial shear being transferred through the annulus concrete.

The analysis for the thermal loads with ANSYS incorporated a "gap" element to create the effect of a compression with no tension capability boundary between the basemat and annulus concrete. The "gap" element accurately models the actual interface.

2:04 SHIELD BUILDING - ANNULUS CONCRETE INTERFACE

The Shield Building - annulus concrete interface was modelled as a monolithic section, in other words, no slip is assumed to occur along the interface. To evaluate this assumption, the interface shear and normal stresses were reviewed for the critical load combinations. The variation of these stresses along the height of the annulus concrete is shown in Figure 2.3 for the abnormal/extreme environmental condition, which is controlling. From this figure, it is seen that for the region starting above section 1 and extending above section 7, a distance of approximately 12 feet, the normal stresses are entirely compressive. Over this region the maximum vertical shear stress is 108 psi with the average stress of 55 psi. For the region starting just above section 7 extending through 9 (4 feet), the normal stresses are tensile with a peak value of 60 psi accompanied by small values of shear stress (25 psi maximum). Above section 9, (5 feet) the shear stresses increase to a maximum of 227 psi, but these are accompanied by normal stresses at the interface which are compressive. In the lower portion, below section 2 (2.5 feet), the shear stresses increase to a maximum of 212 psi in conjunction with a tensile normal stress of 60 psi. The likelyhood that these stresses would cause debonding at the annulus concrete - Shield Building interface is discussed below.

The Corps of Engineers' report "Investigation of Methods of Preparing Horizontal Construction Joints In Concrete"[5] presents the results of an experimental research program on construction joints. This report presents individual test results of tension and shear capacity across a construction joint that is rough, clean and dry. The age of the specimens at the time of testing was 17 days, at which time the concrete had achieved a compressive strength of approximately 1300 psi. The specimens contained 1-1/2 inch crushed limestone coarse aggregate. The tension values from nine tests ranged from 130 psi to 80 psi with an average of 105 psi. The shear values ranged from 150 psi to 240 psi with an average of 195 psi. The minimum test values were used to establish a reduced Mohr's failure envelope for the interface, and the combined shear and normal stresses from the curves in Figure 2.3 were evaluated with respect to this criteria. From this evaluation it is expected that debonding of the interface will not occur, except perhaps in a local region at the base of the annulus. However, the slip in this area is expected to remain small due to restraint provided by the bonded joint above and the basemat below.

The Corps of Engineers' report(5) also gives conclusions which are useful in defining the surface preparation of the Shield Building for the placement of the annulus concrete. The report concludes that the surface should be rough, clean and dry for best results. To obtain these conditions the Shield Building surface in the annulus was bush hammered to produce a roughened surface with a 1/4" amplitude which will be air cleaned before placement of the annulus concrete.

For composite flexural members, ACI 318-71(2) contains design requirements for shear transfer across the interface of the components which comprise the member. Generally, these provisions permit a shear stress as large as 80 psi to be transferred across the interface without ties, if the interface is intentionally roughened and clean. An exception to this allowable is if tension normal to the interface exists. In this case ties are required to provide a normal clamping stress necessary to develop the shear stress. The interface between the annulus concrete and the Shield Building differs from the interface in a composite flexural member in several respects.

First, for a composite flexural member, if the calculated interface shear stresses exceed the shear strength of the joint, debonding occurs. Slip at the interface occurs and without ties, no clamping mechanism exists to limit the slip or to develop any significant portion of the calculated shear stress at the interface. Consequently, composite action between the components is lost across the entire width of the member and along its length where this condition exists. However, this condition would not occur at the untied interface of the annulus concrete and the

Shield Building. The annulus concrete and Shield Building can be visualized as an inner cylinder contained within an outer cylinder. If debonding of the interface occurs, vertical slippage at the roughened interface between the two cylinders will develop a compressive clamping stress at the interface due to the axisymmetric geometry of the cylinders. This condition will limit slip and transfer shear without ties across the interface.

Another difference between the composite flexural member and the annulus concrete is the variation of the calculated shear stress at the interface. The annulus concrete interface normal and shear stresses plotted in Figure 2.3 are peak values. These values may occur at one location around the circumference, and they decrease away from this location. This differs from a flexural member in that the maximum calculated stresses are uniform across the entire width of the member, and if these stresses exceed the joint capacity composite action for the entire cross section is lost.

Based on the above discussion it is concluded that significant slip at the annulus concrete - Shield Building interface is not expected to occur. Therefore, the assumption in the analysis model that the annulus concrete and Shield Building act as monolithic concrete is reasonable.

The preceding discussion provides the basis for the assumption in the finite element model that the Shield Building and annulus concrete act monolithically. However, an analysis was performed to demonstrate that the stresses in the Containment Vessel are not significantly influenced by this assumption. For the purpose of the analysis, the vessel stresses produced by the long term LOCA load combination were compared for the case of including the 3 ft. Shield Building as a monolithic part of the 5 ft annulus concrete and for the case where the Shield Building is removed from the model.

For the long term LOCA load combination the largest stresses are caused by the accident pressure and temperature loads. By performing a plane stress analysis for these loads, the vessel stresses were obtained. The design pressure of 15 psig was used with a temperature of 115 ^{o}F applied to the vessel. The value of 115 ^{o}F corresponds to the vessel experiencing a temperature increase from its 70 ^{o}F stress free value to the maximum design LOCA temperature of 185 ^{o}F. For these combined loads, the net vessel stress in the hoop direction is compressive and was calculated as 17400 psi for the 8 ft monolithic model and 15700 psi for the model consisting only of the vessel and the annulus concrete. This represents a 10% reduction in vessel compressive stress, which is not significant. However, as seen from the above results, use of the monolothic model actually gives a greater calculated hoop stress in the vessel.

3:00 DESIGN

3:01 LOAD COMBINATIONS

The loading conditions used for the annulus concrete design were the containment loading combinations presented in the FSAR including Appendix 3A and 3B. However, the design has been evaluated using the load combinations specified in Table CC 3230-1 of the ASME Code[4] and the Appendix to NUREG-0800[6].

3:02 VERTICAL REINFORCEMENT

The vertical reinforcement was designed to carry the vertical forces and moments along with the tangential shear forces as defined by ASME Section III, Division 2, Subsection CC 3521.1.1 c. The final design is #18 Grade 60 reinforcing bars on 15 inch centers on both faces. To insure that the vessel and the annulus concrete act together and to spread the reinforcment, the vertical reinforcment next to the vessel is to be placed through holes in the horizontal stiffeners. Figure 3.1 is a copy of a reduced construction drawing of the general steel layout.

Table 3.1 gives steel stress values for each section of the annulus concrete for the critical load combination. The table shows that the stresses in the vertical reinforcement range from small compression to 35.5 ksi in tension. These stress values do not include the tangential shear stress that is transferred to the orthogonal reinforcement. This is discussed later in Section 3:05.

3:03 HORIZONTAL REINFORCEMENT

The horizontal reinforcement was designed to carry the hoop forces and moments and the tangential shear force as defined in

ASME Code, Section III, Division 2, Subsection CC 3521.1.1 c. The final design is #18 Grade 60 reinforcing bars spaced from 6 to 12 inches on centers on both faces. See Figure 3.1.

Table 3.1 shows that the horizontal reinforcement stresses range from small compression to 50.8 ksi tension. Again the tangential shear stress has not been added.

3:04 TRANSVERSE (RADIAL) SHEAR REINFORCEMENT

The horizontal ties (shear reinforcement) were designed to carry the transverse shear force in excess of what the concrete can carry. Although the original design was to ACI-318, it meets the criteria of the ASME Code, Section III, Division 2, Subsection CC 3421.4.1. The ties are #7 bars spaced circumferentially at each vertical bar below the horizontal stiffener #1 and above horizontal stiffener #4 and every other bar in the middle sections. Additional ties were added in regions of attachment plate stiffeners. The vertical distribution of shear ties is as follows:

Below horizontal stiffener #1 -	4 tie elevations
Between horizontal stiffeners #1 & #2 -	4 tie elevations
Between horizontal stiffeners #2 & #3 -	4 tie elevations
Between horizontal stiffeners #3 & #4 -	3 tie elevations
Above horizontal stiffener #4 -	3 tie elevations

3:05 TANGENTIAL SHEAR REINFORCEMENT

3:05.1 Code and SRP Requirements

Using the shear friction provisions of ACI 318-71, the original design included tangential shear in determining the reinforcement requirements in the vertical and horizontal directions, and inclined reinforcement was not provided. However, based on

SRP 3.8.1, inclined reinforcement is required if the tangential shear stress is greater than 40 psi for abnormal/severe environmental loads and 60 psi for abnormal/extreme environmental loads. These limits are very conservative when compared with the ASME Code.

For the minimum reinforcement provided in the annulus concrete, CC3421.5.1(a) of the ASME Code allows 107 psi before inclined reinforcement would be required. However, the maximum calculated tangential shear stress is 83 psi, which occurs for the abnormal/extreme environmental condition; therefore, inclined reinforcement is not required by the Code. The SRP 3.8.1 requirements would result in inclined reinforcement consisting of #5 bars at a 12 inch center to center spacing. This amount of reinforcement seems rather inconsequential relative to the #18 bars provided in the vertical and horizontal directions. This conclusion is confirmed by the results of the analysis described in Section 3:05.3. Here it is shown that the stresses in the orthogonal reinforcement and the strains in the concrete are not significantly reduced by the addition of the #5 inclined bars.

The design of the annulus concrete for tangential shear was based on the shear allowable of the ASME Code rather than the reduced allowables presented in SRP 3.8.1 for two reasons. First, the magnitude of the tangential shear stresses are not as severe as those for a typical concrete containment subjected to the same seismic input. More importantly, the results of recent research indicates that the tangential shear allowables of the ASME Code are conservatively low considering the magnitude of the stresses in the orthogonal reinforcement in the annulus concrete, as discussed below.

3:05.2 Tangential Shear Research

Tests on reinforced concrete specimens containing orthogonal reinforcement and subjected to simultaneous loads creating biaxial tension and tangential shear stresses have been performed at the Construction Technology Laboratories of the Portland Cement Association (PCA) and at Cornell University. The PCA tests were conducted on two (2) feet thick specimens containing #14 and #18 reinforcement. The Cornell test specimens were smaller than those tested by PCA. The results of the PCA tests are reported in Reference 7. The Cornell test results are presented in Reference 8 and summarized in a recent paper(9). This paper compares the Cornell and PCA results with others performed in Toronto and Japan. Table 3.2 presents a comparison of the calculated tangential shear stresses occurring in the annulus concrete with tangential shear strengths based on the conclusions from the Cornell and PCA tests.

In Reference 9, the following expression is proposed as a conservative estimate of the allowable tangential shear stress in orthogonally reinforced concrete:

$$v_c = \sqrt{f'_c}\,(2.7 + 0.006\,\rho f_y\,(1-f_s/f_y)) \qquad (1)$$

where v_c = allowable tangential shear strength (psi)

f'_c = compressive strength of concrete (psi)

ρ = minimum steel ratio of the two orthogonal reinforcements.

f_y = reinforcement yield stress (psi)

f_s = reinforcement stress due to the biaxial forces (psi)

This equation was developed from equal biaxial tension tests. Equation (1) was conservatively applied to the annulus concrete using the stresses and reinforcing ratios presented in Table 3.1. The largest reinforcement stress was taken to exist on both faces and used as f_s in Equation (1). This resulted in the tangential shear strength values shown in columns 3 and 4 of Table 3.2. The tangential shear strength of the section is the minimum of these two values and is shown in column 5. By comparing this with the calculated tangential shear stress appearing in column 2, it is seen that the shear strengths are in excess of the calculated shear stresses by the factors shown in column 9. At the critical section 2, the strength exceeds the calculated shear stress by 172%.

Reference 7 (the PCA tests) concludes that the following expression provides a lower bound estimate of the shear strength of orthogonally reinforced concrete subjected to cyclic loads:

$$v_{so} = 0.90\ \rho f_y\ (1-f_s/f_y) \qquad (2)$$

where v_{so} = lower bound tangential shear strength (psi)

ρ = minimum steel ratio of the two orthogonal reinforcements

f_y = reinforcement yeild stress (psi)

f_s = reinforcement stress due to the biaxial forces (psi)

To limit shear distortions and strains in the reinforcement, a factor of 0.6 is recommended in place of the 0.9 appearing in equation (2).

The report also establishes an upper limit on shear stress resisted by orthogonal reinforcement as:

$$v_{so} = \sqrt{f'_c}\,(7.5 - f_s/14300) \tag{3}$$

where v_{so} = upper limit tangential shear strength (psi)

f'_c = compressive strength of concrete (psi)

f_s = reinforcement stress due to the biaxial forces (psi)

The shear strength for each section of the annulus concrete was calculated using the above expressions. These are presented in columns 6, 7 and 8 of Table 3.2. Column 6 represents the minimum directional shear strength determined by Equation (2). Column 8 presents the shear strength corresponding to limiting shear distortion. Column 7 is the upper bound on shear strength determined by Equation (3). The controlling limit on tangential shear stress is considered to be the distortion limit shown in Column 8. When these values are compared with the calculated shear stress values shown in Column 2, it is seen that, as a minimum, the shear strength exceeds the calculated shear stress by 63%.

The results of these tests reported in References 7 and 9 are considered to be applicable to the evaluation of the ability of the annulus concrete to resist the calculated tangential shear stresses without inclined reinforcement. From these test results it is concluded that sufficient shear strength exists and the shear distortions will be small using only orthogonal reinforcement in the annulus concrete. The conclusion that the shear distortions will remain small was confirmed by applying Duchon's[10] analytical model to the stress conditions shown in

Table 3.1. This is discussed in Section 3:05.3 below. The Duchon model was selected because the research (7) has concluded it to be a reasonable approximation of the shear distortions experienced by completely cracked elements even for a large number of stress reversals.

3:05.3 Duchon Model

To confirm for the current design that the shear distortions remain small without inclined reinforcement, Duchon's (10) analytical model was applied to the stress conditions of the annulus concrete. The input to Duchon model includes the following:

Forces - Vertical
Horizontal
Shear
Concrete Area
Steel Modulus
Concrete Modulus
Reinforcing Ratio - Vertical
Horizontal
Inclined
Angle of Inclined Steel

The vertical and horizontal forces were input as the maximum of the inside or outside face reinforcing bar stress values at the section from Table 3.1, multiplied by the appropriate reinforcement area. At each section, the shear force was input as the product of the tangential shear stress from column (2) of Table 3.2, times the concrete section area.

The Duchon model was also used to evaluate the effect of the addition of the #5 inclined bars which would result from the requirements in SRP 3.8.1. The results from these analyses on the factored load case are shown in Table 3.3. Columns (2), (4), and (7) are the results for the current design with no inclined reinforcement. Columns (3), (5), (6), and (8) are the results with #5 bars at a spacing of 12 inches and inclined 45° in both directions. Adding the inclined reinforcement reduces the vertical and horizontal reinforcement stresses by an averge of 7%. This reduction is not large enough to justify the addition of inclined reinforcement considering that the orthogonal reinforcement in the current design is not overstressed. For the #5 inclined bars in the model, some reach yield locally as shown in column (6) of Table 3.3. This means that the stress carried by the inclined reinforcement would not be as great as that indicated in Table 3.3 for sections where the inclined reinforcement yields. To be theoretically correct, the Duchon model would have to be revised to set all inclined reinforcement stress levels above yield (60 ksi) to 60 ksi, and then re-evaluate the equilibrium equations. This correction was not considered important and was not made for these analyses.

The lower allowable concrete shear stresses in SRP 3.8.1 produces a requirement for inclined reinforcement. This reinforcement is intended to control shear distortions, which in turn limits the strains in the reinforcement and containment liner. It is believed that this intent of the SRP is met by the current design. The distortional shear strains predicted by the Duchon model are shown in columns (7) and (8) of Table 3.3. The PCA test results from Reference 7 indicate that the Duchon model gives a reasonable approximation of the shear distortions experienced by completely cracked elements even for a large number of stress reversals. Column (7) shows that the distortional shear strain values range from 0.00147 rad to 0.00331 rad, with an average of 0.00217 rad for the current design. These values are small, and the 0.00217 rad average value is less than one-half of the ultimate values of shear distortion measured in the PCA tests in Reference 7. Comparing these results with those in column (8), it is seen that the effect of the #5 inclined reinforcement is to reduce the distortional shear strains by approximately 8%. This reduction is not significant considering that the distortional shear strains in the current design are not large. The addition of the inclined steel would only slightly improve the distortional shear strains, but not enough to offset the problems associated with placing the inclined reinforcement.

3:05.4 Conclusion on Tangential Shear

As discussed above, the current annulus concrete design for tangential shear meets all of the requirements of ACI 318-71 and ASME Section III, Division 2. The design does not meet the reduced allowable shear provisions of SRP 3.8.1. However, it has been shown that the current annulus concrete design meets the intent of the SRP to require a design with adequate shear strength and limited shear strains. This was demonstrated from an evaluation of the design using tangential shear test results obtained by PCA (7) and Cornell (9), and by applying the Duchon analytical model (10).

3:06 REINFORCING STEEL STRAIN LIMITS

The ASME Code Section III, Division 2, Subsection CC 3410 generally limits reinforcement strains to the elastic range for factored loads, allowing the strains to go to twice yield only in specified cases. This constraint is more severe than ACI 318 which generally allows the steel to yield under factored loads. Even though the annulus concrete was originally designed to ACI-318, a check of the critical loads indicates that the strain limits of CC 3422 are not violated. Interaction diagrams were developed using the ASME strain limits. Service and factored load combinations were plotted for each section on the interaction diagrams. Figures 3.2 to 3.7 are interaction diagrams with only the critical sections plotted. They show that all strains are within ASME allowables.

3:07 CONCRETE STRAIN LIMITS

Table CC-3421-1 and CC-3431-1 define the concrete stress limits for the ASME Code for Section III, Division 2. The stresses in the annulus concrete are small and fall below the allowables presented. Figures 3.2 through 3.7 also show the concrete stresses to be less than ASME Code allowables.

3:08 TANGENTIAL SHEAR TRANSFER AT BASEMAT

The annulus concrete is not mechanically connected to the basemat; therefore all the tangential shear force must be transferred to either the Containment Vessel or the Shield Building. This transfer of force has been evaluated with respect to the particular code governing the design of each building. This evaluation establishes the adequacy of the Containment Vessel, Annulus Concrete, and Shield Building to carry the tangential shear and ultimately to transfer this tangential shear to the foundation.

The models used for the annulus concrete analysis and design contain the Shield Building; therefore, the ANSYS analysis can be used to evaluate the tangential shear transfer to the basemat. The Containment Vessel model does not contain the annulus concrete; therefore, a special analysis was performed to evaluate the tangential shear transfer to the basemat. The Shield Building must carry 42.5 kips per foot (100 psi) of tangential shear during the critical load combination for the annulus concrete. When this tangential shear is combined with the vertical and horizontal reinforcing stresses for this critical load combination, there is a 16% safety margin over the ACI-318 allowable for the Shield Building. The effect of adding the annulus concrete has a negligible effect on the tangential shear design values. Because the critical load combination is different for the Shield Building and the annulus concrete due to thermal effects, a confirmation analysis was made which indicated that the Shield Building could carry all postulated load combinations within normal safety margins.

The Containment Vessel is required to carry an additional 1.68 kips per inch or 745 psi of tangential shear. The vessel designer (Newport News Industrial Corporation) supplied the basic vessel stresses which were increased by the 745 psi and evaluated. The vessel still meets all ASME Code, Section III, Division I design requirements for stress intensity levels. With allowable at 3Sm = 57.9 ksi the controlling load combination produces only an intensity of 25 ksi.

4:00 MATERIAL, TESTING AND CONSTRUCTION CONSIDERATIONS

4:01 REINFORCING STEEL

Purchasing, placing, and the mechanical (Cadweld) splicing of reinforcing steel bars in the annulus area was performed utilizing the Safety-Related PNPP specifications for concrete and reinforcing steel, without consideration of the ASME Code, Section III, Division 2 rules. However, to demonstrate the extent to which the ASME Code, Section III, Division 2, technical requirements were met, a third party, an Authorized Nuclear Inspector (ANI), was brought on-site by the Constructor. The ANI has reviewed all material certification and construction procedures to verify PNPP Specification compliance. Table 4.1, "Reinforcing Steel and Splicing Code Comparison", is presented to indicate the detail to which the ANI reviewed this material and to establish Code compliance. All concerns of the ANI have been addressed and resolved, such that a letter has been issued stating Specification compliance. It has been further demonstrated that the requirements of ASME Section III, Division 2, NCA-3461, which requires the Constructor to survey, qualify and audit certain suppliers, has been met with respect to the Code's intent, as related to reinforcing steel and Cadweld splices. This was accomplished by producing combined Owner and Contractor records showing inspections and audits of these suppliers. This approach is used because the cost to remove and replace reinforcing steel according to the ASME Code has been estimated to be $20 million.

