Consumers Power Company

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Operating Reactors Branch No 5
Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

DOCKET 50-155 - LICENSE DPR-6 -
BIG ROCK POINT PLANT - SEP TOPIC
III-6, SEISMIC DESIGN CONSIDERATIONS

Consumers Power Company letter dated August 16, 1981 provided the report entitled "Seismic Safety Margin Evaluation - Big Rock Point Nuclear Plant Facilities" (August 1981 Revision) by D'Appolonia Consulting Engineers. This report contained a seismic analysis of major plant structures, primary coolant system (PCS) piping and supports, and portions of other plant systems. NRC letters dated January 19, 1982 and July 27, 1982 requested additional information relating to the seismic review program for Big Rock Point Plant and the D'Appolonia analysis in particular. Consumers Power Company's response to each request, with the exception of Question 5 from the July 27, 1982 letter, is provided in the attachment to this letter. A response to Question 5 will be provided as soon as possible.

A meeting was held between the NRC staff, Consumers Power Company and D'Appolonia Consulting Engineers on August 24, 1982 to discuss the seismic review program for Big Rock Point Plant. During this meeting, considerable time was spent in discussing the D'Appolonia model of the reactor building and, in particular, the modeling assumption which treated the concrete internal structures as a monolithic structure. Consumers Power Company continues to believe that the modeling approach taken by D'Appolonia is appropriate for the structures being evaluated. However, additional justification of this assumption is being prepared and will be submitted to the staff for review in the very near future.

At the same meeting, the subject of soil structure interaction and structural damping values used in the D'Appolonia analysis were also discussed (Reference 6 attached). Our detailed response to these issues is provided in the attachment and summarized here. Consumers Power Company feels that additional studies of variations in soil parameters are not necessary for two reasons: (1) the results of completed studies indicate that overall damping values used

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were conservative; and, (2) the results of completed parametric studies of soil properties for the reactor building indicate that the effects of such variations for the turbine and service building will be small and well within acceptable levels for engineering accuracy. Since the parametric studies indicate that relatively few structural members are overstressed, we have concluded that additional studies are not necessary. Consumers Power Company is committed to perform the necessary structural modifications to correct the overstressed conditions (Reference 2 attached). Although additional analysis may show that the conditions are acceptable, it is our belief that both the plant and public safety are best served by spending available funds to improve the physical plant. In addition, an evaluation of the safety-related components and systems contained within the turbine and service buildings (this includes the control room, the electrical equipment room and portions of the fire system) indicates that safe shutdown of the reactor can be achieved even if these other systems were to fail due to gross structural failures of the related buildings. This evaluation assumes that the PCS piping will remain intact and that necessary operator actions to reach cold shutdown can be performed from outside the control room. Modifications planned as a result of the NRC's fire protection rule (10 CFR 50, Appendix R) are intended to provide the ability to achieve cold shutdown from outside the control room assuming an all-encompassing control room fire. Specifically, we plan to install an alternate shutdown panel (Consumers Power Company March 19, 1981 letter) which will be located adjacent to the post-incident room and will: (1) contain necessary plant process instrumentation; (2) provide capability for remote control of the emergency condenser system; and, (3) contain provisions for restoring PCS makeup in a timely fashion.

Our letter of August 5, 1982 provided a current status of our efforts and a general schedule for the remaining work on the seismic issue. In summary, the program is either on-going or complete in several areas, including: (1) reanalysis of the PCS loop and supports; (2) development of computer models to evaluate piping systems; and, (3) analysis of block walls and mechanical and electrical equipment anchorages. We plan to provide a detailed schedule and scope for all remaining work on SEP Topic III-6 in the near future.

Thomas C. Bordine
for

David J VandeWalle
Nuclear Licensing Administrator

CC Administrator, Region III, USNRC
NRC Resident Inspector-Big Rock Point

Attachment

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ATTACHMENT

Consumers Power Company
Big Rock Point Plant
Docket 50-155

Response to NRC Letters
of January 19, 1982 and
July 27, 1982 Regarding
the Big Rock Point Seismic
Review Program (SEP Topic III-6)

September 20, 1982

70 pages

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QUESTION NRC-1: Provide your responses to the January 19, 1982 NRC letter requesting additional information.

RESPONSE:

The responses to the January 19, 1982 letter are submitted herewith. A total of 11 comments or items were included by the reviewer (Lawrence Livermore Laboratory). For clarity, these items have been designated by LLL-n, where n is the item number in the list. The questions posed by the USNRC in their letter of July 27, 1982 are marked as NRC-n.

Of the 11 comments slated in the letter of January 19, 1982, Comments LLL-3 and LLL-6 require no response as they are general comments on the part of the reviewer. Therefore, no response to these items has been provided.

QUESTION NRC-2: Wherever overstresses were predicted in the August 1981 D'Appolonia structural analysis, identify all such areas and/or items, and provide the details of your disposition and resolution of each. Specifically address any assumed load redistribution and its effect on your analyses results.

RESPONSE:

The seismic safety margin evaluation of the Big Rock Point plant structures has indicated the following deficiencies with respect to the structure's capability to withstand an earthquake having a zero period ground acceleration of 0.12g and matching the USNRC Regulatory Guide 1.60 response spectrum:

- Two steel columns (Column Nos. E_A-8 and D_A-7) have been judged to have marginal buckling capacities in the passageway area.

- A stress concentration probably exists in the Primary Coolant Loop (PCL) at the junctions of 4-inch diameter cross ties and the 24-inch diameter downcomer. Possible undesirable displacements are also present in this area.

- The potential for uplift at some of the steel column bases exists between the steel column and its base plate. This uplift is small and can generally be resisted by the existing footing and base plate connections. However, for Column Nos. J-1 and H-1 in the service building, the uplift force is significant and under the stipulated loading conditions may lead to failure at their column bases.

- Overstressing of some of the bracing in the northeast corner of the service building has been predicted.

For Column Nos. E_A-8 and D_A-7, the absence of any lateral bracing to resist lateral movements has been judged to be the cause of marginal factors of safety. Lateral bracing will be provided in this area.

For the potential stress concentration problem in the PCL, a reanalysis is currently planned using the SEP site-specific spectrum. On the bases of the results of the previous analysis, no overstress with respect to supports is anticipated. If, in the reanalysis, overstress is determined to occur in the 4-inch cross-tie the significance of the condition will be reviewed and modification will be made if deemed necessary.

For potential uplift at Column Nos. J-1 and H-1 and the overstressed bracing in the northeast corner of the service building, Consumers Power believes that the most cost-effective solution will be upgrading of the deficient structural elements. Upgrading will provide the required tensile and compressive capacities for the bracing in the northeast corner of the service building. To eliminate the potential uplift at Columns J-1 and H-1, additional bracing between Columns G-1 and H-1 will be provided to distribute equally the loads calculated to act on the bracing between Column Lines J-1 and H-1. This will eliminate the uplift conditions which were calculated to exist beneath Columns J-1 and H-1.

The additional lateral bracing provided in the passageway area will result in the modification of the floor response spectra presently shown in D'Appolonia's report. The effect of other structural modifications on the response of the structure is considered to be insignificant, as the modifications will produce insignificant changes in the overall stiffness and mass of the structure complex.

QUESTION NRC-3: Provide the Addendum to the August 1981 D'Appolonia reports.

RESPONSE:

The Addendum to the D'Appolonia report was developed by D'Appolonia at the request of Consumers Power. This Addendum is titled as a Commentary and is aimed primarily for informal explanations of D'Appolonia's report to Consumers Power personnel who were unfamiliar with seismic evaluation studies. Consumers Power believes that this Commentary will provide minimal additional information beyond that already submitted to the NRC and is, in no way, contradictory to the results and conclusions of the report. Therefore, Consumers Power does not plan to provide this Commentary to the NRC.

QUESTION NRC-4: Identify any analyses which will be performed using the SEP Site Specific Spectrum for Big Rock Point and provide the criteria and method to be employed, and corresponding justification thereof.

RESPONSE:

It is the intent of CPCo to use the SEP Site Specific Spectra to generate amplified response spectra wherever a response spectra is expected to be used in the future. More specifically, it is expected that Site Specific Spectra will be generated at all locations where spectra were previously generated from the Regulatory Guide 1.60 earthquake. Additional amplified spectra will be generated on the primary coolant loop, steam drum and reactor vessel.

It is not anticipated that any stress analysis of structures previously evaluated by the Regulatory Guide earthquake will be re-evaluated by the Site Specific Spectra. However, the primary coolant loop will be re-analyzed with a time history associated with the site specific spectra. It is expected that piping, pipe supports, mechanical equipment and electrical equipment will be evaluated with the site specific spectra. The masonry wall analysis has utilized the site specific spectra as has the analysis of the control room panels which were evaluated under the auspices of IE Information Notice 80-21.

The methodologies associated with the generation of amplified response spectra will be consistant with those employed in development of spectra from the Regulatory Guide earthquake. The structural damping employed in amplified spectra development will reflect an awareness of the overall stress levels determined from the stress analyses already performed from the Regulatory Guide earthquake.

QUESTION NRC-6: Soil springs have not been varied in accordance with SEP guidelines, and your bases for the higher structural damping values used in your analyses (given that high stresses in structures are local) have not been adequately justified. You should demonstrate, using (1) a ±50 percent variation in shear modulus; (2) the SEP Site Specific Spectrum; (3) rigorously justified corresponding levels of assumed structural damping, that the August 1981 D'Appolonia results are conservative for the Reactor and Turbine Buildings. Parametric studies of these phenomena for these two structures may be used. Where this approach is used and results are presented at various points in these structures, comparative graphs using identical scales should be employed or the results should be plotted on the same graph. You may extend your conclusions drawn for these studies to other structures. In addition, you should quantify the effect of including uncertainties (e.g., floor spectra peak broadening) on the D'Appolonia Reactor Coolant Loop analysis.

RESPONSE:

The question consists of four parts which may be broken down as follows:

- Adequacy of analysis with respect to the intent of SEP guidelines on soil-structure interaction as regards possible effects of ±50 percent variation in soil shear modulus;
- Use of SEP site-specific spectrum as opposed to the spectrum used in our analysis;
- Adequacy of analysis with respect to the structural damping values used in the evaluation of stresses and the corresponding structural damping levels which have been used to develop the floor response spectra at various locations of the structures; and
- Quantification of effects of uncertainties in the analysis of the Primary Coolant Loop (PCL).

