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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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.

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.

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

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 10^6 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 10^6 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.

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

amplified region of the spectra, thus resulting in further reduction of wall accelerations.

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 factor for SSE conditions). The American Institute of Steel Construction (AISC) Steel Construction Manual, 8th Edition and the provisions of PVNGS FSAR, Section 3.8 are the sources for the allowable stress values for the steel reinforcing plates. In 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

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	1.5×10^6 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 ACl 531-79 and SRP 3.8.4 (NUREG-0800)
Steel plate allowables	Per AISC, 8th Edition

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Steel plate allowables	Per AISC, 8th Edition

TABLE 2

LOWER BOUND FIRST MODE FREQUENCIES
OF
MODIFIED MASONRY WALLS

WALL LOCATION	FREQUENCY (Hz)	
	OBE	SSE
North	8.4	7.3
Center	9.8	8.6
South	9.2	8.9

[illegible]

[illegible]		[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

TABLE 3

STRESS SUMMARY FOR MODIFIED MASONRY WALLS

COMPONENT		STRESS (psi)			
		0.1g OBE		0.2g SSE	
		MAXIMUM CALCULATED	ALLOWABLE	MAXIMUM CALCULATED	ALLOWABLE
Masonry Compression		330	(1) 333	660	(1) 833
Reinforcement Tension		11,200	(1) 24,000	21,100	(1) 48,000
Reinforcement Bond		95	(1) 120	165	(3) 180
Plate	Compression	1,850	(2) 10,000	2,870	(2) 16,000
	Tension	2,630	(2) 22,000	3,850	(2) 35,000

(1) Allowable stresses based on ACI 531-79 and Appendix A to SRP 3.8.4.
(2) Allowable stresses based on AISC Steel Construction Manual, 8th Edition.
(3) Allowable stress increased using NRC recommended value of 1.5.

[illegible]