4:02 CONCRETE SUPPLY AND PLACEMENT

Specification SP-14, "Supply of Concrete", which is the construction specification for all the nuclear safety related concrete for the PNPP has been revised to meet all ASME Code Section III, Division 2 requirements as provided in Tables 4.2 and 4.3. Table 4.2 "Concrete Code Comparison" is a compilation, section by section, of the comparisons between the Code rules and the revised SP-14 rules. In addition, concrete testing requirements are compared in Table 4.3. Additional review of Code sections including quality assurance, personnel qualifications, vendor surveillance, and an independent review by a third party, ANI, have further established CEI's ability to meet the intent of Code mandated practices in these areas. For these reasons CEI's Site Organization will continue to operate the concrete batch plant; thereby, taking advantage of over seven (7) years of experience in supplying nuclear safety related concrete. This is in contrast with the ASME Code which states that the Constructor shall control the batch plant. No improvement in quality can be achieved by following this requirement; in fact, some reduction in quality could occur if the Constructor were required to control or supply a batch plant for the small quantities of concrete required for the annulus.

Upon discharge from the transit mix truck, the plastic concrete will be conveyed, placed, consolidated, cured, and tested in full compliance with ASME, Section III, Division 2 as required by the certified construction specification SP-801. SP-801 was specifically prepared for the annulus concrete placement.

4:03 TESTING

The Perry containment is scheduled to undergo a Structural Integrity Test (SIT) in accordance with the rules of ASME Section III, Division 1, Subsection NE-6000. There are currently no rules in the ASME Code for the structural testing of the annulus concrete portion of the containment shell. However, rules for such a test have been proposed as a revision to the ASME Code Case N-258, and the Perry Containment SIT will comply with these proposed rules in addition to those of NE-6000. The proposed provisions require that displacement measurements and concrete crack inspections be performed to a limited extent. The displacement requirements call for radial displacements to be measured on the vessel near the top of the annulus concrete at four azimuths. The crack inspections are to be performed on a 40 square ft. area of the annulus concrete. The acceptance criteria are to be in accordance with ASME Section III, Division 2, Subsection CC-6000. Also, strain measurements are required in the region of the annulus concrete near the base slab and in the vicinity of the largest penetration in the annulus concrete.

5:00 CONCLUSION

The concrete and reinforcing steel individually and collectively as a unit meet fully the ASME Code, Section III, Division 2[4], except purchasing, placing and the mechanical (Cadweld) splicing of reinforcing steel bars and the concrete supply. As indicated in Sections 4:01 and 4:02 the full intent of the Code has been followed with respect to these areas. The design approach presented here is the best possible considering the specifics of the Perry Containment Vessel, Shield Building and annulus concrete. The final design developed from this approach is capable of safely carrying all postulated loads and load combinations.

6:00 REFERENCES

1. ASME Boiler and Pressure Vessel Code, 1974 Edition with Summer 1974 Addenda.

2. ACI 318-71 Bulding Requirements For Reinforced Concrete.

3. ASME Code Case N-258 - "Design of Interaction Zones for Concrete Containments Section III, Division 2" March, 1980.

4. ASME Boiler and Pressure Vessel Code, 1980 Edition with Summer 1981 Addenda.

5. U.S. Army Engineer Waterways Experiment Station - "Investigation of Methods of Preparing Horizontal Construction Joints for Concrete" Tech. Report No. 6-518 July 1959 - Corps of Engineers.

6. NUREG-0800 - SRP 3.8.1 "Concrete Containment" Rev 1, July 1981.

7. Oesterle, R.G. and Russell, H.G. - "Shear Transfer in Large Scale Reinforced Concrete Containment Elements." Construction Technology Laboratories, Portland Cement Association - NUREG/CR-2450, Dec 1981.

8. Perdikanis, P.C.; White, R.N.; Gergely, P. - "Strength and Stiffness of Tensional Reinforced Concrete Panels Subjected to Membrane Shear, Two-Way Reinforcing" - Department of Structural Engineering, Cornell University - NUREG/CR-1602 July 1980.

9. Cowley, White, Hilmy and Gergely - "Design Considerations for Concrete Nuclear Containment Structures Subjected to Simultaneous Pressure and Seismic Shear" presented at Session 53, 6th SMIRT Conf. Paris, 1981.

10. Duchon, N.B. - "Analysis of Reinforced Concrete Membrane Subject to Tension and Shear", ACI Journal, Proc. Vol. 69, No. 9, Sept 1972 pp 578-583.

7:00 LIST OF FIGURES

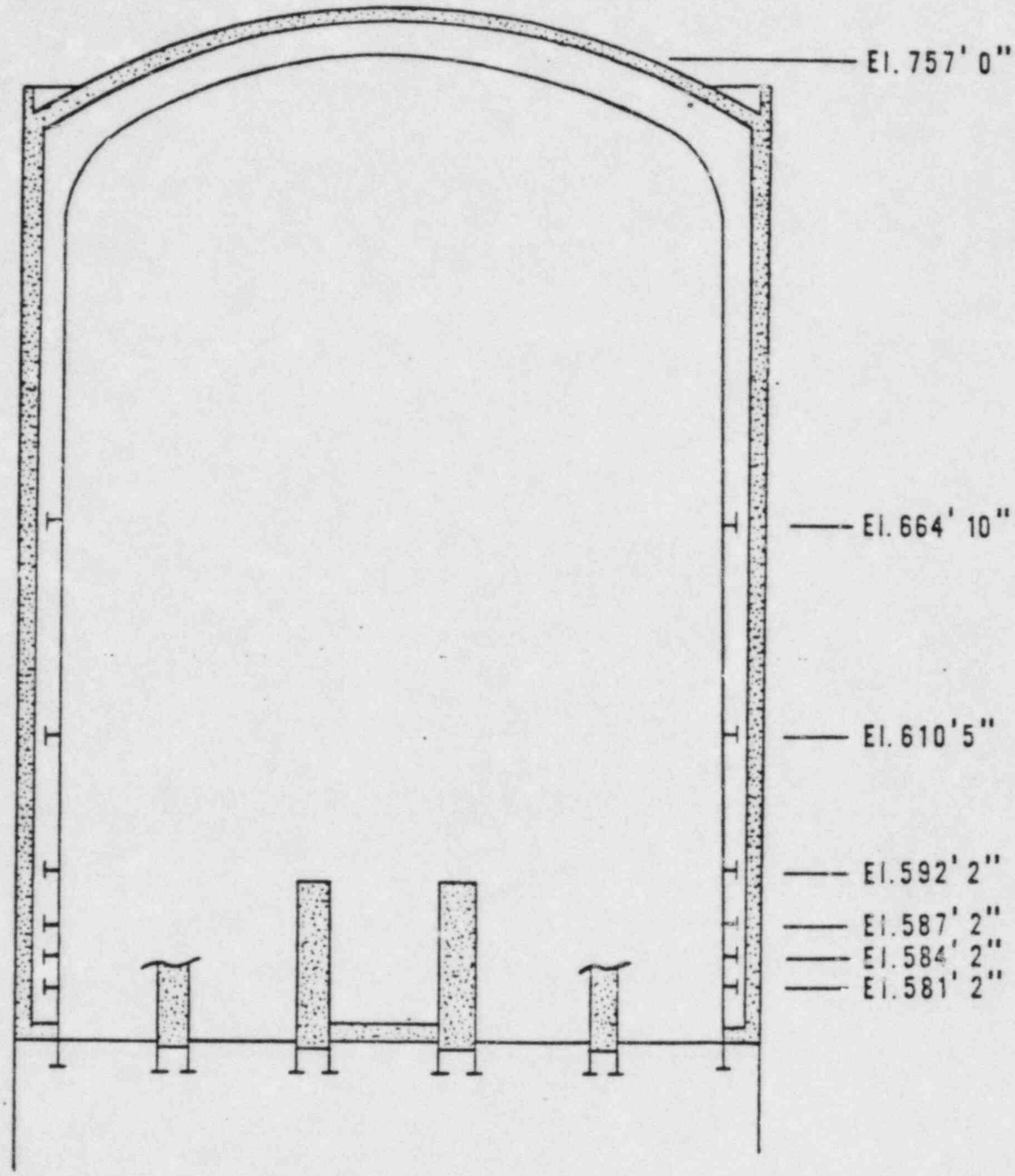

Figure 1.1 Containment - Shield Building

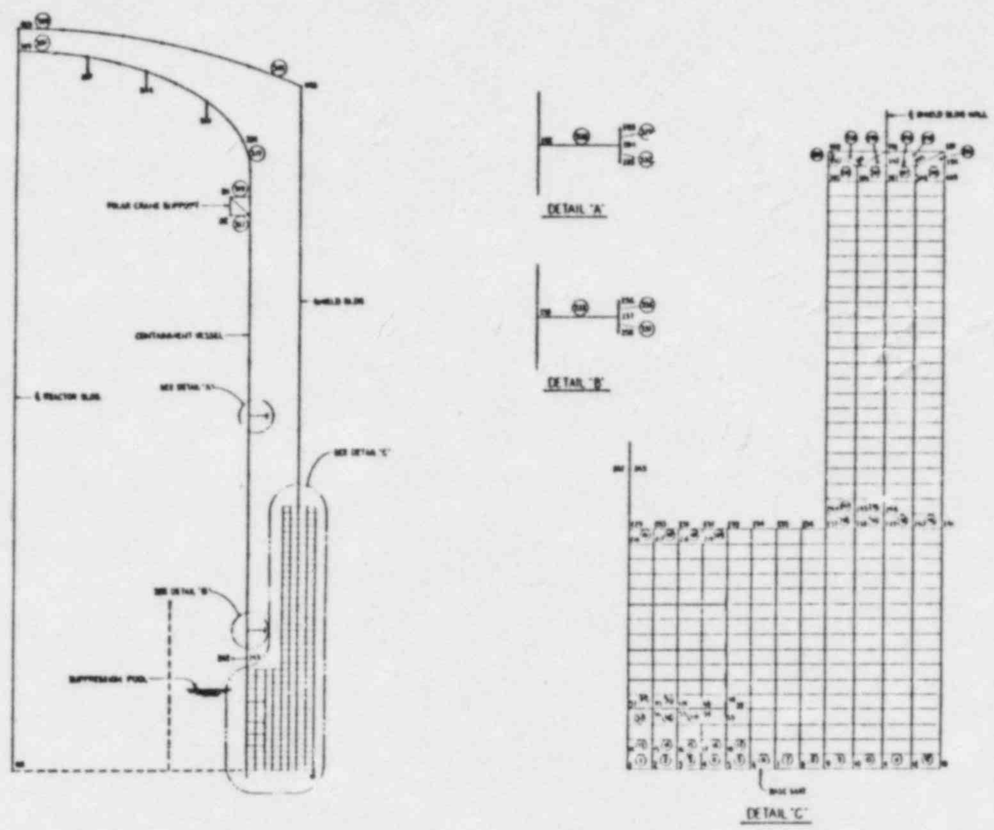

Figure 2.1 ASHSD2 Model

Section No.

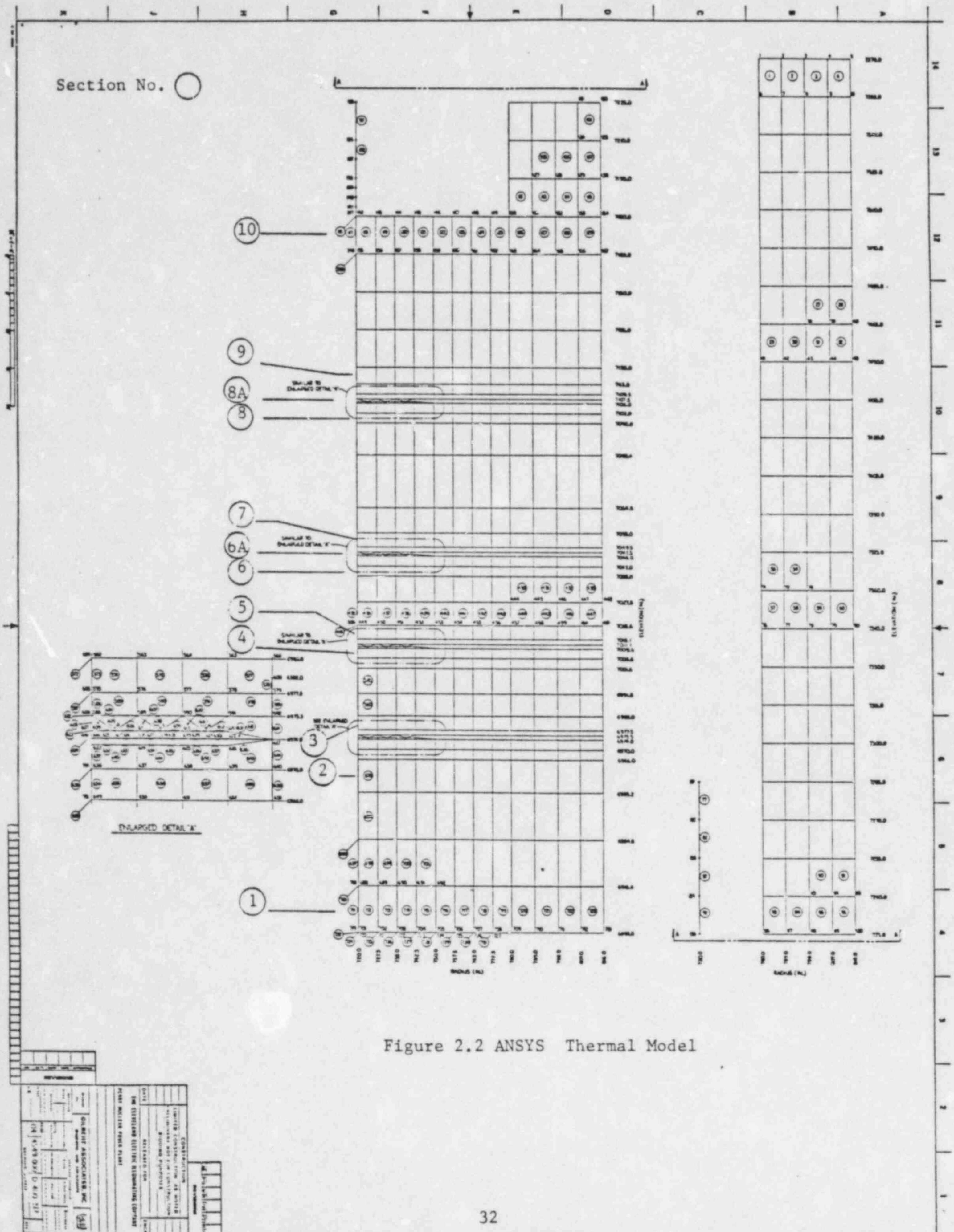

Figure 2.2 ANSYS Thermal Model

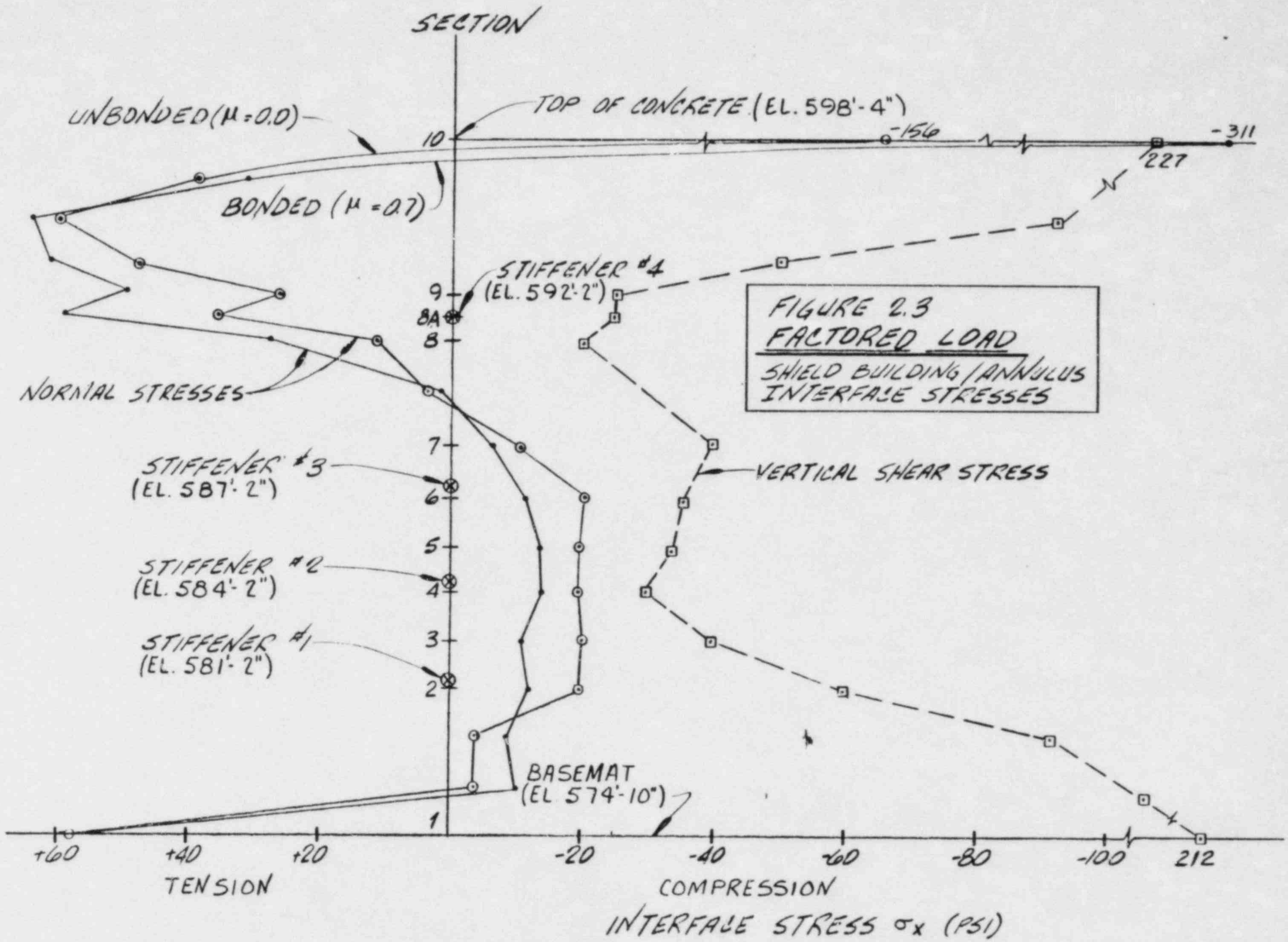

FIGURE 2.3
FACTORED LOAD
SHIELD BUILDING/ANNULUS INTERFACE STRESSES

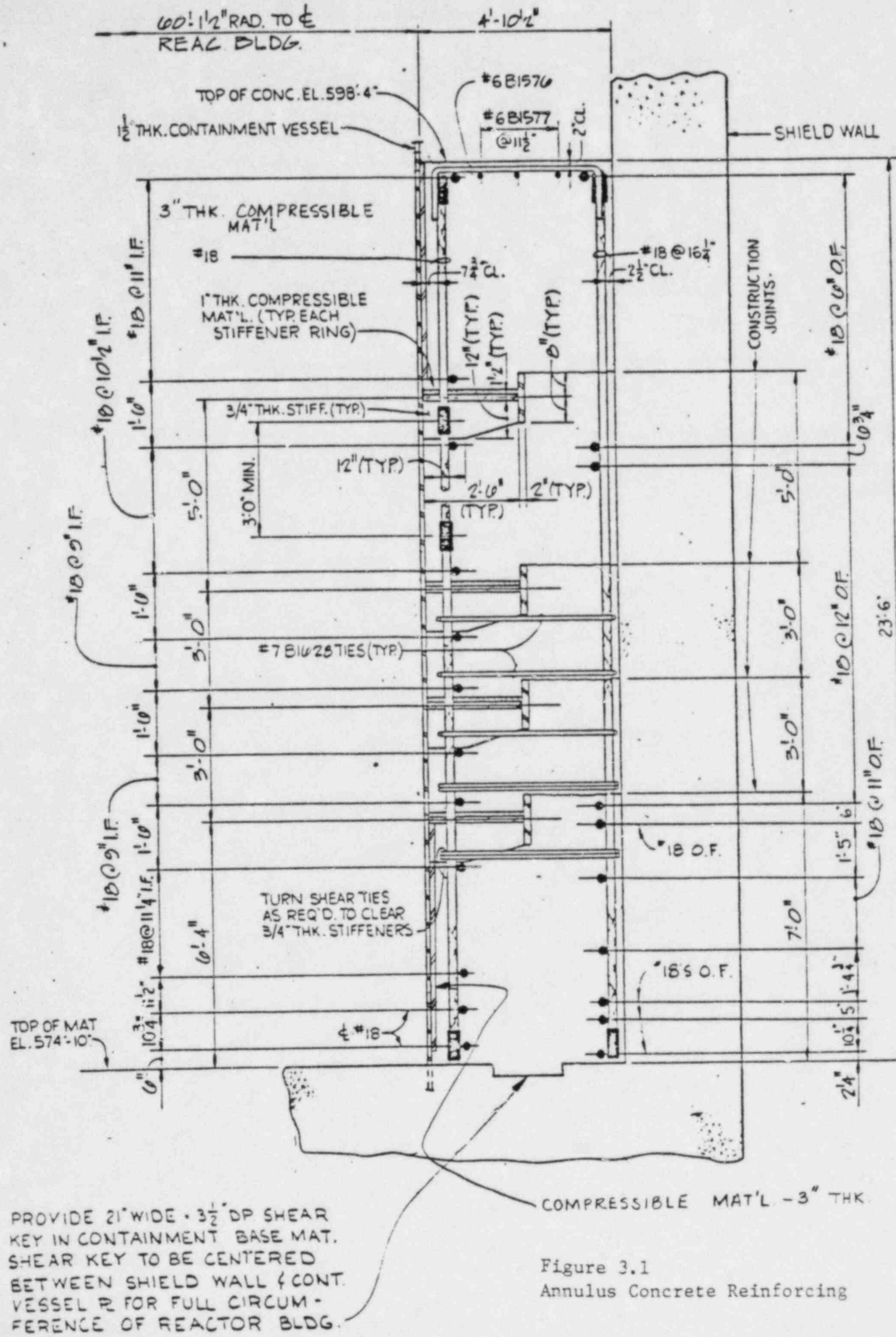

Figure 3.1
Annulus Concrete Reinforcing

NOTE:

TWO POINTS ARE PLOTTED FOR EACH SECTION REPRESENTING THE COMBINATION OF + AXIAL AND ± MOMENTS

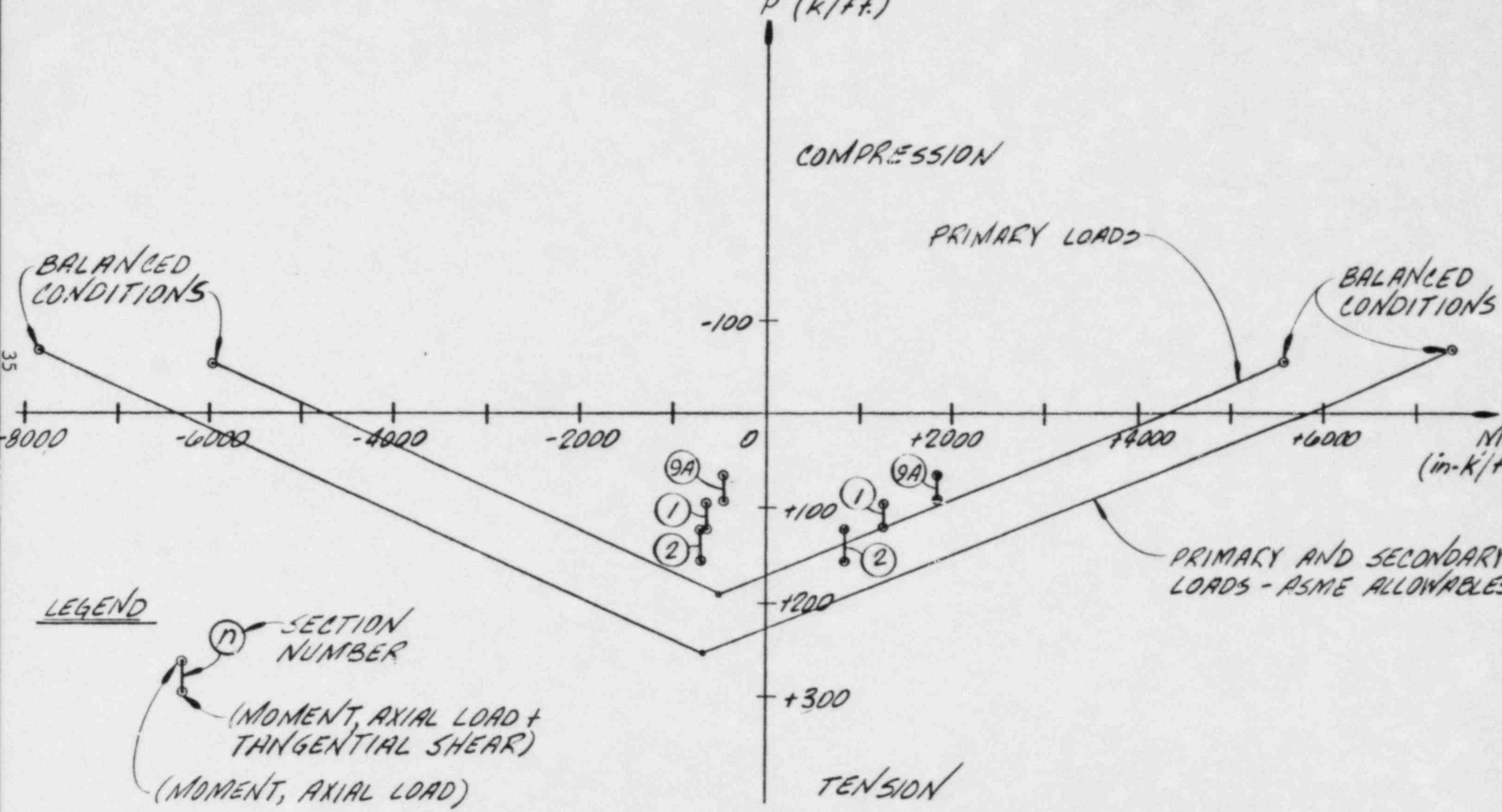

FIGURE 3.2
VERTICAL STEEL - SERVICE LOADS

NOTE:

FOUR POINTS ARE PLOTTED FOR SECTION 7 REPRESENTING THE COMBINATIONS OF ± AXIALS AND ± MOMENTS.

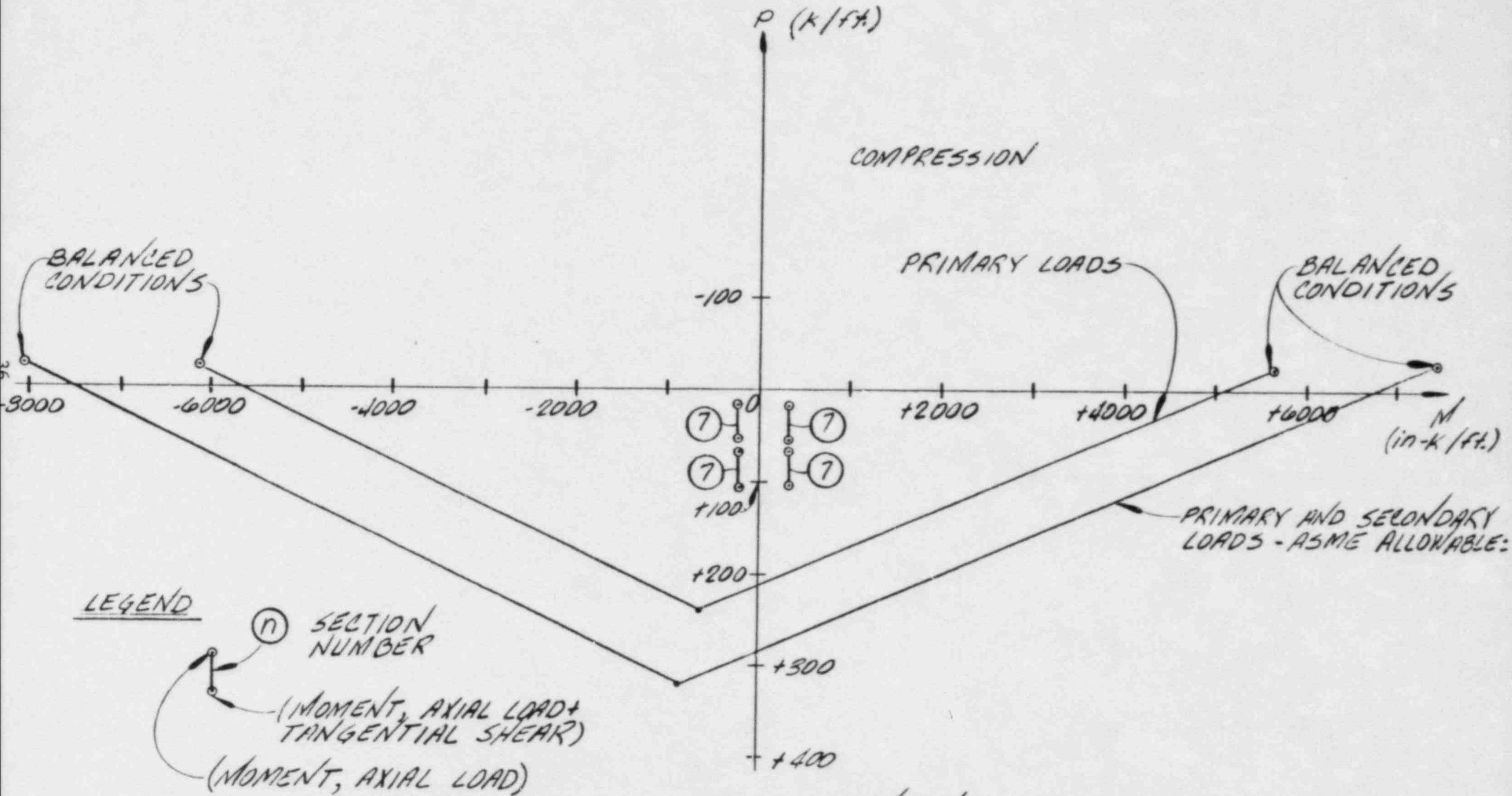

FIGURE 3.3
HORIZONTAL STEEL - BELOW ELEV. 590'-6" - SERVICE LOADS

NOTE:

TWO POINTS ARE PLOTTED FOR SECTION 9A REPRESENTING TWO COMBINATIONS OF + AXIAL AND ± MOMENTS.

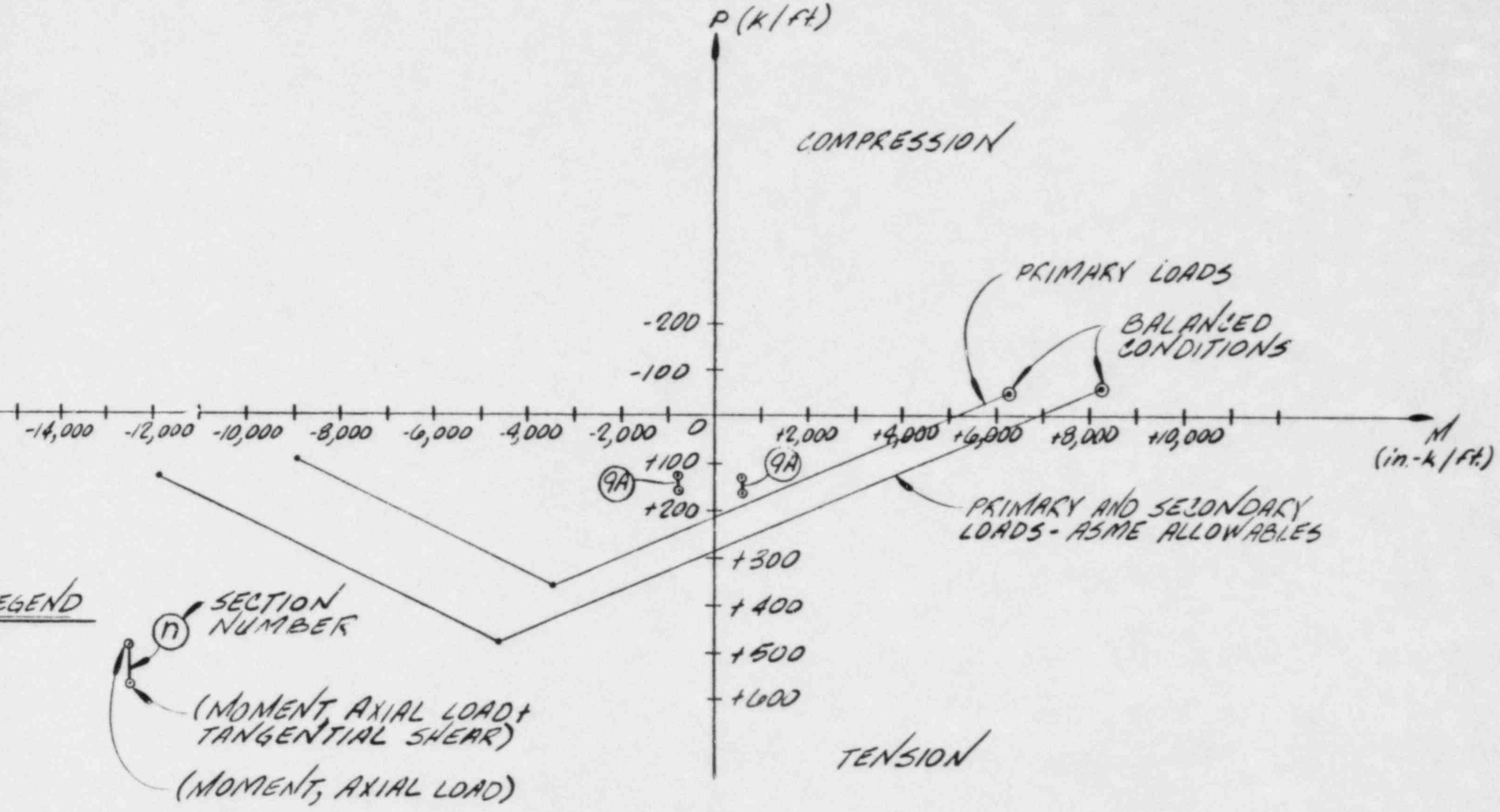

FIGURE 3.4

HORIZONTAL STEEL - ABOVE ELEV. 590'-6" - SERVICE LOADS

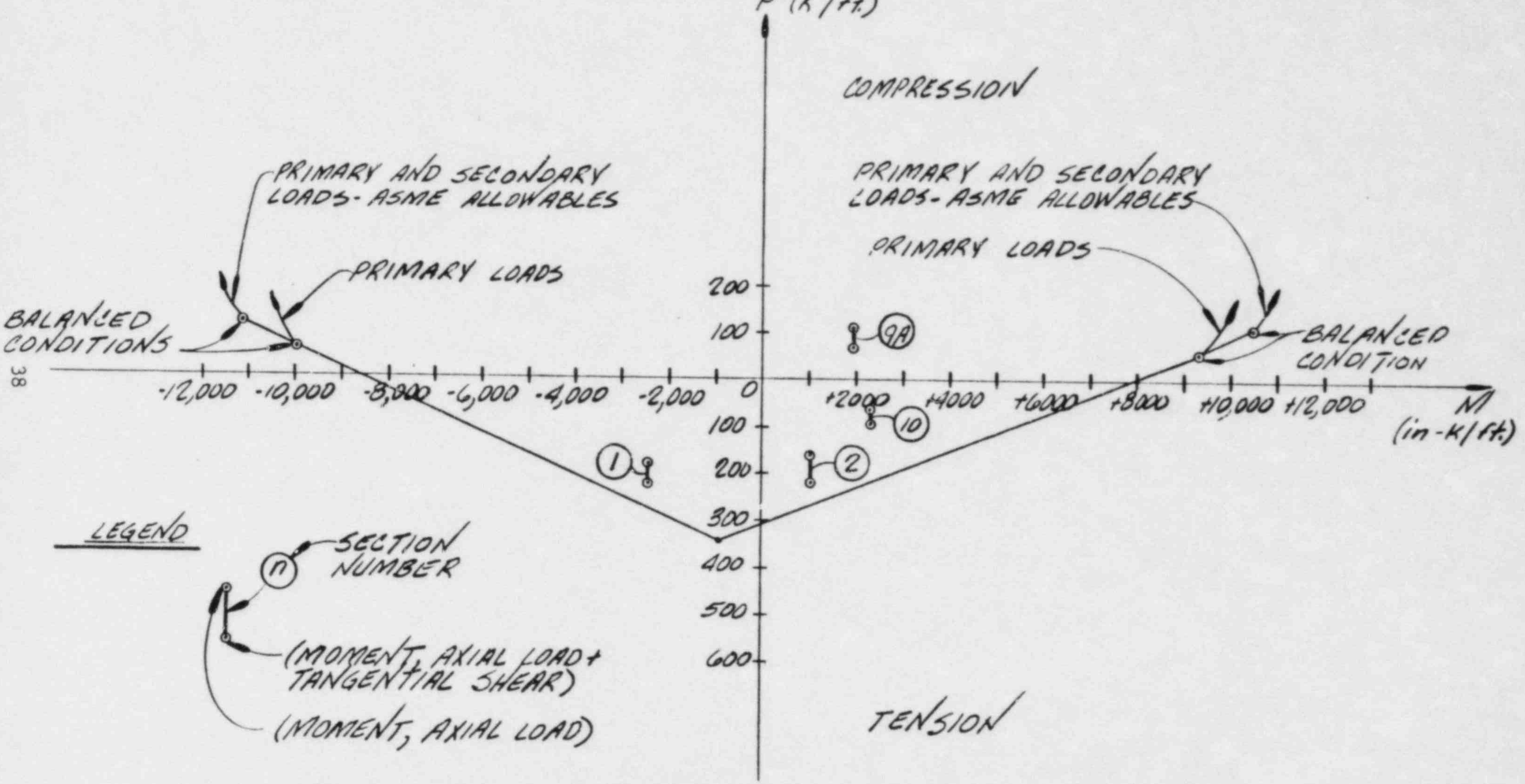

FIGURE 3.5
VERTICAL STEEL-FACTORED LOADS

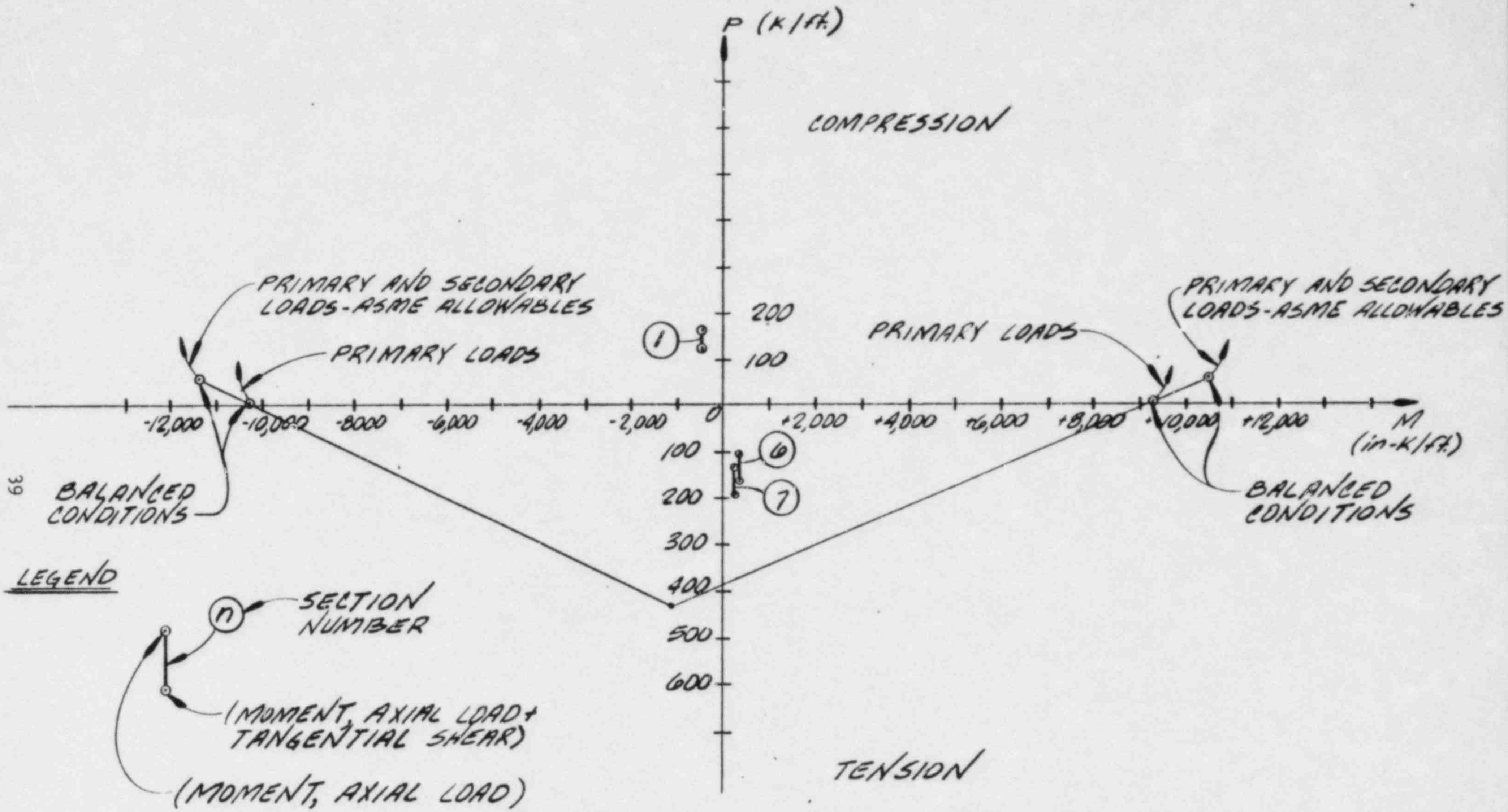

FIGURE 3.6

HORIZONAL STEEL - BELOW ELEV. 590'-6" - FACTORED LOADS:

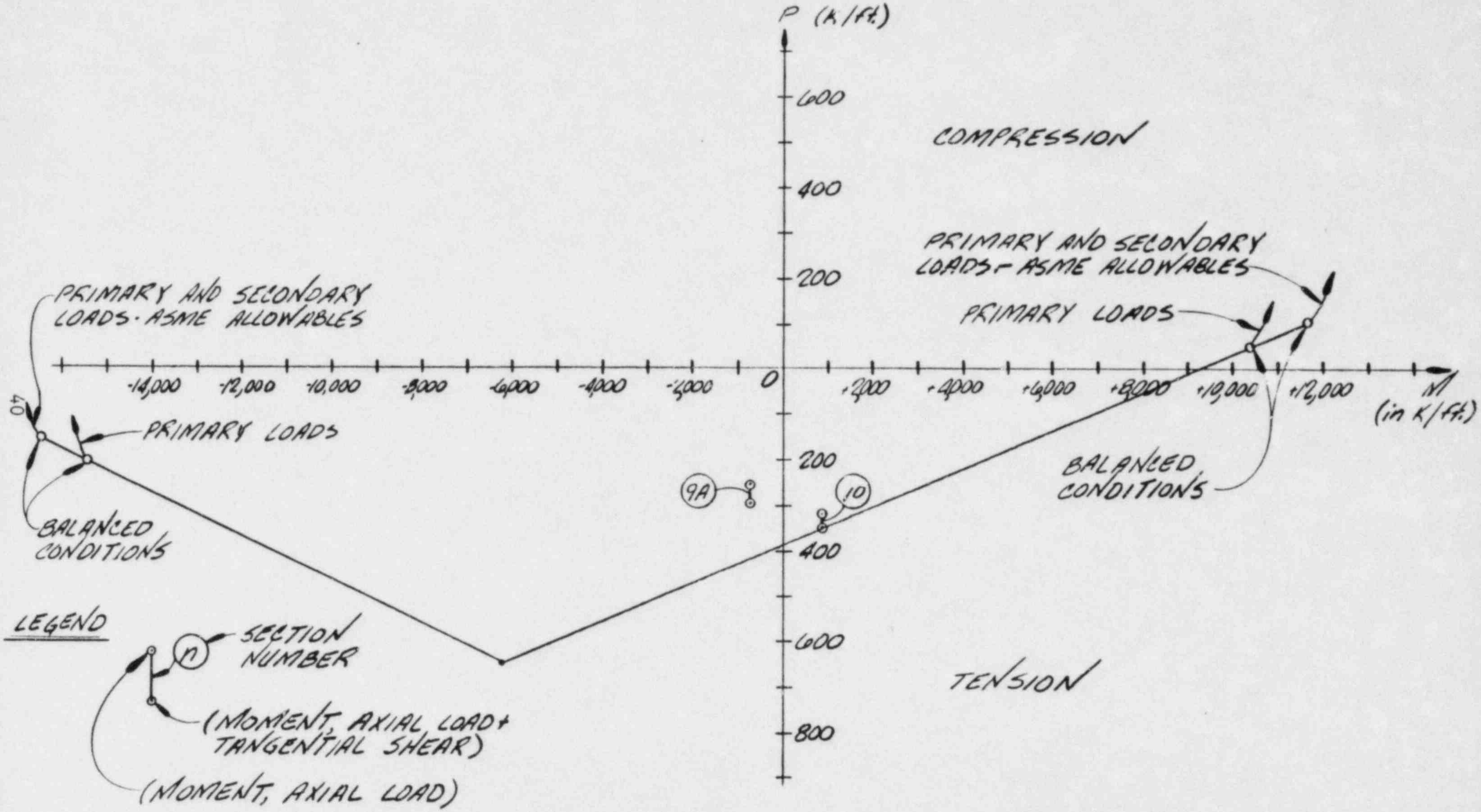

FIGURE 3.7

HORIZONTAL STEEL - ABOVE ELEV. 590'-6" - FACTORED LOADS

8:00 LIST OF TABLES

Table 3.1 Reinforcing Steel Stresses Excluding Tangential Shear

Section No (1)	Reinforcing Stress - Tension (ksi)			
	Vertical (2)		Horizontal (3)	
	Inside Face	Outside Face	Inside Face	Outside Face
1	14.9	41.2	C	C
2	35.5	15.2	0	0
3	31.2	27.1	6.1	3.7
4	29.1	25.4	8.3	6.6
5	26.9	24.0	12.9	10.2
6	26.7	23.0	17.0	13.0
7	24.2	21.8	20.8	16.1
8	24.4	18.5	29.4	11.2
9	19.0	16.2	33.4	13.0
9A	16.3	C(4)	40.1	16.0
10	26.3	C	50.8	14.6

Notes

(1) See Figure 2.2.

(2) Reinforcing ratio is 0.009.

(3) Reinforcing ratio is 0.011 for Sections 1-7 and 0.017 for Sections 8-10.

(4) Small compression.

Table 3.2 Calculated Tangential Shear Strength Based on Cornell[9] and PCA[7] Tests

Section No[a]	Perry Tangential Shear (b) psi	Cornell Tests Tangential Shear Strength - psi			PCA Tests Tangential Shear Strength - psi			Ratio-Tangential Shears Tests/Perry	
		Vertical	Horizontal	Minimum	Minimum (c)	Minimum Upper Bound[d]	Limited Distortion	Cornell Minimum	PCA Limited Distortion
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	57	203	365	203	152	253	102	3.56	1.79
2	81	220	365	220	199	275	132	2.72	1.63
3	81	233	343	233	233	291	156	2.88	1.93
4	82	239	335	239	250	299	167	2.91	2.04
5	82	246	318	246	268	308	179	3.00	2.18
6	83	246	303	246	270	309	180	2.96	2.17
7	83	254	290	254	290	318	193	3.06	2.33
8	82	253	319	253	288	298	192	3.08	2.34
9	78	269	296	269	332	283	222	3.45	2.85
9A	62	277	259	259	305	257	203	4.18	3.27
10	41	248	199	199	141	216	94	4.85	2.29

Notes:

(a) See Figure 2.2

(b) Peak Values

(c) Minimum value of vertical and horizontal

(d) Conservative bound of minimum values

TABLE 3.3 - RESULTS OF DUCHON[10] ANALYSES FOR THE FACTORED LOAD CASE
WITH (W)* AND WITHOUT (W/O) INCLINED REINFORCEMENT

Section **	Vertical Reinforcement Stress (ksi)		Horizontal Reinforcement Stress (ksi)		Inclined Reinforcement Stress (ksi)	Concrete Distortional Shear Strain (Rad)	
	W/O	W #5	W/O	W #5	W #5	W/O	W #5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	45.3	43.1	17.0	14.7	55.3	.00200	.00182
2	42.9	40.5	21.5	19.2	59.2	.00221	.00203
3	39.9	37.3	25.6	23.3	61.1	.00231	.00213
4	31.6	29.1	28.4	26.3	56.7	.00217	.00200
5	33.4	30.7	32.3	30.0	62.1	.00237	.00219
6	38.1	35.1	36.0	33.4	69.8	.00266	.00245
7	37.0	33.8	38.7	36.0	71.2	.00271	.00250
8	34.7	31.6	41.9	40.1	72.7	.00274	.00254
9	31.8	28.7	44.3	42.6	71.8	.00269	.00250
9A	30.4	26.6	56.1	54.4	79.5	.00293	.00269
10	32.4	28.6	52.8	51.1	78.6	.00291	.00267
Avg.	36.1	33.2	35.9	33.7	67.1	.00252	.00232
% Decrease	-	8.0	-	6.1	-	-	7.9

*Inclined reinforcement is at 45° and spaced 12" on centers, both directions.

**See Figure 2.2 for location of sections.

TABLE 4.1

REINFORCING STEEL AND SPLICING - CODE COMPARISON

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-2300 Material (Reinforcing Systems).					
CC-2310(a)	Material used for reinforcing systems shall conform to ASTM A-615	SP-663 2:05.1, 2:06		X		
CC-2310(b)	Material to be used for bar to bar splices shall conform to ASTM A513, A519, A579	SP-202 1:07.3		X		
CC-2320	Reinforcing system shall be traceable to CMTR during production and transit	SP-663 2:07		X		
CC-2330	Special material testing.					
CC-2331.1	One full diameter tensile bar of each bar size shall be tested per each 50 tons or fraction	SP-663 2:06.1		X		
CC-2331.2	Acceptance standard is ASTM A615	SP-663 2:06.1		X		
	If specimen fails - two retest.	SP-663 2:06.3			X	Single retest. Review of all material test reports show no failures.
CC-2332	Bend test					
CC-2332.1(a)	Per ASTM 615	SP-663 2:06.1		X		
CC-2332.1(b)(1)	One full size specimen per heat	SP-663 2:06.1	X			
CC-2332.1(b)(2)	Tested at ambient	ASTM A615		X		
CC-2332.1(b)(3)	Tested around a 9d pin	Not Addressed	X			Tested around an 8d pin
CC-2332.2	Acceptance standards					
CC-2332.2(b)	Absence of transerve cracking	SP-663 2:05.1	X			
	If specimen fails - two retest.	SP-663 2:06.2.1			X	Single retest - review shows no failures.
CC-2333	Chemical analysis - reported in accordance with A615	SP-663 2:05.1		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

TABLE 4.1

REINFORCING STEEL AND SPLICING - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-4300 Fabrication and Construction (Reinforcing Systems).					
CC-4320	Bending or reinforcing steel	SP-663 2:08.4		X		
CC-4321.1	Standard Hooks					
CC-4321.2	Diameter	SP-663 2:08.4		X		
CC-4322	Stirups, tie hooks, and bend other than standard hooks	SP-663 2:08.4		X		
CC-4324	Bending					
CC-4323.1	All bars shall be cold bent	SP-663 2:08.2		X		
	Examination of bends	SP-663 2:08.6		X		Inspected once per shift.
CC-4323.4	Tolerances per Fig. CC-4323-2 or 3	SP-663 2:08.4			X	Final acceptance is based on as-built field condition.
CC-4330	Splicing or reinforcing bars					
CC-4331.1	As required or permitted by designer	SP-202 1:07.1		X		
CC-4331.2	Permitted types of splices	SP-202 1:07.2		X		
CC-4332	Lap Splices	SP-202 1:07.2		X		
CC-4333	Mechanical Splices					
CC-4333.1.1	Required qualification - splicers	SP-202 1:08.2		X		
	Required qualification - splicing procedure	Not Addressed			X	PNPP utilized ERICO's proven splicing procedure
CC-4333.1.2	Maintenance and certification of records	SP-202 1:08.1.10		X		
CC- 4333.1.3	Splicing prior to qualification is not permitted	SP-202 1:08.2		X		
CC-4333.2	Splice system qualification requirements	Not Addressed			X	ERICO's long history of acceptable test results is an industry standard.
CC-4333.4	Initial qualification test 2 per splice position	SP-202 1:08.2		X		
CC-4333.5	Continuing splice performance tests					

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

TABLE 4.1

REINFORCING STEEL AND SPLICING - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
CC-4333.5.1	Cointuing series of testing shall be performed	SP-202 1:09	X			
CC-4333.5.2	Splice samples	SP-202 1:09.1 & 1:09.2		X		
CC-4333.5.3(a)	Frequency - 1 test per 100 splice	SP-202 1:09.3	X			
CC-4333.5.4	Tensile testing requirements	SP-202 1:09.4		X		
CC-4333.5.4(a)	Tensile strength shall equal or exceed 125% yield	SP-202 1:09.4.1		X		
CC-4333.5.4(b)	Running average of 15 shall equal or exceed minimum tensile	SP-202 1:09.4.2		X		
CC-4333.5.5	Substandard tensile test result					
CC-4333.5.5(a)	Failure in bar - investigate with fabricator	SP-202 1:09.5.1			X	Report to owner - only difference.
CC-4333.5.5(b)	Failure in splice	SP-202 1:09.5.2	X			
CC-4333.5.5(c)	Running average tensile strength failure	SP-202 1:09.5.3		X		
CC-4333.5.5	When splicing is resumed, frequency started anew	SP-202 1:09.5.4		X		
CC-4333.6	Recording of tensile test results	SP-202 1:08.1.10		X		
CC-4340	Placing reinforcing					
CC-4341	Supports	SP-202 1:06.4		X		
CC-4342	Tolerances	SP-202 1:06.5		X		
CC-4350	Spacing of reinforcement					
CC-4351	Layers	SP-14 5:07.2.3 & ACI 301		X		
CC-4352	Splices	SP-202 1:07		X		
CC-4360	Surface condition	SP-202 1:06.3 & 1:06.4.4		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

TABLE 4.1

REINFORCING STEEL AND SPLICING - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-5300 Construction Testing and Examination (Reinforcing System)					
CC-5300	Examination of reinforcing system					
CC-5320	Acceptance criteria for mechanical splices	SP-202 1:07.3 & 1:08		X		
CC-5321	Sleeve with ferrous filler metal splices					
CC-5321(a)	One sleeve per crew visually examined daily for fit-up	Not Addressed			X	Const. Spec. to be revised. Contractor's procedure required at least one visual examination daily.
CC-5321(b)	All completed sleeves shall be examined for:					
	- filler metal at end and tap hole	SP-202 1:08.1.9		X		
	- check for allowable maximum void	SP-202 1:08.1.9		X		
CC-5340	Examination of bends					
	The bent or straightened surface of bars shall be visually examined for indication of cracks	SP-663 2:08.6		X		Performed at fabricator facility.

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

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Table 4.2

CONCRETE - CODE COMPARISON

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-2200 MATERIAL (CONCRETE AND CONCRETE CONSTITUENTS).					
CC-2220	Concrete Constituents.					
CC-2221	Cement					
CC-2221.1	Material Requirement - shall conform to ASTM C-150, Type II	SP-14 5:06.1	X			SP-14 requires the optional tests plus establishes more conservative values for certain tests.
CC-2222	Aggregates.					
CC-2222.1	Aggregates shall conform to ASTM C-33	SP-14 5:07.1 & 5:07.2		X		
CC-2222.1(b)	Flat and elongated particles - 15% CRD-C119	SP-14 5:07.2.5		X		
CC-2222.1(c)	Optional - Potential Alkali Reactivity of Cement Aggregate Combination Agg. ASTM C-227	SP-14 5:07.2(c)	X			SP-14 requires additional optional tests.
	Optional - Potential Reactivity Aggregates ASTM C-289	SP-14 5:07.2(c)	X			SP-14 requires additional optional tests.
	Optional - Potential Volume Change of Cement Aggregate Combination ASTM C-342	SP-14 5:07.2(c)	X			SP-14 requires additional optional tests.
	Required - Petrographic Examination	SP-14 5:07.2(c)		X		
CC-2222.1(d)	Water Soluble Chloride Content of Aggregates ASTM D-1411	SP-14 5:07.2.8		X		Test has been performed. Test results are less than 10 PPM.
CC-2222.1(e)	Tangential Shear (L.A. Abrasion) Max. 40% ASTM C-131	SP-14 5:18.3.3(i)		X		Review of material Test Reports Max. = 32%.
CC-2222.1(f)	Max. Size of Aggregate	SP-14 5:07.2.3 & ACI 301		X		
CC-2222.4	Aggregate for Grout - Conforms to ASTM C-33	SP-14 5:07.1.1		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Table 4.2

CONCRETE - CODE COMPARISION (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
CC-2223	Mixing Water					
CC-2223.1	Water Shall be Clean with Max. Total Solids of 2000 PPM. ASTM D-1888	SP-14 5:09.1	X			SP-14 requires 1,000 ppm per APHA test.
	Water shall be tested for Chlorides ASTM 512	SP-14 5:09.1.3		X		
CC- 2223.2(a)	Time of setting ASTM C-191	SP-14 5:09.2.1(b)		X		
CC-2223.2(b)	Compressive Strength	SP-14 5:09.2.1(c)		X		
CC-2224	Admixtures					
CC-2224.1	Construction Specification Shall Specify Type, Quantity, and Additional Limits. Each Admixture shall not contribute more than 5 PPM, by weight of Chloride Ions to total concrete constituent	SP-14 5:04.9		X		Our present admixtures contribute less than 5 PPM by weight.
CC-2224.2.1	Air Entraining Admixtures shall conform to ASTM C-260	SP-14 5:08.1		X		
CC-2224.2.3	Chemical Admixtures shall conform to ASTM C-494	SP-14 5:08.2		X		
CC-2230	Concrete Mix Design					
CC-2231.1	Properties of Concrete which influence the Design shall be established in the Construction Specification	SP-14		X		
CC-2231.2	Chloride Content of Cement Paste shall not exceed 400 ppm by weight	SP-14 5:04.10		X		
CC-2231.3	Applicable Concrete Properties in Table CC-2231-1 shall be defined in Const. Spec.	SP-14 5:02		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Table 4.2

CONCRETE - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
CC-2232	Selection of Concrete Mix Proportions					
CC-2232.1	Trial Mix Design Proportions	SP-14 5:04.2		X		
CC-2232.2	Strength Tests	SP-14 5:04.2		X		
CC-2232.3	Durability					
CC-2232.3.1	W/C shall not be exceed 0.53 for Concrete Expose to Freezing Temperatures.	SP-14 5:10.1	X			SP-14 requires a maximum W/C ratio of 0.50.
CC-2240	Cement Grout					
CC-2241	Constituent for Cement Grout					
CC-2241.1	Cement shall conform to ASTM C-150	SP-14 5:06.1	X			SP-14 requires the optional tests plus establishes more conservative values for certain tests.
CC-2241.2	Aggregate shall conform to ASTM C-33	SP-14 5:07.2		X		
CC-2241.3	Water shall conform to CC-2223	SP-14 5:09		X		
CC-2250	Marking and Identification of Concrete Constituents					
CC-2251	Cement shall be sealed and tagged before leaving supplier showing lot number, specification, grind date and type	SP-14 5:06.5.4 SP-14 5:06.10		X		
CC-2252	Aggregate shall be identified to size, source, and specification	Not Addressed		X		Presently addressed in Nonmetallic Material Manufacturer's QA Program.
CC-2253	Admixture tanks shall be labeled with name, specification, and storage requirements.	Not Addressed		X		Nonmetallic Material Manufacturer's QA Program requires labeling all but storage requirements. QA manual being revised. Storage requirements have been labeled.