Background Information

At the time of initiation of the SEP at Big Rock, the following objectives were mutually agreed upon (Reference: letter to Mr. David A. Bixel of Consumers Power from Mr. Victor Stello, Jr., USNRC, dated December 1, 1977) by Consumers Power and the USNRC relative to the scope of the SEP:

1. Reassess the safety margins of the design and operation of selected older operating nuclear power plants.

2. Establish documentation which shows how each operating plant reviewed in the SEP compares with current criteria on significant safety considerations, and which provides a basis for acceptance of any departures from these criteria.

3. Provide the capability to make integrated and balanced decisions with respect to any required safety improvements.

4. Identify and resolve significant safety deficiencies early in the SEP, if such deficiencies exist.

5. Efficiently use available personnel and minimize NRC and licensee resource requirements to perform the SEP.

On January 15, 1979, USNRC directed Consumers Power to develop documentation for seismic design basis (Reference: letter to Mr. David A. Bixel , Consumers Power from Mr. Victor Stello, USNRC). Accordingly, work on this documentation was immediately started. Prior to actual start of the analytical work, a meeting between Consumers Power, its consultant, and the USNRC was held in Bethesda, Maryland on July 26, 1979. In that meeting, USNRC directed that the design criteria as embodied in NUREG/CR-0098 (Newmark and Hall, 1978) be considered as the governing criteria. On the topic of soil-structure interaction, Newmark and Hall had cited DAP-TOP-1 developed by D'Appolonia[1] as a reference document.

About 18 months later, in December 1980, the analytical work was completed. About the same time, the SSRT recommendations, with respect to soil-structure interaction, were provided to us as SEP guidelines on this subject. In other words, lacking specific guidelines and criteria

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[1] D'Appolonia Consulting Engineers, Inc., 1975, "Soil-Structure Interaction for Nuclear Power Plants," Report DAP-TOP-1, May.

from USNRC, but following the overall spirit and intent of SEP as outlined in the above quoted letter, Consumers Power proceeded, in good faith, along a conservative, rational path, using criteria and guidelines recommended by consultants prominent in the field who, in fact, were the authors of the Topical Report cited by Newmark and Hall.

A brief summary comparison of the main input parameters to the SEP seismic analysis effort conducted by Consumers Power is given in the following:

COMPARISON OF ANALYSIS CRITERIA

SUBJECT	ANALYSIS CRITERIA	
	CONSUMERS CRITERIA (July 1979)	USNRC CRITERIA (December 1980, and later requests)
Safe Shutdown Earthquake	0.12g	0.105g
Response Spectra	R.G. 1.60	Site Specific but lower than R.G. 1.60
Structural Damping (for Stress Analyses)	R.G. 1.61 Concrete 7% Welded Steel 4% for Reactor Bldg. Concrete 10% Bolted Steel 10% for other Bldgs.	Concrete 3 to 5% Welded Steel 2 to 3% Concrete 3 to 5% Bolted Steel 5 to 7%
Soil Damping (for Stress Analyses)	for Reactor Bldg. 50% of calculated values--all modes (typically 11 to 35%) for other structures 10% maximum	75% of calculated damping in translational modes and 100% of calculated damping in rotational modes. (typically 18 to 50%)
Structural Damping (for Floor Response Spectra Generation)	for Reactor Bldg. 7% for Concrete 4% for Welded Steel for other structures 7% for both Concrete and Bolted Steel	Concrete 3 to 5% Welded Steel 2 to 3% Bolted Steel 5 to 7%
Soil Damping (for Floor Response Spectra Generation)	for Reactor Bldg. same criteria as used in the stress analysis. For all other structures, 7% maximum	Same as Above
Soil Shear Modulus Variation	±50% (Reactor Bldg.* only using R.G. 1.60 spectra and 0.12g)	±50% (Reactor and Turbine Bldgs.)

*Adopted April 1982.

The subsequent text discusses the most significant differences in criteria described above as well as a summary of our findings. We wish to point out that, on April 5, 1982 in a telephone conversation with Consumers Power, its consultants, and the USNRC, Consumers Power was directed to demonstrate that the floor response spectra developed in the seismic safety margin evaluations include adequate conservatism such that the use of the floor response spectra in the subsequent analyses of subsystems (e.g., equipment) can be properly justified with respect to variations in soil parameters and structural damping within levels of engineering accuracy. In spite of the fact that a great deal of conservatism had been used in the original analysis, as evident in the above summary, Consumers Power Company provided such justifications in a report, "Parametric Study, Soil-Structure Interaction, Big Rock Point Nuclear Power Plant," prepared by D'Appolonia Consulting Engineers, Inc., in April 1982.

Shear Modulus Parametric Studies

Generally speaking, Consumers Power acknowledges the potential value of considering parametric variations of soil shear modulus for "deep soil sites," particularly if the soils are soft. However, the Big Rock Point site is not a "deep soil site." The subsurface material beneath the foundation consists primarily of 20 to 30 feet of glacial till overlying a limestone formation. The shear wave velocities in the till range from 1,200 feet per second at the top of the layer to 2,700 feet at the interface of the till and limestone. These velocities indicate a shallow, highly competent soil layer with properties significantly stiffer than most soil sites. In fact, many soft rocks have shear wave velocities in this range. The shear wave velocity in the underlying limestone ranges from approximately 3,300 feet per second in the broken zones to about 7,000 feet per second in the competent zone. The overall effect, if significant, of varying soil parameters at such a site will be most predominant on massive concrete structures, such as the reactor building.

In the study requested by th[illegible]C on April 5, 1982, a set of parametric studies was conducte[illegible]-50 percent variations in soil modulus.

It is believed that even if one can justify any parametric studies, a more narrow banded variation, approximately ±20 percent variation in the shear modulus, is appropriate. However, the ±50 percent variation in the soil shear modulus to define the upper and lower bounds of the measured values was employed in the analysis.

The results of this parametric study examined at three significant levels of the structures indicated that, even due to ±50 percent variation in soil shear modulus, the recommended floor response spectra are adequately conservative within normal levels of engineering accuracy. Specifically, we found that the modal frequencies did not vary by more than 10 percent in the first five modes of the structure extending to a frequency of 17 Hertz.

Structural Damping

The following summary compares the structural damping values used by Consumers, those recommended by Newmark and Hall (1978) at yield and at one-half yield, and the corresponding SSE values as specified in USNRC Regulatory Guide 1.61 (1973).

COMPARISON OF DAMPING (PERCENT OF CRITICAL*) VALUES

	Consumers Criteria (July 1979)		Newmark and Hall		USNRC R.G. 1.61 (percent)
	Reactor Building (percent)	Other Buildings (percent)	At Yield (percent)	At 1/2-Yield (percent)	
Concrete					
Stress Analysis	7	10	7 to 10	2 to 5	7
Floor Response Spectra	7	7			
Welded Steel					
Stress Analysis	4	-	5 to 7	2 to 3	4
Floor Response Spectra	4	-			
Bolted Steel					
Stress Analysis	-	10	10 to 15	5 to 7	7
Floor Response Spectra	-	7			

*See previous table for soil damping values.

The generally higher values as recommended by Newmark and Hall for SEP analysis are in recognition of certain nonlinearities which are not rigorously considered in linear elastic analyses (Levin , 1980).[2] Thus, for active components and other deformation limited items, increased damping levels are considered to more realistically account for energy absorption in the context of overall linear response.

The reactor building analysis conducted in April 1982 was done using a time-history analysis technique. In this analytical model, damping in soil was included explicitly along with structural damping estimated in accordance with the recommendations in Regulatory Guide 1.61. As detailed in the "parametric study" report, the soil damping was calculated in an extremely conservative manner when such values are compared with those calculated in accordance with the SSRT recommendations. Thus, parametric studies conducted using the reactor building analytical model for variations in structural damping along with the more liberal values of soil damping as recommended by the SSRT are conservative even for very low values of structural damping.

Effects of Parametric Studies on Turbine Building Analysis

The turbine building is a widespread structure requiring a highly complex structural analysis involving a number of substructures. Parts of this structure are primarily reinforced concrete while other parts are steel framed. Soil-structure interaction at the bases of steel column footings is negligible and, therefore, was not included in the basic analysis. Correspondingly, their damping effects were also excluded. For the concrete portions of the structure, soil springs were calculated and were included as part of the analytical model in the basic analysis. However, explicit incorporation of soil damping at the base of these concrete structures was not practical due to the complex model required for this analysis. Therefore, a conservative approximation was made whereby concrete portions of the structure were assumed to have the same overall structural damping as the steel portions of the structure and

[2] Levin , Howard, A., 1980, "NRC Systematic Evaluation Program, Seismic Review.

soil damping was also taken to be the same as assumed for steel. This approximation is exceptionally conservative, especially since the effects of soil damping will impart additional damping to the overall structure damping. The results of the reactor building analysis also indicate that such an assumption is conservative as the effects of radiation and material damping will develop lower response to the massive parts of the turbine building complex. Finally, because the parametric study on variations of soil-structure interaction on the reactor building (the most massive building in the plant) indicated that a variation of ±50 percent on the soil modulus led to negligible variation in the response, it follows that the same conclusion applies to the turbine building.

The analyses already performed are adequate for making integrated and balanced decisions with respect to required safety improvements for the reactor and turbine buildings. Additional analyses are not in conformance with the original objectives of the SEP whereby minimization of the licensee's resource requirements is a consideration. Consumers Power contends that adequate parametric variations have been conducted and no additional analyses are required because:

- The results of completed parametric studies utilize structural damping values which are already conservative.
- The results of completed parametric studies for the reactor building, a building which maximizes amplification effects of parametric variation, indicate that the modal frequencies did not change by more than 10 percent for modal frequencies in the first five fundamental modes of the structure extending to a frequency level of 17 Hertz.

Effects of Uncertainties in Primary Coolant Loop Analysis

The PCL is currently undergoing a reanalysis. The analysis already completed for the PCL indicates that except for the four-inch cross-tie, the stress levels in the piping system are significantly lower than the allowable stress for SSE condition. These stress levels are expected to be reduced significantly due to site-specific earthquake input to be used for this analysis. Following performance of this analysis, the stress levels in the piping system will be reexamined.

QUESTION NRC-7: Provide the details of your evaluations of the acceptability of those members for which AISC column and beam buckling criteria (both local and gross) were exceeded taking credit for no increase in the normal AISC limits without the 1.33 increase for considering earthquake loads.

