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April 27, 1984

United States Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: Mr. George W. Knighton, Chief
Licensing Branch 3
Office of Nuclear Reactor Regulation

SUBJECT: Beaver Valley Power Station - Unit No. 2
Docket No. 50-412
NRC Structural Design Audit

Gentlemen:

In letter 2NRC-4-018, dated February 27, 1984, we provided you with a response or a schedule for providing a response to each of the 28 NRC Structural Design Audit Action Items and to each of the NRC Structural Engineering Section's review comments on BVPS-2 Standard Review Plan differences. Attached are the responses that were scheduled to be provided by April 27, 1984 (Action Items 1, 6, 13, 14, 15, 16, 21, 22, and 28, and SRP Sections 3.3.1.II.3 and 3.7.3.II.7) and three of the responses that were scheduled to be provided by June 15, 1984 (Action Items 5, 11, and 24).

If you have any questions on this matter, please contact J. D. O'Neil at (412) 787-5141.

DUQUESNE LIGHT COMPANY

By E. J. Woolever
E. J. Woolever
Vice President

JDO/wjs
Attachment

cc: Mr. G. Walton, NRC Resident Inspector (w/a)
Mr. M. Lacitra, Project Manager (w/a)

SUBSCRIBED AND SWORN TO BEFORE ME THIS 26th DAY OF April, 1984.

Elva D. Lesondak
Notary Public

ELVA D. LESONDAK, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

B001
1/1

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 26th day of April, 1984, before me, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge.

Elva G. Lesondak
Notary Public

ELVA G. LESONDAK, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

NRC STRUCTURAL AUDIT ACTION ITEMS

1. The applicant intends to demonstrate that the site-specific response spectra approved by the NRC's Geosciences Branch will be comparable to that shown on FSAR Figure 3.7B-1 and any significant differences will be addressed and justified. The applicant will also address the deviations of the vertical spectra from those given in Regulatory Guide 1.60 as it is applicable to the BVPS site.

   Response:

   This item is addressed in the attached responses to NRC Geosciences Branch questions 230.2, 230.3, and 230.6, and NRC Structural Engineering Section question 220.4. The separate report referred to in these responses, entitled "Seismic Design Response Spectra, BVPS-2", will be provided by June 1, 1984.

NRC Letter: September 19, 1983

Question 230.2 (SRPs 2.5.2.3, 2.5.2.4, 2.5.2.5 and 2.5.2.6)

According to the FSAR, the BVPS-2 seismic design parameters are based upon a response spectrum anchored to a 0.125g ZPA which is different from the Regulatory Guide 1.60 spectrum. The documents quoted for the development of the above seismic design spectrum are BVPS-2 PSAR Appendixes 2C and 2D. BVPS-2 Appendix 2C recommends a 0.10g design earthquake. BVPS-2 Appendix 2D recommends a Housner response spectrum normalized to 0.125g which was obtained from an estimated amplification factor of 3.5 combined with a maximum ground acceleration of 0.035g. In addition to these multiple assumptions, there are several factors mentioned in the FSAR which have not been adequately discussed with respect to the influence that these factors have on the seismic design criteria for the plant. For example, FSAR Table 3.7B-12 indicates variations in depth of soil over bedrock from 35 feet to 100 feet. FSAR Figures 2.5.4-2 through 2.5.4-9 indicate significant differences in density of soils underlying the Category I structures.

Describe how the above information was used to determine the seismic design criteria for each of the Category I structures. For example, describe the free field foundation acceleration assumed for the seismic design of Category I structures. Describe how the established free field foundation acceleration was augmented to accommodate for soil amplification or reduction.

Response:

Refer to revised Section 2.5.2, Amendment 6, and a separate report entitled, Seismic Design Response Spectra, BVPS-2, which will be submitted to the NRC under separate cover in response to NRC Structural Design Audit Action Item 1.

NRC Letter: September 19, 1983

Question 230.3 (SRPs 2.5.2.3, 2.5.2.4, 2.5.2.5, and 2.5.2.6)

The site is considered to be located in the Appalachian Plateau Tectonic Province. The largest historic earthquake in this tectonic province was determined to be the November 6, 1926, S.E. Ohio earthquake. This determination was obtained from intensity listings shown in the FSAR Table 2.5.2-2. Using the Standard Review Plan procedure for deriving seismic design criteria from intensity data, the MMI=VI-VII intensity listed for the 1926 earthquake would indicate a Regulatory Guide 1.60 response spectrum anchored to 0.10g zero period acceleration which may be modified to reflect local site conditions.

In recent safety reviews, the staff has relied upon site specific spectra to evaluate the seismic design criteria. The reason being that site specific spectra are more in accord with the controlling earthquake size, frequency spectrum, and local site conditions. For example, using the Nuttli/Herrmann (1978) relationship, the site specific spectra for a MMI=VI-VII intensity earthquake could be developed from the 84th percentile spectra of a suite of appropriate earthquake records of magnitude m = 5.0 + 0.5. In addition, a direct estimate of magnitude may be obtained from the information listed in the updated FSAR Table 2.5.2-2. In the event that appropriate records are not currently available, a site specific spectrum may be determined by modifying a rock site specific spectrum to accound for local soil amplification characteristics of the site (refer to Midland OL-SER, Clinton OL-SER).

1. Using the guidelines described in the Standard Review Plan (1981) compare the BVPS-2 design spectra to the appropriate intensity based on Regulatory Guide 1.60 spectra. Describe the effects of local site conditions and discuss exceedences, if any.

2. Based upon your estimate of the appropriate magnitude of the Safe Shutdown Earthquake prepare Site Specific Spectra in accordance with guidelines described above. Compare these spectra with the design spectra for the plant and discuss exceedences, if any. Include in your discussion the effects of the following variations in parameters which influence the ground motion estimates:

   a. Variation in shear velocity in the soil layers which depend upon the composition, depth, and/or densification of soil layers under the Category I structures.

   b. If appropriate, compare results of layered soil analysis programs to methods other than those used in Appendix 2D, such as SHAKE.

Response:

Refer to revised Section 2.5.2, Amendment 6, and a separate report entitled, Seismic Design Response Spectra, BVPS-2, which will be submitted to the NRC under separate cover in response to NRC Structural Design Audit Action Item 1.

NRC Letter: September 19, 1983

Question 230.6

According to the FSAR section 3.7B.1.1 the vertical design response spectra are taken to be two-thirds of the horizontal design response spectra. Discuss the adequacy of the vertical response spectra with respect to the Regulatory Guide 1.60 procedures for determining the vertical response spectra (reference Regulatory Guide 1.60, Table II). Include in your discussion relevant information obtained from the recent eastern U.S. and Canada earthquake records (1982 New Hampshire and New Brunswick earthquakes).

Response:

Refer to revised Section 2.5.2, Amendment 6, and a separate report entitled, Seismic Design Response Spectra, BVPS-2, which will be submitted to the NRC, under separate cover in response to NRC Structural Design Audit Action Item 1.

NRC Letter: September 15, 1983

Question 220.4 (Section 3.7B.1.1 SRP 3.7.1.II.1a)

Referring to Item 3.7.1.II.1a discussed in "Priority Review of Beaver Valley 2 Standard Review Plan Differences," the site specific response spectra and the values of vertical design response spectra should be addressed in Section 2.5.

Response:

Refer to revised Section 2.5.2, Amendment 6, and a separate report entitled, Seismic Design Response Spectra, BVPS-2, which will be submitted to the NRC under separate cover in response to NRC Structural Design Audit Action Item 1.

5. For each of the three key structures identified in Item 4, consider the base mat shear forces due to earthquakes and assess the impact of including additional accidental torsional effects, and show that the structural elements are adequate for these effects.

Response:

The auxiliary building is considered to be representative of the seismic Category I structures at BVPS-2; as such it was chosen for the purpose of assessing the impact of including the accidental torsional effects.

The resultant torsion at the base of the structure was calculated by assuming the resultant story shears to be offset from the center of rigidity by 5 percent of the maximum building dimension perpendicular to the direction of the applied story shear. This resultant torsion was distributed to the walls, at the base of the structure, in proportion to their relative stiffnesses and distances from the center of rigidity.

A summary of results is presented in Table 5.1. It can be seen from Table 5.1 that the effects of accidental torsion are very small and the structures have adequate strength to carry this arbitrary loading.

Table 5.1

Wall Designation	V_{old} (kips)	As_{old} (in^2/ft)	V^T_{accid} (kips)	V_{new} (kips)	As_{new} (in^2/ft)	$As_{provided}$ (in^2/ft)
A	915	0.72*	65	980	0.72*	3.74
B	6867	1.32	402	7269	1.39	3.74
C	4035	1.08*	157	4192	1.08*	1.20
D	5256	0.72*	314	5570	0.72*	3.74
E	4165	0.72*	236	4401	0.72*	3.74

V_{old} = Shear in wall due to story shears and actual eccentricity.

V^T_{accid} = Shear in wall due to specified accidental eccentricity.

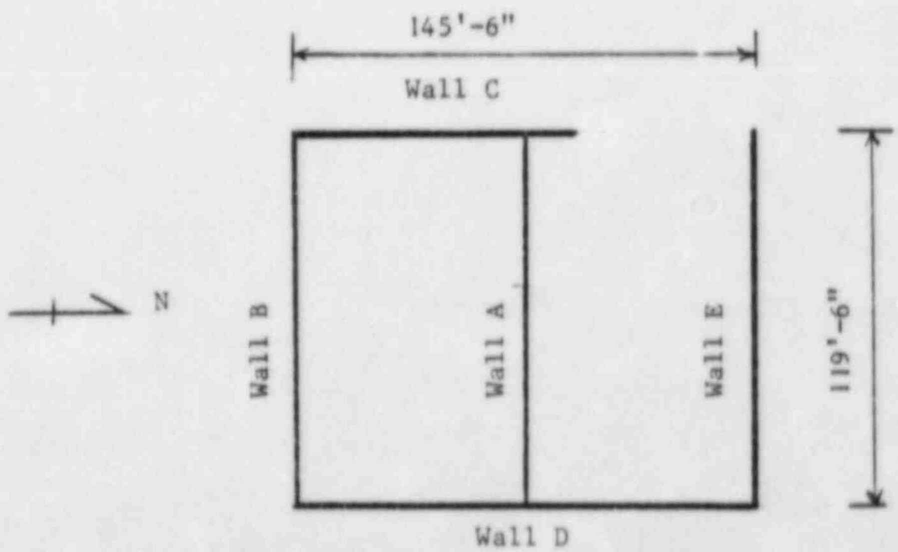
$V_{new} = V_{old} + V^T_{accid}$.

