TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401

400 Chestnut Street Tower II

January 22, 1982

Director of Nuclear Reactor Regulation
Attention: Ms. E. Adensam, Chief
Licensing Branch No. 4
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Ms. Adensam:

In the Matter of the Application of	)	Docket Nos. 50-390
Tennessee Valley Authority	)	50-391

Enclosed for NRC review is TVA's final response to S. A. Varga's letter of April 21, 1980 concerning Category I Masonry Walls. This enclosure also provides responses to NRC questions provided by letter from E. G. Adensam to me dated December 7, 1981.

If you have any questions concerning this matter, please get in touch with D. P. Ormsby at FTS 858-2682.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

L. M. Mills

L. M. Mills, Manager
Nuclear Regulation and Safety

Sworn to and subscribed before me
this 22nd day of January 1982
Paulette H. White
Notary Public
My Commission Expires 9-5-84

Enclosure

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Enclosure

ENCLOSURE

WATTS BAR NUCLEAR PLANT UNITS 1 AND 2
MASONRY WALLS

References:
1. Letter from L. M. Mills to A. Schwencer dated February 12, 1981.
2. Letter from L. M. Mills to E. Adensam dated August 20, 1981.
3. Letter from L. M. Mills to E. Adensam dated September 14, 1981.
4. Letter from E. Adensam to H. G. Parris dated December 7, 1981.

TVA has responded to the NRC information request on Category I Masonry Walls employed by plants under construction twice previously. Our first response, reference 1, dealt with reinforced masonry walls while our second response, reference 2, dealt with unreinforced masonry walls. In reference 2 we stated that our final response would: (1) evaluate all our masonry walls for all loads including those addressed in the NRC Bulletin 80-11 and (2) demonstrate our method of analyzing unreinforced, unmortared walls for seismic loads. However, since that time TVA has received additional questions from the NRC (reference 4) concerning previous responses. Therefore, this final response to the subject information request will provide the material previously promised as well as a response to the NRC's specific questions in reference 4. It should be noted that TVA has addressed some of the NRC's questions (reference 4) in a letter sent to the NRC (reference 3) which defines and provides justification for the differences between TVA's unreinforced masonry wall design criteria and the NRC's interim and final SEB criterias. This matter was discussed in a telephone conference call with the NRC on December 7, 1981. It was agreed that TVA would answer questions 3a and 3b of reference 4 by referring the NRC to reference 3.

Question

1. In reference 1 the response to question 2 indicated that load combinations for dead plus live, dead plus live plus operating basis earthquake, and dead plus live plus safe shutdown earthquake were the only load combinations considered. Explain and justify the exclusion of load combinations for tornado wind and pressure loads associated with the postulated pipe break.

Response

1. Since the transmittal of TVA's first response, reference 1, to the information request, TVA has revised the WBN Design Criteria for Reinforced Concrete Block Walls (appendix A of reference 1) to address all the NRC Bulletin 80-11 loading concerns. The following design loading conditions have been added to the criteria.

$$S = D + L + W_t$$
$$S = D + L + 1.5\ P_a$$
$$S = D + L + 1.25\ P_a + 1.0\ (Y_j + Y_m + Y_r) + 1.25\ E$$
$$S = D + L + P_a + 1.0\ (Y_r + Y_j + Y_m) + E'$$
$$S = D + L + F$$

TVA is evaluating all the reinforced masonry walls for these additional loading conditions. If any of the reinforced masonry walls are found to be structurally inadequate to resist these additional loads, they will be restrained.

TVA has also completed an evaluation of all unreinforced masonry walls for all applicable loading conditions in accordance with the Design Criteria for Evaluation of Unreinforced Masonry Walls Constructed from Solid Concrete Blocks (appendix A of reference 2). We are in the process of externally restraining 17 walls as a result of this analysis.

Question

2. The following questions refer to appendix A to reference 1 describing the working stress design allowables permitted for reinforced masonry walls at Watts Bar.

   a. The values indicated are not related to the type of stresses that occur in masonry construction (i.e., axial or flexural compression, bearing, shear in masonry, shear in reinforcement, etc.). Please elaborate on the allowable stresses used and indicate whether or not the values used conform to the requirements of ACI 531-79, Building Code Requirements for Concrete Masonry Structures. If any of the allowables do not conform to ACI 531-79, indicate the difference and provide justification.

Response

2a. The following table compares ACI 531-79 code allowable stresses to those used in the WBN Design Criteria for Reinforced Concrete Masonry Walls. Calculations of ACI Code allowable stress values, Fa, Fm, and vm are based on formulas from chapter 10, "Allowable Stresses" of the ACI code. The value f'_m = 1350 pounds per square inch (lbs/in^2) used in these formulas is taken from table 4.3 of ACI 531-79 for type S mortar and 2000 lbs/in^2 compressive test strength on net area of type N hollow-core block.

Load Case I: D+L	ACI 531-79 Code (lbs/in^2)	WBN Criteria (lbs/in^2)
Fa*	296	810
Fm	445	810
vm	40	47
Fs	24000	24000
Load Case II: D+L+E		
Fa*	394	810
Fm	594	810
vm	54	47
Fs	32000	30000
Load Case III: D+L+E'		
Fa*	Not addressed in code	1350
Fm		1350
vm		78
Fs		54000

As can be seen, in most cases, the allowable stresses used in the WBN Design Criteria do not conform to ACI Code 531-79 values. Allowable stresses Fa, Fm, and vm in the WBN design criteria are all based on compressive strength of mortar (1800 pounds per square inch) since it is the weakest component of the reinforced block wall. Justification and deviation of the allowable stress values in the WBN criteria are discussed further in response to questions 2b and 2c.

*Fa based on 8-foot-high by 8-inch-thick wall.

TVA's design criteria also allows an increase in the allowable stresses for walls in which the cores of every block are "filled." This increase is to account for the additional strength given to the wall by filling the cores with concrete (compressive strength of 3000 pounds per square inch at 28 days).

Question

2b. Provide allowable stresses for mortar used in bed and collar joints.

Response

2b. Type S mortar was used on reinforced walls at WBN. This mortar has a compressive strength of 1800 lbs/in^2. The value 1800 lbs/in^2 was used in the calculation of allowable compressive and shear strengths for the load cases in appendix A of reference 1. Stresses referring to "block" in this criteria were actually referring to the weaker compressive strength of the hollow core block or the mortar. Since type S mortar has a lower compressive strength (1800 lbs/in^2) than that of a hollow core block (2000 lbs/in^2 on net area) mortar was the limiting value used in calculating the allowable stresses. For load cases I and II, the allowable compressive stress of 810 lbs/in^2 was calculated utilizing the ACI 318-63 code formula

$$fc = .45\ f'_c$$

where fc = 810 lbs/in^2 when f'_c = 1800 lbs/in^2 for mortar.