RESPONSE:

Evaluation of Column Stability

The analytical basis for checking stability of columns under combined bending and axial loading conditions is presented in Section 5.5.1.2 of the "Seismic Safety Margin Evaluation" Report, Vol. I, Review 1. The general philosophy is presented below followed by a summary discussion of results.

For the design of columns under combined bending and axial load, AISC recommends three equations to evaluate interaction effects; the three equations may be separated into two groups. In the first group where the axial compression is significant, two equations are to be simultaneously satisfied to account for secondary moment magnification effects and to satisfy the yield criterion. In the second group, where the axial compression is relatively small, secondary effects are neglected. According to the AISC code, the column is adequate when the sum of the ratios included in the interaction equations is less than or equal to unity. Relative to safety, this corresponds to a minimum factor of safety against failure approximately equal to 1.67 (Salmon and Johnson, 1971)[1]. Thus, as in accordance with the Standard Review Plan, Section 3.8.4, an increase in the allowable stresses against buckling by a factor of 1.6 may be permitted under SSE conditions. However, most of the columns that were analyzed were associated with an interaction factor less than unity, which was computed using no increase in allowable stresses for combined dead load and SSE conditions. Further, it is recognized that the stress analysis based on

[1] Salmon, C. G. and J. E. Johnson, 1971, Steel Structures - Design and Behavior, Intext Educational Publishers.

the selection criteria of structural members as presented earlier does not include all structural columns. Therefore, a computed interaction factor greater than unity has been treated with caution. Whenever an interaction factor greater than unity has been obtained, the behavior of the column has been investigated under dynamic lateral loading conditions with respect to its frequency response at various natural frequencies of the structure.

If, on the basis of the foregoing analyses, it is concluded that the higher interaction factor has been obtained because of inadequate (or absence of) lateral restraint, the column has been defined to have marginal safety on the basis of prudent engineering judgment.

Table NRC-7-1 presents the summary results of the columns analyzed. As may be seen in Table NRC-7-1, the maximum value of the interaction factor obtained without any increase in the allowable stresses due to earthquake loading conditions was obtained for Column E_A-8. On the basis of inspection of the column geometry and the floor response spectra obtained at this location, Column E_A-8 was judged to have marginal safety because of the absence of any lateral bracing in the area.

For all other columns, the interaction factors obtained using similar procedures were less than one. Therefore, the columns were considered to be adequate.

Evaluation of Beam Stability

The beams of the following structures were checked:

- Fuel Cask Loading Dock - The crane girder was specially analyzed because of the possible effect of acceleration level on the stress capacity of the girder. In this analysis the allowable stress was taken to be 0.66 F_y - 22 ksi, in accordance with the AISC code stipulations with respect to buckling in compression flange.

- Screenhouse - The crane girder was checked to determine the acceleration level that would be required to develop stresses in the beam equal to the allowable stress. The allowable stress for bending was 20 ksi; and the allowable net dynamic factor was 2.34 after accounting for gravity and frequency effects.

- Passageway - One beam in the passageway was checked. The allowable stress used was 0.6 F_y.

Beams were checked primarily for critical structural elements, such as a crane girder, or at a critical location of a structure, e.g., passageway in the sercvice building. Table NRC-7-1 provides summary of allowable stresses used for three beams checked in the analysis. As may be seen, the beams are adequate without the need for increasing the allowable stresses given in the AISC code for static loading conditions.

TABLE NRC-7-1

SUMMARY RESULTS
STEEL COLUMN ANALYSIS

BUILDING	COLUMN	MAXIMUM INTERACTION FACTOR	LOCATION
Turbine	A-1	0.841	Column Line A-1
	A-6	0.331	Column Line A-6
Service	J-1	0.81	Column Line J-1
	J-6	0.27	Column Line J-6
	E_A-8	1.3	Column Line E_A-8 in the Passageway (cable penetration room)
Fuel Cask Loading Dock	3-A	0.476	Column Line 3-A
Sphere Ventilation Room	A-1	0.15	Column Line 1-A
	B-1	<0.15	Column Line 1-B
Intake Structure	-	0.99	

TABLE NRC-7-2

SUMMARY RESULTS
STEEL BEAM ANALYSIS

BUILDING		BEAM SIZE	CRITERION	FACTOR OF SAFETY
Passageway	El. 604.5 Mode 22-28	8B13	$0.6\ F_y$	1.9
Loading Dock	Crane Girder	36W194 15C33.9	$0.66\ F_y$	1.67
Screenhouse	Crane Girder	10W21	$0.61\ F_y$	2.92

QUESTION NRC-8: Provide the details of the bases for your evaluations of steel angles considering only tension stresses over the gross area.

RESPONSE:

The allowable stress levels in braces were checked on gross area in conformance with the AISC (1980) code. The general behavior of such braces as described in the code is described below.

It has been observed that a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the scale of reduction of gross area and the mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states. The above behavior has been recently recognized by the AISC (1980) code. [1]

To prevent failure of a member loaded in tension, AISC (1980) has imposed a factor of 1.67 against yielding of the entire member and 2.0 against fracture of its weakest effective net area.

It is clear that the portion of the member occupied by the net area at fastener holes has a negligible length relative to the total length of the member; thus, yielding of the net area at fastener holes does not constitute a limit state of practical significance.

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(1) AISC (1980), Manual of Steel Construction, Eighth edition, American Institute of Steel Construction, Inc.

The mode of failure is dependent upon the ratio of effective net area to gross area and the mechanical properties of the steel. The boundary between these modes, according to the provisions of AISC (1980), is defined by the equation

$$A_e/A_g = 0.6F_y/0.5F_u$$

where A_e = effective net area of an axially loaded tension member,
A_g = gross area of an axially loaded tension member,
F_y = specified minimum yield stress of steel, and
F_u = specified minimum tensile strength of steel.

When $A_e/A_g \geq (F_y/0.833\ F_u)$, general yielding of the member will be the failure mode. When $A_e/A_g < (F_y/0.833\ F_u)$, fracture at the weakest net area will be the failure mode.

For ASTM A7 steel, the specified minimum yield strength is 33 ksi and the tensile strength is 60 to 72 ksi (AISC, 1953).[2] Therefore, $(F_y/0.833\ F_u)$ = 0.66, assuming the specified minimum yield strength, is equal to 60 ksi.

For ASTM A36 steel, the minimum yield stress is 36 ksi and the tensile strength is 58 to 80 ksi. Therefore, $(F_y/0.833\ F_u) = 0.745$, assuming the specified minimum yield strength, is equal to 58 ksi.

The field connections were performed using 3/4-inch-diameter ASTM A325 high-strength bolts. The ratio of A_e/A_g for a representative section (2L-3 x 2 - 1/2 x 1/4), assuming two bolts at connections, is approximately equal to 0.65. For connections with three bolts or more, the ratio of A_e/A_g is 0.74. Considering the actual factors of safety provided for the two failure conditions, tension on the gross area will govern and was so adopted in the analysis.

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(2) AISC (1953), Steel Construction Manual, American Institute of Steel Construction, Inc.

QUESTION NRC-9: Provide the details of your evaluations of column bases. In addition, demonstrate that they are adequate to resist any additional loads (above those predicted by your analyses) due to the redistribution of loads from overstressed members.

RESPONSE:

Stress analysis for the column bases summarized in Table NRC-9-1 was performed to determine:

- Anchor bolt stresses
- Concrete bearing stress

Base plate thickness was not checked because the plate thickness generally provided for column connections is equal to or greater than one inch. The loads on the column bases are generally small; therefore, the plate thicknesses provided are considered adequate.

In the analysis of base plates, the increase in bolt stresses due to prying action was not investigated. Table NRC-9-1 shows a summary of typical uplift values obtained at the bases of the columns analyzed. The net uplift, with the exception of Column No. J-1 in the service building, is low. Table NRC-9-2 summarizes the base plate dimensions. As may be seen on this table, the plate thicknesses generally are greater than one inch. Therefore, because of the plate thicknesses and uplift values, prying action is not significant. Furthermore, a generic study of the influence of base plate flexibility upon bolt loads is described in Reference 1. This study indicates that the effects of plate flexibility may be important for cases where the distance from the edge of the attached column to the edge of the base plate is greater than three times the base plate thicknes. The edge distance for the base plates generally is within this limit which shows that the prying action should not be significant.

1. Teledyne Engineering Services, 1979, "Summary Report, Generic Response to USNRC I&E Bulletin 79-02, Base Plate/Concrete Expansion Anchor Bolts, Technical Report 3501-2, August.

TABLE NRC-9-1

AXIAL LOADING
STEEL COLUMN ANALYSIS

BUILDING	COLUMN NUMBER	SEISMIC (lbs)	STATIC (lbs)	NET UPLIFT (lbs)
Turbine	A-1	51,550	40,310	11,240
	A-6	19,840	13,440	6,400
Service	J-1	50,820	12,360	38,460
	J-6	15,600	5,830	9,780
	E_A-6	3,280	6,740	-3,460
Fuel Cask Loading Dock	3-A	29,800	38,700	-8,900
Sphere Ventilation Room	A-1	850	1,450	-610
	B-1	260	1,250	-990

Note: Negative signs indicate no uplift.