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Table 4.2

CONCRETE - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-4200 FABRICATION AND CONSTRUCTION (CONCRETE)					
CC-4220	Storing, batching, mixing and transporting.					
CC-4221.1	Stockpiling and storing aggregate.	SP-14 6:09.1 & 6:11.10 ACI 301		X		
CC-4221.2	Storage; Cement & Admixture.	SP-14 6:09.1 & 5:07.4		X		
CC-4222	Batching					
CC-4222.1	Distribution					
	1) Conform to ACI-304	SP 14 6:11		X		SP-14 references ACI 301 requirements. ACI 301 and 304 requirements are consistent.
	2) Only accepted material used	SP-14 5:18.3		X		Our present practice is to conduct the aggregate testing the day before.
CC-4222.2	Measuring					
	1) By weight - Cement & Aggregates	SP-14 6:11.3		X		
	2) By volume - H_2O	SP-14 6:11.5		X		
	3) Free moisture correction shall be accounted for	SP-14 5:11.5		X		
	4) Tolerances per ASTM C-94	SP-14 6:11.9		X		
CC-4223.1	Mixing - per ASTM C-94	SP-14 6:11.11		X		
CC-4223.2	Operation of mixer per ASTM C-94	SP-14 6:11.10 & ACI 301		X		ACI-301 Sect. 7.2.2 gives same requirements as ASTM C-94.
CC-4224.1	Conveying from mixer to point of placement	SP-14, SP-801 5:05.5		X		Specs satisfy code requirements.
CC-4224.2	Conveying equipment	SP-801 5:05, SP-14 6:09		X		Specs satisfy code requirements.

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Tabel 4.2

CONCRETE - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
CC-4225	Depositing					
CC-4225.1	General	SP-801 5:05.6		X		
CC-4225.2	Continuity	SP-801 5:05.6		X		
CC-4226	Consolidation					
CC-4226.1	General - per ACI-309	SP-801 5:05.7		X		
CC-4240	Curing					
	(A) Moist & protected through minimum curing period	SP-801 5:05.9		X		
	(D) When mean daily temperature is below 40°F, conc to be at least 50°F & moist for 7 days	SP-801 5:05.9		X		
CC-4250	Formwork and Const. Joints					
CC-4251.1	General - properly designed braced and tied	SP-801 5:05.2		X		
CC-4251.2	Design of formwork - ACI-347	SP-801 5:05.2		X		
CC-4251.3	Use of liner as formwork	SP-801 5:05.2		X		
CC-4252	Construction joints located as shown on drawings	SP-801 5:05.3		X		
CC-4260	Cold and hot weather conditions	SP-14 15:3.1 SP-801 5:05.10		X		
CC-4270	Repairs to concrete - as directed by designer and per CC-4252 of code.	SP-801 5:06.6		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Table 4.2

CONCRETE - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	CC-5200 CONSTRUCTION TESTING AND EXAMINATION (CONCRETE).					
CC-5200	Concrete examinations					
CC-5210	General	SP-801		X		Authorized Inspector will have access to batch plant.
CC-5220	Concrete Constituents					
CC-5221.1	Cement Requirements	SP-14 5:18.3.7	X			Option tests are required plus more conservative values are established for certain tests.
CC-5221.2	Testing frequency	See modified Table CC-5200-1		X		
CC-5223.1	Admixture requirements ASTM C-494	SP-14 5:18.3.5 5:04.1c		X		
CC-5223.2	Testing frequency	See modified Table CC-5200-1		X		
CC-5224.1	Aggregate requirements	SP-14 5:04.1.8, 5:18.3.3		X		
CC-5224	Testing frequency	See modified Table CC-5200-1		X		
CC-5225.1	Mixing water requirements	SP-14 5:18.3.4		X		
CC-5225.2	Testing frequency	See modified Table CC-5200-1		X		
CC-5231	Concrete, sampled to ASTM C-172	SP-801 5:06.4		X		
CC-5232.1	Slump requirements to ASTM C-143	SP-801 5:06.4		X		
CC-5232.2	Testing frequency	SP-801 5:06.4		X		
CC-5233.1	Temperature requirement	SP-801 5:06.4		X		
	Air content to ASTM C-173 or ASTM C-231	SP-801 5:06.4		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Table 4.2

CONCRETE - CODE COMPARISON (Continued)

CODE SECTION	SUBJECT	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
	Unit weight to ASTM C-138	SP-801 5:06.4		X		
CC-5233.2	Testing frequency	SP-801 5:06.4		X		
CC-5234.1	Compressive strength cylinders ASTM C-31 or ASTM C-39	SP-801 5:06.4		X		
CC-5234.2	Evaluation and acceptance	SP-801 5:06.5		X		

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

DW38/B/7/jg

Tabel 4.3

MODIFIED TABLE CC-5200-1
ASME CODE/PNPP SPEC. COMPARISON OF CONCRETE RELATED TEST FREQUENCIES

MATERIAL	REQUIREMENTS AND METHOD	FREQUENCY	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
CEMENT	Standard chemical prop. ASTM C-114	Each 1200T	SP-14 5:18.3.7	X			Optional test required.
	Fineness ASTM C-204 or ASTM C-115	Each 1200T	SP-14 5:18.3.7	X			Maximum fineness of 4,000 CM²/SRAM.
	Auto clave expansion ASTM C-151	Each 1200T	SP-14 5:18.3.7		X		
	Compressive strength ASTM C-109	Each 1200T	SP-14 5:18.3.7	X			Minimum 4,500 psi at 28 days.
	Time of setting ASTM C-266 or ASTM C-191	Each 1200T	SP-14 5:18.3.7		X		
AGGREGATE	Gradation ASTM C-136	Each 1000 C.y.	SP-14 5:18.3.3.A	X			Daily test.
	Moisture ASTM C-566	Twice Daily during production	SP-14 5:18.3.3.B		X		
	Material finer than #200 ASTM C-117	Each 1000 C.y.	SP-14 5:18.3.3.C	X			Daily test.
	Organic impurities ASTM C-40	Each 1000 C.y.	SP-14 5:18.3.3.D	X			Daily test.
	Flat and elongated particles CRD C-119	Monthly	SP-14 5:18.3.3.I		X		
	Friable particles ASTM C-142	Monthly	SP-14 5:18.3.3.E		X		
	Light weight particles ASTM C-123	Monthly	SP-14 5:18.3.3.F		X		
	Specific gravity and absorption ASTM C-127 or ASTM C-128	Monthly	SP-14 5:18.3.3.M		X		
	L.A. Abrasion ASTM C-131 or ASTM C-535	Every 6 months	SP-14 5:18.3.3.H		X		
	Potential reactivity ASTM C-289	Every 6 months	SP-14 5:18.3.3.J		X		
	Soundness ASTM C-88	Every 6 months	SP-14 5:18.3.3.K		X		
	Water soluble chloride ASTM D-1411	Every 6 months	CP-14 5:18.3.3.O		X		Testing program has started.

(+) Exceeds Section III, Division 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

Tabel 4.3

MODIFIED TABLE CC-5200-1
ASME CODE/PNPP SPEC. COMPARISON OF CONCRETE RELATED TEST FREQUENCIES

MATERIAL	REQUIREMENTS AND METHOD	FREQUENCY	CORRESPONDING PNPP CONSTRUCTION SPEC.	+	=	-	REMARKS
WATER & ICE	Effect on compressive Str. ASTM C-109	Every 6 months	SP-14 5:18.3.4		X		Testing program has started.
	Effect on setting time ASTM C-191	Every 6 months	SP-14 5:18.3.4		X		Testing program has started.
	Total solids ASTM D-1888	Every 6 months	SP-14 5:18.3.4		X		Testing program has started.
	Chlorides ASTM D-512	Monthly	SP-14 5:18.3.4		X		Testing program has started.
ADMIXTURE	Uniformity - infrared spectrophotometry, PH and solids per ASTM C-494	Each load	SP-14 5:18.3.5		X		Spectrophotometry, PH, Specific Gravity and Total Solids tests are conducted.
CONCRETE	Mixer uniformity ASTM C-94	Initially and every 6 months	SP-14 5:18.3.1.A		X		
	Compressive strength ASTM C-39 or CRD C-84	1 set every 100 cy 1 set a day for each class	SP-801 5:06.4		X		
	Slump ASTM C-143	1st batch & every 50 cy.	SP-801 5:06.4		X		
	Air Content ASTM C-173 or C-231	1st batch & every 50 cy	SP-801 5:06.4		X		
	Temperature	1st batch & every 50 cy	SP-801 5:06.4		X		
	Weight/Yield ASTM C-138	Daily during production	SP-801 5:06.4		X		

(+) Exceeds Section III, Devision 2 Requirements (=) Meets Code Requirements
(-) Construction Specification Insufficient

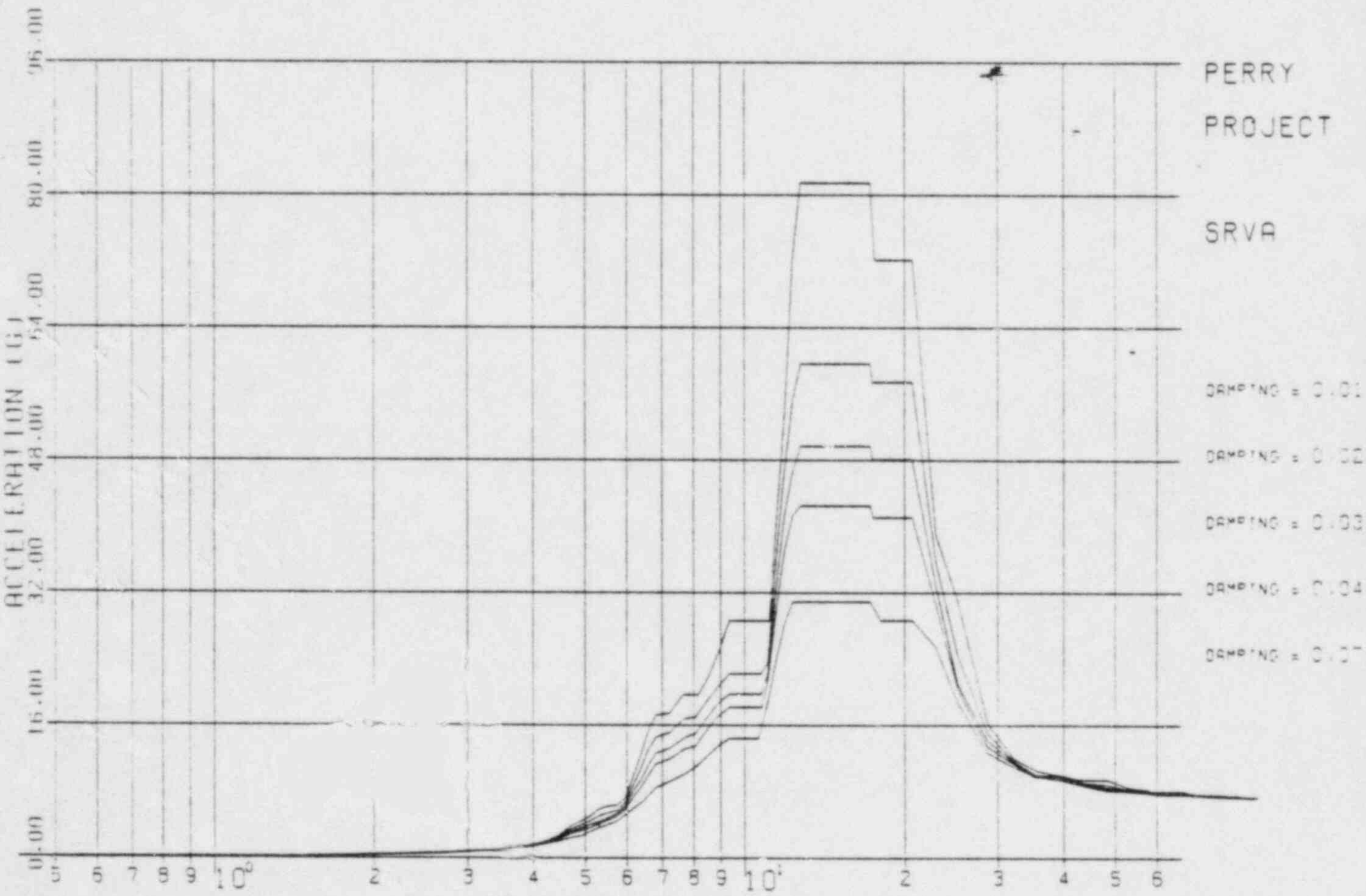
DW38/D/2/jg

APPENDIX A

Comparison of SRVD Response Speectra for the Containment Vessel with and without the Annulus Concrete

Response spectra are presented for Elevation 579'-5" (node 155), Elevation 664'-10' (node 272), and Elevation 749'-4" (node 311) in the radial (direction 1), vertical (direction 2), and tangential (direction 3) directions for the General Electric safety relief valve discharge (SRVD) random loading for 19 valves, load case 23. Figures 1-3 are the response spectra for the SRVD analysis which does not include the annulus concrete. These response spectra curves are envelopes of GE random loading: 19 valves - load case 23, 19 valves - load case 32, and 19 valves - load case 46. Load case 23 provided the largest response of the three load cases and therefore these curves can be compared to the response spectra curves presented in Figures 4-6 which are generated from random load 19 valves - load case 23. Some problems may arise since the response spectra from three enveloped load cases are being compared to one individual load case; however, the comparison provides a good indication of the changes caused by the addition of the annulus concrete. Node 155 is located in the suppression pool, node 272 is located on the cylindrical portion of the vessel above the pool, and node 311 is located on the dome.

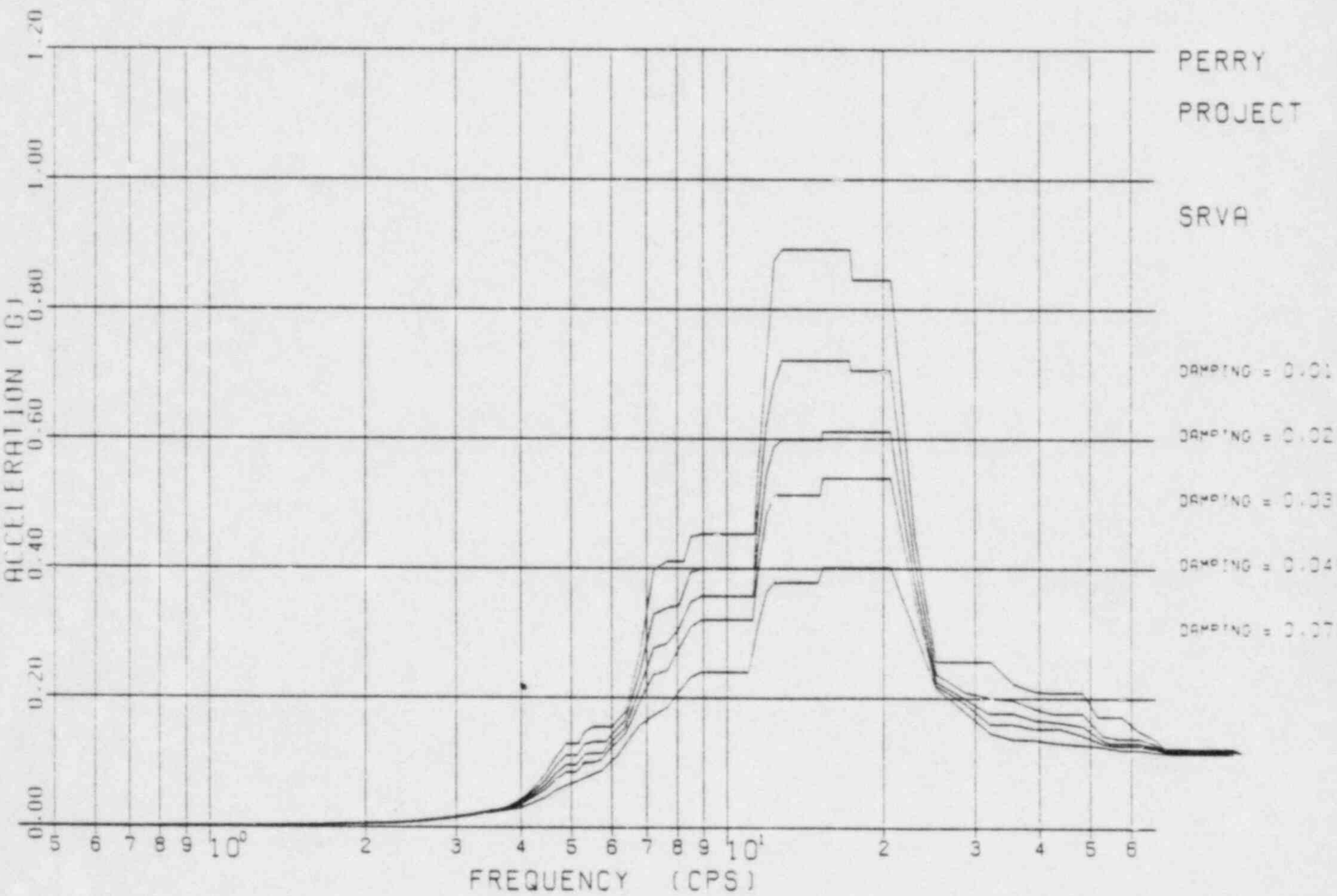
As an example, if Figure 3a is compared to Figure 6a, it is observed that the peak acceleration response for the 1% damping curve was reduced from 10.7 g to 0.44 g. A frequency shift caused by the addition of the annulus concrete occurred. The center of the peak for the analysis which did not include annulus concrete is located at approximately 18.0 Hz (figure 3a) while the center of the peak for the analysis which did include the annulus concrete is located at approximately 25.0 Hz. The additional stiffness provided by the annulus concrete caused a substantial reduction in the acceleration response of the Containment Vessel and a frequency shift in the location of the peak response.