The allowable shear strength of 47 lbs/in^2 was calculated utilizing the ACI 318-63 code formula

$$vc = \sqrt{1.1\quad f'_c}$$

where vc = 47 lbs/in^2 when f'_c = 1800 lbs/in^2 for mortar.

Load case III allowable stress values (except for steel) are the load case I allowables increased by a factor of 1.67. The ACI 318-63 code did not cover the extreme environmental loads that are found on the reinforced masonry walls in category I structures at WBN. In addressing extreme environmental loads, an increase in the allowable stresses (or required strength) of the concrete to 0.75 f'_c was used in the criteria. This is an increase in allowable stress by a factor of 1.67, and thus the allowable compression and shear stresses are allowed the 1.67 increase.

The tensile strength of the mortar was not considered since reinforcing steel is assumed to take all tensile stress.

Allowable stresses for mortar in the collar joints were not considered because TVA has no multiwythe reinforced walls at WBN.

Question

2c. Justify the 25-percent allowable stress increase in reinforcing steel permitted for load case II (D + L + E). The "SEB Criteria for Safety-Related Masonry Wall Evaluation" does not permit increases in allowable stresses for operating base earthquake loads.

Response

2c. TVA's reinforced masonry wall design criteria (appendix A, reference 1) was issued in 1973. TVA used the ACI 318-63 code in the development of the reinforced masonry wall criteria. The 1963 ACI code allowed a 33-percent increase in allowable tensile steel stress for wind and seismic loadings. TVA increased the allowable tensile stress of steel by only 25 percent (from 24,000 lbs/in^2 to 30,000 lbs/in^2) from load case I (D + L) to load case II (D + L + E) in appendix A of reference 1. Therefore, TVA's allowable tensile steel stress in the original design of the block walls was more conservative for earthquake design than the allowable tensile steel stress required in the 1963 ACI code. TVA has since evaluated the reinforced walls for load case II (D + L + E) without increasing the allowable tensile steel stress by 25 percent (i.e., f_s = 24,000 lbs/in^2). The evaluation showed the as-built walls to be adequate to resist the design loading while maintaining a tensile stress in the steel less than 24,000 lbs/in^2.

Question

3. The following questions refer to appendix A of reference 2.

   a. Section 3.3.1, Service Load Combinations, excludes consideration of the operating base earthquake. Explain and justify this exclusion.

Response

3a. See previous TVA response as provided in reference 3.

Question

3b. Section 3.3.2, Extreme Environmental and Abnormal Loads, omits the load factor of 1.25 from load cases including pressure from pipe rupture and the safe shutdown earthquake. Also, the load case "D + 1.5Pa" is omitted from consideration. Explain and justify these omissions.

Response

3b. See previous TVA response as provided in reference 3.

Question

3c. In sections 3.4.6.1 and 3.4.6.2 explain why tensile strength and shear strength were not considered in the analysis of continuous vertical joints and bed joints.

Response

Section 3.4.6.1 of TVA's Design Criteria for Evaluation of Unreinforced Masonry Walls Constructed from Solid Concrete Blocks (appendix A of reference 2) deals with walls laid in stacked bond. This comes directly from ACI 531-79 code which states in section 10.3.2, "Where masonry is laid in stack bond, the tensile strength of masonry, grout, and mortar shall not be used in vertical continuous joints and the shear strength of masonry, grout, and mortar shall not be utilized to transfer concentrated loads across vertical continuous joints."

Section 3.4.6.2 of appendix A to reference 2 states, "The tensile strength of the mortar shall not be considered in the analysis of the bed joint on the top of the masonry wall." This section refers only to mortar at the top bed joint. Because of the difficulty of placing mortar in this top joint and the possibility for a crack to develop at the mortar-concrete interface, a condition existed that might not be conducive to transferring tensile stresses longitudinally through the wall to the superstructure, and thus tensile bond was not considered.

Question

4. In the dynamic analysis of both reinforced and unreinforced masonry walls indicate whether or not consideration of cracked walls was made. Discuss how this consideration was made and the effect on the period of the walls during a seismic event.

Response

4. The methods used in the analyses and design of reinforced concrete block walls are similar to those used to design other seismic category I reinforced structures, i.e., beams, walls, slabs. Natural frequencies and dynamic load calculations are determined from analyses of uncracked sections. The detail design is based on a cracked section using ACI 318-63, with f'_c based upon the mortar strength.

   For unreinforced mortared masonry walls the assumption of an initially elastic section is made and the dynamic forces acting on the walls are calculated using classical methods. If the detailed stress analysis indicates that the allowable tensile stress of the wall is exceeded, structural steel restraints are provided. The restraints are designed as an elastic system for the full force of the wall and the walls are not considered to have any inherent load resisting capability. Therefore, a frequency analysis and dynamic load calculation based on a cracked section is not required.

Question

5. Provide a description of the analysis and evaluation of unreinforced, unmortared masonry walls. Provide sample calculations.

Response

5. The method of analyzing unreinforced unmortared walls is included in attachment A, Design Criteria for Evaluation of Unreinforced Masonry Walls Constructed from Solid Concrete Blocks. Sample design calculations are also provided in attachment B.

ESS '810609 203

TENNESSEE VALLEY AUTHORITY

Division of Engineering Design

ATTACHMENT A

PLANT: WATTS BAR NUCLEAR PLANT

Detailed

Design Criteria

For

EVALUATION OF UNREINFORCED MASONRY WALLS CONSTRUCTED FROM SOLID CONCRETE BLOCKS

Design Criteria No: WB-DC-20-30

Issue Date: May 21, 1981

	Revision RO	R1	R2	R3	R4	R5
	Date 5-21-81	DEC 21 1981				
Prepared	R.E. Field Jr. / C.K. Buck	REF / CKB				
Supervised	M.G. Maxwell	MGM				
Reviewed	C. Glidewell	CG				
Submitted	Ruben O. Hendry	ROH				
Approved	S.C. Barnett	[illegible]				
Reviewed NSSS Vendor						

TVA 10566 (D-9-74)

Title: Evaluation of Unreinforced Masonry Walls Constructed from Solid Concrete Block	REVISION LOG WBN-DC-20-30

Revision No.	DESCRIPTION OF REVISION	Date Approved
1	1. Increased the spectral response curve to be used from 2 percent to 7 percent damping for seismic analysis of walls in section 3.2.3. 2. Added the definitions of Q and M to section 4.2 for "Loadings in one direction." Also deleted the definitions of P_t and W_t from this section. 3. Revised figure 4.0-2.	