TABLE NRC-9-2

COLUMN BASE PLATE DIMENSIONS

BUILDING	COLUMN	COLUMN SIZE	BASE PLATE SIZE (inches)	DRAWING NO.
Loading Dock	2-A	14WF61	12x16x1-1/2	C-322
	1-A	27WF145	14x27x1-3/4	C-322
	3-A	27WF102	14x27x1-3/4	C-322
	1-B	27WF145	14x27x1-3/4	C-322
	2-B	14WF61	12x16x1-1/2	C-322
	2-C	8WF17	6-1/2x8x1	C-323
	3-B	27WF102	14x27x1-3/4	C-322
	2_a-A	8WF24	6-1/2x8x1	C-322
	2_a-B	8WF24	6-1/2x8x1	C-322
	2_a-C	8WF17	6-1/2x8x1	C-323

TABLE NRC-9-2
(Continued)

Building	COLUMN	COLUMN SIZE	BASE PLATE SIZE (inches)	DRAWING NO.
Turbine	E-1	24WF76	12x34x1-3/4	C-201
	E-2	24WF76	12x34x1-3/4	C-201
	E-3	24WF76	11x24x1-1/2	C-201
	E-4	24WF76	11x24x1-1/2	C-201
	E-5	24WF76	11x24x1-1/2	C-201
	E-6	24WF76	11x24x1-1/2	C-201
Service	F-2	10WF33	10x12x1	C-201
	G-2	10WF33	10x12x1	C-201
	F-3	12WF53	16x20x1-1/2	C-201
	G-3	10WF33	10x12x1	C-201
	F-4	12WF53	16x20x1-1/2	C-201
	F-5	14WF87	23x24x2	C-201
	G-4	10WF33	10x12x1	C-201
	G-4.1	10WF33	10x12x1	C-201
	G-5	14WF74	18x24x2	C-201
	G-6	8WF31	9x9x3/4	C-201
	H-5	10WF33	10x12x1	C-201
	H-4.1	10WF33	10x12x1	C-201
	J-5	10WF33	10x12x1	C-201
	G-5.1	10WF33	10x12x1	C-201
	H-5.1	10WF33	10x12x1	C-201
	J-4.1	10WF33	8x10x3/4	C-201

TABLE NRC-9-2
(Continued)

BUILDING	COLUMN	COLUMN SIZE	BASE PLATE SIZE (inches)	DRAWING NO.
Service	J-5.1	10WF33	10x12x1	C-201
	H-6	10WF33	10x12x1	C-201
	J-6	10WF33	10x12x1	C-201
Passageway	E_A-8	10WF33	12x12x1	C-222
	D-6	14WF30	10x16x1	C-201
	D_A-7	6WF15.5	8x8x3/4	C-222
Turbine	D-1	16WF50	11x18x1	C-201
	C-1	16WF50	11x18x1	C-201
	C-6	14WF30	10x16x1	C-201
	B_A-1	16WF50	11x18x1	C-201
	B-6	14WF30	10x16x1	C-201

TABLE NRC-9-2
(Continued)

BUILDING	COLUMN	COLUMN SIZE	BASE PLATE SIZE (inches)	DRAWING NO.
Turbine	A-1	24WF76	12x34x1-3/4	C-201
	A-2	24WF76	12x34x1-3/4	C-201
	A-3	24WF76	12x34x1-3/4	C-201
	A-4	24WF76	12x34x1-3/4	C-201
	A-5	24WF76	12x34x1-3/4	C-201
	A-6	24WF76	12x34x1-3/4	C-201
Sphere Vent Room	A-2	6WF15.5	6x8-1/2x3/4	C-310
	A-1	6WF15.5	6x9-1/2x3/4	C-310
	B-1	6WF15.5 6C8.2	6x16-1/2x3/4	C-310
	B-2	6WF15.5	6x16-1/2x3/4	C-310
Intake Structure	--	8WF28	12x24x1/2	C-33

QUESTION NRC-10: Justify the acceptability of the method for determining overturning moment resistance as outlined on Page C-16 of the August 1981 D'Appolonia report, Volume III. Provide the details of your calculations of factors of safety against overturning for all structures.

RESPONSE:

The stability of foundations for the Big Rock Point structures have been checked for overturning and sliding conditions. Additionally, the base pressure distribution under the foundation has been computed to determine if possible tension zones occur.

Overturning is an extreme failure condition whereby gross failure of the foundation is caused by rotation about the toe. The factor of safety against overturning is defined as the ratio of all resisting moments acting on the structure about its toe and the resulting overturning moment on the structure due to various loading conditions.

Sliding potential is determined to investigate possible gross lateral movement of the structure on its base. The factor of safety against sliding is defined as the ratio of all resisting forces acting on the foundation against sliding and the horizontal forces acting on the foundation due to various loading conditions.

Base pressure distributions are calculated to determine possible tension zones which indicate the loss of contact area between the foundation and its supporting medium. A no-tension condition indicates that the resultant forces acting on the foundation passes through the middle third of its base width. For cases where tension has been calculated to exist, the total foundation area in tension has been calculated and compared with the total foundation area to examine the severity of the tensile zone developed under the foundation. For all cases examined, the ratio of foundation area in tension to the total area is small.

Considering the conservative nature of the analysis whereby the inertia effects on the structure are considered to act without any reversal of the direction of action, this is considered justifiable.

All analyses have been performed in accordance with generally accepted engineering practices for such computations. Tables NRC-10-1 through NRC-10-6 provide the details of the overall foundation stability analyses. As shown, adequate safety factors, generally significantly higher than the minimum required by the USNRC, have been obtained for both overturning and sliding.

TABLE NRC-10-1

RESULTS OF FOUNDATION STABILITY ANALYSES
CONTAINMENT SHELL AND REACTOR BUILDING

MODE	RESULTS DUE TO COMBINED STATIC AND SEISMIC LOADINGS		
	FACTOR OF SAFETY	MAXIMUM TOE PRESSURE (psf)	MINIMUM TOE PRESSURE (psf)
Overturning	4.7	4,600	470
Sliding	1.7	-	-
Rotation about vertical axis	9.9	-	-

NOTES

1. Factor of safety against overturning $= \frac{\text{Resisting Moment}}{\text{Overturning Moment}}$

2. Factor of safety against sliding $= \frac{\text{Resisting Force}}{\text{Sliding Force}}$.

3. Factor of safety against rotation about vertical axis $= \frac{\text{Resisting Moment}}{\text{Vertical Axis Moment}}$

4. No tension on the foundation occurs.

Reference: D'Appolonia report, August 1981, Seismic Safety Margin Evaluation, Reactor Building, Vol. II, Appendix A.

TABLE NRC-10-2

RESULTS OF FOUNDATION STABILITY ANALYSES
TURBINE GENERATOR PEDESTAL

MODE	STATIC + LOAD CASE I			STATIC + LOAD CASE II			STATIC + LOAD CASE III		
	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)
Overturning	4.5	-8	9,200	4.0	-700	9,000	4.6	43	8,600
Sliding	1.44	-	-	1.45	-	-	1.9	-	-

NOTES:

1. Factors of safety against overturning are the minimum values for overturning about either the x- or y-axis.
2. Factor of safety against overturning = $\frac{\text{Resisting Moment}}{\text{Overturning Moment}}$
3. Factor of safety against sliding = $\frac{\text{Resisting Force}}{\text{Net Driving Force}}$.
4. Negative sign indicates tension.
5. The computed tension of Load Combination II acts [illegible] less than five percent of the total base area and therefore represents no threat to stability.

Reference: D'Appolonia report, August 1981, Seismic [illegible] Margin Evaluation, Turbine Building, Vol. III, Appendix C.

TABLE NRC-10-3

RESULTS OF FOUNDATION STABILITY ANALYSES
CONTROL ROOM AREA

STATIC + LOAD CASE I

MODE	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)
Overturning	7.4	540	4,800
Sliding	1.35	-	-

NOTES:

1. Factors of safety against overturning are the minimum values for overturning about the y-axis.

2. Factor of safety against overturning = $\frac{\text{Resisting Moment}}{\text{Overturning Moment}}$

3. Factor of safety against sliding = $\frac{\text{Resisting Force}}{\text{Net Driving Force}}$.

4. Load Case I provided the worst seismic loading condition.

5. No tension on the foundation occurs.

Reference: D'Appolonia report, August 1981, Seismic Safety Margin Evaluation, Service Building and Office Addition, Vol. III, Appendix D.

TABLE NRC-10-4

RESULTS OF FOUNDATION STABILITY ANALYSES
REINFORCED CONCRETE STACK

MODE	STATIC, LATERAL EARTH PRESSURE, AND STATIC LOAD CASE I			STATIC, LATERAL EARTH PRESSURE, AND STATIC LOAD CASE II		
	FACTOR OF SAFETY	MAX. TOE PRESSURE (psf)	MIN. TOE PRESSURE (psf)	FACTOR OF SAFETY	MAX. TOE PRESSURE (psf)	MIN. TOE PRESSURE (psf)
Overturning	5.9	4,500	840	9.3	3,900	1,400
Sliding	2.6	-	-	2.9	-	-

NOTES:

1. Factor of safety against overturning $= \frac{\text{Resisting Moment}}{\text{Overturning Moment}}$
2. Factor of safety against sliding $= \frac{\text{Resisting Force}}{\text{Sliding Force}}$.
3. No tension on the foundation occurs.

Reference: D'Appolonia report, August 1981, Seismic Safety Margin Evaluation, Reinforced Concrete Stack, Vol. IV, Appendix E.

TABLE NC-10-5

RESULTS OF FOUNDATION STABILITY ANALYSES
FULL CASK LOADING DOCK/CORE SPRAY EQUIPMENT ROOM

MODE	STATIC + SEISMIC LOAD CASE I			STATIC + SEISMIC LOAD CASE II			STATIC + SEISMIC LOAD CASE III		
	FACTOR OF SAFETY	MAX. TOE PRESSURE (psf)	MIN. TOE PRESSURE (psf)	FACTOR OF SAFETY	MAX. TOE PRESSURE (psf)	MIN. TOE PRESSURE (psf)	FACTOR OF SAFETY	MAX. TOE PRESSURE (psf)	MIN. TOE PRESSURE (psf)
Overturning	6.1	2,400	40	4.8	2,600	-30	6.7	2,300	200
Sliding	3.5	-	-	3.8	-	-	6.0	-	-

NOTES:

1. Factors of safety against overturning are the minimum values for overturning about either the x- or y-axis.

2. Factor of safety against overturning = $\frac{\text{Resisting Moment}}{\text{Overturning Moment}}$

3. Factor of safety against sliding = $\frac{\text{Resisting Force}}{\text{Net Driving Force}}$.

4. Negative sign indicates tension.

5. For Load Combination II, the computed tension acts over approximately 0.01 percent of the base area and therefore represents no threat to stability.

Reference: D'Appolonia report, August 1981, Seismic Safety Margin Evaluation, Fuel Cask Loading Dock/Core Spray Equipment Room, Vol. VI, Appendix C.