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 1

FOR LOAD CASES 19231, 19321, 19461

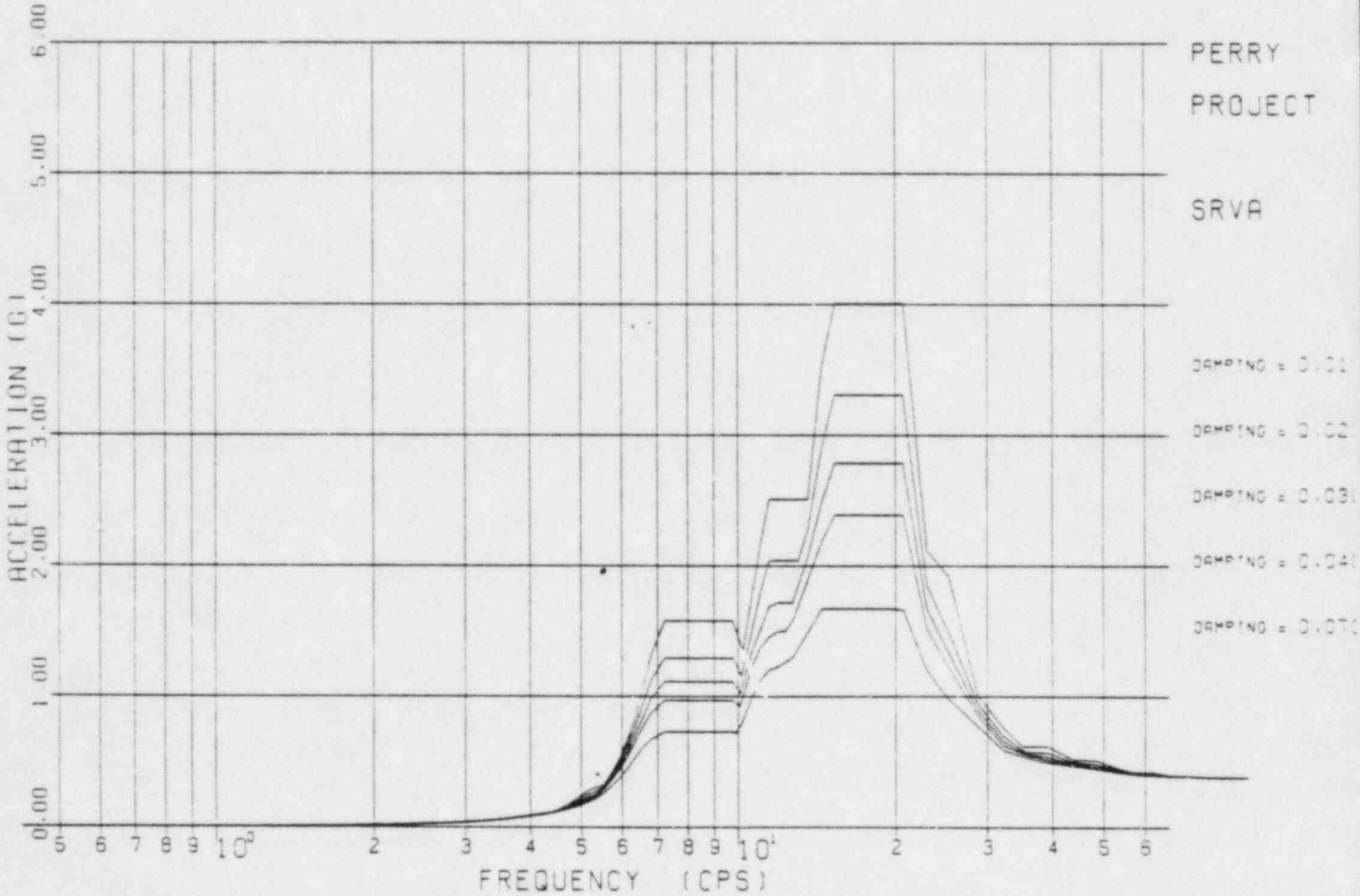
FIGURE 1a RADIAL RESPONSE SPECTRA AT EL. 579'-5" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 2

FOR LOAD CASES 19231, 19321, 19461

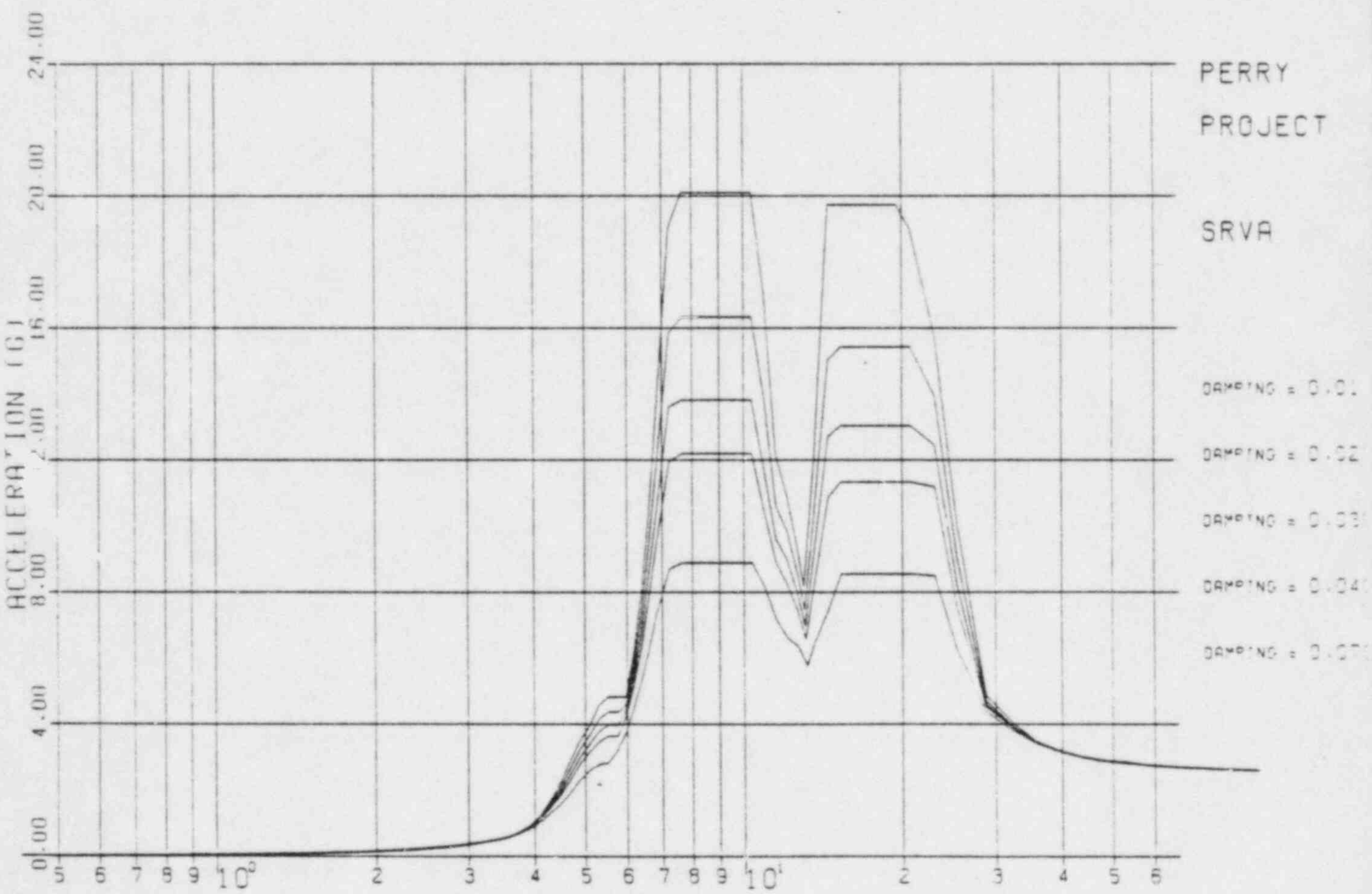
FIGURE 1b VERTICAL RESPONSE SPECTRA AT EL. 579'-5" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 3

FOR LOAD CASES 19231, 19321, 19461

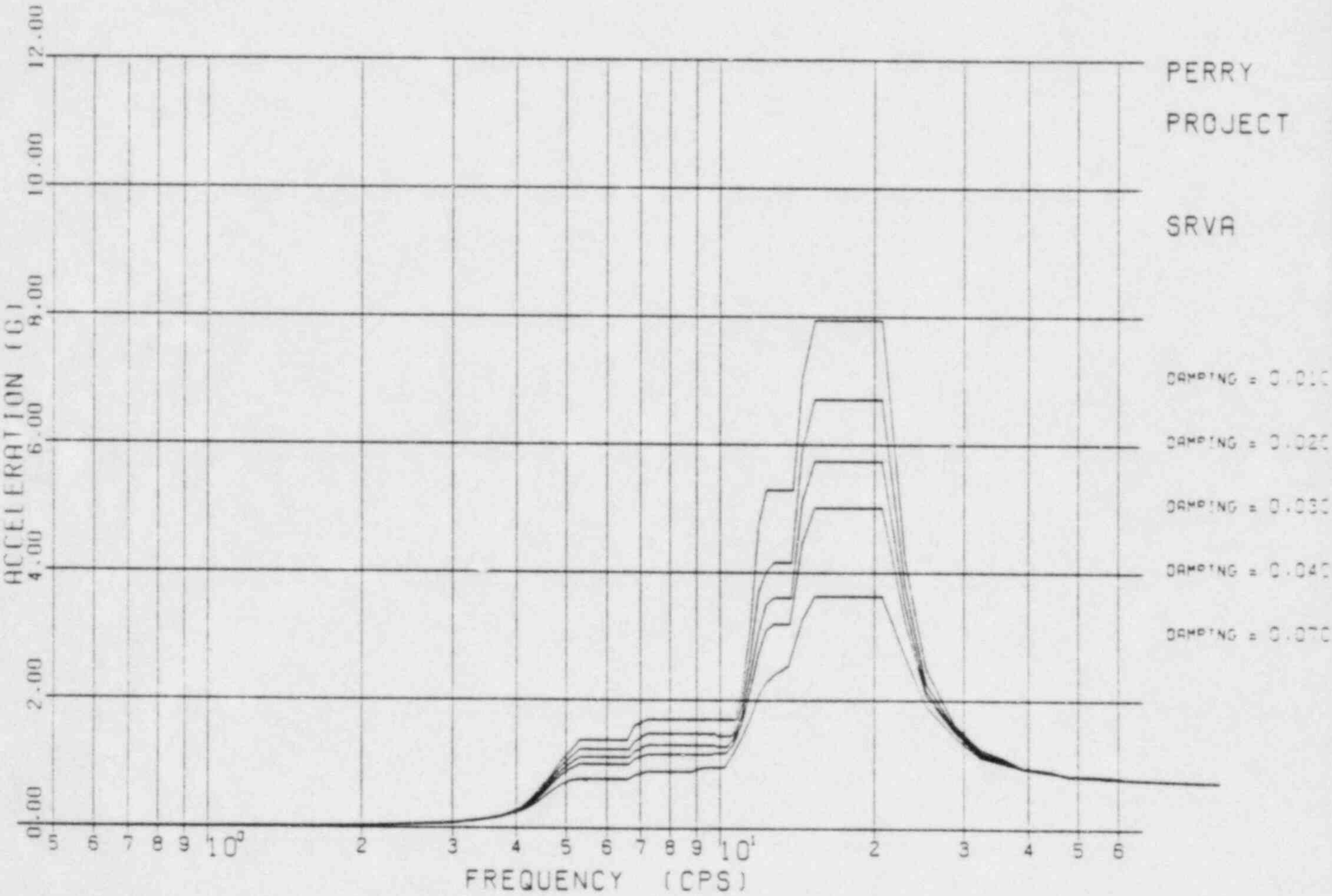
FIGURE 1c TANGENTIAL RESPONSE SPECTRA AT EL. 579'-5" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 272 IN DIRECTION 1

FOR LOAD CASES 19231, 19321, 19461

FIGURE 2a RADIAL RESPONSE SPECTRA AT EL. 664'-10" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 272 IN DIRECTION 2

FOR LOAD CASES 19231, 19321, 19461

FIGURE 2b VERTICAL RESPONSE SPECTRA AT EL. 664'-10" NO ANNULUS CONCRETE

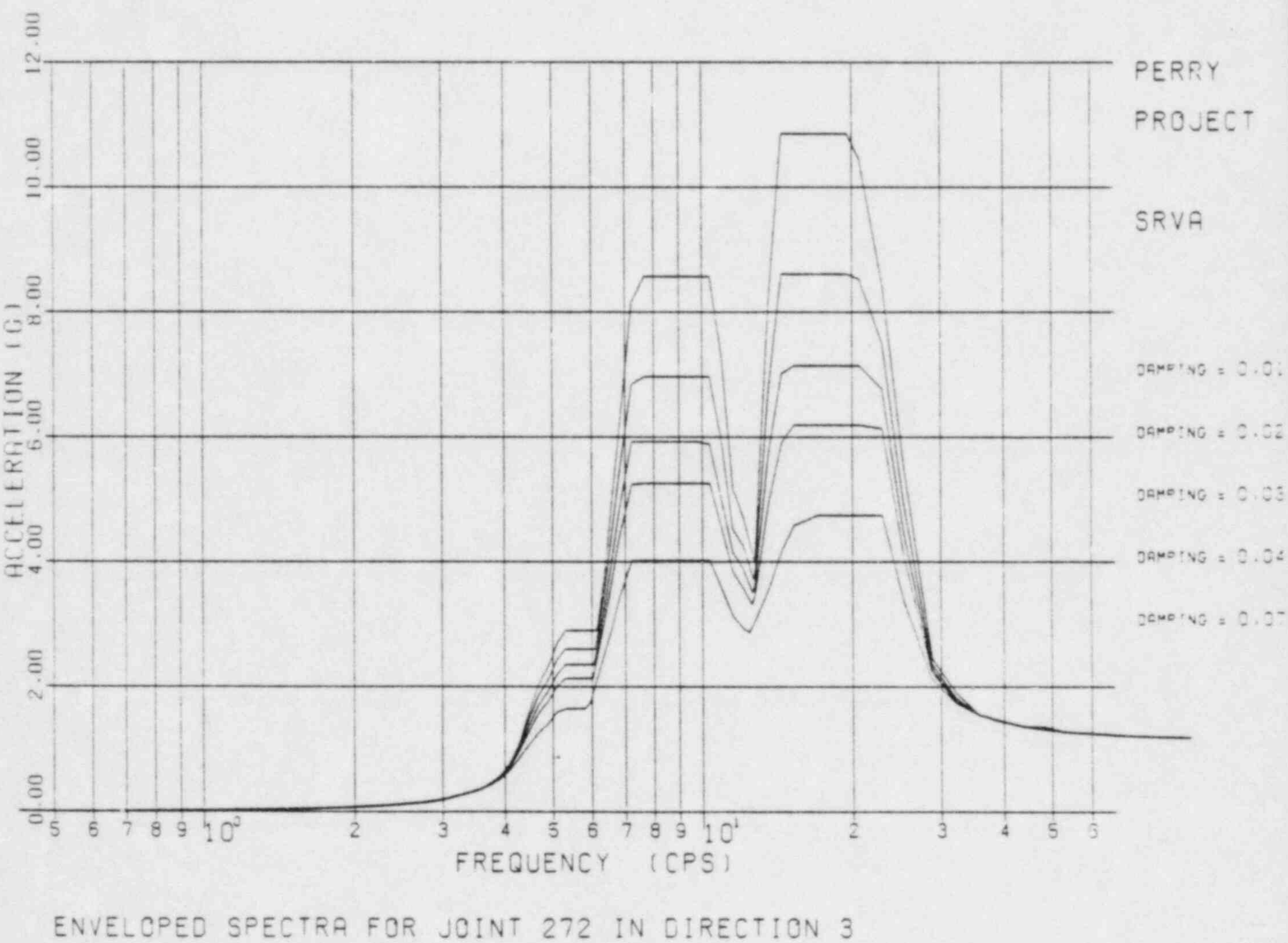
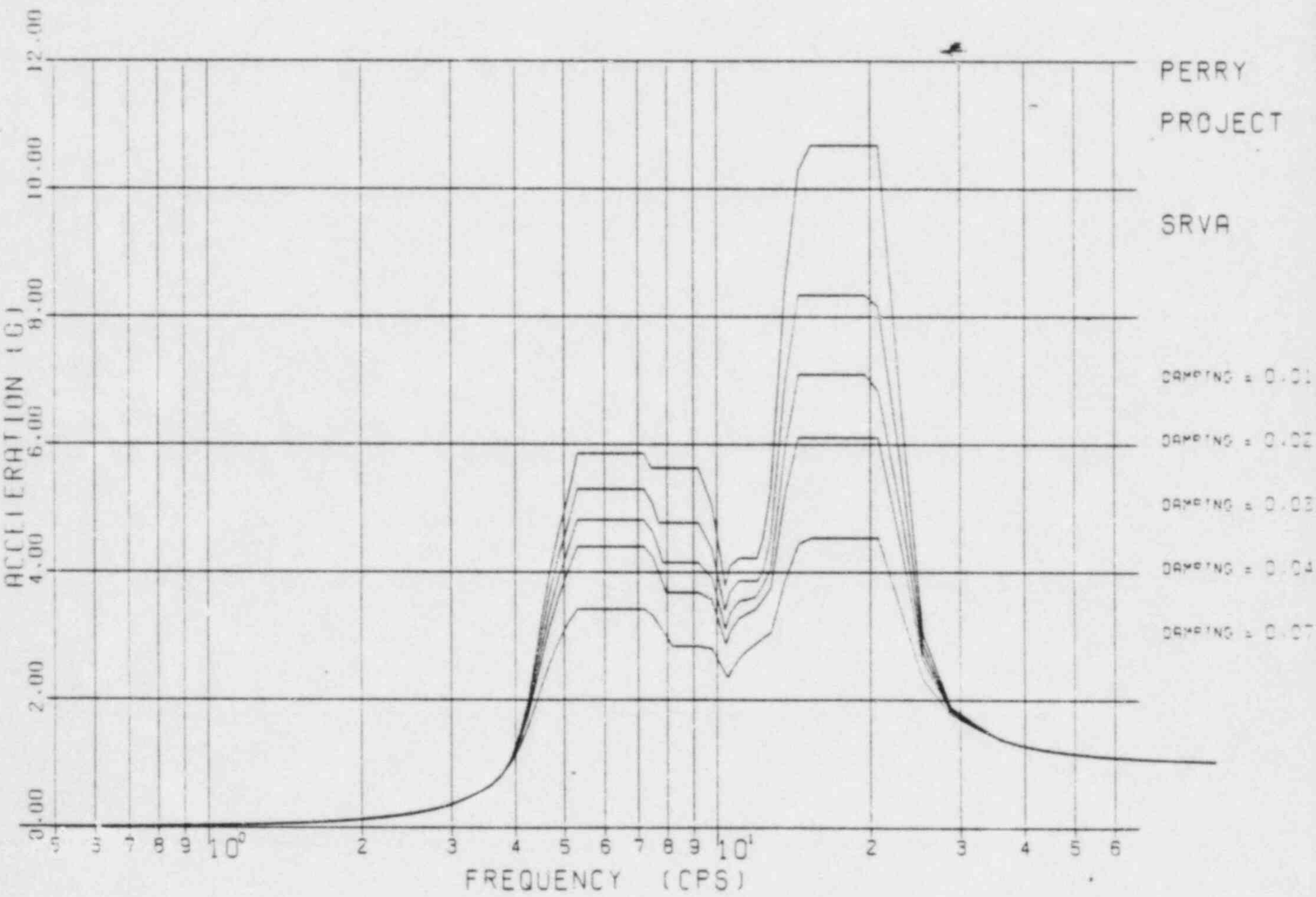


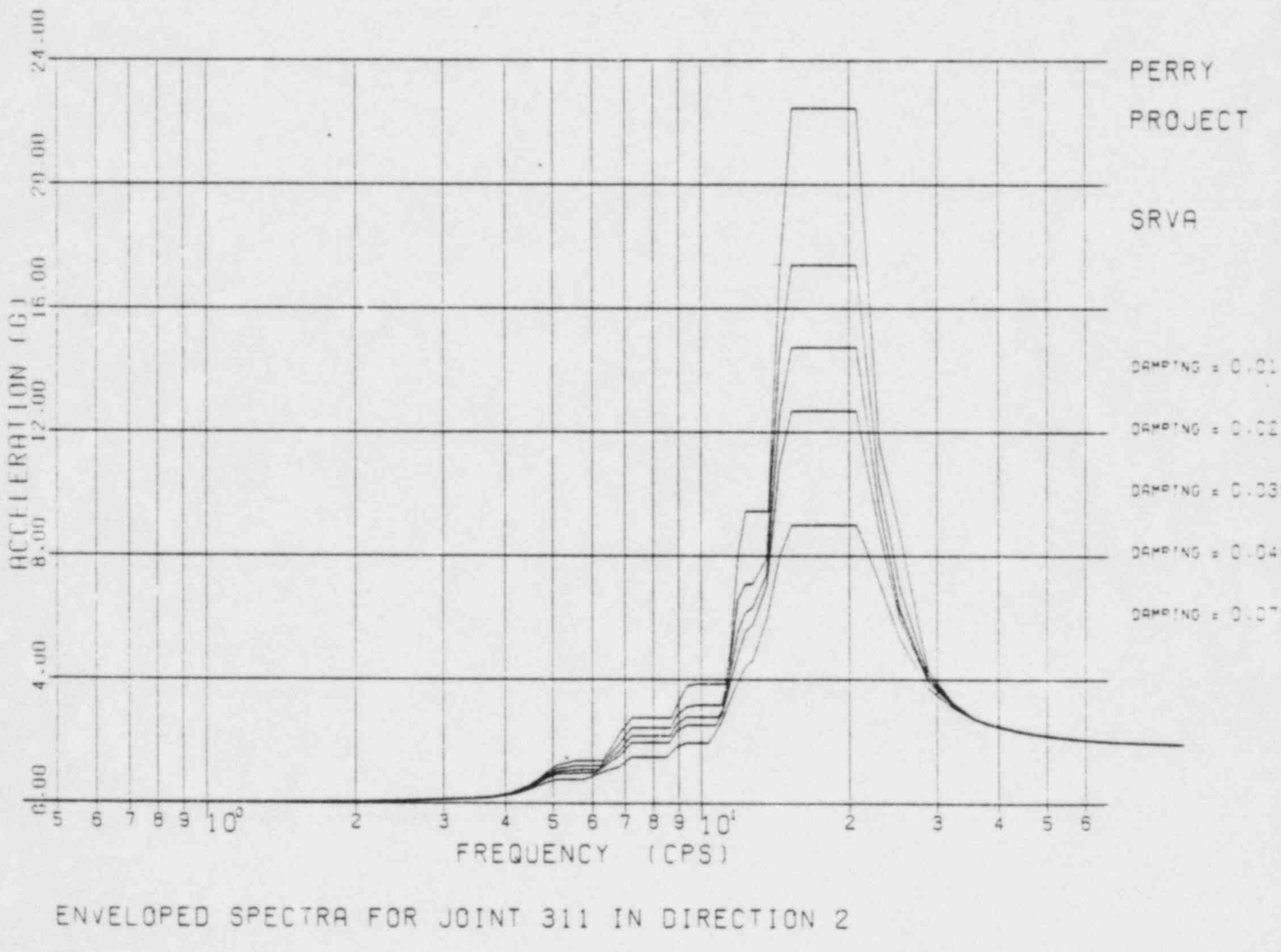
FIGURE 2c TANGENTIAL RESPONSE SPECTRA AT EL. 664'-10" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 1