TVA 10567 (EN DES-6-80)

COORDINATION LOG

Plant: Watts Bar Nuclear Plant

Design Criteria No: WB-DC-20-30

Design Criteria For: EVALUATION OF UNREINFORCED MASONRY WALLS CONSTRUCTED FROM SOLID CONCRETE BLOCKS

Revision: 1

R – Denotes review

A – Denotes approval

Thermal Power Engineering Branches

MEB R	MEB A	EEB R	EEB A	CEB R	CEB A
				NES	NES
				DC	[illegible]
				JAE	JAE
				RDG E	RDG E
				GBN	[illegible]
					DED
				RJH	RJH

NEB R	NEB A	R	A	R	A
[illegible]	[illegible]				
CWP	CWP				
I	[illegible]				
	[illegible]				
	JAR/EHT				

Thermal Power Design Project

Project Manager R	Project Manager A
[illegible]	[illegible]

MD R	MD A	ED R	ED A	CD R	CD A
				PKL	PKL
				HDH	HEH
				IHK	HK
				PCG	PCG
				WDL	WDL
				JPL	JPL
				AJ	AJ

Architectural, Hydro, & Special Projects Engineering & Design Branches

AD R	AD A	ME&D R	ME&D A	CE&D R	CE&D A	EE&D R	EE&D A

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1.0 SCOPE

This evaluation criteria shall apply to the evaluation of unreinforced masonry walls constructed from solid concrete masonry units. The criteria addresses both mortared and unmortared joint conditions and gives specific details where differences occur.

2.0 PURPOSE

The purpose of this criteria is to establish a guide to gather and evaluate information in regard to the Nuclear Regulatory Commission's (NRC) "Information Request on Category I Masonry Walls" (NEB 800514 255) and NRC IE Bulletin 80-11. By using this criteria and the data gathered by a field survey, each unreinforced masonry wall shall be evaluated for its effect upon safety-related equipment should that wall fail. If the field survey indicated that a wall would not damage any safety-related equipment by its failure, no further action will be necessary for that wall. However, if the field survey indicated that a wall could damage safety-related equipment by its failure, the wall shall be evaluated for its structural ability to withstand combinations of the following: dead loads, impact or compartmental pressurization loads such as missile, pipe whip, pipe break, jet impingement, or tornado depressurization, flooding, and seismic loads described herein. However, for pipe break, unless the safety-related equipment is required following that specific break, no protection is necessary. If the evaluation determines that a wall can withstand the design loads, no further action will be required for that wall. However, if the evaluation indicates that a wall could not withstand any of the design loads, corrective action shall be taken to prevent damage to any safety-related equipment. This may be accomplished by designing and installing a restraint mechanism which will prevent the wall from failing or by designing and installing a barrier to protect the safety-related equipment from failure of the wall. If a restraint system is required, its design including anchorage shall conform to reference 5.5.

3.0 EVALUATION BASIS

3.1 Materials

3.1.1 Concrete Blocks

The solid concrete masonry units shall be conservatively assumed to conform to the requirements of American Society for Testing and Materials (ASTM) "Solid Load-Bearing Masonry Units," Designation C145-71, Grade S-II, unless records are available to substantiate that the masonry units conform to the requirements of the higher grades.

3.1.2 Mortar

The mortar shall be conservatively assumed to conform to the requirements of ASTM "Mortar for Unit Masonry," Designation

C270, type N, unless records are available to substantiate that the mortar used conformed to the requirements for types S or M.

3.2 Loads

3.2.1 Dead Loads (D)

D - Dead loads or their related internal moments and forces. The dead load shall be based on the density of the solid masonry units being 135 pounds per cubic foot (lb/ft^3).

3.2.2 Live Loads (L)

L - Attachments to the unreinforced masonry walls shall not be allowed. In the event that attachments are presently being utilized on the walls, corrective action must be taken to ensure their removal and relocation.

3.2.3 Seismic Load (E')

E' - Loads generated by the safe shutdown earthquake (SSE)

The seismic analysis will consider two types of block walls: unreinforced, mortared walls as discussed in section 3.2.3.1, and unreinforced, unmortared block walls as discussed in section 3.2.3.2.

3.2.3.1 Mortared Block Walls

Unreinforced, mortared block walls shall be dynamically analyzed on a case-by-case basis as necessary. Parametric studies or "worst case" walls may be utilized for analysis purposes as desired. Unless it can be verified that the top block is structurally restrained or adequately mortared, the wall shall be analyzed as a simple cantilever. Otherwise, the wall shall be analyzed as a propped cantilever.

3.2.3.1.1 Walls Analyzed As a Simple Cantilever

In a typical analysis, a unit width of wall shall be assumed to act as a cantilever. The following steps shall be followed to determine dynamic loads:

Step 1. The natural frequency of the wall shall be calculated as follows:

$$f_1 = \frac{(0.597)^2\pi}{2L^2}\sqrt{\frac{EI}{m}} \quad ,n=1$$

$$f_n = \frac{(n-\frac{1}{2})^2\pi}{2L^2}\sqrt{\frac{EI}{m}} \quad ,n>1$$

where,

n = Mode number
f = Frequency, Hertz (Hz)
m = Mass per unit length of wall for unit width ($lb\text{-}sec^2/in^2$)
E = 1,000,000 lb/in^2
I = Moment of inertia of unit width of wall (in^4)
L = Height of wall (in)

All frequencies ≦ 33 Hz shall be calculated and retained. In the vertical direction, the wall will exhibit rigid body behavior and a frequency ≧ 33 Hz is assumed.