TABLE NRC-10-6

RESULTS OF FOUNDATION STABILITY ANALYSES
SCREENHOUSE/DIESEL GENERATOR ROOM/DISCHARGE STRUCTURE

MODE	STATIC + LOAD CASE I			STATIC + LOAD CASE II			STATIC + LOAD CASE III		
	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)	FACTOR OF SAFETY	MIN. TOE PRESSURE (lbs/ft^2)	MAX. TOE PRESSURE (lbs/ft^2)
Overturning	13.2	330	2,000	5.8	200	2,200	11.8	300	2,100
Sliding	2.1	-	-	4.8	-	-	3.8	-	-

NOTES:

1. Factors of safety against overturning are the minimum values for overturning about the x-axis.

2. Factor of safety against overturning = $\frac{\text{Resisting Moment}}{\text{Overturning Moment}}$

3. Factor of safety against sliding = $\frac{\text{Resisting Force}}{\text{Net Driving Force}}$

4. No tension on the foundation occurs.

Reference: D'Appolonia report, August 1981, Seismic Safety Margin Evaluation, Screenhouse/Diesel Generator Room/Discharge Structure, Vol. VII, Appendix H.

QUESTION NRC-11: Summarize the bases for your selection of dynamic degrees of freedom.

RESPONSE:

Two computer programs, ANSYS (DeSalvo and Swanson, 1979) and DAPSYS (D'Appolonia Services, 1979), have been used in the analysis of various structures for the Big Rock Point plant. In the DAPSYS computer program, the solution for eigenvalues is performed for the complete stiffness and mass matrices generated through the assembly of the element matrices. Therefore, dynamic degrees of freedom (DDOF) are not specified.

In the ANSYS computer program, the sizes of the overall stiffness and mass matrices are first reduced through the Guyan reduction technique. In this method, the total structure matrices are reduced to a more manageable size by redefining the structure through a judicious selection of only its important degrees of freedom such that the lowest natural modes of the structure are accurately represented. The choice of these "active" degrees of freedom are specified in the ANSYS computer program through a DDOF list. The general rules used in the specification of the DDOF list for the analysis of the structure were:

- o Translational degrees of freedom at each floor level of the building
- o Locations evenly distributed along a planar frame
- o Locations of heavy masses
- o Free edges of a structure
- o Elbow or corner locations of a piping network.
- o Points describing unusual features or topology of the model.

Following performance of a mode-frequency analysis with the preliminary choice of the degrees of freedom, the frequencies and mode shapes are reviewed to develop an understanding of its dynamic behavior and to verify that the selected degrees of freedom represent an adequate range of calculated frequencies. If the review indicates the necessity of a further increase or decrease in the list of DDOF, the mode-frequency analysis is repeated with the revised degrees of freedom. The model is finalized when such review indicates that an adequate number of degrees of freedom has been specified.

QUESTION NRC-12: Define and justify the adequacy of your criteria for identification of significant weights to be included in the D'Appolonia analyses. The justification should specifically include consideration of the adequacy of the criteria to allow for appropriate determination of local structural effects.

RESPONSE:

The licensee has reviewed the list of equipment weights which were considered in the anlysis. On the basis of review, it is confirmed that the list is exhaustive and accounts for the significant equipment weights in the plant structures. The 5,000-pound cut-off referred to in the report corresponds to the basic guideline for typical weights to be included. However, items of lesser weight were usually included in the analysis when their location was judged important and their weight known. It is estimated that the sum of masses of nonstructural components and equipment which were not included in the model constitute less than one percent of the total mass.

QUESTION NRC-13: Provide the results of your review of the effects of floor flexibilities on equipment response. In addition, describe in detail how floor spectra are modified in your piping, equipment, and component evaluations to account for member or structure flexibility between their attachment points and the points at which spectra have been derived.

RESPONSE:

Concrete floor slabs will be investigated with respect to floor flexibilities by means of appropriate hand calculations wich consider the structural configuration of the slab and the boundary conditions. If a slab is supported across its width by structural members, it may be analyzed for its natural frequencies as a panel spanning the support members. In addition, the slab is also analyzed for its natural frequencies by considering the overall slab system to act as a single panel. If the concrete slab system is orthotropically reinforced, the natural frequency of the slab will be based upon the orthotropic flexural rigidity of the slab system. Once the natural frequencies of the slab system are determined, the bending moments for the slab system may then be calculated and compared with the moment required to cause cracking. If cracking occurs, the frequencies are recalculated considering the effects of cracking whereby the stiffness of the cracked concrete section is revised in accordance with the procedure as outlined in ACI 318-77 (1977).

It is expected that a program will be developed to evaluate floor flexibilities as part of Consumers Power's evaluation of seismic integrity of piping, equipment and components. Specifically, the floors either supporting safety-related subsystems or floors whose failure during an earthquake may pose a danger to any subsystem, will be cataloged. The effects of floor flexibility will then be concentrated for these floors only. It should be noted that when computing the overall response of the various structures, the flexibility of the floors is considered a local effect; and, therefore, the flexibility does not affect the response of the structure. In addition, because the floor response spectra were generated at column locations of the structure, effects of floor flexibilities will not affect the generated floor response spectra.

In the performance of the analysis for floor flexibility, analyses will consider integrity to maintain safety; and, therefore, stress analyses of such floors will be performed. In the event that floor integrity evaluation is performed separately from that of equipment, piping, and components, the decoupling criteria presented below will be utilized.

For evaluation of integrity of piping, equipment, and components, effects of floor flexibilities will be included through suitable representation of the supporting floor systems for each system to be analyzed. The floor response spectra presently developed at the corners of the floors will then be used as input to the analytically modeled floor system.

Decoupling Criteria

Mathematical uncoupling of the equipment/support structure system can be justified if the mass and stiffness of the supported subsystems are such that they do not appreciably affect the dynamic response of the supporting system models. The method generally used to decouple systems and components from the major structures assumes that the supported system or component can be rigidly lumped into its supports simply as a function of the ratio of the supported mass to the supporting mass. For such systems or components which are comparatively rigid with respect to the supporting structure , the equipment is usually lumped into the supporting structure mass if the equipment mass is less than one-tenth of that of the supporting mass, i.e., the mass ratio is 0.10 or less. The response of the rigidly lumped systems and components are then characterized by the appropriate floor response spectra.

When the frequency ratio of the supported subsystem to the support structure is considered, the decoupling criteria for subsystems must include both the mass and the frequency ratios. Therefore, the criteria which include both the mass and frequency ratios as given in USNRC Standard Review Plan, Section 3.7.2 (1975) will be followed. The criteria are summarized as follows:

- If $R_m < 0.01$, decoupling can be done for any R_f
- If $0.01 \leq R_m \leq 0.10$, coupling should be done if $0.8 \leq R_f \leq 1.25$
- If $R_m > 0.10$, an approximate model of the subsystem should be included in the primary system model

where

$$R_m = \frac{\text{Total mass of the supported subsystem}}{\text{Mass that supports the subsystem}}$$

$$R_f = \frac{\text{Fundamental frequency of the supported subsystem}}{\text{Frequency of the dominant support motion}}$$

QUESTION NRC-14: Provide the details and corresponding bases for your determination of "stick model" member properties for the actual structural member assemblages. In addition, provide corresponding information for your determination of individual structural member forces from those resulting from your analysis of the "stick model"

RESPONSE:

In the development of stick models, the mass of the structure has been lumped at various nodal locations of the model. The locations of nodes have been selected on the basis of floor elevations, and the element properties connecting two successive node points in a stick model define the stiffness properties of the structure between two floor levels.

In developing the stick models for the various building components, it was assumed that shear stiffness would be provided mainly by the walls running in the direction of seismic input motion. Therefore, only those walls parallel to the input motion were considered in calculating the shear areas. The total area obtained using this procedure was divided by a shear coefficient factor (e.g., 1.2 for rectangular sections) to calculate the total effective shear areas in a particular direction. Furthermore, walls in the perpendicular direction were assumed to offer only flexural stiffness and were thus included in computing the area moments of inertia of each beam element. Torsional stiffness at any level was calculated as an assemblage of open and closed sections. The relative distance between the center of rigidity and center of mass was considered in the development of the stick models. When the relative distance was large, as for the spent fuel pool, the eccentricity of the mass was incorporated into the stick model to account for coupling between horizontal and torsional response.

Stiffness properties were computed on the basis of sectional properties at midpoint elevations between lumped masses. Most of the plant structures for which stick models were developed comprise of massive, prismatic walls having few openings. During the development of the stick model, each wall was carefully examined for the presence of large openings. Small openings, e.g., pipe penetration in the walls, were considered to have insignificant effects on the wall stiffness. For openings which were relatively large, e.g., door openings, the stiffness of the wall was calculated assuming the opening to extend all along the wall height. Thus, the calculated stiffness properties are representative of average stiffness existing between the two floor elevations.

In the reactor building model, two separate stick models were used, one for the containment shell and the other for the internal structure. The model of the internal structure consisted of a stick with its centroidal locations dictated by the distribution of mass at various horizontal sections cut through the structure. The spent fuel pool and steam drum enclosure are located at a significant distance from the center of mass of the rest of the internal structure. They were, therefore, modeled as eccentric masses at their own centroidal locations.

The screenhouse structure has been modeled by representing the structure as four stick models interconnected by rigid links. Axial, shear, and bending properties of each beam section have been included in the development of the element stiffness properties. The analytical model developed represents the expected behavior of the structure under seismic excitation and also includes consideration of torsion of the structure due to its asymmetric configuration.

All six components of global seismic forces, i.e., three forces and three moments were considered in obtaining the individual structural member forces. The seismic shear on a wall was computed based on the assumption that only the walls oriented parallel to the direction of the earthquake were effective in resisting the global seismic shear in that direction. The global seismic shear was distributed in proportion to the shear stiffness of the walls. The global seismic force in the vertical direction was distributed in proportion to the cross-sectional areas of the walls. This force was treated either plus or minus relative to the axial load due to gravity. Additionally, the global moments about the horizontal axes were resolved into positive or negative axial forces in the walls in accordance with the area moments of inertia about the appropriate horizontal axes.

To obtain shear on the walls due to the global torsional moment, the total torsional moment at the appropriate section was distributed to individual torsional elements in proportion to their relative torsional stiffness. The shear flow on each individual element comprising a torsional group was then obtained in accordance with general engineering practice based on classifying the element as an open section or a closed section.

QUESTION NRC-15: Describe the methods by which the interactions of in-plane and out-of-plane loading on walls and other concrete elements are considered.

RESPONSE:

For analysis of walls, two types of forces have been considered in the analysis. They are:

- In-plane forces resulting from the analysis of the stick model.
- Out-of-plane forces, which are primarily local forces, e.g., seismic inertial forces, hydrostatic and hydrodynamic loadings, static and seismic lateral earth pressure.

