DESCRIPTION OF PROPOSED ENHANCEMENTS

TO THE

PVNGS CONTROL BUILDING ELEVATION 74'-0"

MASONRY WALLS

FOR THE

ARIZONA NUCLEAR POWER PROJECT

PALO VERDE NUCLEAR GENERATING STATION

UNITS 1, 2 AND 3

OCTOBER 1986



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DESCRIPTION OF PROPOSED ENHANCEMENTS TO THE PVNGS CONTROL BUILDING ELEVATION 74'-0" MASONRY WALLS

1. INTRODUCTION

In response to an NRC requirement⁽¹⁾ to strengthen the PVNGS Control Building masonry walls at Elevation 74'-0", a wall upgrade has been designed using conservative analysis techniques and assumptions.

The modifications consist of a series of vertical steel plates that sandwich the wall and are bolted together to form a composite section.

The modifications assure that wall masonry, reinforcement, and bond stresses will remain within conservative allowable limits under both OBE and SSE conditions. The analysis and design basis for the modification, as well as the resulting stresses, are described in the following sections.

2. SUMMARY AND CONCLUSIONS

The modification of the Control Building Elevation 74'-0" masonry walls consists of the addition of a series of steel plate assemblies that both strengthen and stiffen each of the three wall segments. Each plate assembly consists of a pair of vertical steel plates, one on each side of the wall, connected by pretensioned through bolts (see Figures 1 and 2). Plate length and spacing considers the location of existing penetrations, attachments and location of maximum out-of-plane bending. Torquing of the through bolts provides a friction connection between the plate and masonry surfaces that permits the transfer of shear forces resulting from out-of-plane bending.

In developing the design, anticipated methods of implementation and potential impact to existing safety related components and equipment were taken into account. These considerations resulted in the "sandwich plate" design which provides for ease of installation.

The modification design was based on finite element model analysis to determine the response of the walls. The steel plates increase the stiffness of the walls thus increasing the wall frequency to a range outside the peak spectral acceleration. This results in all stresses remaining below NRC established allowables. These enhancements will serve to strengthen the walls and increase the existing design margins. R

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3. DESCRIPTION OF MODIFICATIONS

The purpose of the modifications of the masonry walls at Elevation 74'-0" of the Control Building is to strengthen the wall and increase wall design margin for seismic loading conditions. This has been achieved by developing a modification that stiffens the walls, thereby reducing seismic response and limiting deflection and cracking.

The subject walls are located at Elevation 74'-0" of the Control Building, oriented along the north-south center line and separate the two essential air handling rooms (see Figure 1). The walls are non-shear, non-load bearing partitions subject to vertical and lateral seismic inertial loads due to their own weight and the weight of light attachments (instrumentation tubing, conduit, junction boxes, etc.).

The wall modification consists of a series of vertical steel plate assemblies connected to the wall by through bolts (see Figures 1 and 2). Each assembly consists of two plates located on opposing sides of the wall and connected by pretensioned threaded rods (through bolts) that clamp the plates together. The plate assemblies become an integral part of the wall, permitting composite action, when the through bolts are torqued allowing the transfer of shear stresses resulting from out-of-plane bending. The plates vary in length from approximately 8 to 17 feet long, are centered at approximately the midheight of the wall, and are spaced approximately 3 to 6 feet apart horizontally. The spacing and length of the plate assemblies are varied in order to avoid existing penetrations and minimize the relocation of existing wall attachments. To preclude inadvertent cutting of main vertical reinforcement, the through bolts are installed only through the center webs of the masonry units.

In order to minimize potential impacts on the plant and to assure the safe implementation of these modifications, the design and construction of the modifications has been optimized to include the following considerations:

- 1. Manageable size pieces of steel plates for easier installation.
- 2. Flexibility in placement of the steel plates to accommodate physical restrictions caused by existing components and, therefore, minimizing relocations.
- 3. Bolted construction utilizing the least amount of bolts required, thereby minimizing drilling operations, and minimizing in-plant welding.