FOR LOAD CASES 19231, 19321, 19461

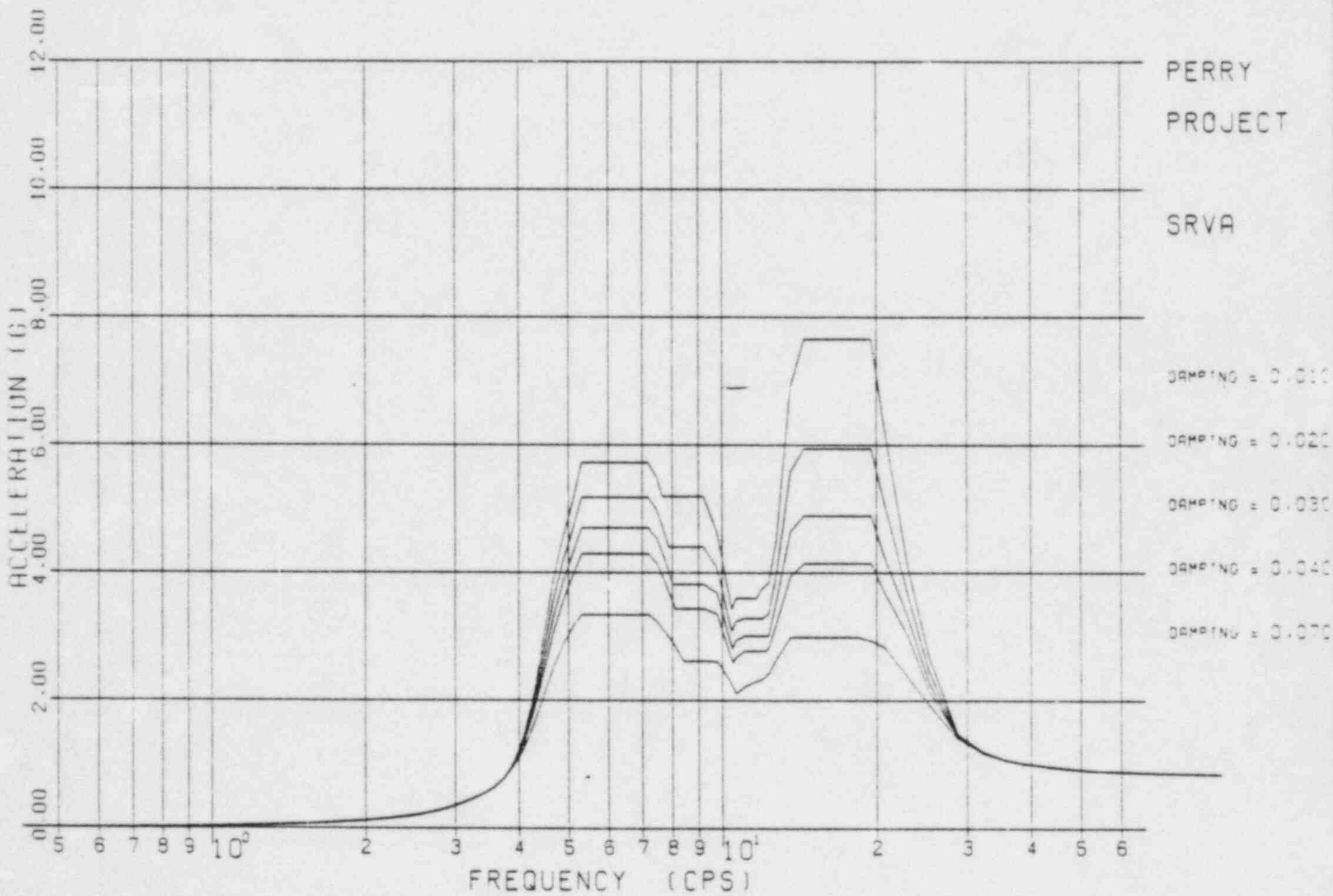
FIGURE 3a RADIAL RESPONSE SPECTRA AT EL. 749'-4" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 2

FOR LOAD CASES 19231, 19321, 19461

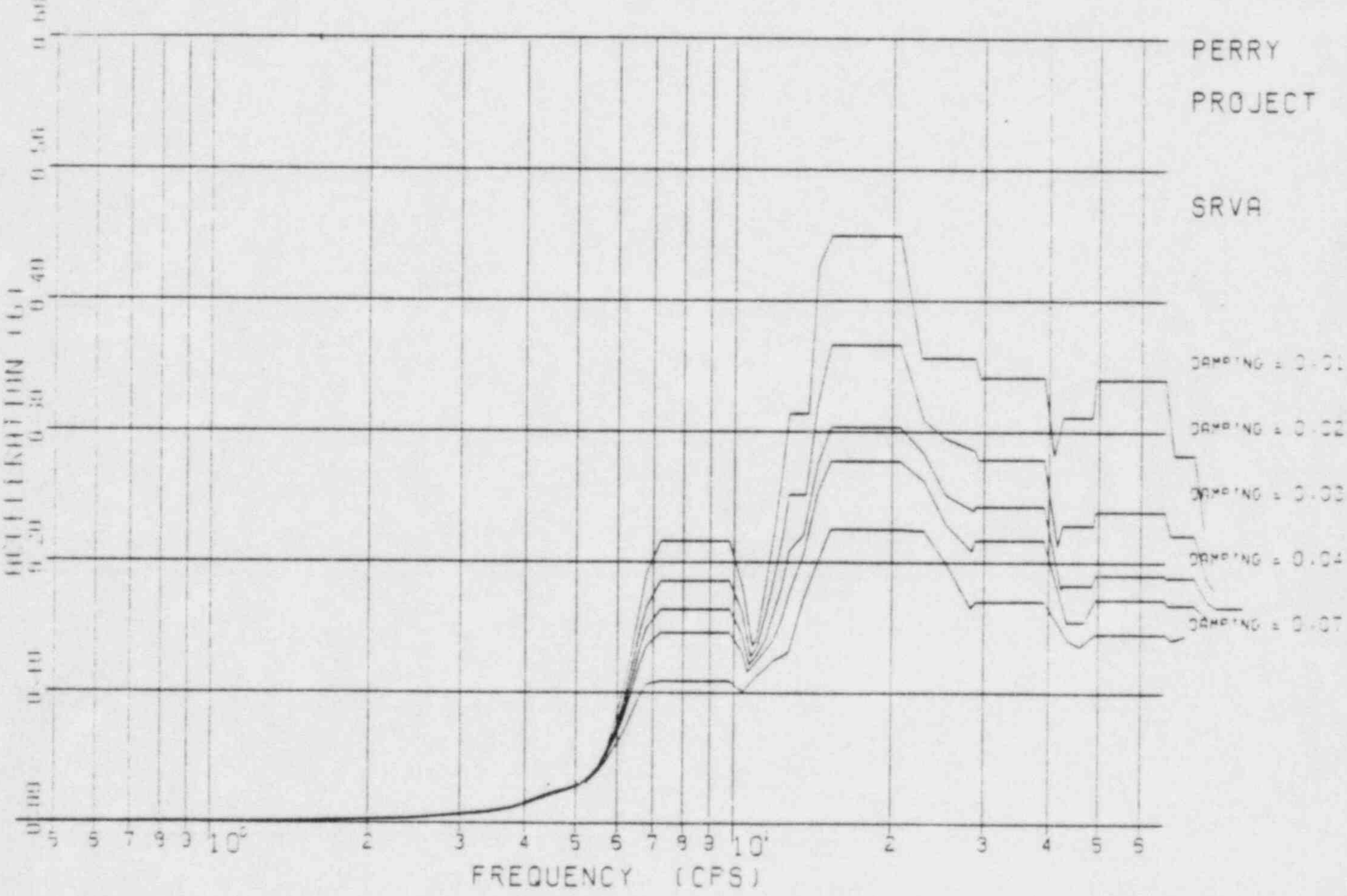
FIGURE 3b VERTICAL RESPONSE SPECTRA AT EL. 749'-4" NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 3

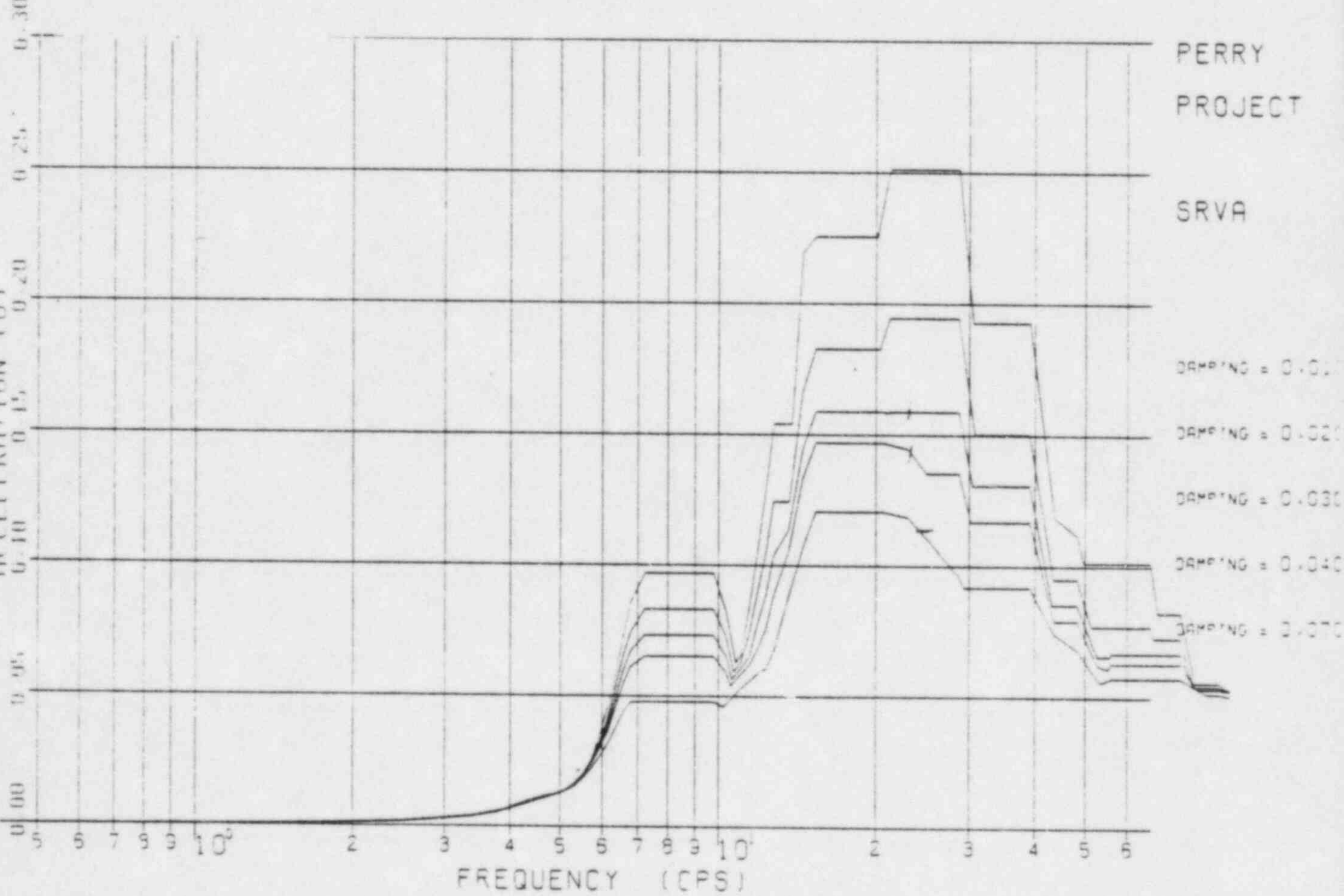
FOR LOAD CASES 19231, 19321, 19461

FIGURE 3c TANGENTIAL RESPONSE SPECTRA AT
EL. 749'-4"
NO ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 1
FOR LOAD CASES 22222

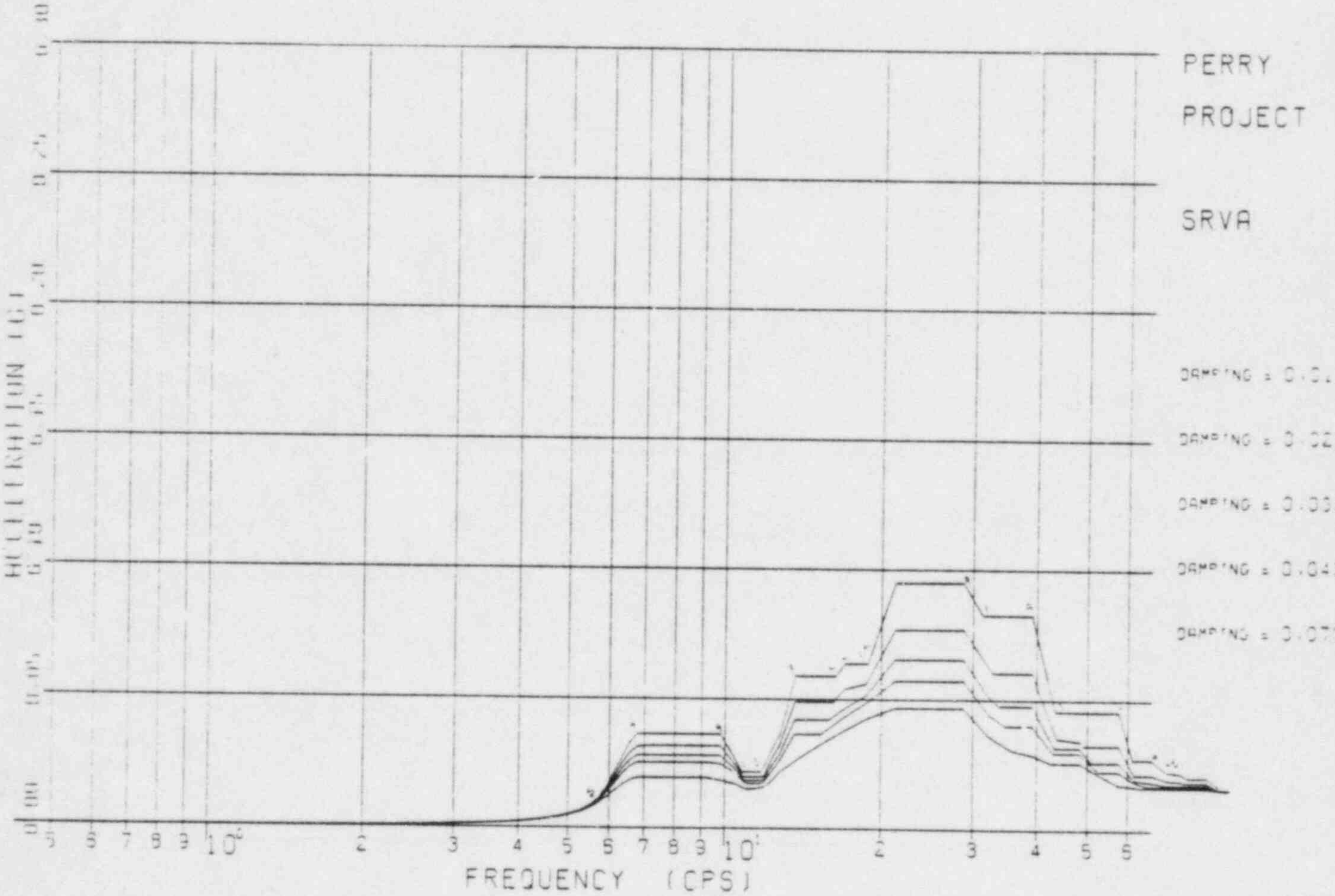
FIGURE 4a RADIAL RESPONSE SPECTRA AT
EL. 579'-5"
ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 2

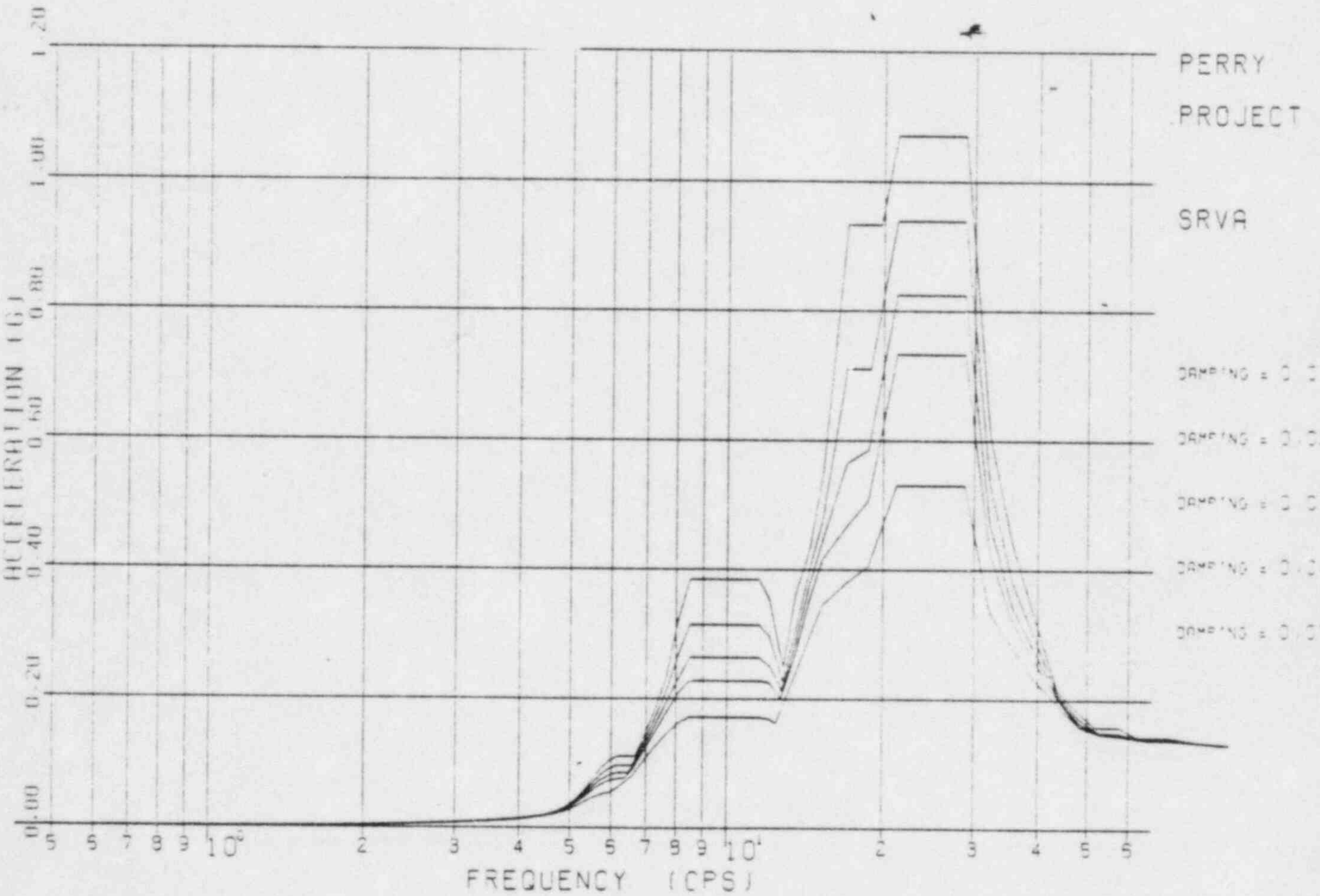
FOR LOAD CASES 22222

FIGURE 4b VERTICAL RESPONSE SPECTRA AT EL. 579'-5" ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 155 IN DIRECTION 3
FOR LOAD CASES 22222