The analysis for in-plane and out-of-plane loadings on the wall has then been performed in accordance with ACI-349-76 (1976). Consideration of in-plans and out-of-plane shears as used in the analysis is typically described below.

In-plane shear on the walls results from the gross seismic transverse shear and the gross seismic torsional moment acting on the cross sections of the seismic-structural model. The resulting shear stress was checked against permissible shear stress for walls, as given in the ACI code. For simplicity, the effect of vertical compression was neglected recognizing that this is a conservative assumption. The seismic shear stress in all cases analyzed was determined to be below that permitted by the ACI code.

Out-of-plane shear forces result from local forces acting perpendicular to the plane of the wall. Shear stress was obtained on the cross sections of the wall and compared against the permissible limits for beams as given in the ACI code. The in-plane and out-of-plane shear forces were thus checked independently in accordance with the recommendation of ACI 349-76 (1976). It should be noted that the interaction of in-plane axial forces was considered when checking the design capacity of a concret element subjected to out-of-plane shear force or bending moment.

The results of analyses indicate that the calculated in-plane and out-of-plane shear forces are very low compared to the nominal allowable shear stress of concrete. However, in recognition of possible interaction between in-plane and out-of-plane shear actions, the vector sum of the two calculated values has been compared for a critical section (east wall of the spent fuel pool) against the nominal allowable shear stress in the concrete. At the spent fuel pool wall, the resultant shear stress has been obtained as 50 psi, which is well below the allowable nominal shear stress in concrete.

QUESTION NRC-16: Provide justification for allowable bearing capacity.

RESPONSE:

The standard penetration resistance in the subsurface soil varies from a minimum of about 19 blows per foot near the surface to a maximum greater than 100 blows per foot. The majority of the blow count data is above 60 blows per foot. Existing data indicate that little soil testing was performed to determine the various subgrade properties such as bearing capacity. Available triaxial test data indicate that the glacial till has a cohesion between 3 and 4 kg/cm^2 (tsf) and an angle of friction, ϕ, between 30 and 32 degrees resulting in a calculated ultimate bearing capacity between 90 and 120 tsf. At the time of construction, the allowable bearing pressure, as recommended by Soil Testing Services, Inc. (1960), was a maximum of 2.75 tsf. Comparison of the allowable and calculated ultimate values indicates a high degree of conservation is the value of allowable bearing capacity recommended at the time of construction. On the basis of information available for allowable bearing capacity, the allowable bearing capacity recommended in Table 11-1 MAVFAC-DM7 (1971) was adopted by D'Appolonia for foundations supported on glacial till material and is 10 tsf. For locations where the depth of foundation was judged to be shallow and thus not supported on glacial till, bearing capacity calculations were performed combining the available data and data available in the literature.

For the reactor building, the embedment depth permits an increase in the allowable bearing capacity. According to Table 11-2, NAVFAC-DM7 (1971), an increase in allowable bearing capacity of five percent per foot of embedment is permissible. Taking embedment to be approximately 12 feet the resulting allowable bearing capacity is 16 tsf. Considering that the ultimate bearing capacity as calculated from triaxial data is above 90 tsf and that an allowable bearing capacity considering embedment is 16 tsf, the valve of 10 tsf used for the SEP analysis is conservative.

Further, NAVFAC-DM7 recommends a factor of safety for bearing capacity of two to three. Considering the allowable bearing capacity selected by D'Appolonia of 10 tsf and minimum calculated ultimate capacity of 90 tsf, the NAVFAC criteria is exceeded.

QUESTION NRC-17: The stack was checked against ACI 307-69 including the proposed revisions of 1978. Discuss the significance of any differences between ACI 307-69 and ACI 307-79. Evaluate the typical stresses resulting from the shear and moment at the base openings. Are the dowels connecting the base to the stack adequate?

RESPONSE:

A comparison of ACI 307-69 and ACI 307-79 has been made and the two codes have identical design equations and assumptions; except for a minor modification to the equations in ACI 307-79 which considers two layers of vertical reinforcement as opposed to one layer as assumed in ACI 307-69.

The stack was checked for shear at the base considering maximum shear acting in two horizontal orthogonal directions. The resultant in-plane maximum shear stress due to $\frac{VQ}{Ib}$ stress distribution, including the effects of the base openings, was calculated to be 62 psi, which is less than the minimum allowance shear stress of $2\sqrt{f_c}$ = 118 psi for a stack concrete strength of 3,500 psi. The dowels at the base of the stack were also investigated for the effects of combined maximum moments about two horizontal orthogonal directions at the base. The maximum stress in the dowels due to the combined moment is 11,200 psi. Based upon the requirements of ACI 318-77, the necessary ultimate development length for No. 8 dowels is 25 inches for a foundation concrete strength of 2,500 psi (Bechtel, 1961), which is less than the provided development length of 30 inches.

In addition, concrete and rebar stresses were checked at a section at Elevation 595 feet for combined deadweight, temperature, and earthquake loads considering the openings at the base of the stack. All computations were performed in accordance with the procedures recommended in ACI 307-69 and ACI 307-79. The resultant stresses for the concrete and

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Bechtel Corporation, 1961, Specification for Designing, Furnishing and Erecting a Reinforced Concrete Stack, Specification No. 3159C-21, March.

rebar were computed to be 760 psi and 11,700 psi, respectively. These stresses are significantly lower than the allowable stresses of 2,800 psi in concrete (based on 0.8 f_c') and 36,000 psi in steel applicable for SSE conditions as explained in the D'Appolonia Report Volume IV. Appendix E.

QUESTIONS LLL-1: The licensee does not have a detailed written program plan for their seismic reevaluation. The Volume I of the D'Appolonia's report (Reference 1) was reviewed in lieu of a program plan.

RESPONSE:

From the initiation of the SEP, Consumers Power has agreed to perform a seismic reevaluation of safety related plant structures and equipment. However, a detailed program plan for items other than the structures may only be developed following completion of seismic evaluation of the plant structures. This work was completed in August 1981 and the D'Appolonia report on structural evaluation was submitted to the USNRC for review.

Following submission of this report, seismic qualification of plant subsystems, e.g., block walls, mechanical and electrical equipment, and piping and supports, was initiated. In April 1982, Consumers Power became aware of USNRC's concern about possible anomalous site conditions at the Big Rock Point site resulting in an indication of uncertainty as to whether the sample problem earthquake, defined as having a zero period ground acceleration of 0.12g, was conservative. Because the seismic qualification of subsystem depends to a significant degree on the floor response spectra generated at various locations of the structure, and because USNRC's concern indicated uncertainty with respect to the validity of these spectra, the work on these items has been delayed. Consumers Power has recently been informed about the acceptance by the USNRC of the site specific spectra, thus confirming that the sample problem earthquake is conservative.

It should further be noted that, in spite of the abo Consumers Power has developed criteria for piping and equipment analyses which will be submitted in the near future. With respect to electrical equipment, Consumers Power has completed seismic qualification of the electrical equipment in accordance with the recommendations of the NRC IE Information Notice 80-21. Similarly, work has been completed on seismic qualificaiton of block walls.

In summary, development of a complete program plan has not been possible because of uncertainties with respect to the fundamental input parameters that would be needed to complete such a program. However, a considerable amount of work is already in progress or nearing completion in spite of these uncertainties.

QUESTION LLL-2: Soil spring approach was used for SSI analysis. No SSI code is needed. It is not clear whether in-house computer programs were used to calculate the soil spring constants and damping values.

RESPONSE:

A verified D'Appolonia in-house computer program was used to calculate the soil spring constants and radiation damping coefficients for the soil-structure interaction (SSI) analysis. The lumped SSI parameters are computed for a rigid circular disk founded on the surface of an elastic, layered half-space.

The theoretical basis for the analysis is contained in two publications:

1. Richart, F. E., J. R. Hall, and R. D. Woods, 1970, Vibrations of Soils and Foundations, Prentice-Hall, New Jersey.

2. Christiano, P. P., P. C. Rizzo, and S. J. Jarecki, 1974, "Compliances of Layered Elastic Systems," Proceedings of the Institute of Civil Engineers, London, pp. 673-683.

The lumped spring constants, mass ratios, and damping ratios are calculated for each mode of disk vibration from equations presented in Reference 1. The methodology used to account for the effect of layering on the spring constants is taken from Reference 2. The curves presented in this reference, showing cumulative strain energy versus depth for the four modes of disk vibration, are stored in digital format within the computer program. As regards the effect of layering on radiation damping, the calculated damping values were reduced, in proportion to the ratio of soil to rock impedance, to account for the presence of the shallow rock medium beneath the foundation soil.

QUESTIONS LLL-4: The support system of primary coolant loop includes rods or hangers. These kinds of structural elements can generally take very little compressive force. It is not clear how these elements are modeled in the seismic analysis of the primary coclant loop.

RESPONSE:

The spring support systems of the Primary Coolant Loop consist of:

- In-line double rod supports,
- Variable support hangers, and
- Sway braces

The in-line double rod supports consist of two in-line rods connected at their extreme ends to the reactor internal structure. The other two ends of the two rods converge to the piping system support location. The examples of such rods are the down comer support rods or the steam drum supporting system. These kinds of support resist movements through tension only. Therefore, analytically they have been modeled as spring elements. The forces calculated on these numbers are assumed to act in tension on any one of the two rods.

The variable support hangers are preloaded spring hangers. These hangers can resist both tension and compression until the loads are such that the limits of preloaded are exceeded ("topped out condition"). In the analytical model, these hangers have been included as spring elements. Following analysis, the forces of these spring hangers would be examined to check the adequacy of the assumed conditions used in analysis.

The sway braces are double acting springs which resist loads both in tension and in compression. Each of these braces features three inches of travel in either direction. Therefore, these sway braces have also been modeled as spring elements. Following analysis, their maximum movements are examined to check the validity of the analytical modeling assumption.

QUESTION LLL-5: The intake structure is submerged under water. No detailed description is given as to how the structure is modeled to account for the effect of water.