As_{old} = Area of reinforcement required for V_{old}.

As_{new} = Area of reinforcement required for V_{new}.

$As_{provided}$ = Area of reinforcement provided.

*A_{min} = Minimum area of reinforcement required per ACI 318 Section 11.16.4.1 governs ($\rho = 0.0025$).



PLAN

Auxiliary Bldg.

6. Assess the safety factors against sliding and overturning of both the containment and auxiliary building structures by accounting for the three component earthquake input.

Response:

Factors of safety against sliding and overturning that account for three-component earthquake input have been calculated for the auxiliary building. The auxiliary building was chosen as representative of Seismic Category I buildings at BVPS-2.

Maximum floor acceleration responses were calculated in accordance with the acceptance criteria of SRP 3.7.2 (that is, "when the response spectra method is adopted for seismic analysis, the maximum structural responses due to each of the three components of earthquake motion should be combined by taking the square root of the sum of the squares of the maximum codirectional responses caused by each of the three components of earthquake motion at a particular point of the structure"). This method is demonstrated in Table 6.1 for the case of horizontal acceleration response in the north-south direction due to three component earthquake input. This procedure was repeated to obtain the building acceleration response in the east-west and vertical directions.

From these calculated floor accelerations the shear and overturning moments at the base of the structure were obtained by summation of inertia forces. Sliding and overturning loads with respect to both axes, east-west and north-south, of the building have been calculated in this manner.

Factors of safety against sliding and overturning were then calculated as described in FSAR Section 3.8. The results obtained are presented in Table 6.2 for the SSE case. Inspection of resulting loads on the foundation show that the SSE case governed over the OBE case. Sliding forces were resisted by friction at the soil/mat interface. Overturning was resisted by a linear distribution of bearing stress under the foundation mat.

In conclusion, the factors of safety against sliding and overturning have been computed for the auxiliary building considering the three-component earthquake input and are within the limits specified in FSAR Table 3.8-13.

TABLE 6.1

Three-Component Earthquake Response in N/S Direction (ft/sec^2)

Floor Elevation	Response Direction	Contribution From E-W Excitation	Contribution From N-S Excitation	Contribution From Vertical Excitation	SRSS Resultant
710'-6"	N-S	0.36	6.51	0.71	6.56
735'-6"	N-S	0.26	7.83	0.55	7.85
755'-6"	N-S	0.26	9.31	0.47	9.33
773'-6"	N-S	0.32	10.73	0.45	10.74
797'-6"	N-S	1.04	13.60	0.79	13.67

TABLE 6.2

Factors of Safety (SSE Earthquake)

Condition	East/West Direction Factor of Safety	North/South Direction Factor of Safety	BVPS-2 FSAR Limit
Overturning	2.6	3.0	1.1
Sliding	1.6	2.2	1.1

11. Perform a random and limited review of actual design calculations to ensure that the stress and strain levels of key structural elements are consistent with Position C.3 of Regulatory Guide 1.61.

Response:

Position C.3 of Regulatory Guide 1.61 requires that the maximum combined stresses due to static, seismic, and other dynamic loadings, not be significantly lower than the yield stress and 1/2 yield stress for SSE and 1/2-SSE, respectively, if the Regulatory Guide 1.61 dampings are to be used.

A review of the design calculations for the lower walls of the auxiliary building and safeguards building was performed to determine the corresponding stresses. The results of this review show the stresses in the wall (reinforcing steel) to be significantly lower than those identified in Position C.3.

For the 1/2-SSE analysis of structures at BVPS-2, a value of 2 percent of critical damping was used for reinforced concrete structures; this is 50-percent less than the corresponding value identified in Regulatory Guide 1.61. For the SSE analysis, the BVPS-2 damping value is the same as that identified in Regulatory Guide 1.61 (that is, 7.0 percent of critical damping).

The structures at BVPS-2 are founded on soil; as such the dynamic responses due to seismic excitation are dominated by the soil deformations. In fact it can be seen from the results of the dynamic analysis of the soil-structure models that the system dampings (modal dampings) are mainly a function of the soil damping for the predominant modes. To demonstrate this, the soil-structure model of the auxiliary building was reanalyzed for the SSE case, using a damping of 1.0 percent of critical for the structural elements. The soil damping values were not changed. Table 11.1 demonstrates clearly that the seismic responses of BVPS-2 structures are insensitive to variations in steel and concrete structures. Since the results of the two analyses are essentially identical, it is concluded that the values of damping used for the seismic analysis of structures at BVPS-2 are acceptable.

TABLE 11.1

Calculated Accelerations and Displacements

Direction	Elevation (ft)	Accelerations (g) Case I	Accelerations (g) Case II	Displacements (ft) Case I	Displacements (ft) Case II
North-South	797.25	0.43	0.43	0.045	0.046
North-South	772.50	0.33	0.34	0.036	0.037
North-South	754.50	0.29	0.30	0.032	0.032
North-South	735.50	0.24	0.25	0.026	0.027
North-South	709.00	0.20	0.21	0.019	0.020
East-West	797.25	0.45	0.46	0.051	0.052
East-West	772.50	0.33	0.34	0.039	0.040
East-West	754.50	0.28	0.28	0.033	0.034
East-West	735.50	0.23	0.23	0.026	0.026
East-West	709.00	0.20	0.20	0.017	0.018
Vertical	797.25	0.29	0.29	0.017	0.020
Vertical	772.50	0.26	0.26	0.017	0.017
Vertical	754.50	0.26	0.26	0.017	0.017
Vertical	735.50	0.25	0.25	0.016	0.017
Vertical	709.00	0.24	0.23	0.016	0.016

NOTE: Case I = 7.0% structure damping

Case II = 1.0% structure damping

13. Provide technical justification for using a dynamic amplification factor of one for the analysis of pressurization effect on steam generator cubicles. Also, address the basis for not accounting for the cracking effect of the concrete elements during pressurization.

Response:

An elastic dynamic analysis has been performed for the steam generator cubicles when subjected to the time history of pressurization load (Figure 13.1).

The fundamental frequencies of the floor slabs, cranewall, and radial walls of the cubicles have been calculated for both the uncracked and cracked concrete conditions. For the governing structural element (a radial wall) the uncracked and cracked frequencies are 37 Hz and 27 Hz, respectively. The pressure time histories were applied to each of these structural element models and the resulting damped equations of motion were numerically integrated to determine Dynamic Load Factors (DLF). Results of these analyses justify the use of a DLF = 1.0 for the pressure load P throughout the steam generator cubicle (the maximum calculated DLF was 1.04).

A confirmation program is currently underway to verify the design of the containment's internal structure (including steam generator cubicles) for final pressure loads. Methods similar to those described above will be used to determine the appropriate DLFs for the final pressure loadings.