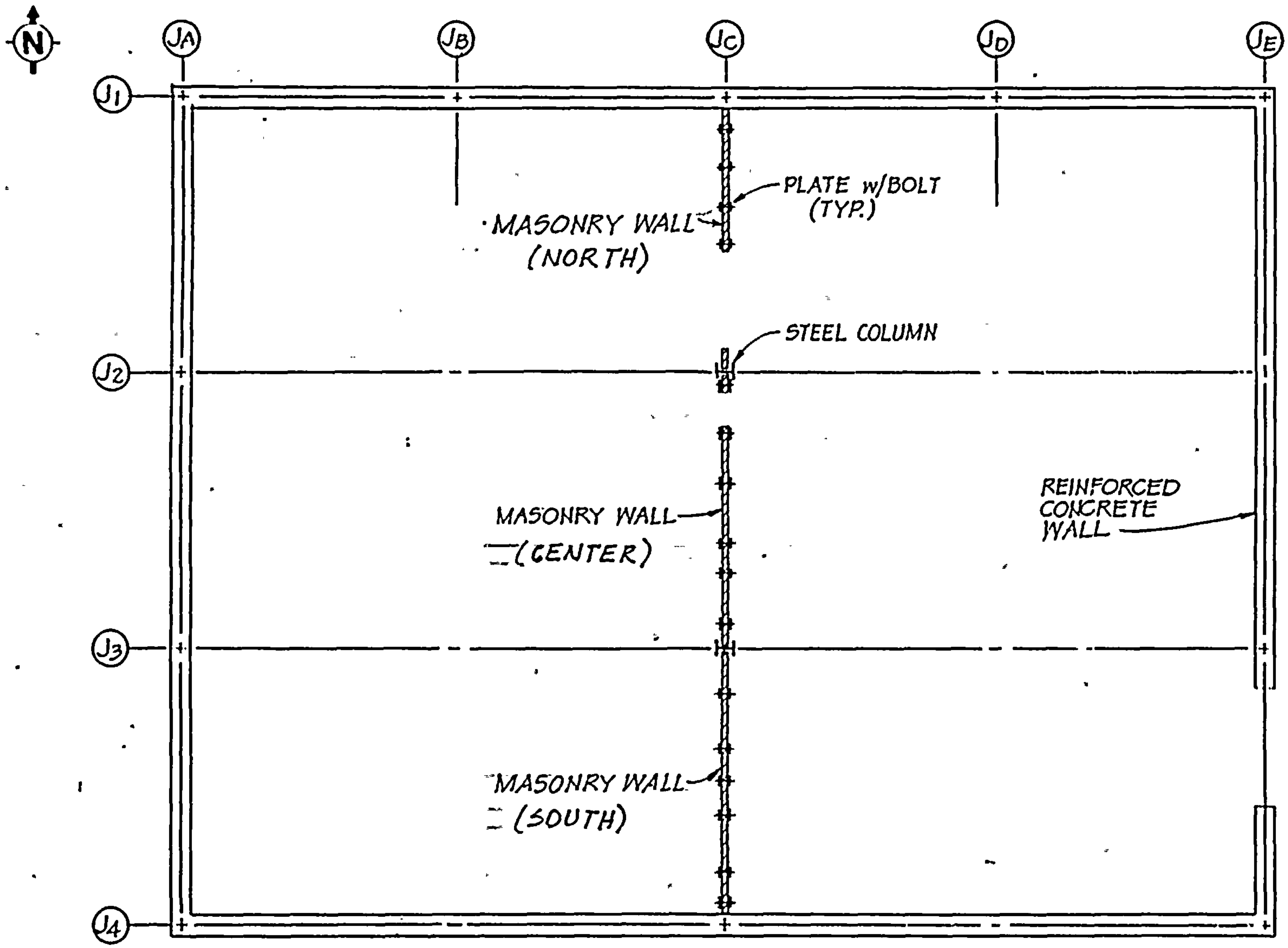

FIGURE 1: MASONRY WALL MODIFICATIONS - CONTROL BUILDING PLAN ELEV. 74'-0"