RESPONSE:

In most cases of practical interest, the earthquake-induced vibrations of submerged structures can be satisfactorily determined under the assumptions that wave action is negligible and that the velocity of the structure relative to the surrounding fluid is sufficiently low that the liquid may be assumed as incompressible, inviscid, and irrotational. Under these conditions the phenomenon can be analyzed by adding, to the mass of the structure (not considering the buoyant effect of the liquid), the mass of a certain volume of liquid, which gives a total "virtual" mass, and then treating the structure as though it stood in air (Newmark and Rosenblueth, 197 [1] This procedure has been followed in the analysis of the intake structure, whereby the virtual mass for each element is considered to be proportional to the square of the element width and the projected cross section in the direction of motion of the structure.

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[1] Newmark, N.M., and E. Rosenblueth, 1971, Fundamentals of Earthquake Engineering, Prentice-Hall.

QUESTIONS LLL-7: In the response spectrum analysis of the containment shell using the in-structure response spectra at the basemat as input, it is not clear how the rotational effect of the basemat is taken into account.

RESPONSE:

Analysis of the containment shell was performed in two stages as discussed in the D'Appolonia Report, Volume II, Appendix A, Attachment A1. In the first stage, a global model of the containment shell, reactor building, and internal structure was used to derive time histories and response spectra at the base of the containment shell. In this model, the containment was represented as a single mass at the top of a shear beam. Rotational degrees of freedom were included in the global model. The second stage of the analysis of the containment shell was detailed stress analysis of the containment using the translational time histories and response spectra at the containment base which were determined from the global model.

The effect of rocking upon response of the containment structure may be separated into two parts. The first part of the rocking response is represented by global vertical translation of the base of the shell as a result of rocking of the foundation. This effect is included in the translational input used for the detailed analysis of the shell. The second part of the rocking response of the containment shell may be represented as a rotational input to the base of the containment shell. In general, rotational base motion may be input for the detailed model of the containment shell. However, the inclusion of rotational input to the base of the shell would create calculational difficulties which could be circumvented through the deployment of a conventional analysis, i.e., one with input comprising only translation. Due to small rotations resulting from soil-structure interaction and due to modeling details discussed below, the rotational base excitiation of the containment shell was excluded in the detailed model.

The effects of translational and rotational input derived from the global model are conservatively represented through the modeling details employed in the foundation base. In the global model, the containment shell is supported by two rigid links; one link is connected between Elevation 584.5 feet at the containment center line and Elevation 573 feet to represent the base mat while a second link is connected between the containment base center line and the reactor internal structure at Elevation 584.5 feet. The second link is included to represent direct coupling of the containment shell and the top of the base mat. To allow for the effects of deformation of the reactor internal structure in the region of Elevation 573 feet to 584.5 feet and to affect the connection between the shell and the base mat, the ends of these rigid links were pinned at the reactor internal node points.

The maximum relative horizontal displacement and rotation at the base of the mat at Elevation 573 feet (Node 656) are 1.1×10^{-3} feet and 0.86×10^{-5} radians, respectively. As the base mat is rigid, the relative translation and rotation at the base of the containment shell at Elevation 584.5 feet (Node 663) are approximately 1.2×10^{-3} feet and 0.86×10^{-5} radians, respectively. Therefore, the support displacement input to the base of the shell should correspond to the above values or their equivalent.

As a result of the modeling scheme described above, the relative displacement at the base of the containment shell input in the analysis of the shell is 1.8×10^{-3} feet of translation and no rotation rather than 1.2×10^{-3} feet of translation plus the rotation of 0.86×10^{-5} radians. The difference in horizontal translation of 0.6×10^{-3} feet is equivalent to a rigid body rocking of the base mat equal to 0.5×10^{-4} radians.

The effects of the rocking due to a rotation of 0.86×10^{-5} radians when analyzing the detailed stresses in the shell may be represented by an equivalent translational input. Considering the shell to act rigidly, the rotation of the base is equivalent to a translation of 0.5×10^{-3} feet at the equator of the sphere. This is equivalent to a rigid body translation of the base of the shell equal to 0.5×10^{-3} feet. This value of 0.5×10^{-3} feet is less than the additional relative displacement of 0.6×10^{-3} feet introduced by the modeling details described above. Hence, modeling approximations introduced at the containment base result in an equivalent input translational motion which adequately represents the effects of rocking.

QUESTION LLL-8: In the response spectrum method of analysis and the model superposition time history analysis, are all modes included in the analysis? If the selected modes are used, what is the criteria for the selection?

RESPONSE:

As mentioned in Volume I, Section 5.1.2, response spectrum analyses were performed to obtain the seismic response of the following structures:

- Sphere ventilating room,
- Screenhouse/diesel generator room/discharge structure,
- Fuel cask loading dock/core spray equipment room,
- Reinforced concrete stack,
- Turbine building comples, and
- Containment shell.

For the analyisis of the first four structures, the DAPSYS (D'Appolonia Services, 1979) computer code was sued, while in the analyses of the turbine building comples and the containment shell, the ANSYS (DeSalvo and Swanson, 1979) computer code was used. Another structure, the intake structure, was analyzed by hand, as it contained only two degrees of freedom.

Response spectrum analyses were performed using a specified response spectrum for each structure in each of three directions of excitation. Maximum displacements, forces, and moments for each mode were calculated. The maximum displacement of the analytical model for a given mode is expresses as:

$$d_n = \frac{S_{vn} \Gamma_n}{\omega_n} \phi_n$$

where

d_n = vector of maximum displacements,

S_{vn} = spectral pseudovelocity at frequency ω_n,

γ_n = participation factor for mode n, and

ϕ_n = model displacement.

The mode coefficient, which is a measure of the significance of the mode to the response of the structure, is defined by the quantity:

$$\frac{S_{vn}\ \gamma_n}{\omega_n}$$

In the DAPSYS computer code, the responses code, the responses of all modes are combined to obtain the maximum displacement. The small ratio of the lowest to the highest mode coefficient is indicative of the degree to which the combinations were complete.

In the ANSYS computer code for a given excitation, those modes associated with a mode coefficient that is a small fraction of the largest mode coefficient (<1/100 for these analyses), are excluded from the combination.

Table LLL-8-1 shows further details of the response spectrum analyses. The table shows, for each direction of excitation, the number of modes combined, the lowest and highest frequencies in the combination, and the ratio of the lowest to the highest mode coefficient.

For the intake structure involving only two degrees of freedom in the analytical model, both modes were included in the combination.

For generation of floor response spectra, all of the calculated modes were included in the model time-history analysis. Table LLL-8-2 shows the number of modes considered and the lowest and highest frequencies in the combination.

TABLE LLL-8-2

MODE COMBINATION CRITERIA
GENERATION OF FLOOR RESPONSE SPECTRA
FLOOR RESPONSE SPECTRUM ANALYSES

STRUCTURE	TOTAL NUMBER OF MODES	FREQUENCY (HERTZ) LOWEST	HIGHEST
Sphere Ventilation Room	35	7.7	101
Screenhouse	24	0.1	103
Fuel Cask Loading Dock	60	4.0	80
Turbine Building Complex	170	2.5	3,946
Containment Shell			
(North-South)	66	7.86	140.8
(East-West)	66	7.83	135.9
(Vertical)	67	17.99	154.0

TABLE LLL-8-1

MODE COMBINATION CRITERIA
RESPONSE SPECTRUM ANALYSES

STRUCTURE	TOTAL NUMBER OF MODES IN COMBINATION			FREQUENCY RANGE (HERTZ) X[1]		Y		Z		RATIO OF LOWEST MODE COEFFICIENT TO HIGHEST MODE COEFFICIENT		
	X	Y	Z	LOWEST	HIGHEST	LOWEST	HIGHEST	LOWEST	HIGHEST	X	Y	Z
Sphere Ventilating Room	35	35	35	7.7	101	7.7	101	7.7	101	4.1×10^{-6}	2.4×10^{-6}	3.4×10^{-6}
Screenhouse	20	20	20	1.1	62.7	1.1	62.7	1.1	62.7	1.5×10^{-7}	1.8×10^{-7}	6.3×10^{-4}
Fuel Cask Loading Dock	30	30	30	4.0	36.2	4.0	36.2	4.0	36.2	9.2×10^{-5}	1.2×10^{-5}	3.9×10^{-4}
Stack	40	40	40	0.7	79.6	0.7	79.6	0.7	79.6	3.4×10^{-18}	1.1×10^{-17}	8.7×10^{-23}
Turbine Building Complex	27	29	53	2.5	21.2	2.5	21.0	2.5	38.9	0.01	0.01	0.01
Containment Shell[2]	1	1	1	7.86	7.86	7.83	7.83	17.99	17.99	1	1	1

(1) Except for the containment shell structure, the X-axis is toward north, the Y-axis is toward west, and the Z-axis is vertical upwards. For the containment shell, X-direction indicates the radial component, Y-direction indicates the tangential component, and Z-direction indicates the vertical component.

(2) For explanation of single mode consideration in the response spectrum analysis of the containment shell, see Volume II, Attachment A1.

QUESTION LLL-9: The topic of possible impact between adjacent structures during earthquake is not addressed although some displacement results were presented in D'Appolonia's report.

RESPONSE:

The impact between two adjacent structures has not been investigated because adequate clearance between two adjacent structures generally exists. Clearances are much greater than the relative displacement of the reactor building mat which is about 0.013 inch. The only possible direct contact where impact may occur is between the pipe tunnel and the reactor building at the footing level. The type of contact and contact area are shown in the attached Figure NRC-9-1. The question of interaction between the reactor building and the adjacent pipe tunnel was addressed previously in response to NRC comments and questions dated August 27, 1979. The effects of interaction of the reactor building, pipe tunnel, and the turbine pedestal/pedestal-mounted equipment were evaluated using a simple three-mass model. The analysis indicated that the force in the tunnel is approximately 15 percent of the base shear acting on the reactor building. The axial stress at the pipe tunnel interface was about 340 psi. Therefore, it is concluded that impact at this interface will not affect the integrity of the structure. In the event that slight spalling of concrete occurs at the interface, no adverse effects on the safety of the plant will result because this interface is at or below grade level.