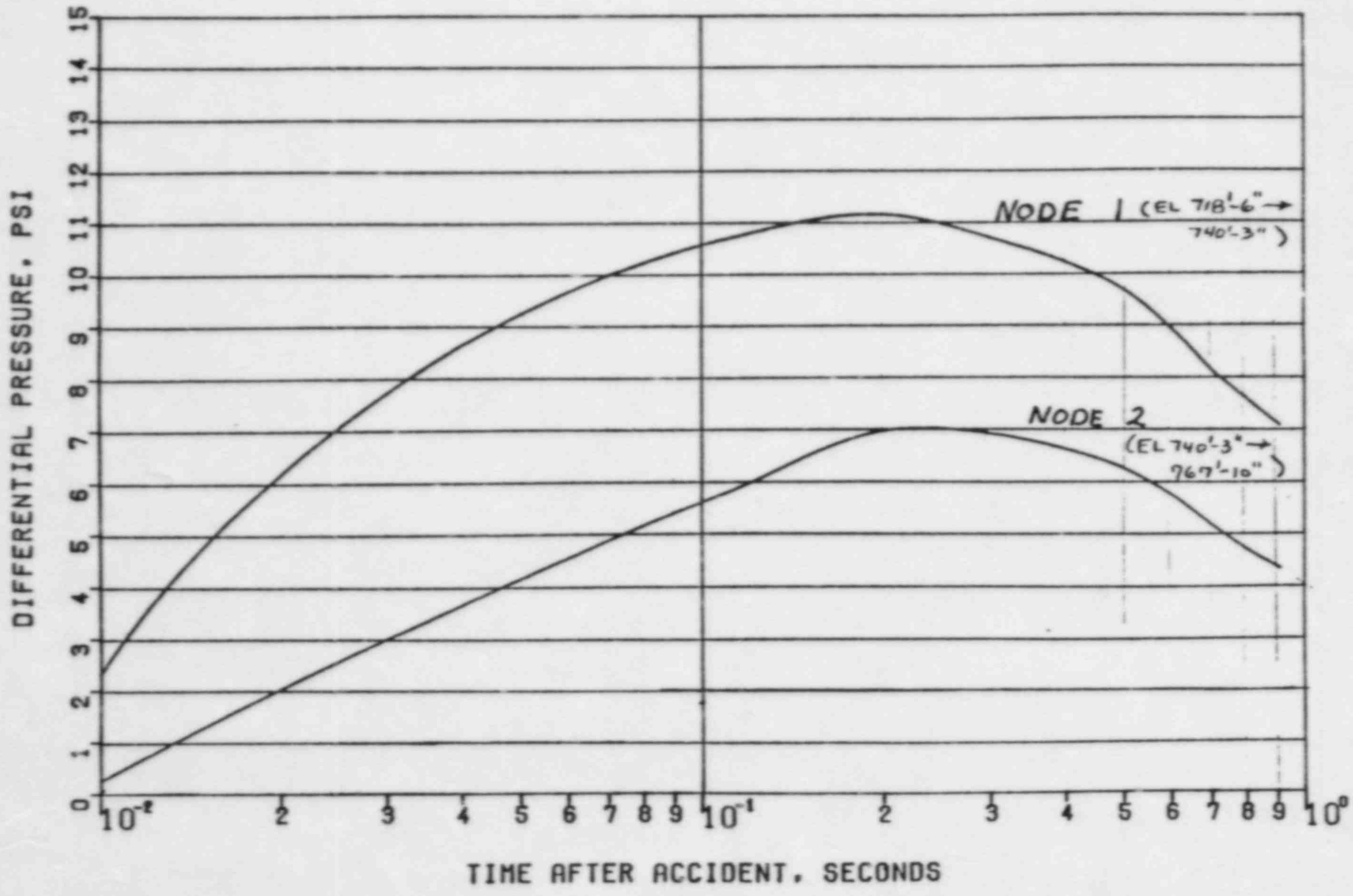


Figure 13.1
DIFFERENTIAL PRESSURE VS. TIME
STEAM GENERATOR CUBICLE, LOOP 1
BEAVER VALLEY POWER STATION UNIT 2

14. The applicant is requested to provide comparisons to demonstrate that the current SRP structural acceptance criteria (3.8.2) for design limits and loading combinations are complied with. Deviations identified by the comparison should be justified.

Response:

As stated in FSAR Section 3.8.2.2 and as defined in the "Codes and Standards" Section of 10CFR50.55a, BVPS-2 is governed by the 1971 edition of the ASME B&PV Code, Section III, Division 1, up through and including the Winter 1972 Addenda. The edition in effect at the time of SRP 3.8.2 Rev. 1 (7/81) was ASME III 1980.

A comparison has been performed to determine the deviations between the current SRP 3.8.2 acceptance criteria for design limits and loading combinations and the corresponding criteria specified for BVPS-2.

The comparison resulted in the finding that the loading combinations and design limits of BVPS-2 are in close agreement with SRP 3.8.2 Rev. 1. The minor deviation noted below results exclusively from differences in the applicable edition of the ASME III Code for BVPS-2 and the applicable edition in effect at the time of Rev. 1 of SRP 3.8.2.

The deviation in the combinations and/or limits is that stress limits for the test condition are reduced somewhat in the 1980 version of the ASME code. Test condition allowable stresses are:

	General Membrane	Local Membrane	Bending and Local Membrane
1971 ASME Code:	0.90 Sy	1.25 Sy	1.25 Sy
1980 ASME Code:	0.75 Sy	1.15 Sy	1.15 Sy

The design of those portions of the BVPS-2 containment covered by ASME Section III, Division 1, is not governed by the test condition; thus this change is not considered significant.

15. The applicant should identify and justify the deviations of its internal structural design from the applicable requirements of the ACI-349 as amended by Regulatory Guide 1.142.

Response:

A review has been performed to identify the significant differences between the requirements for the BVPS-2 design of the internal structure of the reactor containment (FSAR Section 3.8.3), and the requirements of ACI 349-76 as amended by Regulatory Guide 1.142. The results of this review are summarized in Table 15.1; justifications of differences in design requirements are also given.

Table 15.1

Comparison of Concrete Design Criteria
for Containment Internal Structures

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
Chapter 1 General Req.	Requires copies of structural drawings, typical details and specifications to bear the seal of a licensed engineer.	Requires copies of structural drawings, typical details, and specifications be signed by licensed engineer.		BVPS-2 drawings and specifications bear the signatures of licensed engineers.
	Requires inspection by a competent engineer.	Requires inspection by Owner.	Recommends inspectors be experienced and familiar with ACI and ASTM standards.	Inspections are performed by a BVPS-2 Quality Control representative.
Chapter 2 Definitions	Massive concrete not specifically mentioned.	Requires areas to be treated as massive concrete to be identified on drawings or specifications.		Current practice is in accordance with ACI 349. BVPS-2 concrete specifications define the areas to be treated as massive concrete.
Chapter 3 Materials		Excludes use of air-entraining Portland Cement.		BVPS-2 concrete specification meets the intent of ACI 349 by requiring that only Type II low alkali cement be used.

Table 15.1 (Cont'd)

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
	No test reports required.	Requires test report on every cement shipment. No cement can be used prior to receipt of 7 day mill test strengths.		BVPS-2 specifications require mill tests for each shipment. Results are verified for conformance to ASTM C150 before use of cement is allowed.
	Allows the use of light-weight aggregate concrete.	Excludes use of light-weight aggregate concrete.		Lightweight concrete was not used at BVPS-2 for structural applications.
	Materials must conform to applicable ASTM specification.	Testing requirements established for aggregrate, reinforcing steel, and admixtures.		BVPS-2 specifications are in conformance with ACI 349.
	Not required.	Reinforcing steel shall be identifiable by documentation, tags or other means of control, to a specific heat number or heat code until review of the Certified Materials Test Report has been performed.		BVPS-2 specifications are in conformance with ACI 349.
	Allows use of rail steel and axle steel bars. Allows Grade 90 bars.	Requires use of billet steel reinforcing bars of Grade 60 or less, in order to limit crack formation.		Reinforcing steel specification and drawings require that rebar comply with ASTM A615 grade 50 for No. 14's and No. 18's and grade 60 for No. 11's and smaller.

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
Chapter 4 Concrete Quality	Allows proportioning of mix design using water-cement ratio tables in lieu of field experience or trial batch methods of proportioning mix.	Requires that mix design be based on field experience or laboratory trial batches.		BVPS-2 concrete specifications require that mixes be determined by trial batches.
		Provides method of determining water/cement ratio for fly ash mixes.		Fly ash mixes are not used in BVPS-2 structures.
	Requires use of Type V cement for sulfate exposure.	Defines sulfate exposure limit. Requires Type V cement or a fly ash mix for sulfate exposure conditions.		Sulfate exposure is not a problem for the concrete structures at BVPS-2.
	Requires 1 strength test/day/concrete class and at least once for each 150 cu yd of concrete placed or for each 5,000 sq. ft of concrete surface.	Permits test interval increase by 50 yd^3/100 psi lower standard deviation if standard deviation for 30 tests in a class is less than 600 psi. Not less than once per shift.	Similar to ACI 349 except at least once for each 100 cu yd placed unless otherwise qualified.	Current BVPS-2 practice is in accordance with Reg. Guide.
Chapter 5 Mixing and Placing Concrete		Prohibits use of aluminum pipe and chutes for conveying concrete.		BVPS-2 concrete specifications are in accordance with ACI 349.
	Allows partially hardened, contaminated, retempered, or remixed (after initial set) concrete to be used at the discretion of the engineer.	Prohibits use of such material.		BVPS-2 concrete specifications comply with ACI 349.

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
Chapter 6 Form Work Embedded Pipes, and Construction	Requires pressure test of embedded pipe to 50 percent above max. pressure (150 psi min.) for 4 hours.	Requires pressure test of embedded pipe "in accordance with the applicable piping code or standard." Otherwise the same requirements as ACI 318.		BVPS-2 complies with the position taken in ACI 349.
	Limits pressure and temperature of embedded piping to 200 psi and 150°F.	Allows 200°F for localized areas. Allows 350° for accident or short term periods. Allows 650°F for local areas from fluid jet from pipe failure. Allows higher temperatures if supported by test results.		BVPS-2 complies with ACI 349 criteria.
	Requires vertical construction joints to be wetted and coated with neat cement grout before placing next lift.	Requires all joints be shown on plans or approved by Engineer. All construction joints shall be wetted and standing water removed. Grout not required.		BVPS-2 shows all joints on plans and complies to ACI 349. In addition, horizontal construction joints are covered with a minimum 1/2 in. thick starter grout and vertical construction joints are coated with a thick-bodied cement water paste.
Chapter 7	The engineer may specify tolerances. Placement tolerances are stricter than ACI 301.	Tolerances on bar placement liberalized to ACI 301 standards for member sizes used in the internals. Tolerances on minimum concrete cover are larger than ACI 301.		BVPS-2 concrete specification complies with ACI 301 and is more conservative than ACI 349.