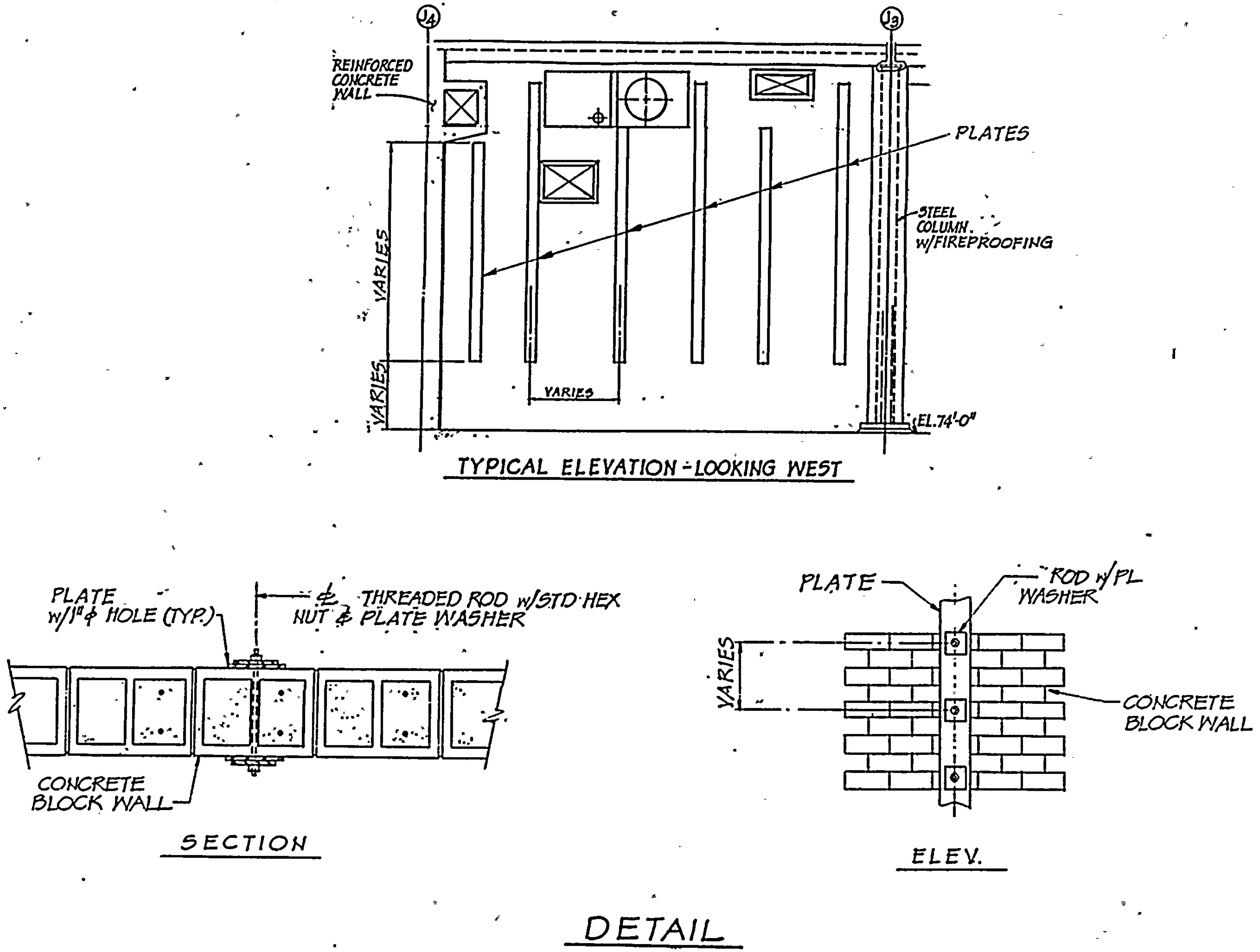

FIGURE 2: MASONRY WALL MODIFICATIONS - TYPICAL ELEVATION AND DETAILS