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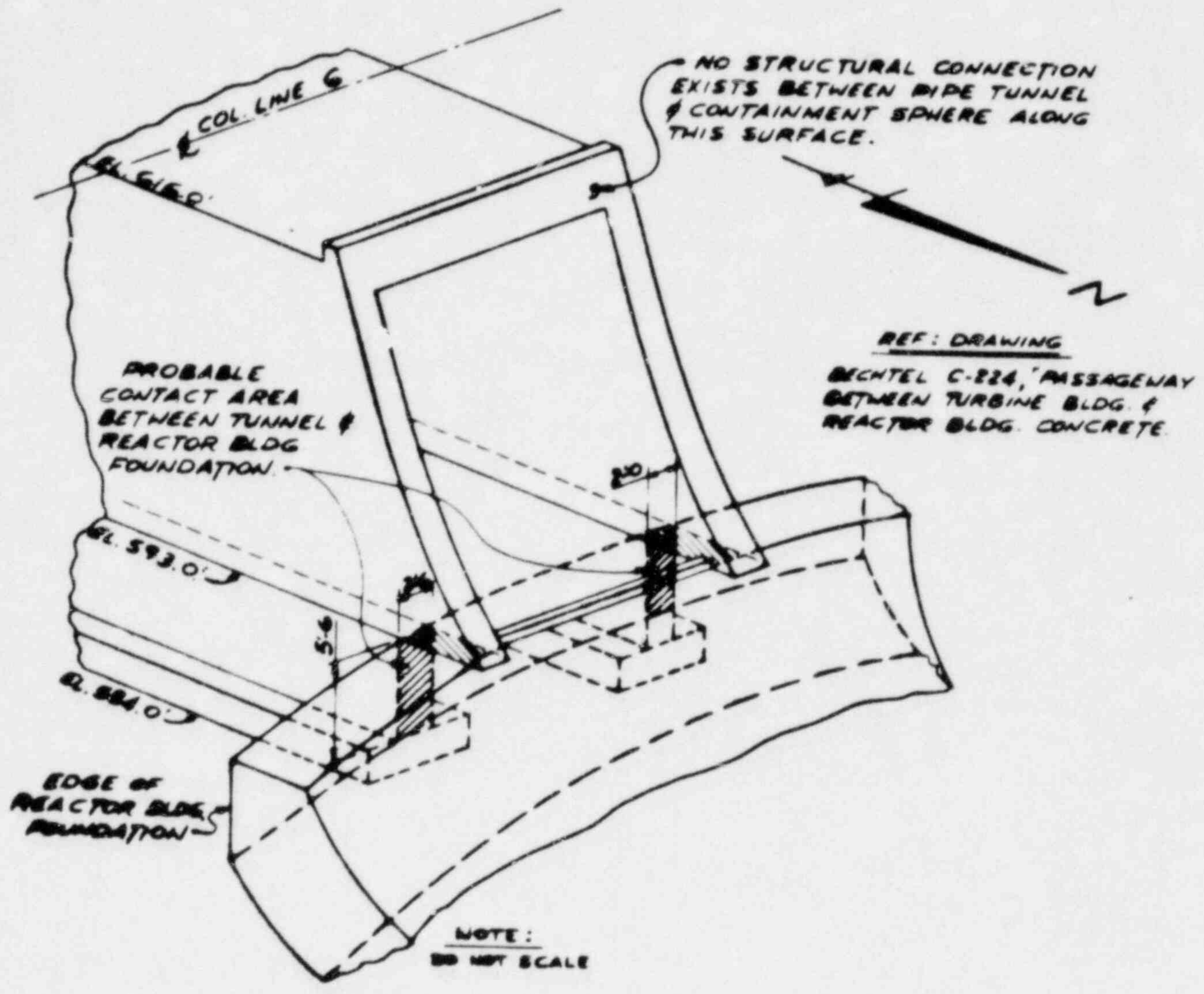

FIGURE LLL-9-1

LINE ISOMETRIC
PIPE TUNNEL-REACTOR BLDG. INTERFACE
BIG ROCK POINT
NUCLEAR POWER PLANT

PREPARED FOR

CONSUMERS POWER COMPANY
JACKSON, MICHIGAN

D'APPOLONIA

QUESTION LLL-10: It is not clear if all D'Appolonia's in-house computer programs were verified and documented.

RESPONSE:

All computer programs used for this work are verified and documented. The procedures used to verify and document programs are part of the overall Quality Assurance Program for the project. The implementation of these procedures was monitored throughout the project by D'Appolonia's Quality Assurance Group and audited by Consumers Power Company Quality Assurance. Documentation and verificaiton procedures are described below.

Computer programs used for this work by D'Appolonia included an industry standard program, ANSYS[1], and several in-house programs. Following is a discussion of the methods used for verification and documentation.

Verification

In-house computer programs are verified using one, or a combination of the three following methods:

1. Preparation of hand calculations which cover the same operations as the computer program and produce equivalent results.

2. Use of an independent computer program which performs the same operations and produces equivalent results.

3. Comparison of the results obtained using the computer program with analyses published in textbooks or journals.

[1] DeSalvo, G. J. and J. A. Swanson, 1979, ANSYS-Engineering Analysis User's Manual, Swanson Analysis Systems, Houston, Pennsylvania.

For Methods 1 or 2 above, the D'Appolonia Quality Assurance Program requires an independent check of the hand calculations and/or the computer program input to provide adequate verification.

External programs, which are widely used and documented, and have verified sample problems, are considered "industry standards," and are not independently verified by D'Appolonia. These programs, such as ANSYS, when implemented without alteration, are tested by duplicating standard sample problems to assure correct operation. Also, they are reviewed by the D'Appolonia Quality Assurance Staff to assure that testing of the programs is adequate.

Documentation

Programs are not approved by the D'Appolonia Quality Assurance Staff for general use, even if a technical verification exists, without user oriented documentation. Documentation of a computer program is paramount because it provides operating instructions for the program, as well as the theoretical basis for the computations. Also, documentation serves as the reference for determining whether the program was used correctly.

Verification and Documentation of Specific Project Computer Programs

ANSYS(1) is a general purpose, proprietary, structural finite element analysis program. It has been used widely by various engineering organizations since 1970. Maintenance of ANSYS by its vendor includes extensive verification testing and providing a detailed user's information manual and a verification manual.

D'Appolonia cannot access the ANSYS code for revision or alteration. Thus, it is used as supplied by the vendor. As a means of verification ANSYS was compared with DAPSYS, as described below by analyzing a project-specific problem. The comparison of results from the two independent programs serves as a verification of the modeling logic for each program for the types of problems analyzed in the Big Rock Point project.

DAPSYS is a general purpose computer program developed by D'Appolonia from the SAPIV computer program of the University of California. It has been used internally for several years and an extensive set of verification test problems exists for DAPSYS. The test problems include a project-specific problem, the response spectrum analysis of the Fuel Cask Loading Dock structure, which demonstrates the correct computation of the absolute sum of the model results for the structure when compared with ANSYS results.

Other test problems for DAPSYS provide verification for other options in the program which were used on the project.

The program HIST1 was used to generate the artificial earthquake time history. The program uses an iterative numerical technique to obtain the artificial time history matching a specified response spectrum. HIST1 has been developed and is maintained by D'Appolonia and was verified using a combination of hand calculations and comparison of results obtained using another program.

The HIST1 response spectra calculation results have been verified by comparison to a documented solution. This included generation of response spectra from a given time history and generation of an artificial time history from a given response spectra. The baseline correction option was verified by comparison of HIST1 to another program and by hand calculations. Capabilities of producing peak response and time of peak response were verified by comparison of HIST1 and ANSYS results. Lastly, computation of the log-log interpolation of design response curves between damping values of 2.0 and 5.0 percent was verified by comparison of HIST1 results to hand calculations.

INSTR, FRESP, SRSS, and WIDDN were used to generate floor response spectra and were verified in conjunction with one another. INSTR provides floor time historics for structures for which response spectrum analyses have been performed. FRESP provides the unsmoothed, computed floor response spectra from the floor time historics. SRSS is utilized to reduce the nine floor response spectra (resulting from three directions of excitation along three directions of excitation along three orthogonal directions) into three spectra, using the SRSS method for summation. WIDDN peak broadens the floor response spectra in accordance with USNRC Regulatory Guide 1.1222. Test problems and accompanying calculations verify each program in the chain and demonstrate that the entire sequence of computer programs works properly. Each program has been individually documented.

INSTR and FRESP were verified by comparing program results to a single-degree-of-freedom oscillator closed-form solution. INSTR and FRESP were additionally verified, in conjunction with SRSS and WIDDN, by solving a three-dimensional frame problem under dynamic loading conditions. The results for INSTR, FRESP, and SRSS were compared to results from the same analysis performed using the computer program DAPSYS. The results from WIDDN were verified by comparison with hand calculations.

WGTMOD2$ is the in-house computer program utilized in the soil-structure interaction analysis. WGTMOD2$ calculates spring constants and radiation damping coefficients for a rigid circular disk founded on the surface of an elastic layered half-space. WGTMOD2$ has been verified by comparison of program results to hand calculations performed using the same equations. The theoretical basis for WGTMOD2$ appears in the reference by Richart, Hall, and Woods[(2)], and an article by Christiano, et al.[(3)].

DSTRESS1 and DSTRESS2 were used on a one-time basis to compute the stresses in the steel reinforcing bars and the concrete at specific points in the stack structure subjected to combined static, temperature, and seismic loading. Computer results were checked and verified by comparison with hand calculations performed independently, as per ACI 307-69, "Specification for the Design and Construction of Reinforced Concrete Chimneys," and its 1978 revisions.

[(2)] Richart, F. E., J. E. Hall, and R. D. Woods, 1970, Vibrations of Soils and Foundations, Prentice-Hall, New Jersey.

[(3)] Christiano, P. P., P. C. Rizzo, and S. J. Jarecki, 1974, "Compliances of Layered Elastic Systems," Proceedings of the Institute of Civil Engineers, London, December, Part 2, pp. 673-683.

QUESTION LLL-11: No formal program plan is available for mechanical and electrical components, piping and supports. The primary coolant loop was analyzed along with the reactor building. Therefore, in this program plan review, the primary coolant loop was reviewed as part of a structure.

RESPONSE:

Consumers Power is currently working on a review criteria for pipe, pipe supports, and mechanical equipment. In the meeting of July 15, 1982, the USNRC transmitted to us the preliminary guidelines for analysis of piping and equipment. A complete program plan is in the process of development and will be submitted in the near future.

Some work has been completed or is in progress. A preliminary evaluation of mechanical equipment on a sampling basis has been conducted and the structural integrity of certain electrical equipment has been completed under the direction of IE Information Notice 80-21. With respect to the analysis of the primary coolant loop, a reanalysis is scheduled and the results will be submitted following its completion.