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
		Specifies minimum reinforcement for nuclear concrete structures.		Reinforcement shown on BVPS-2 drawings conforms to minimum requirements of ACI 349.
		Requires welded splices or positive connections for splicing load carrying reinforcing bars located in regions of membrane tension normal to the splice.		BVPS-2 requires that splices and anchorages comply with ACI 318 criteria. Since membrane tension forces are not a significant factor for BVPS-2 structures, the intent of ACI 349 is met.
Chapter 8 Analysis and Design General Considerations	Gives procedure for Alternate Design Method	Eliminates Alternate Design Method.		Alternate Design Method is not used at BVPS-2.
	Allows use of fillers in concrete joist construction.	Prohibits use of fillers in concrete joist construction.		Fillers are not used at BVPS-2.
Chapter 9 Strength and Serviceability Requirements		Requires consideration of dynamic response of concrete structure, foundation, and surrounding soil.		BVPS-2 practice is in accordance with ACI 349.
		Requires following load combinations applicable to the internals:		BVPS-2 combinations are:

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
	1. U = 1.4D + 1.7L	1. U = 1.4D + 1.7L + 1.7Ro		1. U=0.75(1.4D+1.7CL)
	2. U = 0.75 (1.4D + 1.7L + 1.9E)	2. U = 1.4D + 1.7L +1.7Eo + 1.7Ro	2. U = 1.4D + 1.7L + 1.9Eo + 1.7Ro	2. U=1.4D+1.7L
	3. U = 0.9D + 1.43E			3. U=1.4D+1.7L+1.9Eo
		4. U = D+L+To+Ro+Ess		4. U=0.9D+1.4Eo
				5. U=1.0D+1.0L+1.0Ess
		6. U = D+L+Ta+Ra+1.25Pa	6. U = D+L+Ta+Ra+1.5Pa	6. U=0.9D+1.0Ess
		7. U = D+L+Ta+Ra+1.15Pa +1.0(Yr+Yj+Ym) +1.15Eo	7. U = D+L+Ta+Ra+1.25Pa +1.0(Yr+Yj+Ym)+ 1.25Eo	7. U=D+L+Ta+Ra+1.5Pa
		8. U = D+L+Ta+Ra+1.0Pa +1.0(Yr+Yj+Ym) +1.0Ess		8. U=D+L+Ta+Ra+ 1.25Pa+1.0(Yr+ Yj+Ym)+1.25Eo
		9. U = 1.05D+1.3L+1.05To +1.3Ro	9. U = 1.05D+1.3L+1.3To +1.3Ro	9. U=D+L+Ta+Ra+ Pa+1.0(Yr+Yj+ Ym)+Ess
		10. U = 1.05D+1.3L+1.3Eo+ 1.05To+1.3Ro	10. U = 1.05D+1.3L+1.4Eo +1.3To+1.3Ro	The controlling BVPS-2 loads combinations are identical to those of ACI 349 and Regulatory Guide 1.142, therefore BVPS-2 meets the intent of ACI 349 and Regulatory Guide 1.142.

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
		For combination 7 and 8 local strength can be exceeded for Yr, Yj, and Ym, if no loss of safety-related system results.	For combinations 7 and 8 local strength can be exceeded if no loss of function of any safety-related structures systems or components.	BVPS-2 criteria allow local stresses to be exceeded if there is no loss of function of any safety-related system.
Chapter 10 Flexure and Axial Loads		Specifies minimum requirements for distribution of reinforcement in beams and one-way slabs.		Reinforcement distribution exceeds minimum requirements for the internals structure.
Chapter 11 Shear and Torsion		Establishes punching shear allowables for slabs and walls based on the ratio of the long side to short side of concentrated load or reaction area with consideration given for presence of membrane stresses.		BVPS-2 limits the nominal permissible punching shear stress carried by concrete to $4\sqrt{f'_c}$. Membrane tension forces are not significant and aspect ratios are usually 1.0.
Chapter 12 Development and Splices of Reinforcement		Requires testing of mechanical connections.		BVPS-2 tests for mechanical connections are in accordance with ANSI/ASME N45.2.5-1978.
Chapters 13 thru 17		No significant changes.		

Table 15.1 (Cont'd)

Code Section	ACI 318-71	ACI 349-76	Regulatory Guide 1.142 Rev. 1, Oct 1981	Justification
Chapter 18 Prestressed Concrete				There is no prestressed concrete in BVPS-2 structures.
Chapter 19 Shells	Applies only to thin shell concrete structures.	Applies only to the design of shell concrete structures having thicknesses equal to or greater than 12 in.		Not applicable to BVPS-2 structures.
ACI 349-80 Appendix A Thermal Considerations	Not included in ACI 318.			BVPS-2 meets the requirements of ACI 349.
ACI 349-80 Appendix C Special Provisions for Impulsive and Impactive Forces	Not included in ACI 318.		Sets permissible ductility ratios	BVPS-2 meets the requirements of ACI 349 and Regulatory Guide 1.142.

16. The applicant took some exceptions to the provisions of Regulatory Guides 1.10, 1.55, 1.69, 1.94, 1.115, 1.143. Deviations from these Regulatory Guides should be identified and justified by the applicant.

Response:

BVPS-2 positions on Regulatory Guides 1.10, 1.55, 1.94, and 1.115, which identify and justify deviations from the guides, are presented in FSAR Table 1.8-1. Positions on Regulatory Guides 1.69 and 1.143 are revised as follows. These revisions to Table 1.8-1 will be included in a future FSAR amendment.

TABLE 1.8-1 (Cont)

RG No. 1.69, Rev. 0
FSAR Reference Section 12.3.2

CONCRETE RADIATION SHIELDS FOR NUCLEAR POWER PLANTS (DECEMBER 1973)

Regulatory Guide 1.69 invokes the requirements and recommended practices contained in ANSI N101.6-1972, "Concrete Radiation Shields." The design and construction procedures for Beaver Valley Power Station - Unit 2 (BVPS-2) will meet or exceed the guidance of Regulatory Guide 1.69, with the following alternatives:

1. ANSI N101.6-1972 requires that shop drawings be prepared showing details and dimensions of formwork, and then approved by the responsible engineer before fabrication of the formwork may begin. On BVPS-2, it is the responsibility of field personnel to visually check all formwork. Detail drawings are made only for special applications.

2. Finishing and patching of concrete surfaces after removal of forms will conform to Chapter 9 of ACI-301 rather than Section 8.7.5 of ANSI N101.6. It is not necessary to complete this work within 96 hours after the placing of concrete.

3. Section 8.2.4 of ANSI N101.6 lists the maximum vertical drop of concrete as 5 feet. The maximum vertical drop of concrete during placement operations is 6 feet. Experience has indicated that suitable equipment and provisions are given to prevent segregation of the concrete.

TABLE 1.8-1 (Cont)

RG No. 1.143, Rev. 1

FSAR Reference Sections 3.8.4, 10.4.8, 11.2, 11.3, 11.4, 11.5

DESIGN GUIDANCE FOR RADIOACTIVE WASTE MANAGEMENT SYSTEMS, STRUCTURES, AND COMPONENTS INSTALLED IN LIGHT-WATER-COOLED NUCLEAR POWER PLANTS (OCTOBER 1979)

Beaver Valley Power Station - Unit 2 meets the intent of Regulatory Guide 1.143 with the following alternatives:

Paragraph C.1.1.3

The steam generator blowdown flash tank is located in the turbine building, a nonseismic structure.

Leakage from steam generator blowdown system components located in the turbine building is collected in the turbine building sumps and monitored prior to release to the environment. The turbine building drain release will be automatically secured when the concentrations exceed the maximum permissible concentration (MPC).

Paragraph C.1.2.2

The steam generator blowdown flash tank relief valve discharges to the discharge structure. The structure drains are not capable of being processed by the liquid waste system.

Releases from the steam generator blowdown system flash tank relief valve are anticipated operational occurrences, which release activity to the environment at concentrations equal to or less than that of secondary steam.

Paragraph C.1.2.3

The steam generator blowdown evaporator test tanks and the steam generator blowdown flash tank do not have curbs or elevated thresholds.

Test tanks receive distillate from the steam generator blowdown evaporators and provide facilities for storage and sampling prior to release to the environment. Calculations show that the tanks do not require shielding due to the low activity level of water expected in these tanks. A floor drain is provided in the general area of the test tanks to collect leakage and overflow in the auxiliary building sump for pumping to the liquid waste system. Refer to paragraph C.1.1.3 for the steam generator blowdown flash tank.

TABLE 1.8-1 (Cont)

Paragraph C.1.2.5

The refueling water storage tank is not provided with a dike or retention pond. Drains for the area adjacent to the structure are directed to the river via the storm drains. Tank overflow is directed to the liquid waste system via piping.

The refueling water storage tank is fabricated in accordance with the requirements of ASME III, Class 2. The atmospheric tank is tested full of water to ensure that there are no leaks, and undergoes 100-percent radiographic examination on the shell. Tank overflow is directed to the liquid waste system via the safeguards building sump piping. The radionuclide concentrations of the refueling water storage tank liquid will be determined following each refueling. When the concentrations exceed MPC, periodic surveillance of the tank area will be used to identify any leakage from valves and fittings. This will ensure minimal discharge of radioactive effluent to the storm drain system and subsequently to the environment.

Paragraph C.2.1.1

The gaseous waste system equipment meets or exceeds the codes in Table 1. The gaseous waste delay beds, gaseous waste surge tank, waste gas chiller, overhead gas compressors, piping and valves are designed and fabricated to ASME III. The gaseous waste storage tanks are designed and fabricated to ASME VIII, Division 1.

All equipment manufacturing codes exceed those specified by Table 1 of Regulatory Guide 1.143.

Paragraph C.4.3

The piping downstream of the overhead gas compressors is one-half inch.

The discharge piping for the overhead gas compressors is designed to not impose any unnecessary pressure loss in that portion of the gaseous waste system.

TABLE 1.8-1 (Cont)

Paragraph C.5.1.1

Beaver Valley Power Station - Unit 2 (BVPS-2) does not apply the Regulatory Guide 1.60 design ground response spectra. Instead, a BVPS spectrum is applied as described in Section 3.7B.1.

Paragraph C.5.2.1

Beaver Valley Power Station - Unit 2 does not use Regulatoy Guide 1.60 spectra nor Regulatory Guide 1.61 damping values. Instead, refer to Sections 3.7B.1.1 and 3.7B.1.3.

Paragraph C.5.2.2

Beaver Valley Power Station - Unit 2 complies with this section, except that the spectra referenced in Paragraph C.5.2.1 is used. Refer to Sections 3.7B.1 and 3.7B.2.

Paragraph C.5.2.3

Beaver Valley Power Station - Unit 2 uses the modal time-history technique to generate floor response spectra. Refer to Section 3.7B.2.

Paragraph C.5.2.4

Beaver Valley Power Station - Unit 2 uses ACI-318-71. This was the code in effect at the time of design. Any differences between ACI 318-71 and ACI 318-77 are considered to be insignificant.

Paragraph C.6

Quality assurance programs used for the manufacture of the equipment used in the radwaste management systems are in accordance with the codes and standards specified in the equipment purchase specifications.