Step 2. Each frequency calculated in Step 1 shall be broadened by ±10 percent. Using the 7 percent damping floor response spectrum curve from the appropriate published Civil Engineering Branch (CEB) Report, a horizontal acceleration value corresponding to 0.9 f_n, 1.0 f_n, and 1.1 f_n for each calculated mode (n = 1, 2, 3, ...) ≦ 33 Hz, shall be determined. The largest of the three accelerations determined for each mode shall be retained for use in Step 3. The vertical acceleration shall be determined from the structural response acceleration (ZPA) curve contained in the appropriate report. |R1

Step 3. The retained horizontal acceleration for each mode from Step 2 shall be combined using the square-root-of-the-sum-of-the-squares-method (SRSS) as follows:

$$a_r = \sqrt{a_1^2 + a_2^2 + \ldots a_n^2}$$

where,

a_n = maximum modal horizontal acceleration for the n^{th} mode.

a_r = SRSS acceleration

Step 4. The calculated acceleration a_r, shall be multiplied by the deadweight of the wall and applied as a uniform static load in the direction normal to the wall. A vertical load shall be determined by multiplying the vertical acceleration determined in Step 2 by the weight of the unit width of the wall. The seismic stress (σ_E) in the wall is given by:

$$\sigma_E = \pm\left(\left|\frac{P}{A}\right| + \left|\frac{Mc}{I}\right|\right)$$

where Mc/I is the bending stress due to the horizontal acceleration and P/A is the axial stress due to the vertical acceleration. Since the earthquake is cyclic in nature, the calculated forces are assumed to act in either direction. Section 3.4.7 of this criteria must be met in the combined stress state.

Step 5. If, from Step 4, it is determined that restraints are required to prevent failure of a wall, the wall restraints shall be designed for the loads produced by the accelerations calculated in Steps 2 and 3. First, select the structural shape and size restraint to be used and assume a 4-foot initial spacing. Then multiply the weight of the restraint plus the weight of the block wall (tributary width) by the acceleration given in Step 3 and apply these forces as a uniform load to the restraint. Designs that result in an unrealistically large restraint or closely spaced restraints may be coordinated with CEB personnel for further analysis on a case-by-case basis. Unless otherwise justified, restraints shall be placed on both sides of a wall.

Step 6. In lieu of performing a detailed dynamic evaluation, a factor of 1.5 times the peak horizontal acceleration value of the appropriate 7 percent damping floor response spectrum curve may be used for the horizontal accelerations of the wall. The vertical acceleration will be as |R1 defined in Step 2. Steps 4 and 5 shall then be performed to design the restraint. If Step 6 results in an unrealistically large restraint or closely spaced restraints, the detailed dynamic evaluation shall be performed.

3.2.3.1.2 Walls Modeled As a Propped Cantilever Beam.

In a typical analysis, a unit width of the wall shall be assumed to act as a propped cantilever. The following steps shall be followed to determine dynamic loads.

Step 1. The frequency of the wall shall be calculated as follows:

$$f_n = \frac{(n + \frac{1}{4})^2 \pi}{2L^2} \sqrt{\frac{EI}{m}}$$

where the parameters are defined in Step 1 of section 3.2.3.1.1. All frequencies $\leqq$ 33 Hertz (Hz) shall be calculated and retained. In the vertical direction, the wall will exhibit rigid body behavior and a frequency > 33 Hz is assumed.

Step 2 through Step 6 will be the same as those given in section 3.2.3.1.1.

3.2.3.2 Unmortared Block Walls

Unmortared, unreinforced walls will normally require restraints. To determine if restraints are required, an acceleration value of 1.5 times the peak of the appropriate 7 percent damping floor response spectrum curve can be used to evaluate the stability of the blocks within the wall. If restraints are shown to be required, a more detailed evaluation should be performed to design the restraint system. R

Seismic loads for unmortared, unreinforced walls shall be determined as follows:

Step 1. First select the structural shape and size restraint to be used and assume an initial spacing of one restraint every three feet.

Step 2. Using the structural properties of a restraint and considering the tributary width of the wall to be restrained as an added mass, the frequency is determined using the following formula for a simply supported beam (reference 5.4).

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{EI}{m}}$$

where f_1 = the first mode of the restraint and supported wall mass
E = 29,000,000 lb/in^2 (steel)
m = Mass per unit length of restraint and tributary wall mass (lb-sec^2/in^2)
I = Moment of inertia of restraint (in^4)
L = Length of restraint (in)

Because of the difference in the relative mass of the restraint and the block wall, and the probable nonlinear movement of the block wall, the higher modes of the restraint and added mass will not exist and therefore can be neglected.

Step 3. The calculated frequency shall be broadened by ±10 percent, and the acceleration corresponding to the three frequencies ($0.9f_1$, f_1, $1.1\ f_1$) shall be determined from the appropriate 7 percent damping floor response spectrum curve. The largest of these accelerations shall be retained. | R

Step 4. The acceleration (from Step 3) shall be multiplied by the weight of the restraint and tributary wall weight supported by the restraint, and the resulting force applied as a uniform load to the restraint. The induced stress in the restraint shall be compared to the allowable stresses as given in reference 5.5. The stability of the blocks between restraints shall be checked using the calculated acceleration to ensure that they remain in place. Unless justified, restraints shall be provided on both faces of a wall.

Step 5. Any change in the restraint structural configuration or spacing due to the stress or stability evaluation shall require a redetermination of the dynamic characteristics of the revised configuration. Steps 2, 3, and 4 shall be repeated.

Step 6. In lieu of performing a detailed dynamic analysis, an acceleration value equal to 1.0 times the peak of the appropriate 7 percent damping floor response spectrum curve may be used in the design of the restraining system. Step 4 shall then be performed to design the restraint. | R determined using the following formula for a simply supported beam (reference 5.4).

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{EI}{m}}$$

where f_1 = the first mode of the restraint and supported wall mass

E = 29,000,000 lb/in^2 (steel)

m = Mass per unit length of restraint and tributary wall mass ($lb\text{-}sec^2/in^2$)

I = Moment of inertia of restraint (in^4)

L = Length of restraint (in)

Because of the difference in the relative mass of the restraint and the block wall, and the probable nonlinear movement of the block wall, the higher modes of the restraint and added mass will not exist and therefore can be neglected.

Step 3. The calculated frequency shall be broadened by ±10 percent, and the acceleration corresponding to the three frequencies ($0.9f_1$, f_1, $1.1\ f_1$) shall be determined from the appropriate 7 percent damping floor response spectrum curve. The largest of these accelerations shall be retained. | R1

Step 4. The acceleration (from Step 3) shall be multiplied by the weight of the restraint and tributary wall weight supported by the restraint, and the resulting force applied as a uniform load to the restraint. The induced stress in the restraint shall be compared to the allowable stresses as given in reference 5.5. The stability of the blocks between restraints shall be checked using the calculated acceleration to ensure that they remain in place. Unless justified, restraints shall be provided on both faces of a wall.

Step 5. Any change in the restraint structural configuration or spacing due to the stress or stability evaluation shall require a redetermination of the dynamic characteristics of the revised configuration. Steps 2, 3, and 4 shall be repeated.