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4. DESIGN AND ANALYSIS

A. Methodology and Assumptions

The masonry wall modification is designed as a composite section consisting of reinforced masonry and steel plates. The plate assemblies are located to stiffen and strengthen the critical sections of each wall to reduce stresses under seismic conditions. The plate assemblies and wall were analyzed by use of a finite element model of each of the three walls, using response spectra techniques. The design spectra used were developed from the published floor response spectra by scaling to 0.1g for OBE and 0.2g for SSE and enveloping the response at elevations 74'-0" and 100'-0".

A two-dimensional finite element model was utilized in order to accurately represent the stiffening plates in their actual locations and thus take into account variations in plate length and spacing. The 22 foot high by 27 foot long wall segments were modeled using plate elements 12 inches thick, 16 inches wide, and 8 or 16 inches high. The models include all major penetrations and door openings (see Figures 3, 4, and 5). The steel plate assemblies were modeled by calculating the stiffness of the transformed steel/masonry section and appropriately increasing element stiffness in the areas corresponding to the plate assembly locations. The boundary conditions of the wall models were chosen to reflect the as-built connection details. Wall mass includes the dead weight of the wall, plate assemblies, and attachments.

The plate element properties used in the response spectrum analysis represented either fully cracked or uncracked sections, depending on the level of applied moment. Each analysis was initiated by assuming all masonry elements were uncracked and had the stiffness of the gross section. When resulting moments were determined to cause cracking in a masonry element, the properties of that element were changed to represent a fully cracked section (i.e., masonry cracked to neutral axis, tension carried by reinforcement and where applicable steel plate). This iterative process was repeated until an equilibrium condition was reached. The moment required to crack an element was calculated based on the 1985 Uniform Building Code (UBC) modulus of rupture value for masonry (f_r) of 97 psi. In addition to simplyfing the analysis, the use of only uncracked and fully cracked elements yielded conservative results, since the increased tensile capacity of the grout and the stiffness of partially cracked sections were not considered (i.e., no 3-stage behavior was assumed).

Because dowel placement was confirmed by documented inspection during construction, as-designed "d" values were used for doweled elements. For all remaining elements the cracked stiffness was , r , r

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determined using the average as-built "d" distance values obtained from PVNGS Non-Conformance Report CJ-5343.

The minimum expected PVNGS masonry compressive stress, (f'_m) , is 2000 psi. However, for stress calculations, the conservative value of $f'_m = 1500$ psi was used per UBC-79.

For modulus of elasticity, the American Concrete Institute (ACI) 531-79 Code and UBC-79 specify a value of 1000 f'_m . Utilizing the expected masonry compressive strength, the modulus of elasticity value, E_m , would be 2.0 x 10⁶ psi. However, it is known that the code equation may be unconservative and therefore to address NRC concerns⁽¹⁾ an E_m value of 1.5 x 10⁶ was used, which is 750 x f'_m considering the expected f'_m value. See Table 1 for major design and analysis parameters.

The methodology and assumptions used conform to the requirements of the PVNGS Final Safety Analysis Report (FSAR), Section 3.7 and Bechtel Topical Report BC-TOP-4A, "Seismic Analysis of Structures and Equipment for Nuclear Power Plants" (referenced in FSAR Sections 1.6, 3.7 and 3.8). The seismic response for each wall segment was calculated for 0.1g OBE, 4% damping, and 0.2g SSE, 7% damping conditions.

Utilizing the maximum moment obtained from the response spectra analysis, stresses in each of the wall components (e.g., masonry, reinforcement and steel plates) were calculated following working stress design methods.

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B. Summary of Results

The seismic response of each wall is primarily dependent on the first mode (fundamental) frequency. The calculated first mode frequency for each wall segment, for both OBE and SSE conditions, is listed in Table 2. The amplified regions for the horizontal OBE and SSE wall specific (east-west direction, envelope of elevation 74'-0" and 100'-0") response spectra occur at frequencies less than about 6 Hz. By comparison of this value to the modified wall first mode frequencies contained in Table 2, it can be seen that the wall frequencies are higher than the frequency at which the amplified region begins.