21. With respect to the issue of the rate of pressure drop for tornado definition, please provide a quantitative technical justification demonstrating that the intent of Regulatory Guide 1.76 is fully met. Also, address the adequacy of using a single degree of freedom modeling in assessing the structural response of a plate subject to the above indicated pressure drop loading.

Response:

The nonlinear pressure drop profile specified in Section 3.3.2.1 of the BVPS-2 FSAR is identical to the pressure drop profile resulting from the tornado wind model specified in WASH-1300. As stated in Regulatory Guide 1.76, the values for the design basis tornado characteristics are taken from the WASH-1300 document. Therefore, the pressure drop profile specified for BVPS-2 clearly meets the intent of Regulatory Guide 1.76.

The dynamic load factors for a series of single-degree-of-freedom systems, subjected to the pressure drop time history (Figure 21.1), were determined. The resulting DLF's were compared (Figure 21.2) to those resulting from the linear pressure drop profile (Figure 21.1) suggested in NRC Question 451.2.

A review of the Category I structures was performed to determine the wall or roof panel that would have the lowest natural frequency when modelled as a single-degree-of-freedom dynamic system. The review determined that the lowest resulting natural frequency is approximately 9.0 cps.

It is seen from Figure 21.2 that for systems with natural frequencies greater than 4 cps the resulting DLF's are essentially the same for both pressure drop profiles.

Therefore, since the lowest natural frequency of any panel, (9 cps), is significantly above 4 cps, either profile will result in the same equivalent pressure load. Thus, it can be concluded that the intent of Regulatory Guide 1.76 has been met. The panels can be adequately modelled as single-degree-of-freedom systems based on the procedures presented in Biggs (1964) because the loading is nonfluctuating in nature and is applied uniformly over the surface of the panel.

References:

1. WASH-1300. "Technical Basis for Interim Regional Tornado Criteria," (UC-11) USAEC, Washington, D.C.
2. Biggs, John M. 1964. "Introduction to Structural Dynamics," McGraw Hill Book Company.

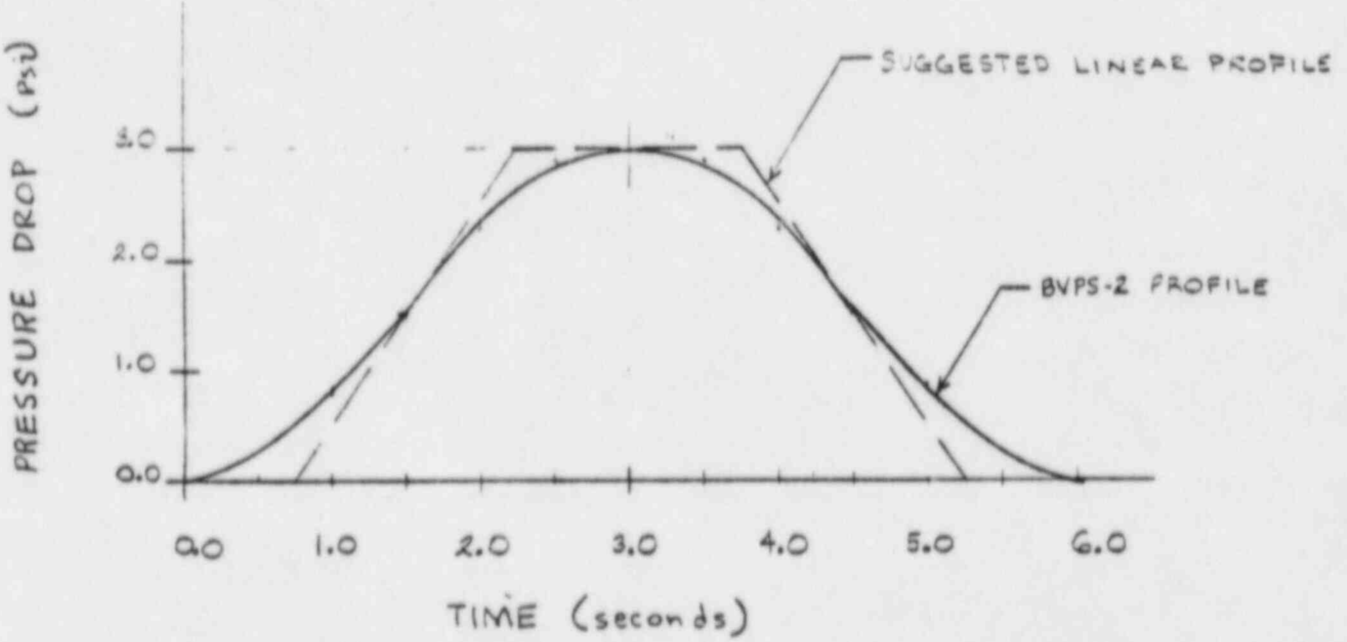


FIGURE 1

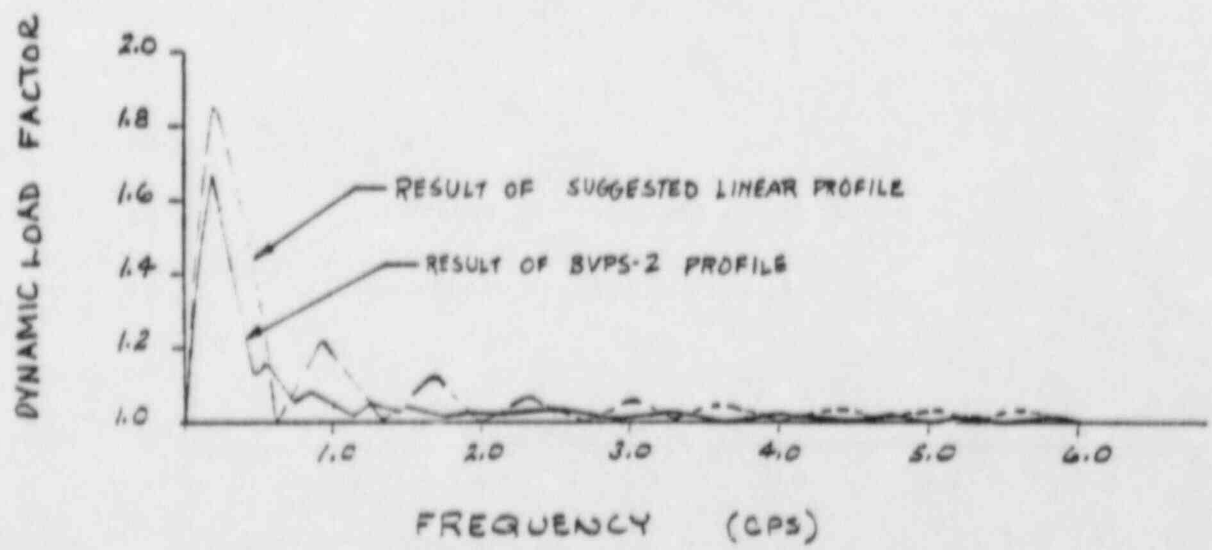


FIGURE 2

Figures 21.1 and 21.2

Pressure Drop and
Dynamic Load Factor
Beaver Valley Power Station
Unit 2

22. Evaluate the (11) Category I tanks with respect to the criteria of SRP Section 3.8.4 in order to demonstrate their design adequacy. The evaluation should consider the following parameters:

    1. Use realistic modeling of the tanks, accounting for the flexibility of the tanks.

    2. Consider any surrounding soil embedment as well as pipe anchoring load effects.

    3. Consider both sloshing and overturning effects.

    4. Evaluate the adequacy of the tanks against buckling failure.

    5. Ensure that foundations, anchoring bolts, and other connections to the tanks are adequately designed.

    For any deviations from the above SRP Section 3.8.4 criteria, use as-built material strength, applicable test results, and design conservatisms to justify the deviations.

    Response:

    1. The eleven Category I tanks were designed in accordance with ASME III Subsection NC for the Class 2 tanks and Subsection ND for the Class 3 tanks.

       The effects of tank wall flexibility have recently been addressed using Housner's deformable tank wall model (Haroun and Housner 1981). This procedure assumes the hydrodynamic pressures developed in ground-supported tanks during an earthquake is defined by the combination of a long-period convective component (related to sloshing), a short-period flexible component which is caused by the vibration of the tank walls, and a rigid liquid pressure component which moves simultaneously with the horizontal acceleration at the base of the tank.

       The two large relatively thin-walled tanks (refueling water storage tank and the primary demineralized water storage tank) with low fundamental impulsive frequencies had sufficient margin in the existing design so that the increased loads due to wall flexibility could be sustained. The remaining nine smaller diameter relatively thick-walled tanks had fundamental impulsive frequencies where spectral accelerations in the rigid range apply. In this frequency range, tank wall flexibility analysis is not required.

    2. Ten of the eleven tanks are above ground so the surrounding soil embedment is not applicable. The one buried tank (fuel oil day tank) is encased in concrete. Pipe anchoring load effects for either the buried or above ground piping have been factored into the designs.

3. For the refueling water storage tank and the primary demineralized water storage tank, overturning moments and base shears were computed by combining the sloshing mode with the flexible mode and rigid mass mode as described in Item 1 above.

   Sloshing effects (U.S. Atomic Energy Commission 1963) were also considered in the analysis of the boric acid tank and quench spray chemical addition tank to determine overturning moments and base shears. Sloshing effects were not considered for the remaining seven tanks which are either horizontally mounted or relatively slender tanks, and, therefore, little or no mass participates in the sloshing mode.

4. Buckling failure has been addressed in the analyses of the refueling water storage tank and the primary demineralized water storage tank, which are large flat-bottomed tanks. The designs of these tanks are such that the compressive stresses are less than the allowable compressive stress (based on the allowable compressive stresses of NC/ND 3133.6 increased in accordance with Table NC/ND 3821.5-1). Tank wall buckling is not considered a credible failure mode for the remaining nine tanks due to their geometry (relatively small diameter thick-walled tanks) with spectral accelerations in the rigid range.