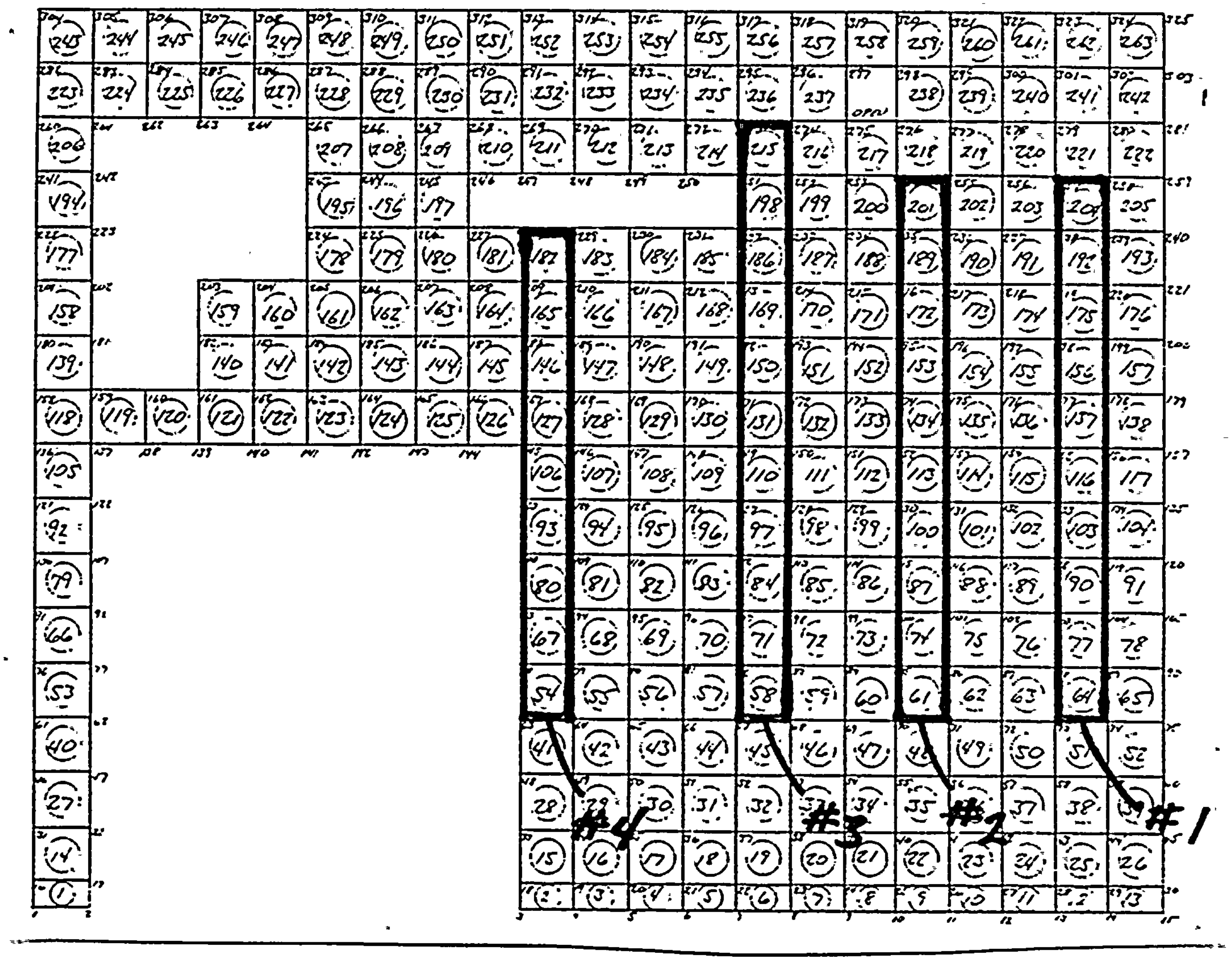

FIGURE 3: NORTH PANEL FINITE ELEMENT MODEL

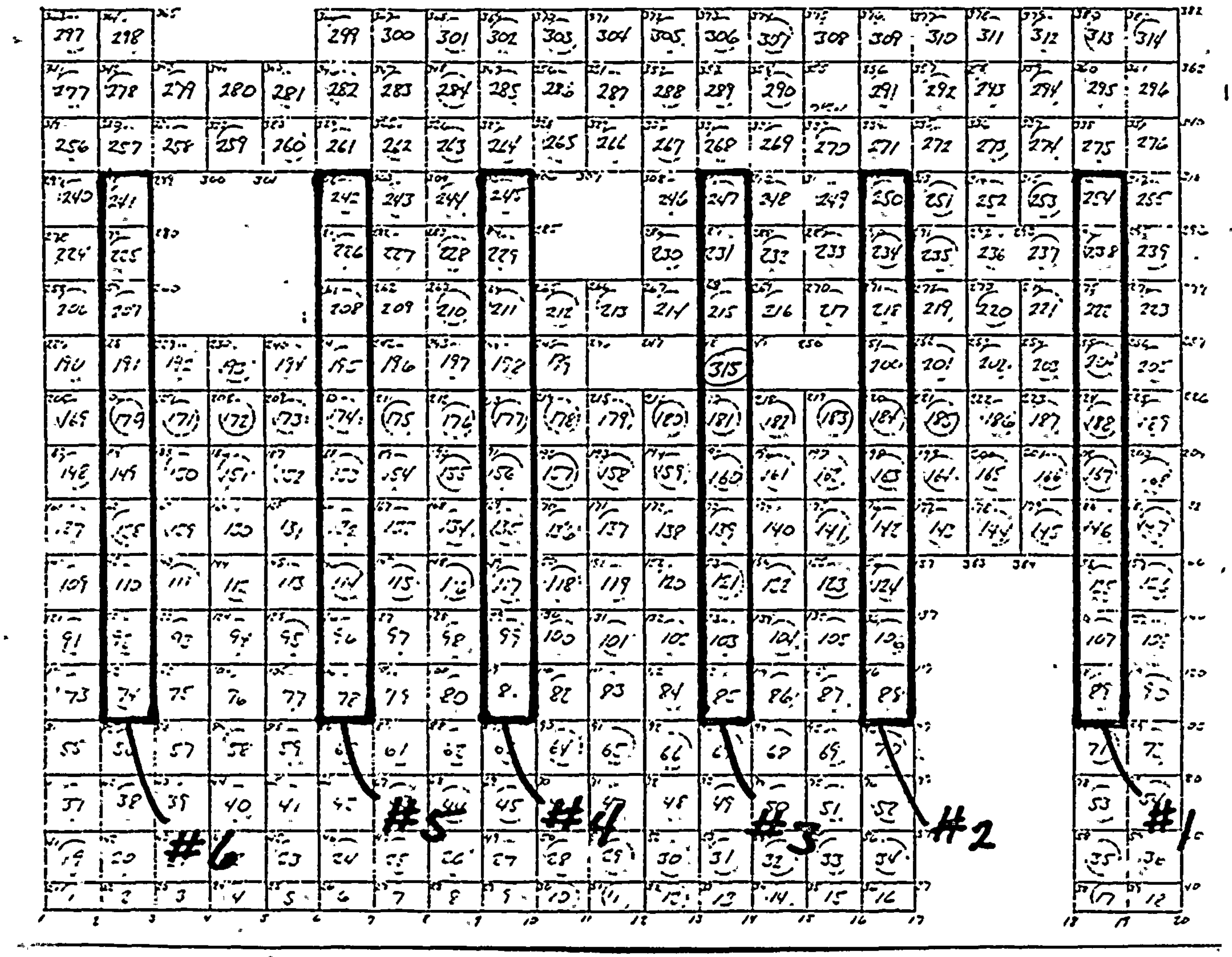