Step 6. In lieu of performing a detailed dynamic analysis, an acceleration value equal to 1.0 times the peak of the appropriate 7 percent damping floor response spectrum curve may be used in the design of the restraining system. Step 4 shall then be performed to design the restraint. | R1

Step 7. Any proposed restraint that cannot be modeled as a simple beam shall be discussed with appropriate CEB personnel.

3.2.4 Pipe Break Loads (P_a, Y_j, Y_m)

P_a - Pressure equivalent static load within or across a compartment generated by the postulated break, and including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_j - Jet impingement equivalent static load on a structure generated by the postulated break, and including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_m - Missile impact load on a structure generated on or during the postulated break, as from pipe whipping and including an appropriate dynamic load factor to account for the dynamic nature of the load.

3.2.5 Tornado Loads (W_t)

W_t - Loads generated by the design tornado specified for the plant. Tornado loads on the masonry walls are due to tornado-created differential pressure.

3.2.6 Flood Loads (F)

F - Flooding equivalent static load on a structure generated by compartment flooding.

3.3 Load Combinations

Unreinforced concrete block walls shall be evaluated as defined in section 3.2. The horizontal and vertical loads used in the design of the wall shall be applied in combinations as prescribed in the following sections.

3.3.1 Service Loads

For loads encountered during normal plant startup, operation, and shutdown, the following load combination shall be considered:

(1) $S = D$

3.3.2 Extreme Environmental and Abnormal Loads

For extreme environmental and abnormal loads due to the safe shutdown earthquake, flood, tornado, or high energy pipe break accident, the following load combinations shall be considered:

(2) $S = D + E'$
(3) $S = D + P_a + Y_j + Y_m$
(4) $S = D + W^t$
(5) $S = D + F^t$

In load combination (3) the maximum values of P_a, Y_j, and Y_m should be used unless a time history analysis is performed to justify otherwise.

In the above load combinations, S is the required section strength based on the working stress design method and the allowable stresses defined in section 3.4.

3.4 Allowable Stresses

Allowable stresses shall be as given below for load combination (1) of section 3.3.1. These values may be increased 33 percent for load cases (2) through (5) provided the increased values do not exceed the stated maximums.

3.4.1 Compressive Strength

3.4.1.1 For walls with mortared joints, the compressive strength of the masonry wall, f_m', shall be taken as 700 pounds per square inch (lb/in^2) for an assumed compressive strength of the masonry units of 1000 lb/in^2 and Type N mortar. If records are available, as stated in section 3.1, a higher value of f_m' may be used as determined from table 4.3 of American Concrete Institute (ACI) "Building Code Requirements for Concrete Masonry Structures" ACI 531-79.

3.4.1.2 For walls with unmortared joints, the compressive flexural strength of the masonry wall, f_m', shall be taken as the compressive strength of the masonry units (3000 lb/in^2) unless records are available to substantiate a higher value. For unmortared block walls, f_m' shall be substituted for f_c' of the alternate design method.

3.4.2 Axial Stress

The allowable compressive stress due to axial loading on the wall shall not exceed

$$F_a = 0.225f_m' \; [\, 1 - (h/40t)^3] \text{ but } \leq 1000 \text{ lb/in}^2,$$

where h (effective height) and t (nominal thickness) are as defined in section 9.4.7 and 9.4.8 of ACI 531-79.

3.4.3 Flexure

The allowable flexural compressive stress shall be

3.4.3.1 $F_m = 0.33 f_m'$ but $\leqq 1200$ lb/in^2 for mortared walls.

3.4.3.2 $F_m = 0.45 f_m'$ but $\leqq 1350$ lb/in^2 for unmortared walls.

3.4.4 Shear

The allowable shear stress for solid concrete blocks with mortared joints shall be

$$v_m = 1.1 \sqrt{f_m'} \quad \text{but} \leqq 50 \text{ lb/in}^2.$$

The allowable shear stress for solid blocks with unmortared joints shall be

$$v_m \leqq f_s$$

where f_s is the static friction. The coefficient of friction shall be taken as 0.7. f_s = 0.7 x normal force.

3.4.5 Tensile Stress

The allowable tensile stress in mortared joints due to bending shall be

$F_t = 1.0 \sqrt{m_o}$ $\leqq$ 40 lb/in^2 normal to the bed joints,

$F_t = 1.5 \sqrt{m_o}$ $\leqq$ 80 lb/in^2 parallel to the bed joints in running bond.

where m_o is the compressive strength of the mortar.

3.4.6 Limitations of Stresses

3.4.6.1 Neither the tensile strength nor the shear strength of the mortar shall be considered in the analysis of vertical continuous joints.

3.4.6.2 The tensile strength of the mortar shall not be considered in the analysis of the bed joint on the top of the masonry wall.

3.4.6.3 If construction inspection records conforming in general to the requirements outlined in section 4.5.2 of ACI 531-79 are not available, the allowable stresses in compression shall be reduced by one-third and the allowable stresses in tension and shear reduced by one-half.

3.4.7 Combined Stress

For combined stresses due to bending and axial loads, the following shall be met:

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} \leq 1.0$$

where f_a is the calculated axial compressive stress in masonry and f_m is the calculated flexural compressive stress in masonry.

4.0 ANALYSIS

4.1 Mortared Block Walls

Masonry walls with mortared joints may be analyzed as a propped cantilever if adequate bond exists between the top block and the supporting structure, or if restraints are added at the top of the wall. Otherwise, the walls shall be analyzed as a cantilever beam. If the calculated stresses exceed the allowable stresses using the Working Stress Design Method of ACI 531-79, the walls shall be restrained. Restraints shall be analyzed as either a simple beam or a plate hinged on four sides if all four sides are restrained.

For multi-wythe walls which are subjected to seismic loads, the wythes shall be assumed to act independently of each other unless they are connected by ties or other mechanical means. Composite action of two or more wythes should not be assumed unless an analysis of the mechanical ties connecting the wythes is performed and the ties are deemed sufficient to assure the wythes act together. For pipe break, tornado, missile, or flood loads (loading in one direction), the loads shall be assumed to act on the external wythe and the load distributed through each successive wythe if there is no air space between the wythes. If an air space exists between the wythes, the external wythe shall be assumed to carry the total load.

4.2 Unmortared Walls

Masonry walls with unmortared joints shall be analyzed as a cantilever beam using stability analysis. Where restraints are required, the portion of the wall between the restraints shall be analyzed as a simple beam.