5. The design of the foundations is in accordance with ACI guidelines as specified in Section 3.8.5. Anchor bolts and other connections are designed within the allowable limits of the AISC code as specified in Section 3.8.4.

References

1. Haroun, M. A. and Housner, G. W. 1981. "Seismic Design of Liquid Storage Tanks," Journal of Technical Councils of ASCE, April 1981.

2. U.S. Atomic Energy Commission 1963. "Nuclear Reactors and Earthquakes," TID-7024, Washington, D.C.

3. Coates, D. W. "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria - CR 1161, Lawrence Livermore Laboratory.

24. Perform a simplified stick model dynamic analysis of the cable tunnel accounting for the overburden and embedment effects. Demonstrate that the provisions of SRP Section 3.8.4 are met. For any deviations from the SRP criteria identified, provide a justification considering the as-built material strength and conservatisms. Also, account for any significant relative ground motions upon the tunnel seismic responses.

Response:

The original soil-structure model was modified and reanalyzed to further consider embedment and overburden effects on the cable tunnel. The original structure model consisted of two lumped masses (6 degrees-of-freedom per mass) interconnected by a weightless elastic beam member. The lumped masses represent the floor and roof slabs plus the walls in between. The elastic beam member represents the stiffness characteristics of the wall system. The founding material (soil) was modeled as elastic springs, based on elastic half-space theory (FSAR Section 3.7B.2).

The original model was modified as follows:

1. Overburden was accounted for by increasing the mass properties at the roof elevation.

2. Embedment was accounted for by modifying the elastic half-space soil springs by the correction factors developed by Kausel and Roesset (1975) and Kausel, Whitman, et al. (1978).

Analyses of the modified soil-structure model show that the fundamental frequency has increased by approximately 13 percent. This frequency change is within the limits of peak widening of the amplified response spectra used at BVPS-2 (+25 percent and -20 percent based on period).

The structural acceleration and displacement values from the reanalysis are lower than those used in the design of the cable tunnel. Thus the design is based on conservative results.

To address the effect of relative ground motions upon the cable tunnel design, an analysis was performed to determine the stresses resulting in the structure from relative ground displacements. This analysis was based on Kausel (1969), Newmark (1971), and Newmark and Rosenblueth (1971). It was assumed that the motion of the tunnel was the same as that of the surrounding soil. The resulting stresses are within the elastic limits for both axial and bending strains due to relative ground motions.

References:

1. Kausel, E. and Roesset, J. M. 1975. "Dynamic Stiffness of Circular Foundations," Journal of the Engineering Mechanics Division, ASCE, December 1975.

2. Kausel, E., Whitman, R. V., et al. 1978. "The Spring Method for Embedded Foundations," Nuclear Engineering and Design 48 (1978), North-Holland Publishing Company.

3. Kausel, T. R. 1969. "Earthquake Design Criteria for Subways" Journal of the Structural Division, ASCE, June 1969.

4. Newmark, N. M. 1971. "Earthquake Response Analysis of Reactor Structures," Nuclear Engineering Design 20 (1972), North-Holland Publishing Company, November 8, 1971.

5. Newmark, N. M. and Rosenblueth, E. 1971. "Fundamentals of Earthquake Engineering," Prentice Hall, Incorporated.

28. Please provide a simplified justification to demonstrate that T_A applicable to the cubicles in the containment need not be included in the design.

Response:

The BVPS-2 FSAR specifies in Section 3.8.3.3 loading combinations for design of the reactor containment interior concrete structures. It is stated therein that accident loads considered in the loading combinations control design of the internal concrete structures.

These accident loads are dynamic and decrease in magnitude with time. For the design of the cubicles peak values of P_a, R_a, Y_j, Y_r, and Y_m are applied simultaneously as they reach their maximum values within fractions of a second after initiation of an accident. This is demonstrated on Figure 28.1 for the pressure loading P_a. The temperature term T_a is considered in the cubicle design with these loads but its contribution to the state of stress at this early, but governing, time is insignificant.

A plot of the temperature T_a for the ambient air conditions inside and immediately outside the cubicles is presented on Figure 28.2. It is noted here that within 20 seconds after initiation of the accident, the ambient air temperatures inside and outside of the cubicle are identifical. This will lead to symmetric thermal stress without bending in the walls and floor slabs of the cubicles. With time and this uniform temperature throughout the containment internals, stress-free thermal growth will occur.

Because of the presence of the other early time dynamic accident loads the evaluation of stresses due to T_a immediately following the accident must be evaluated. A thermal analysis was performed to evaluate the resulting thermal gradient through the concrete wall/floor slab which conservatively considers the concrete surface temperature to be equal to that of the ambient air temperature. The analysis is based upon a thermal transient (Figure 28.3) applied to the inside surface of the crane wall which reaches a higher peak (350 °F) than the actual BVPS-2 transient for T_a . Although this transient reaches a higher peak it can still be used to demonstrate the T_a temperature profile through the concrete as a function of time (Figure 28.4). The depth of significant temperature penetration within the first 20 seconds is only a fraction of an inch. The maximum compressive thermal strain can be conservatively estimated as:

$$\epsilon_{thermal} = \alpha \Delta T$$

Where

α = 5.5×10^{-6} in./in./°F

ΔT = $T - T_{ref}$

T = maximum concrete temperature (275°F)

T_{ref} = stress-free temperature (70°F)

$\epsilon_{thermal} = 0.001 < \epsilon_{concrete\ allowable} = 0.003$

The results of this analysis demonstrate that the effects of T_a are of minor importance in the design of the cubicles, compared to the other governing accident loads.