FIGURE 4: CENTER PANEL FINITE ELEMENT MODEL

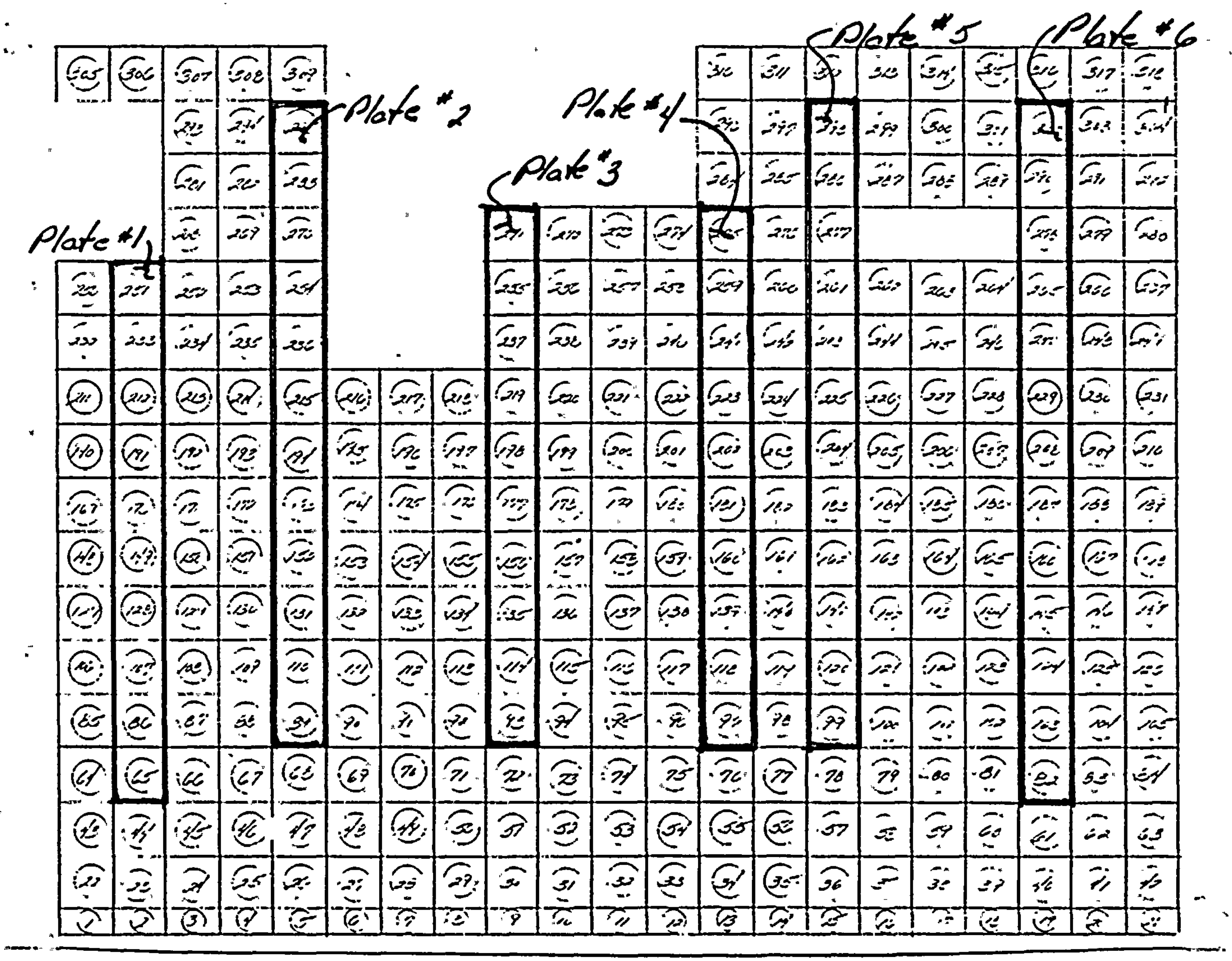

FIGURE 5: SOUTH PANEL FINITE ELEMENT MODEL

CONTROL BUILDING RESPONSE SPECTRUM
0.1G OBE HORIZONTAL
ENVELOPE OF EL. 74' & EL. 100'