The calculated frequencies, as given in Table 2, are shown on Figures 6 and 7 together with the wall specific spectra and the spectra used in the modification design. Since conservative assumptions are made regarding the parameters which determine the frequencies (moment of inertia, modulus of rupture, and modulus of elasticity), the calculated frequencies are considered to be lower bound estimates. Variations that could be expected in these parameters will increase the wall frequencies, away from the • • • • • • •

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The conservatisms associated with wall frequencies and the margin between the wall specific spectra and design spectra provide assurance that upper bound loads have been utilized to design the wall modifications. Therefore, these modifications would significantly increase the existing seismic design margin of the walls.

The maximum calculated masonry, reinforcement, bond, and steel plate stresses for all walls are summarized in Table 3 for both OBE and SSE conditions. For comparison purposes, the corresponding allowable stresses are also listed. The allowable stresses are based on f'_m = 1500 psi. Allowable masonry and reinforcement stresses are in accordance with ACI 531-79 and the provisions of Appendix A to SRP 3.8.4 (NUREG 0800, July 1981). To compensate for possible variations due to the absence of full in-process inspection, applicable allowable stresses have been reduced per ACI 531-79 guidelines. Allowable bond stresses conform to ACI 531-79 requirements and the recommendations of the NRC (1.5 increase The American Institute of Steel factor for SSE conditions). Construction (AISC) Steel Construction Manual, 8th Edition and the provisions of PVNGS FSAR, Section 3.8 are the sources for the In allowable stress values for the steel reinforcing plates. determining plate allowable compression stresses, the slenderness effects are taken into account.

As can be seen from Table 3, all calculated stresses are within the prescribed allowable values, for both OBE and SSE conditions. Therefore, it is concluded that the Control Building masonry walls at elevation 74'-0", modified as described herein, will provide additional margin and meet NRC acceptance criteria and will perform their intended function under postulated seismic conditions.

5. REFERENCES

(1) Letter from E. A. Licitra, NRC, to E. E. Van Brunt, Jr., ANPP, Dated October 6, 1986. Subject: Palo Verde Masonry Walls , , ũ

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TABLE 1

Major Parameters Used in the Design and Analysis of Masonry Wall Enhancements

Parameter	Description
Element size	16" x 16" x 12" (some 16" x 8" x 12")
Element stiffness	Uncracked (gross) section or fully cracked section
Boundary conditions	Reflect as-built connection configuration
Masonry compressive strength, f'm	2000 psi, expected 1500 psi, for stress calculations
Masonry modulus of elasticity, E _m	1.5 x 10 ⁶ psi
Masonry modulus of rupture, f'm	97 psi
Rebar yield strength	60,000 psi
Rebar location	Main reinforcement: as-built "d" Dowel reinforcement: as-designed "d"
Steel plates	1/2" and 3/4" thick, ASTM A36
Through bolts	5/8" and 7/8" diameter, ASTM A36 or A307
Response spectra	Published floor response spectra (scaled to 0.1g OBE and 0.2g SSE,) enveloped for elevation 74' and 100'
Masonry stress allowables	Per AC1 531-79 and SRP 3.8.4 (NUREG-0800)
Steel plate allowables	Per AISC, 8th Edition

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Parameter	Description
Element size	16" x 16" x 12" (some 16" x 8" x 12")
Element stiffnes	Uncracked (gross) section or fully cracked section
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Masonry compressive strength, f'm	2000 psi, expected 1500 psi, for stress calculations
Masonry modulus of elasticity, E _m	1.5 x 10 ⁶ psi
Masonry modulus of rupture, f'm	97 psi
Rebar yield strength	60,000 psi
Rebar location	Main reinforcement: as-built "d" Dowel reinforcement: as-designed "d"
Steel plates	1/2" and 3/4" thick, ASTM A36
Through bolts	5/8" and 7/8" diameter, ASTM A36 or A307
Response spectra	Published floor response spectra (scaled to 0.1g OBE and 0.2g SSE,) enveloped for elevation 74' and 100'
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Steel plate allowables	Per AISC, 8th Edition