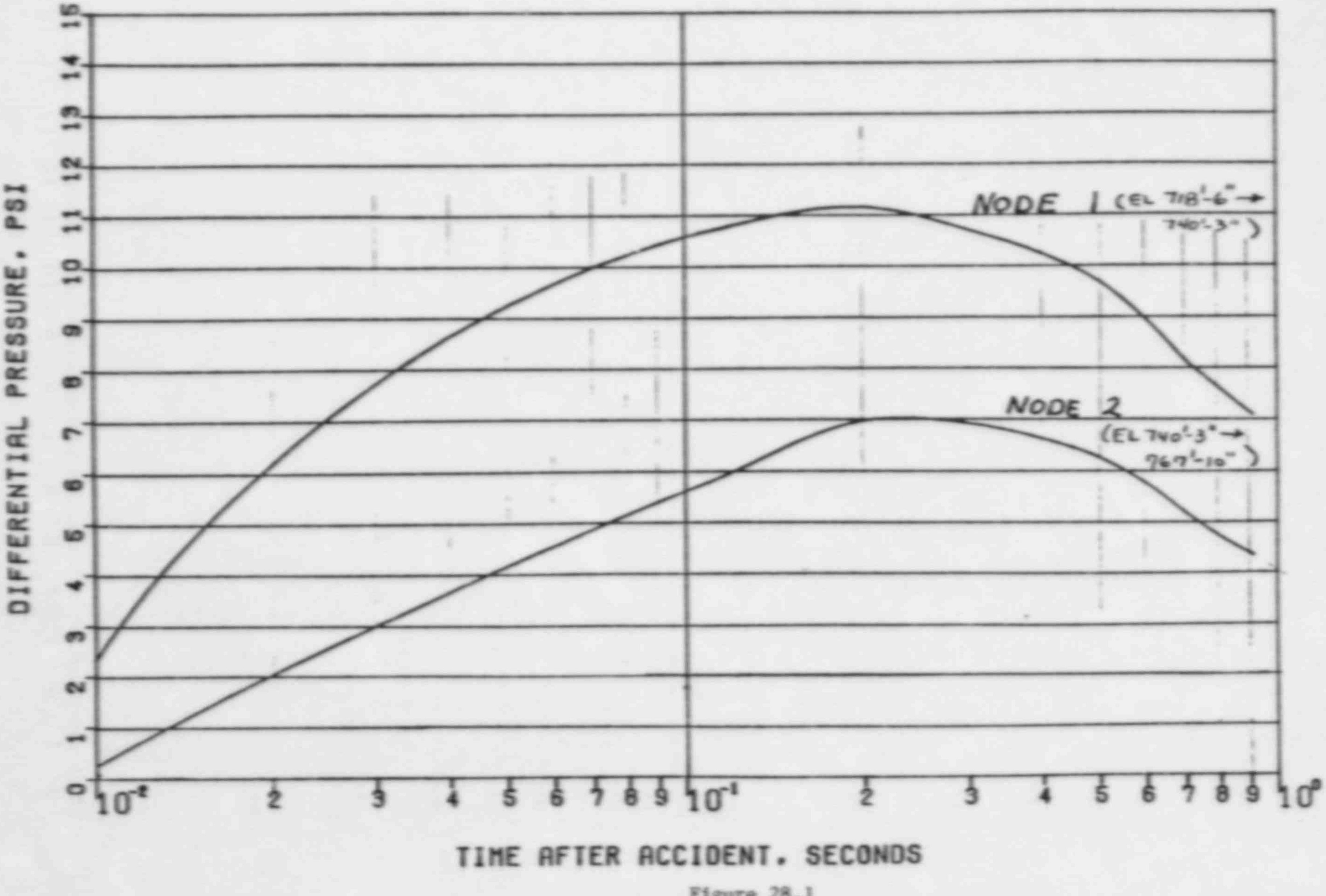


Figure 28.1
DIFFERENTIAL PRESSURE VS. TIME
STEAM GENERATOR CUBICLE, LOOP 1
BEAVER VALLEY POWER STATION UNIT 2

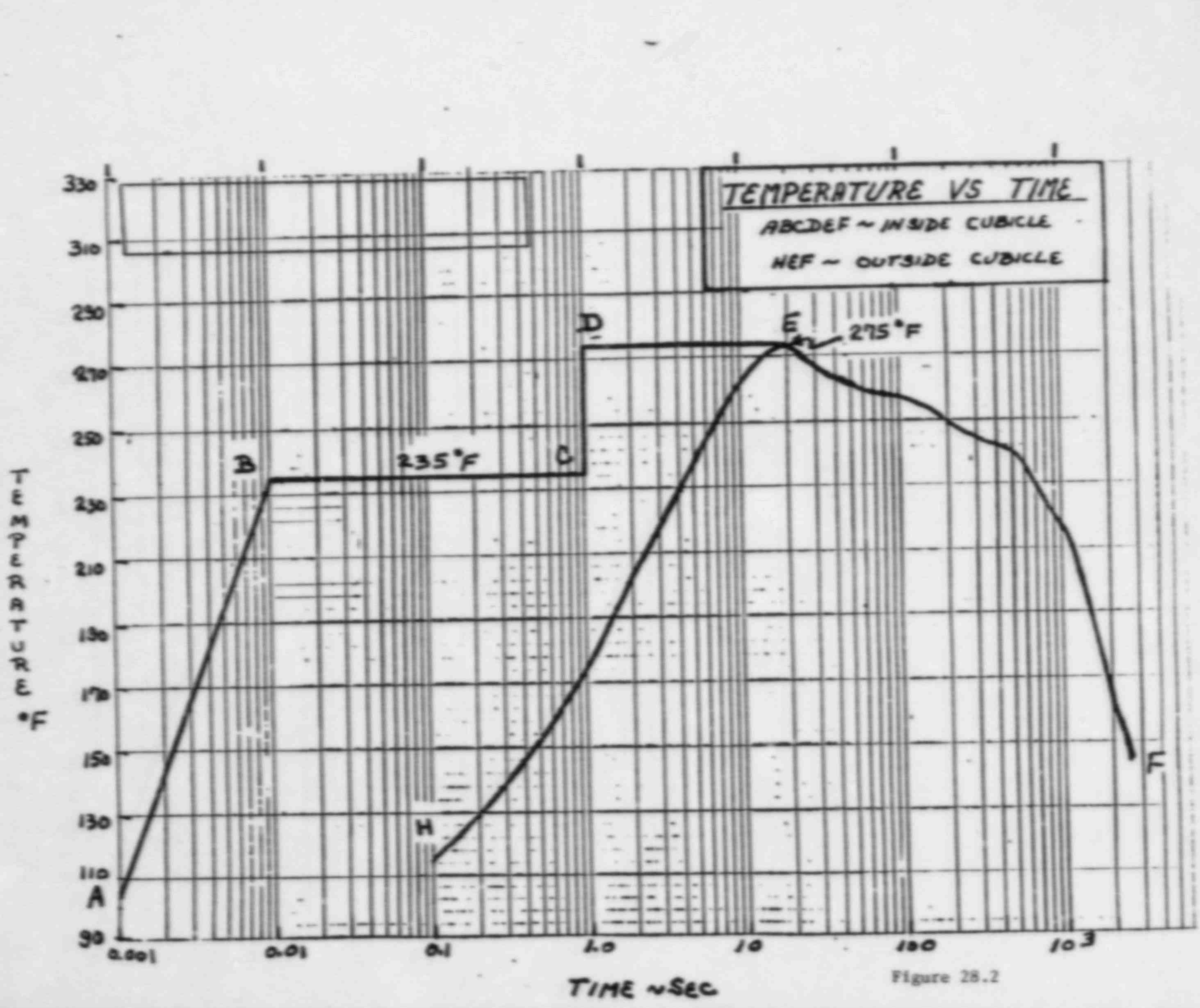

TEMPERATURE VS TIME
ABCDEF ~ INSIDE CUBICLE
HEF ~ OUTSIDE CUBICLE
TEMPERATURE °F
330
310
290
270
250
230
210
190
170
150
130
110
90
A
B
C
D
E
F
H
235°F
275°F
0.001
0.01
0.1
1.0
10
100
10^3
TIME ~ SEC

Figure 28.2

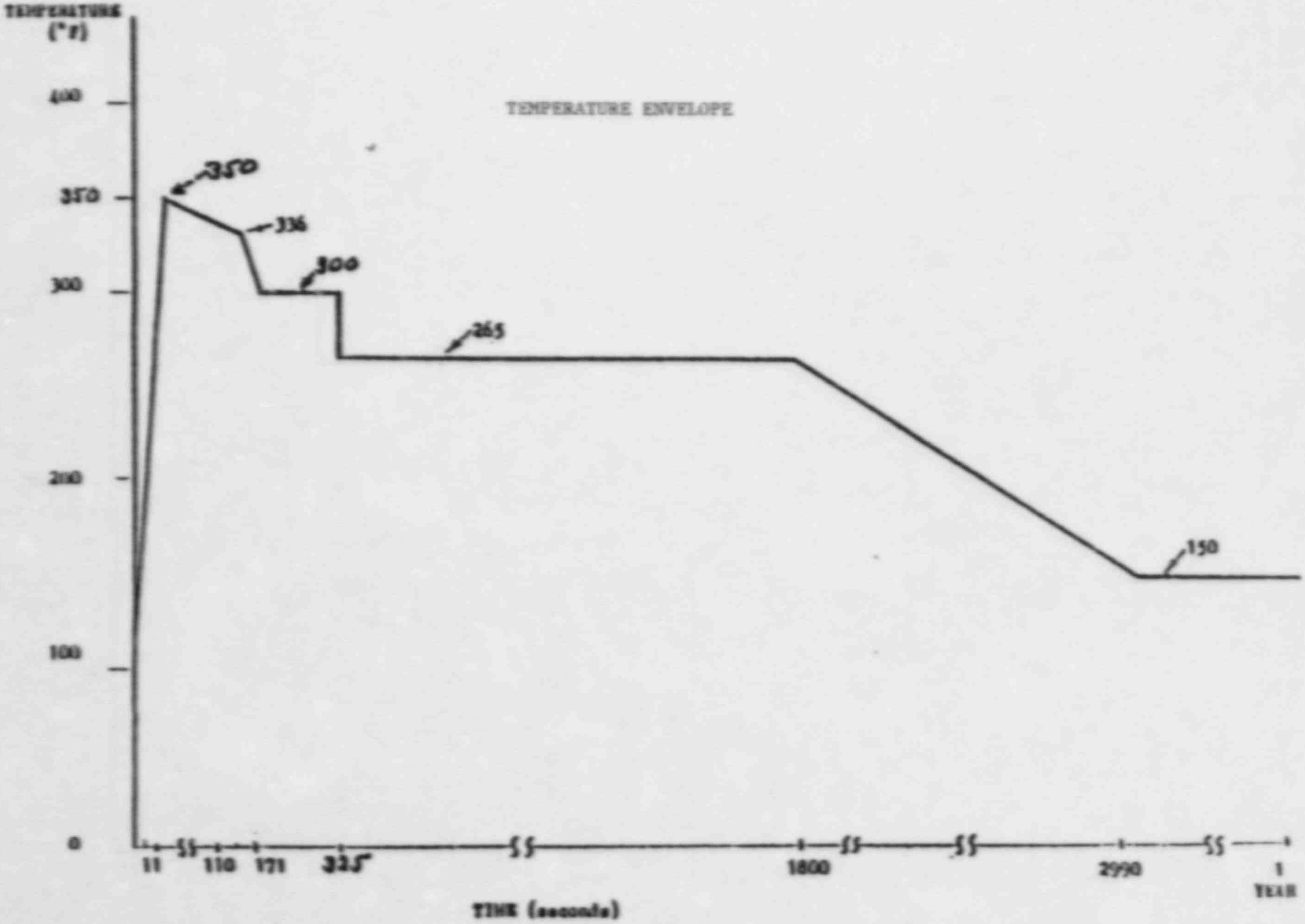
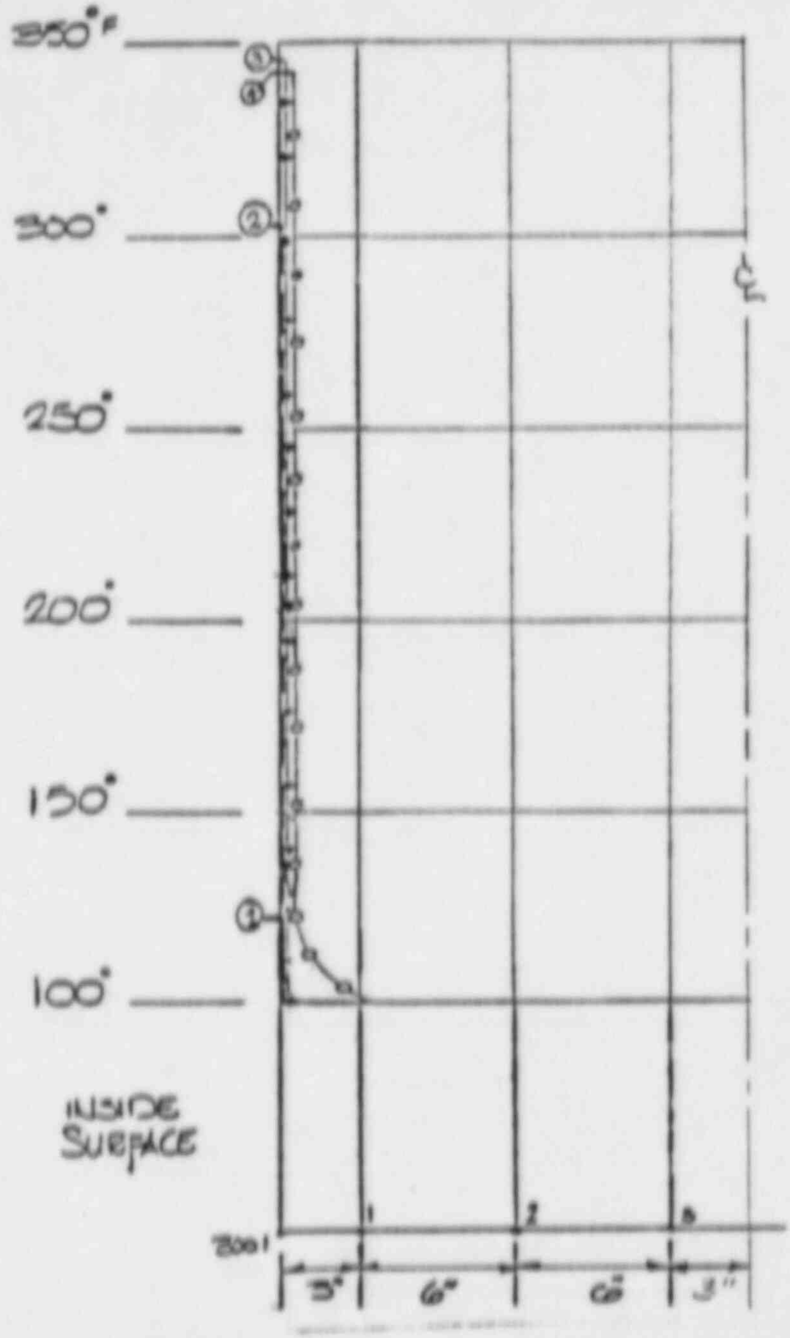