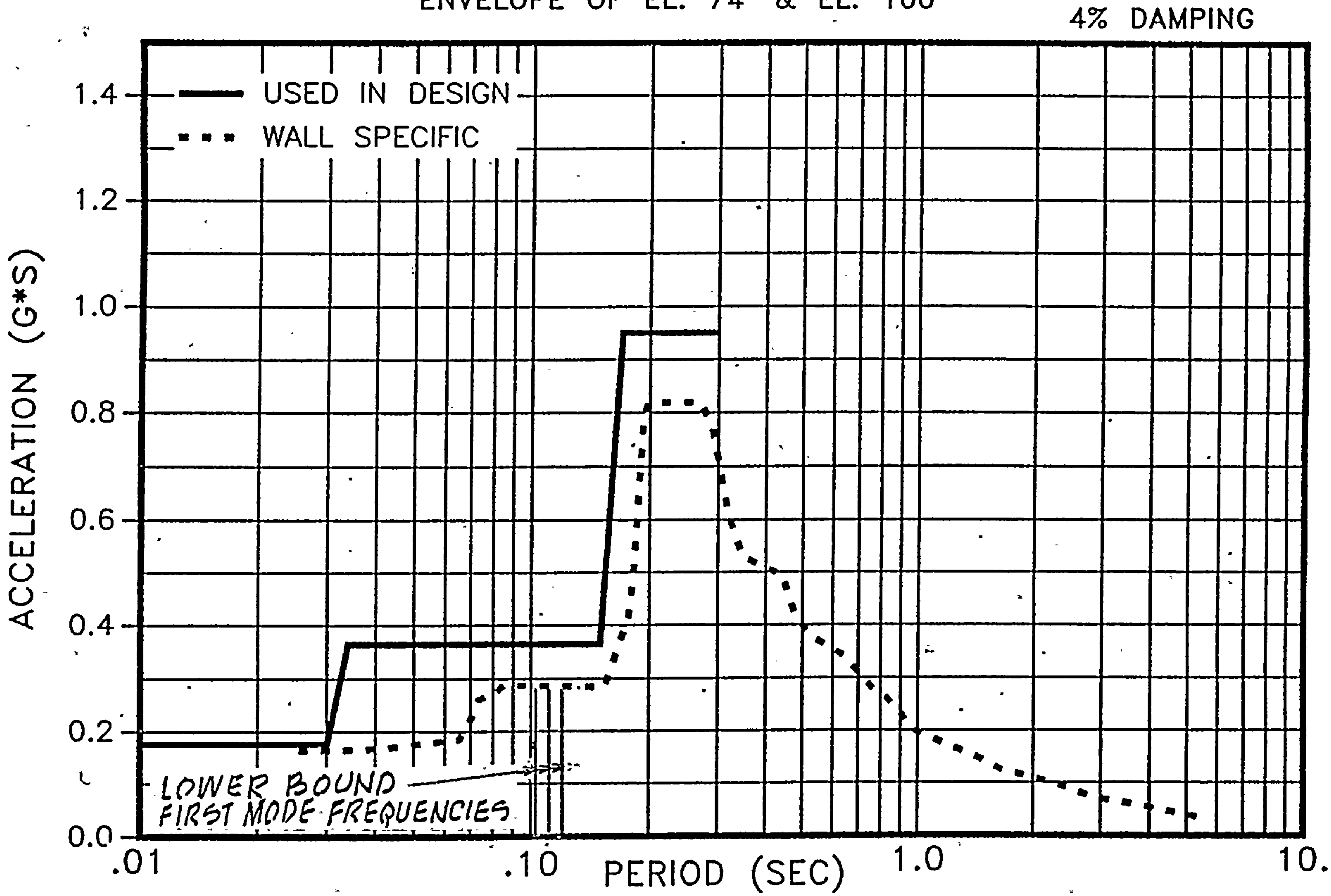

FIGURE 6

CONTROL BUILDING RESPONSE SPECTRUM
0.2G SSE HORIZONTAL
ENVELOPE OF EL. 74' & EL. 100'