For multi-wythe walls subjected to seismic loads, the wythes shall be assumed to act independently. Externally applied loads in one direction such as pressure loads may be distributed equally to the wythes and each wythe analyzed individually for multi-wythe walls without an air space between the wythes.

The evaluation of unmortared walls shall be as follows:

Seismic Evaluation (Reversible Loading)

The walls shall be evaluated for all forces as shown in figure 4.0-1(b), where

P = Axial force applied to top face of block (including vertical seismic effects)
W = Weight of individual block (including vertical seismic effects)
V_1 = Shear force on top face of block
V_2 = Shear force on bottom face of block
f_{s1} = Static frictional force at top face of block ($\mu = 0.7$)
f_{s2} = Static frictional force at bottom face of block ($\mu = 0.7$)
N = Normal vertical force at bottom face of block
d = Distance of normal force (N) from front face of block
M = Applied moment due to external loads
L = Span between lateral supports

such that the moment formulated by the normal force N, and its moment arm d, will resist the moments which result from P, W, V_1, V_2, f_{s1}, f_{s2}, and M when moments are summed about point A (see figure 4.0-1(b)) while the normal force N, remains within the plane of the wythe (d < width of a single block).

The compressive stress on the bottom of the block caused by the normal force N, shall be evaluated as shown in figure 4.0-1(c) to ensure the stress involved does not exceed the allowable stress given in Section 3.4.3, that is

$$f_c < F_m$$

Pipe Break, Missile, Tornado, and Flooding Evaluation (Loading in One Direction)

The walls shall be evaluated for all forces as shown on figure 4.0-2(b) where

P = Axial force applied to top face of block
W = Weight of the individual block
f_{s1} = Static frictional force at top face of block ($\mu = 0.7$)
f_{s2} = Static frictional force at bottom face of ($\mu = 0.7$)
N = Normal vertical force at bottom face of block
L = Span between lateral supports
w = Equivalent uniform static load or concentrated load with an appropriate dynamic load factor
Z = Equivalent point load
Q = Z/number of wythes
M = Applied moment due to external loads

such that the static frictional forces (f_{s1} and f_{s2}) shall not be exceeded (see figure 4.0-2(b)) and the overturning moment does not exceed its internal resisting moment (see figure 4.0-2(c)).

5.0 REFERENCES

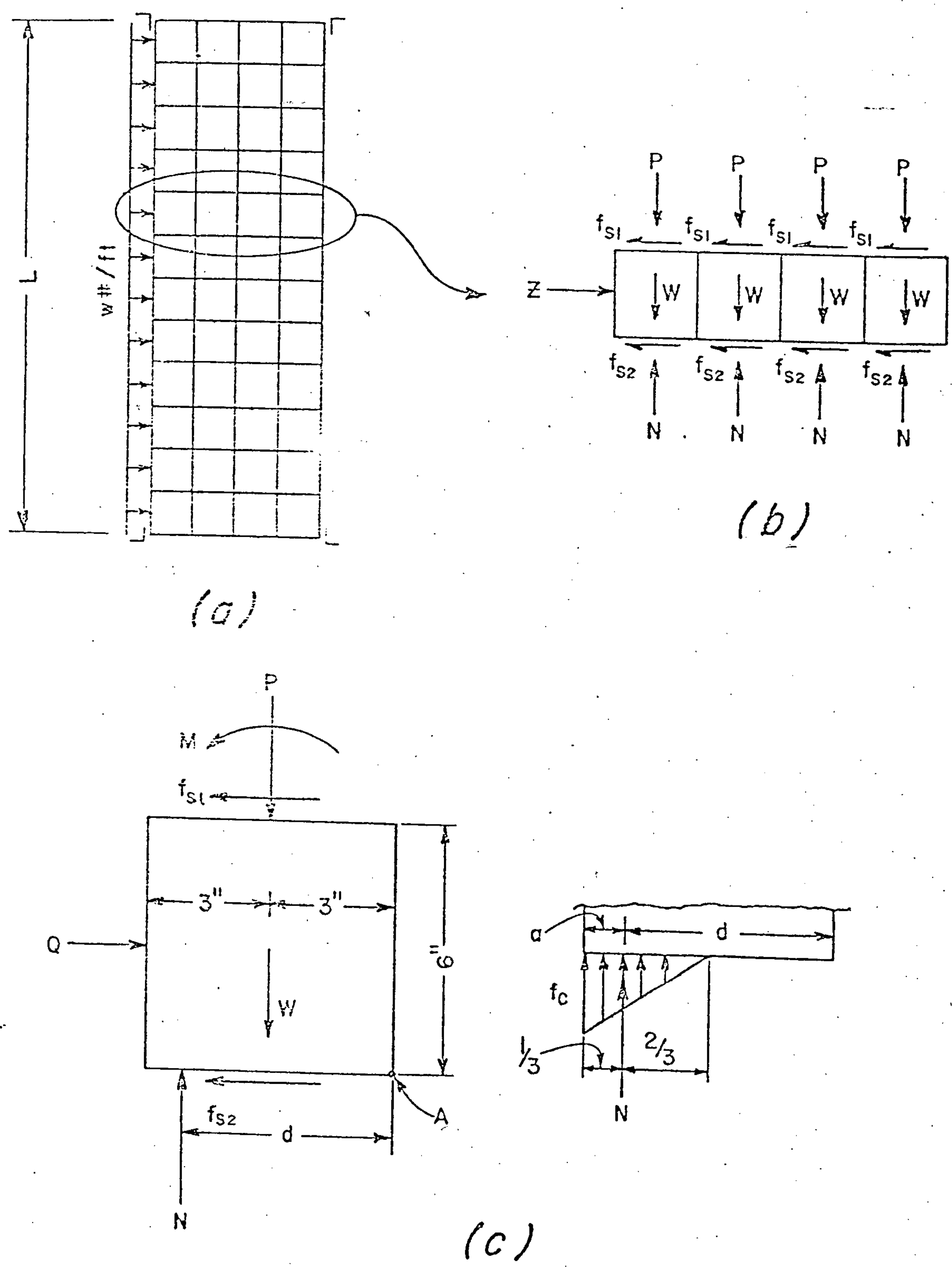
5.1 American Society for Testing and Materials (ASTM) "Solid Load-Bearing Concrete Masonry Units," Designation C145-71.