Figure 28.3
Typical Thermal Transient

Beaver Valley Power Station-
Unit 2



SYMMETRIC ABOUT ℄

THERMAL GRADIENT CRANE WALL (3' THICK)

		SURFACE TEMP	INTERNAL TEMP (1ST NODE)
①	2 SECONDS	122.73	100.00
②	10 SECONDS	306.55	100.10
③	30 SECONDS	347.45	100.06
④	60 SECONDS	343.21	101.22

Figure 28.4

Summary of Thermal Analysis

Beaver Valley Power Station - Unit 2

NRC STRUCTURAL AUDIT ACTION ITEMS

The following responses are provided in consideration of the Structural Engineering Section's comments on BVPS-2 Standard Review Plan differences contained in the NRC letter dated September 22, 1983.

SRP Section:
3.3.1.II.3 - With respect to the definition of wind loading, the Beaver Valley 2 plant referred to ASCE Paper No. 3269 for derivation of wind pressure on structures. The pertinent SRP Section 3.3.1 Revision 2, however, considers the ANSI A 58.1 code as the acceptable base document for defining wind loads.

The applicant should demonstrate that the wind load definition and procedures from ASCE Paper No. 3269 are as conservative as those of the ANSI A 58.1 code.

Response:

For a fastest-mile wind speed 30 ft above ground of 80 mph (FSAR Section 3.3.1.1), the effective velocity pressures determined in accordance with ASCE Paper No. 3269, which are used for BVPS-2, are comparable to those required by ANSI A58.1-1972. Figure 1 shows the values from ANSI 58.1, Table 5, Exposure C as well as those used for BVPS-2.

As can be seen, the BVPS-2 velocity pressures for structures up to 50 ft wide (measured perpendicular to wind direction) exceed ANSI A58.1 pressures in all cases. For structures 50 to 100 ft wide, BVPS-2 exceeds ANSI A58.1 except for an insignificant band from approximately 42 to 50 ft above ground. For structures 100 to 150 ft wide, BVPS-2 exceeds ANSI A58.1 except for minor bands from 30 to 50, 100 to 150, and 360 to 400 ft above ground. The BVPS-2 pressures for structures greater than 150 ft wide are generally less than those of ANSI A58.1.

To address structures greater than 150 ft wide, where BVPS-2 velocity pressure is less than ANSI A58.1, a review has been performed for the service building, considering a wind in the north-south direction. The service building is 186 ft wide and 70 ft high (FSAR Section 3.8.4.1.7). The review was performed by determining the velocity pressure in accordance with the procedure given in ANSI A58.1, Appendix A6.3.4.1. (If used for structures of lesser width, this procedure would not alter the comparison made above.) This procedure applies a "dynamic approach to the action of wind gusts." Using this procedure, the resulting velocity pressures, up to the building height, are approximately 10 percent less than those given in ANSI A58.1, Table 5, Exposure C (Figure 3.3.1.II.3-1). Also, these velocity pressures are within approximately 10 percent of the BVPS-2 pressures.

Therefore, while there is a difference in the manner in which building response to wind gusts is considered by ANSI A58.1 and ASCE Paper No. 3269, there is no significant difference between the wind velocity

pressures used for BVPS-2 and those determined in accordance with ANSI A58.1-1972.

For parts and portions of buildings and structures, ANSI A58.1 requires that greater velocity pressures be postulated for tributary areas of less than 1000 sq ft. Larger pressure coefficients are required to be applied at corners, ridges, eaves, etc. These are intended to provide adequate margin in the design of conventional exterior sheathing to resist localized wind loads. The exterior panels of seismic Category I structures at BVPS-2 are characteristically reinforced concrete 24-inches thick. These exterior panels are adequate to withstand local wind loads within the structural acceptance criteria for load combinations that include wind load.

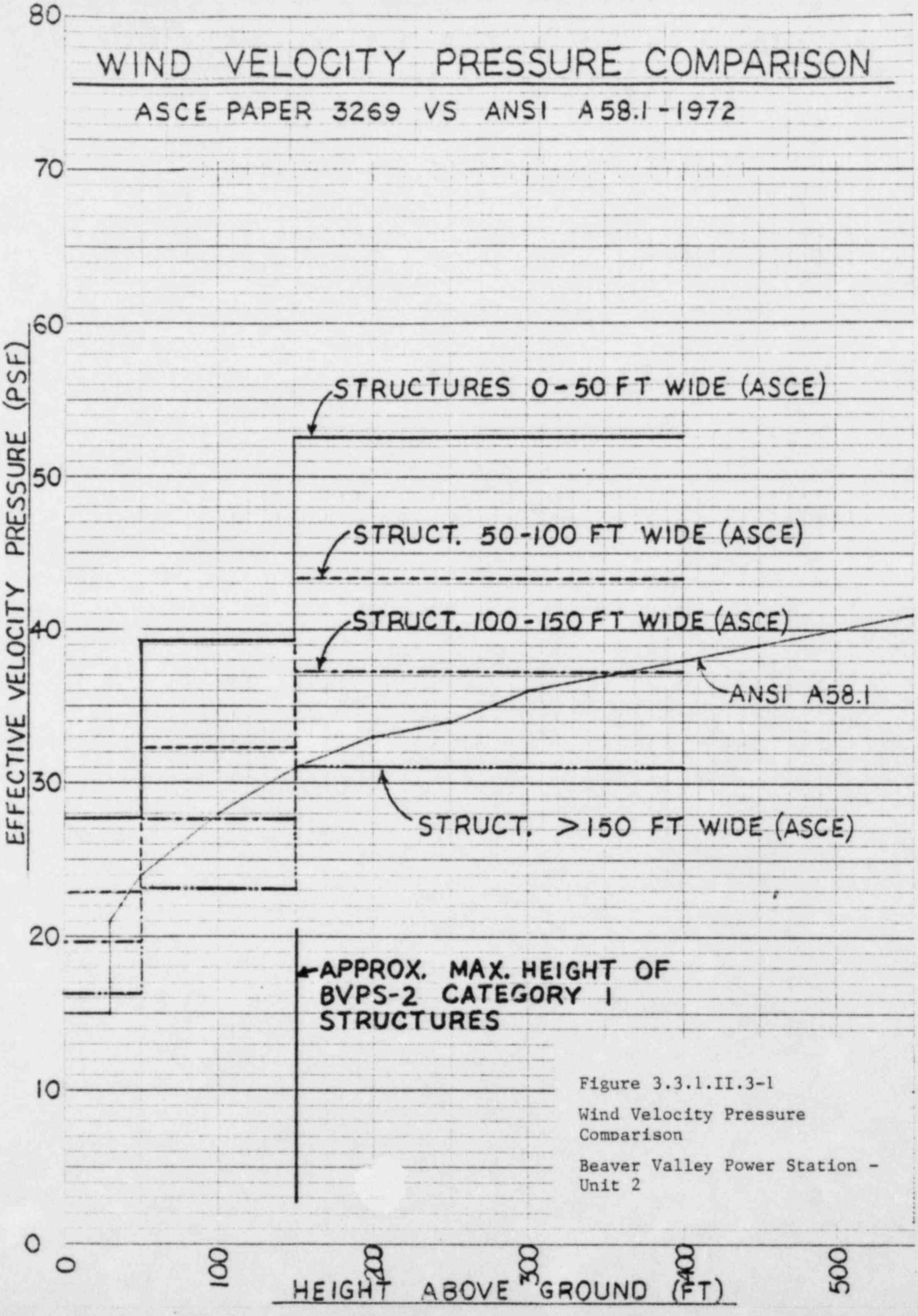


Figure 3.3.1.II.3-1

Wind Velocity Pressure Comparison

Beaver Valley Power Station - Unit 2

NRC STRUCTURAL AUDIT ACTION ITEMS

SRP Section:

3.7.3.II.7 - The SRP specifies that closely spaced modes be combined in accordance with Regulatory Guide 1.92. The applicant stated that the Westinghouse methods were used for combining closely spaced modes. The applicant should provide technical data to show the equivalency of the above two approaches or justify the deviation.

Response:

The method of combining closely spaced modes for items within SWEC's scope is in accordance with Regulatory Guide 1.92, Rev. 1, dated February 1976.

Westinghouse, when combining closely spaced modes, used an alternative method to those endorsed by Regulatory Guide 1.92, Rev. 1. The Westinghouse "Epsilon Factor" method, which is described in FSAR Section 3.7N.2.7, has been used only for items supplied and qualified by Westinghouse. This method has previously been reviwed by the NRC during Mechanical Engineering Branch reviews on the Seabrook, Shearon Harris, Comanche Peak, Catawba, and SNUPPS plants. The staff has accepted the "Epsilon Factor" method.