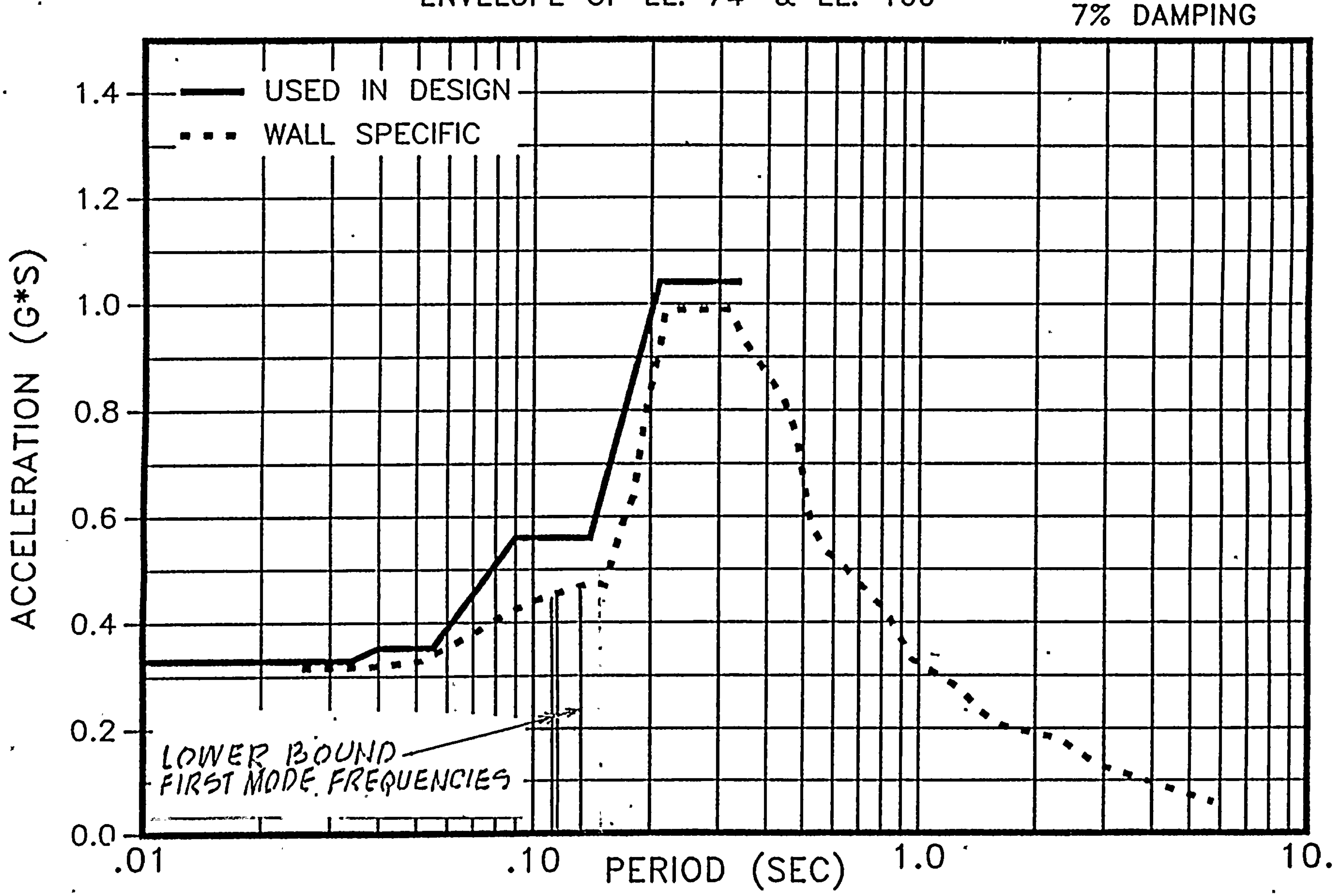

FIGURE 7

October 20, 1986

DISTRIBUTION
Docket File LPDR
PRC System PDR
PD7 Reading
JLee (2)

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 ____________.

☒ ~~Monthly~~ Bi-Weekly Notice; Applications and Amendments to Operating Licenses Involving no Significant Hazards Considerations, dated 9/24/86 (See page 33961).

☐ Application and Safety Analysis Report, Volume ____________.

☐ Amendment No. ______ to Application/SAR dated ____________.

☐ Construction Permit No. CPPR- ______, Amendment No. ______ dated ____________.

☐ Facility Operating License No. ______, Amendment No. ______, dated ____________.

☐ Order Extending Construction Completion Date, dated ____________.

☐ Other *(Specify)* ____________

Office of Nuclear Reactor Regulation

Enclosures:
As stated

cc: See next page

OFFICE	PD7						
SURNAME	JLee						
DATE	10/20/86						

NRC FORM 318 (1/84) NRCM 0240

[illegible]

Mr. E. E. Van Brunt, Jr.
Arizona Nuclear Power Project

Palo Verde

cc:
Arthur C. Gehr, Esq.
Snell & Wilmer
3100 Valley Center
Phoenix, Arizona 85073

Mr. James M. Flenner, Chief Counsel
Arizona Corporation Commission
1200 West Washington
Phoenix, Arizona 85007

Charles R. Kocher, Esq. Assistant
Council
James A. Boeletto, Esq.
Southern California Edison Company
P. O. Box 800
Rosemead, California 91770

Mr. Mark Ginsberg
Energy Director
Office of Economic Planning
and Development
1700 West Washington - 5th Floor
Phoenix, Arizona 85007

Mr. Wayne Shirley
Assistant Attorney General
Bataan Memorial Building
Santa Fe, New Mexico 87503

Mr. Roy Zimmerman
U.S. Nuclear Regulatory Commission
P. O. Box 239
Arlington, Arizona 85322

Ms. Patricia Lee Hourihan
6413 S. 26th Street
Phoenix, Arizona 85040

Regional Administrator, Region V
U. S. Nuclear Regulatory Commission
1450 Maria Lane
Suite 210
Walnut Creek, California 94596

Kenneth Berlin, Esq.
Winston & Strawn
Suite 500
2550 M Street, NW
Washington, DC 20037

Ms. Lynne Bernabei
Government Accountability Project
of the Institute for Policy Studies
1901 Que Street, NW
Washington, DC 20009

Ms. Jill Morrison
522 E. Colgate
Tempi, Arizona 85238

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

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

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