5.2 ASTM "Mortar for Unit Masonry," Designation C270.

5.3 American Concrete Institute (ACI) "Building Code Requirements for Concrete Masonry Structures," ACI 531-79.

5.4 Introduction to Structural Dynamics, John M. Biggs, 1964, Chapter 4.

5.5 Design Criteria for Miscellaneous Steel Components for Seismic Class I Structures, WB-DC-20-21.



ANALYSIS OF SOLID SHIELD BLOCK WALLS

FIGURE 4.0-2 (R1)

WCG-2-45

CONCRETE BLOCK SHEILDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

COMPUTED PCG DATE 12-4-81
CHECKED P.M. DATE 12-16-81

ATTACHMENT B

PURPOSE: TO INVESTIGATE THE STABILITY OF UNMORTARED, UNREINFORCED MASONARY WALLS IN THE WBNP REACTOR BUILDING IN THE EVENT OF A SAFE SHUTDOWN EARTHQUAKE.

REFERENCES:

1) DESIGN CRITERIA FOR "EVALUATION OF UNREINFORCED MASONARY WALLS CONSTRUCTED FROM SOLID CONCRETE BLOCKS" WB-DC-20-30 R1

2) TVA DWG 48N943

3) TVA DWG'S 41W732-2, -4

4) AISC STEEL CONSTRUCTION MANUAL (1978 Ed.)

5) DYNAMIC EARTHQUAKE ANALYSIS OF THE INTERIOR CONCRETE STRUCTURE FOR WBNP REACTOR BUILDING

6) ROGER FIELD, CIVIL ENGINEERING BRANCH, x2762

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

COMPUTED PCG DATE 12-10-81
CHECKED P.M. DATE 12-16-81

PROBLEM DISCUSSION:

TWO UNMORTARED, CONCRETE BLOCK WALLS IN THE WBNP REACTOR BUILDING MUST BE INVESTIGATED TO DETERMINE IF THEY WILL FAIL IN THE EVENT OF A SAFE SHUTDOWN EARTHQUAKE, POSSIBLY DAMAGING SAFETY RELATED EQUIPMENT. THE CALCULATIONS WERE CONDUCTED ACCORDING TO DESIGN CRITERIA WB-DC-20-30 R1. ALTHOUGH THIS GUIDE CALLS FOR 7% DAMPING, A 5% DAMPING CURVE WAS USED FOR THE VERTICAL ACCELERATION. THIS CONSERVATIVE APPROACH WAS USED DUE TO THE LACK OF A 7% DAMPING RESPONSE SPECTRUM.

CONCLUSION:

THE TWO UNMORTARED CONCRETE BLOCK SHIELDING WALLS IN THE WBNP REACTOR BUILDING WILL NOT FALL IN THE EVENT OF A SAFE SHUTDOWN EARTHQUAKE.

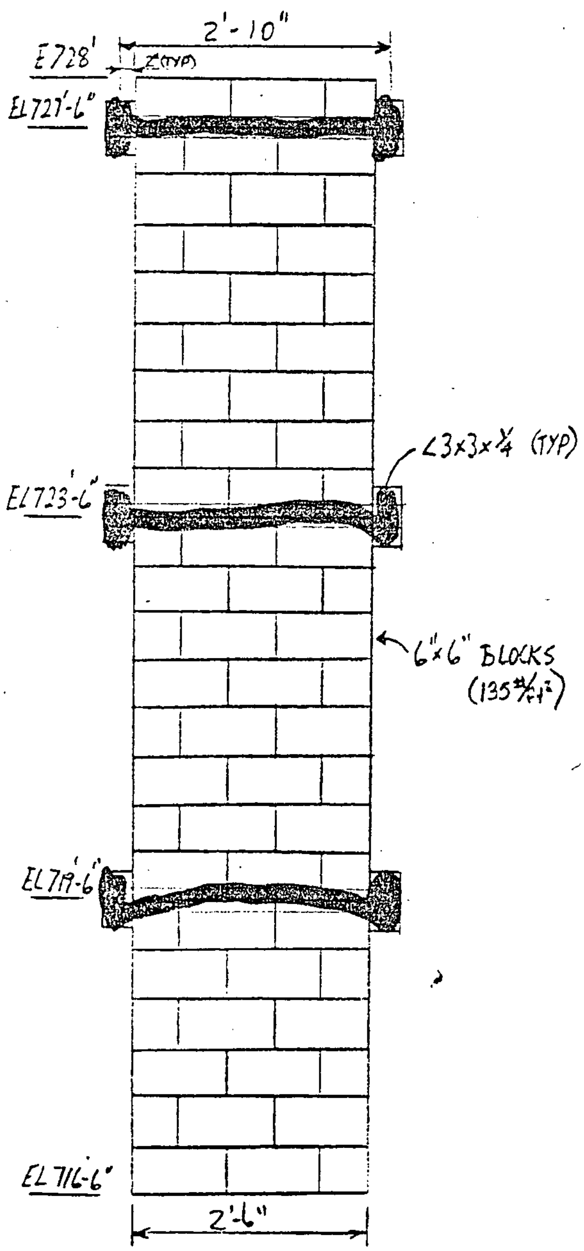
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

COMPUTED PCG DATE 12-11-81
CHECKED P.M. DATE 12-16-81

CHECK THE ADEQUACY OF THE RESTRAINTS FOR THE WALL AT AZ 106°, EL 716'-6". SEE DWGS 48N943 & 41W732-4. THE WALL IS 2½ FEET THICK & 12 FEET TALL.

3.2.3.2 UNMORTARED BLOCK WALLS ~ CONSIDER FULL THICKNESS OF WALL BEING ACCELERATED IN ALL RESTRAINT CALCULATIONS (conservative)

SECTION A-A



STEP 1

RESTRAINT ~ ∠3x3x¼ w/ 4' TRIBUTARY WALL HEIGHT

STEP 2

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{m}}$$

$L = (30" + 2\times2") = 34"$ (RESTRAINT LENGTH)

$E = 29{,}000{,}000\ \#/in^2$ (MOD. OF ELASTICITY)

$I = 1.24\ in^4$ (MOM. OF INERTIA)

$m = \frac{5\#/' + 4'\times2.5'\times135\#/ft^3}{32.2\ ft/sec^2}\times\frac{1 ft^2}{144 in^2} = .2922\ \frac{\#\cdot sec^2}{in^2}$ (RESTRAINT & TRIBUTARY WALL MASS)

$$f_1 = \frac{\pi}{2(34)^2}\sqrt{\frac{29{,}000{,}000\times1.24}{.2922}} = 15.073\ \frac{cyc}{sec}$$