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TABLE 2

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LOWER BOUND FIRST MODE FREQUENCIES OF MODIFIED MASONRY WALLS

WALLFREQUENCY (Hz)LOCATIONOBESSENorth8.47.3Center9.88.6South9.28.9

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TABLE 3

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STRESS SUMMARY FOR MODIFIED MASONRY WALLS

			STRESS	5 (psi)			
		0.1g	OBE	0.2g SSE			
		MAXIMUM		MAXIMUM			
C	OMPONENT	CALCULATED	ALLOWABLE	CALCULATED	ALLOWABLE		
Maso	nry		(1)		(1)		
Co	mpression	330	333	660	833		
			(1)		(1)		
Rein	forcement	11,200	24,000	21,100	48,000		
Те	nsion		-		<u> </u>		
Rein	forcement		(1)		(3)		
Bo	nd	95	120	165	180		
			(2)		(2)		
Plate	Compression	1,850	10,000	2,870	16,000		
		,	(2)	1 / I	(2)		
	Tension	2,630	22,000	3,850	35,000		

(1)

Allowable stresses based on ACI 531-79 and Appendix A to SRP 3.8.4. Allowable stresses based on AISC Steel Construction Manual, 8th (2) Edition.

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Allowable stress increased using NRC recommended value of 1.5. (3)



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FIGURE 1: MASONRY WALL MODIFICATIONS - CONTROL BUILDING PLAN ELEV. 74'-0"

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FIGURE 2: MASONRY WALL MODIFICATIONS - TYPICAL ELEVATION AND DETAILS

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FIGURE 3: NORTH PANEL FINITE ELEMENT MODEL



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FIGURE 4: CENTER PANEL FINITE ELEMENT MODEL

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FIGURE 5: SOUTH PANEL FINITE ELEMENT MODEL

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October 20, 1986

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DOCKET NO(S). 50-528 Mr. E. E. Van Brunt, Jr. Executive Vice President Arizona Nuclear Power Project P. O. Box 52034 Phoenix, Arizona 85072-2034
SUBJECT: ARIZONA PUBLIC SERVICE COMPANY, ET AL PALO VERDE NUCLEAR GENERATING STATION, UNIT NO. 1
The following documents concerning our review of the subject facility are transmitted for your information.
Notice of Receipt of Application, dated
Draft/Final Environmental Statment, dated
Notice of Availability of Draft/Final Environmental Statement, dated
Safety Evaluation Report, or Supplement No, dated
Notice of Hearing on Application for Construction Permit, dated
 Notice of Consideration of Issuance of Facility Operating License, dated Bit Reckly Monthly Notice; Applications and Amendments to Operating Licenses Involving no Significant Hazards Considerations, dated9/24/86 (See page 33961).
Application and Safety Analysis Report, Volume
Amendment Noto Application/SAR dated
Construction Permit No. CPPR, Amendment Nodated
Facility Operating License No, Amendment No, dated
Order Extending Construction Completion Date, dated
Other (Specify)

Office of Nuclear Reactor Regulation

Enclosures: As stated

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CC: See next page

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Mr. Charles B. Brinkman, Manager Washington Nuclear Operations Combustion Engineering, Inc. 7910 Woodmont Avenue Suite 1310 Bethesda, Maryland 20814

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Mr. Ron Rayner P. O. Box 1509 Goodyear, AZ 85338

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Chairman Arizona Corporation Commission Post Office Box 6019 Phoenix, Arizona 85003

Arizona Radiation Regulatory Agency ATTN: Ms. Clara Palovic, Librarian 4814 South 40 Street Phoenix, Arizona 85040

Mr. Charles Tedford, Director Arizona Radiation Regulatory Agency 4814 South 40 Street Phoenix, Arizona 85040

Chairman Maricopa County Board of Supervisors 111 South Third Avenue Phoenix, Arizona 85003

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