FIGURE 4c TANGENTIAL RESPONSE SPECTRA AT EL. 579'-5" ANNULUS CONCRETE

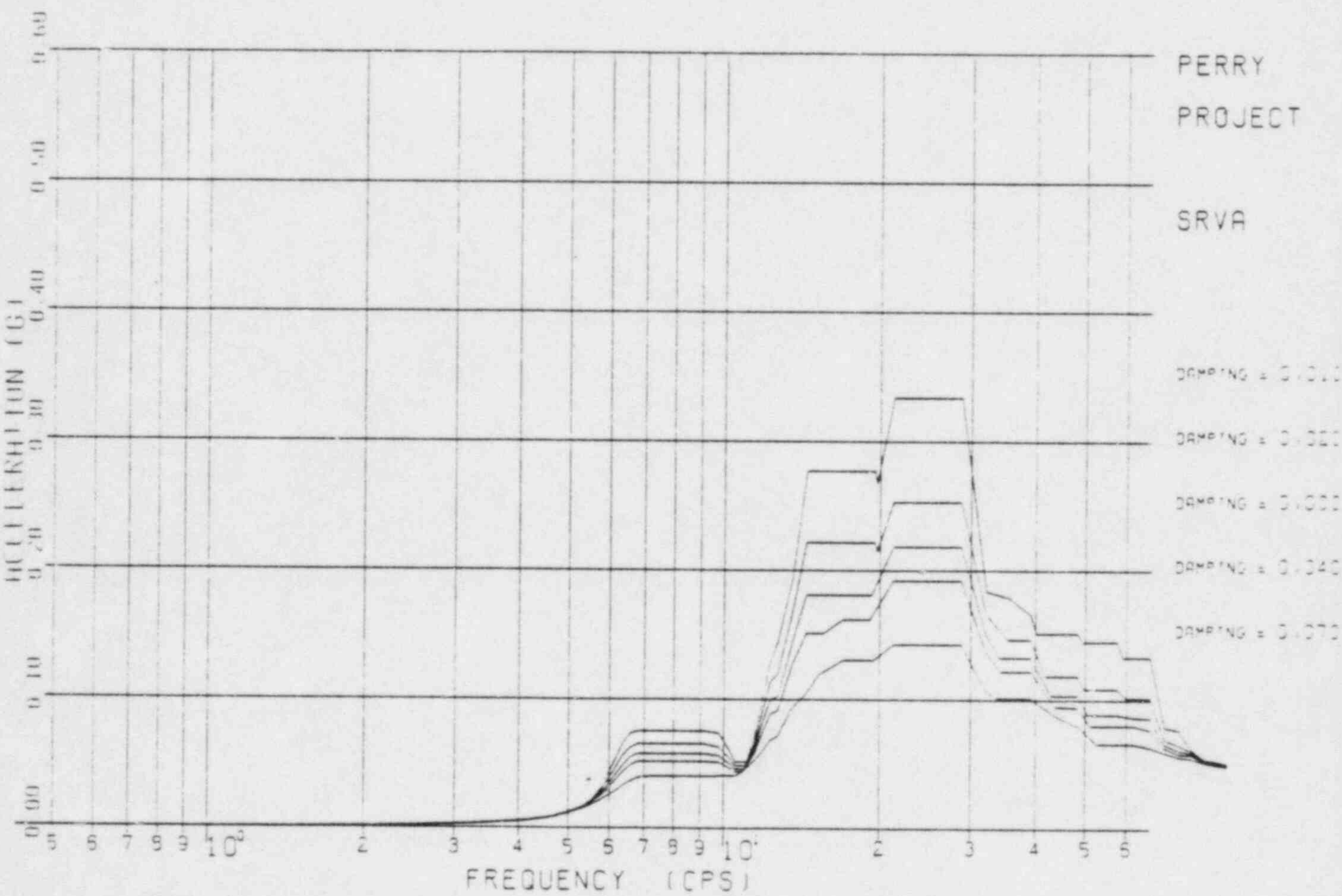


ENVELOPED SPECTRA FOR JOINT 272 IN DIRECTION 1

FOR LOAD CASES 22222

FIGURE 5a

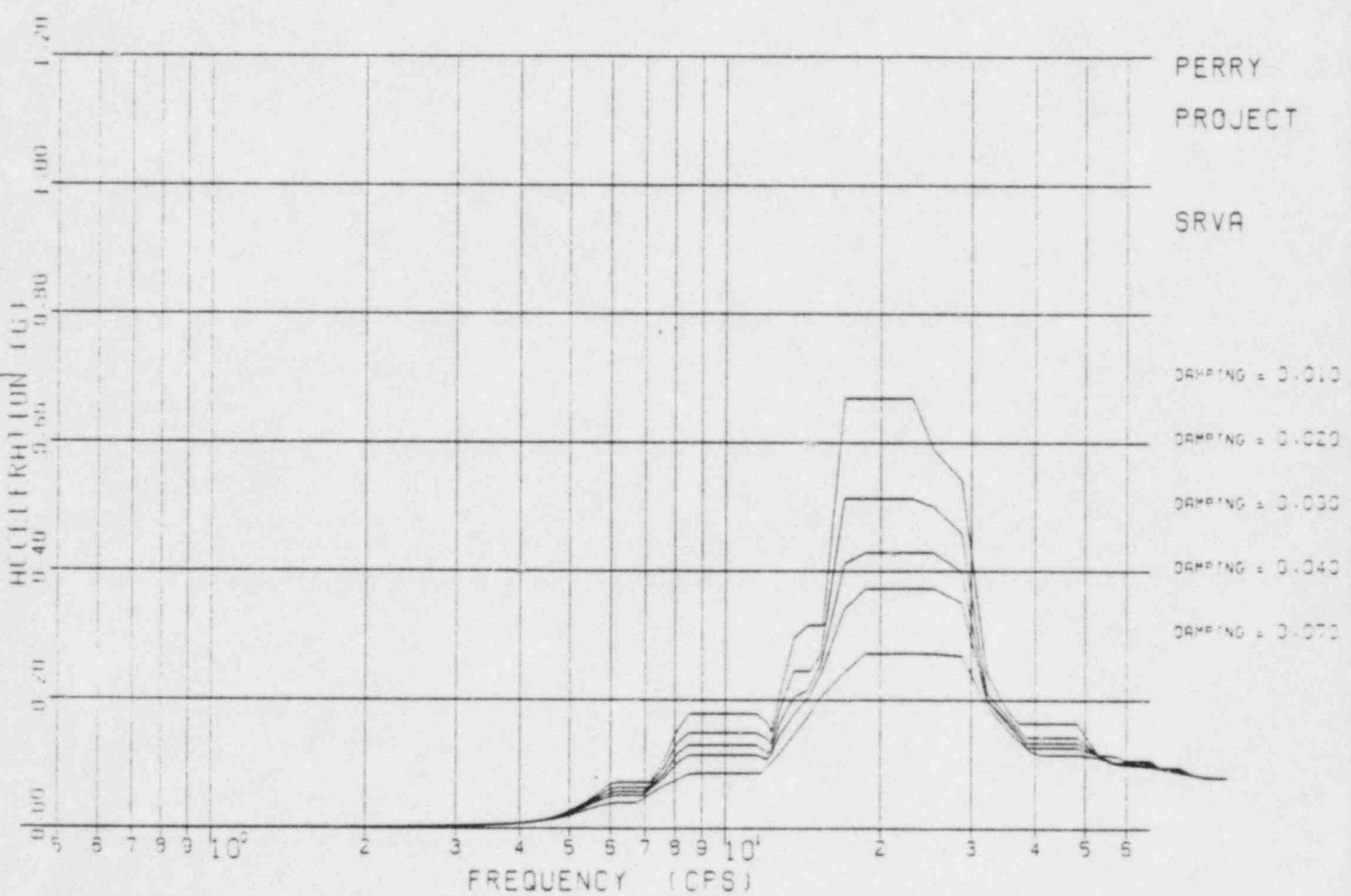
RADIAL RESPONSE SPECTRA AT
EL. 664'-10"
ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 272 IN DIRECTION 2

FOR LOAD CASES 22222

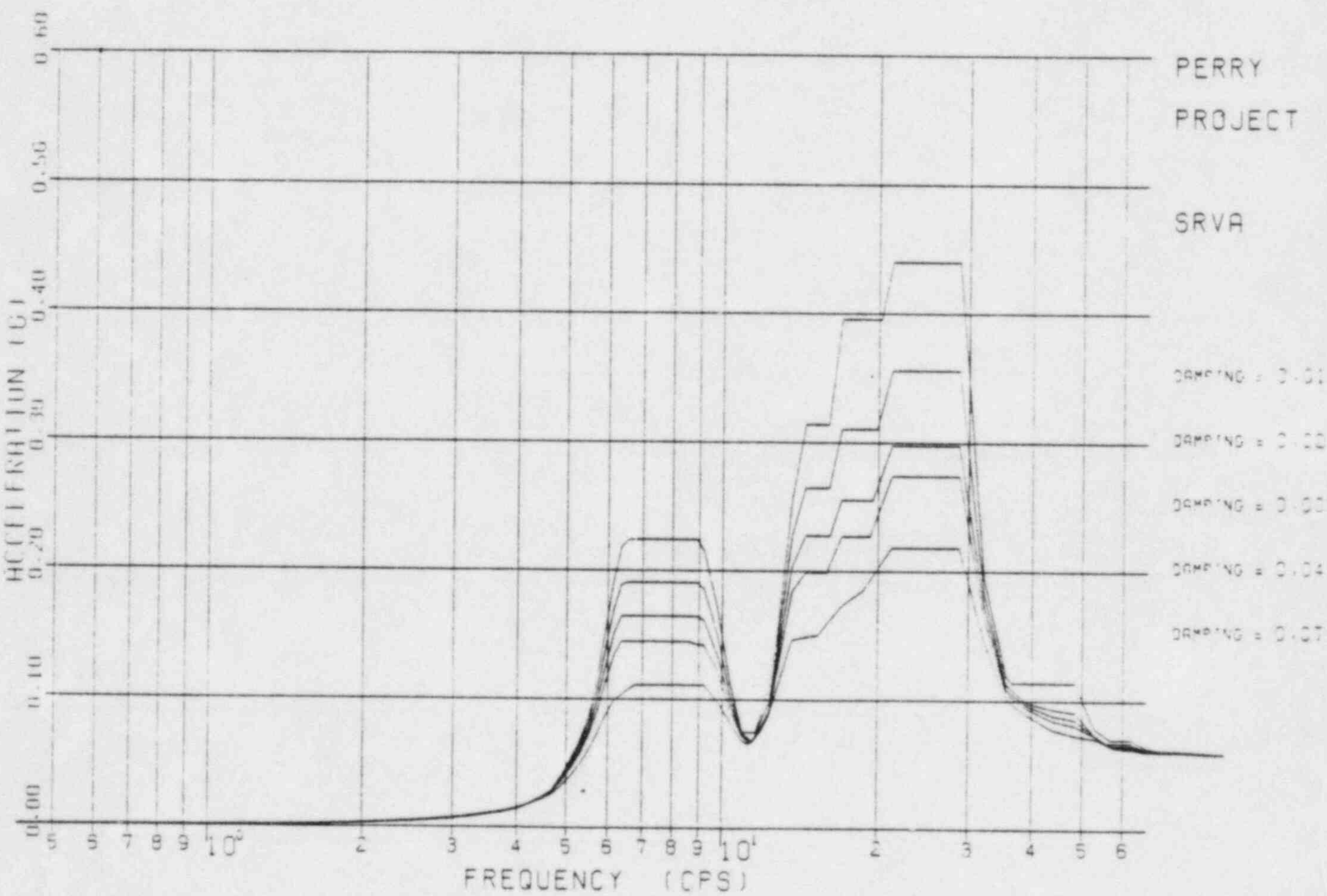
FIGURE 5b VERTICAL RESPONSE SPECTRA AT EL. 664'-10" ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 272 IN DIRECTION 3

FOR LOAD CASES 22222

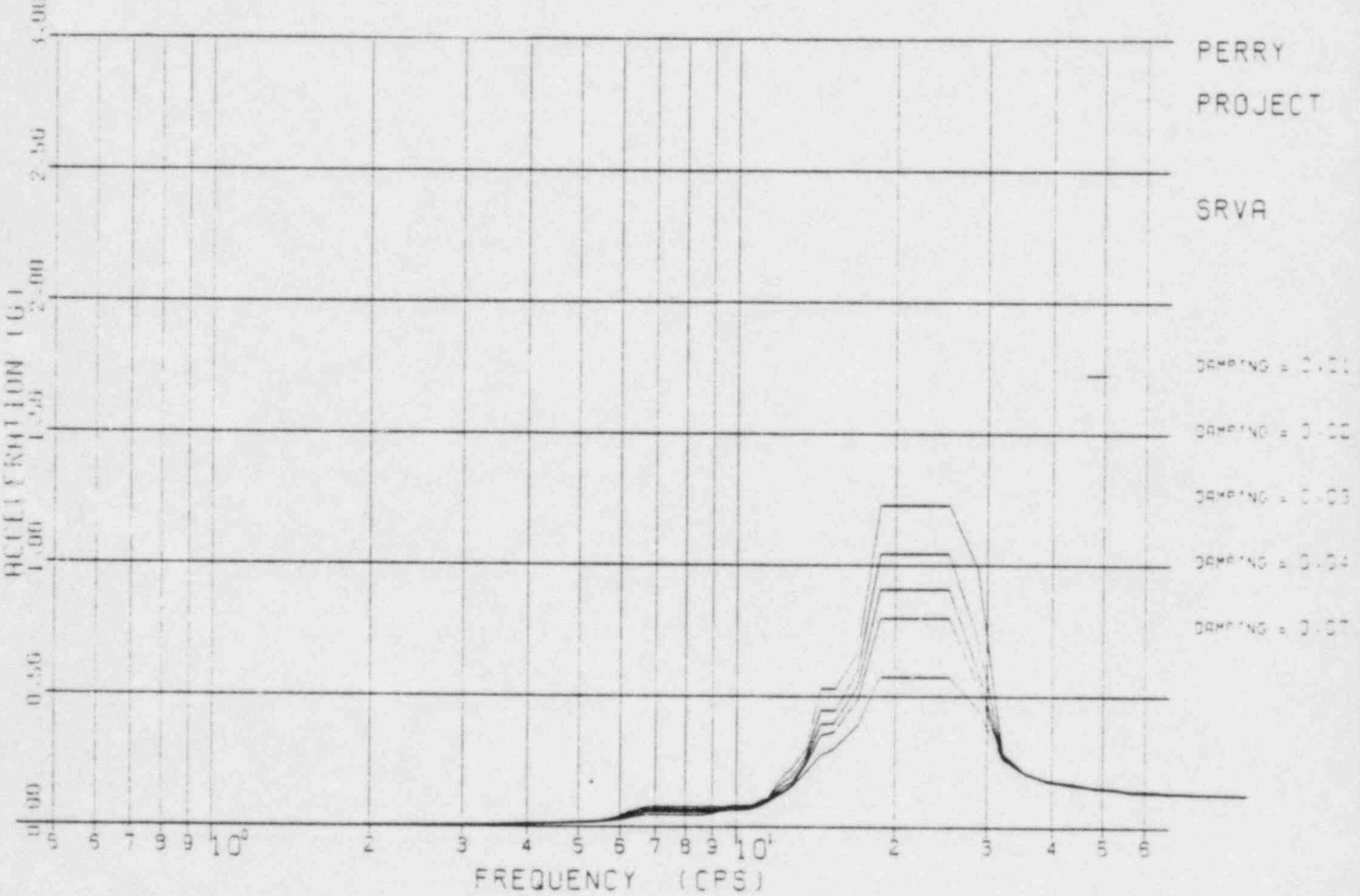
FIGURE 5c TANGENTIAL RESPONSE SPECTRA AT EL. 654'-10" ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 1
FOR LOAD CASES 22222

FIGURE 6a

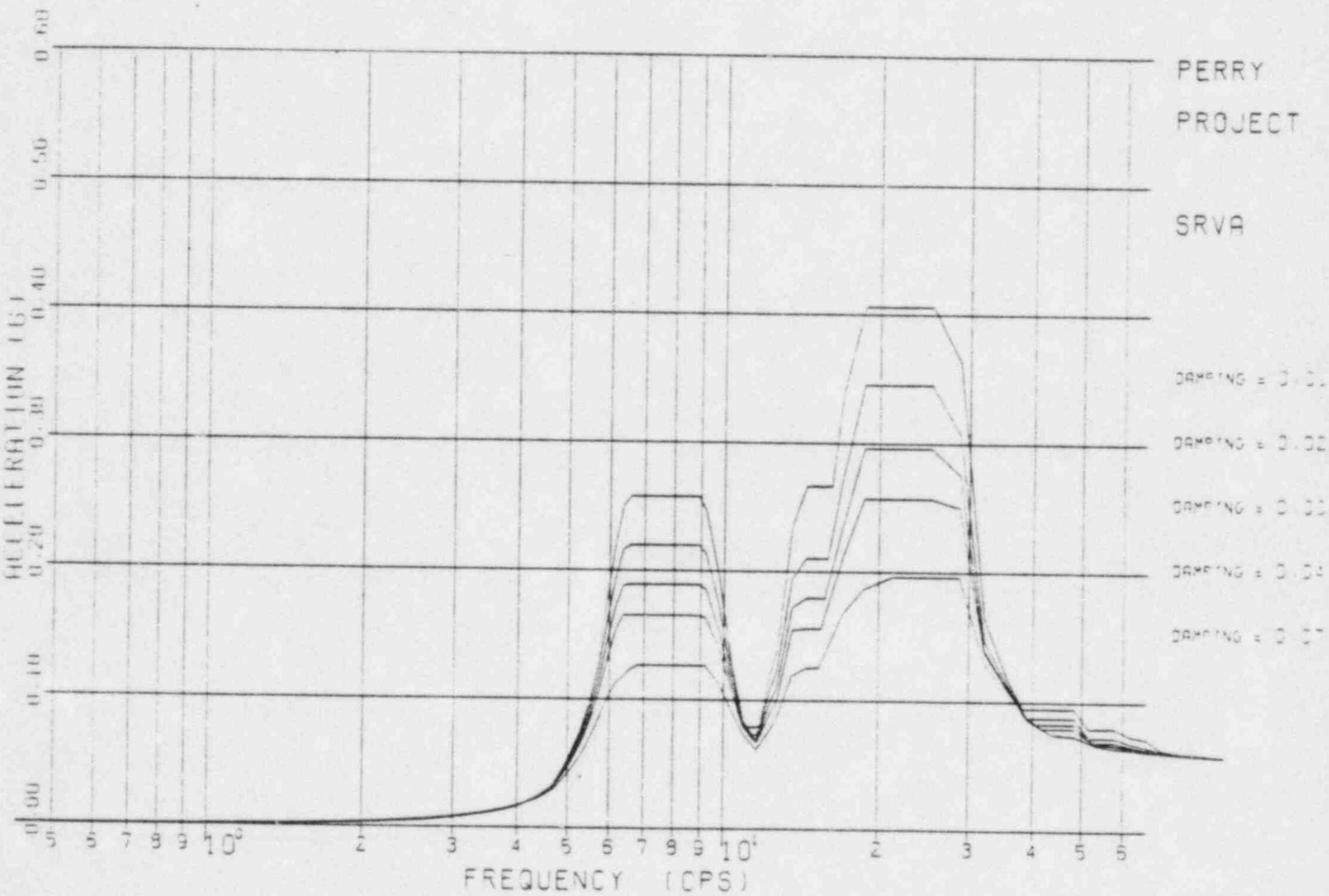
RADIAL RESPONSE SPECTRA AT
EL. 749'-4"
ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 2

FOR LOAD CASES 22222

FIGURE 6b — VERTICAL RESPONSE SPECTRA AT EL. 749'-4" ANNULUS CONCRETE



ENVELOPED SPECTRA FOR JOINT 311 IN DIRECTION 3

FOR LOAD CASES 22222

FIGURE 6c TANGENTIAL RESPONSE SPECTRA AT EL. 749'-4" ANNULUS CONCRETE