STEP 3

$1.1f_1 = 16.58\ \frac{cyc}{sec}$ $T = .06\ sec$

$.9f_1 = 13.57\ \frac{cyc}{sec}$ $T = .07\ sec$ ←

$f_1 = 15.073\ \frac{cyc}{sec}$ $T = .07\ sec$

HORIZONTAL ACCELERATION IS .26 FROM ROGER FIELD, CEB, x2762

STEP 4

$w = E = .26 \times [5\#/' + 4' \times 2.5' \times 135\ \#/ft^3] = 352\#/ft$

$M = \frac{w\ell^2}{8} = \frac{352 \times 2.83^2}{8} = 352^{\#\text{-}ft} = .352^{k\text{-}1}$

$f_b = \frac{.352 \times 12}{.577} = 7.32^{ksi}$

$F_b = .9 \times 36 = 32.4^{ksi}$ (SSE ALLOWABLE)

Since $7.32^{ksi} < 32.4^{ksi}$ RESTRAINT IS OK IN BENDING

CHECK TENSION IN ½"Φ BOLTS

$\text{LOAD/BOLT} = \frac{(.352^{k}/')(2'\text{-}6'')}{2} = .44^{k}/\text{BOLT}$

$\text{STRESS} = \frac{.44^k}{\pi(\frac{.5}{2})^2} = 2.04^{ksi} < 20^{ksi}$ ALLOWABLE ∴ BOLTS OK

CONCRETE BLOCK [illegible]IELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

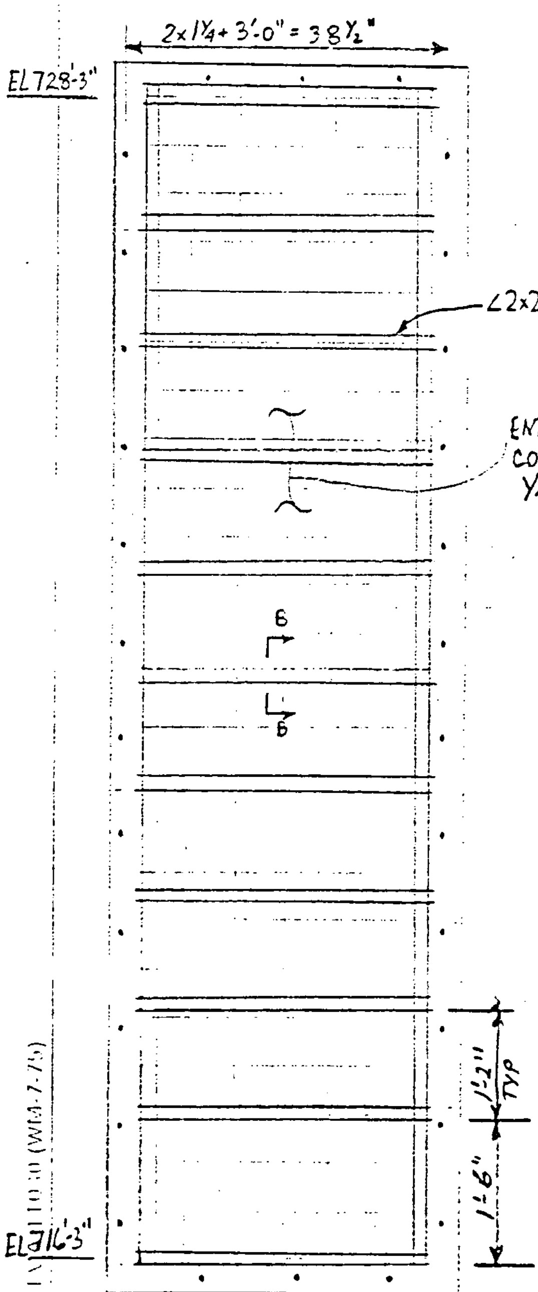
COMPUTED PCG DATE 12-11-81
CHECKED P.M. DATE 12-16-81

MK'S 1 & 2

CHECK TENSION IN ½"ϕ BOLTS:

OK BY INSPECTION (SEE CALCS. ON A-A)

3.2.3.2 UNMORTARED BLOCK WALLS

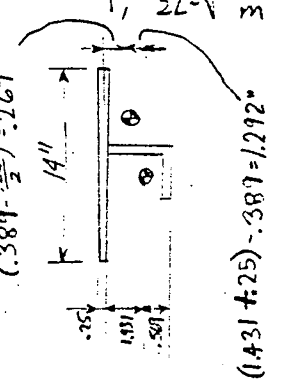


STEP 1

RESTRAINT ~ ¼" PL w/ ∠2x2x3/16 STIFFENERS w/ $\frac{1'\text{-}6" + 1'\text{-}2"}{2}$ = 1'-4" MAX. TRIBUTARY WALL HEIGHT

STEP 2

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{m}}$$



B-B (TYP. SECTION)

$$\Sigma M_{BOT\,PL} = (.715)(1.431+.25)+(.25)(14)\left(\frac{.25}{2}\right) = 1.639$$

$$\text{CENTROID} = \frac{1.639\ in^2\text{-}in}{(.715+3.5)} = .389"$$

$$I = .272+(.715)(1.292)^2+\frac{(14)(.25)^3}{12}+(14)(.25)(.269) = 2.408\ in^4$$

$$m = \frac{(1.33')(2.5')(135\ \#/ft^3)+2.44+(14)(\tfrac{1}{4})(.28\ \#/in^3)\times\frac{12\ in}{ft}}{32.2\ ft/sec^2}\times\frac{1\ ft^2}{144\ in^2} = .0999\ \frac{\#\text{-}s^2}{in^2}$$

$$f_1 = \frac{\pi}{(2)(38.5)^2}\sqrt{\frac{29000000\times 2.408}{.0999}} = 28.02\ \frac{cyc}{sec}$$

STEP 3

$f_1 = 28.02$ cyc/sec $T = .04$

$.9f_1 = 25.22$ cyc/sec $T = .04$ ←

$1.1f_1 = 30.82$ cyc/sec $T = .03$

HORIZONTAL ACCELERATION IS .26 (SEE NOTE ON p4)

STEP 4

$w = E' = .26 \times [14.2 + 1.33 \times 2.5 \times 135] = 120.4\#/ft$

$M = \frac{.1204 \times 3.17^2}{8} = .151^{k-1}$

$f_b = \frac{Mc}{I} = \frac{.151 \times 12 \times 1.861}{2.408} = 1.40\ ksi < 32.4\ ksi$ ∴ RESTRAINT IS ADEQUATE FOR STRESS

WCG-2-45

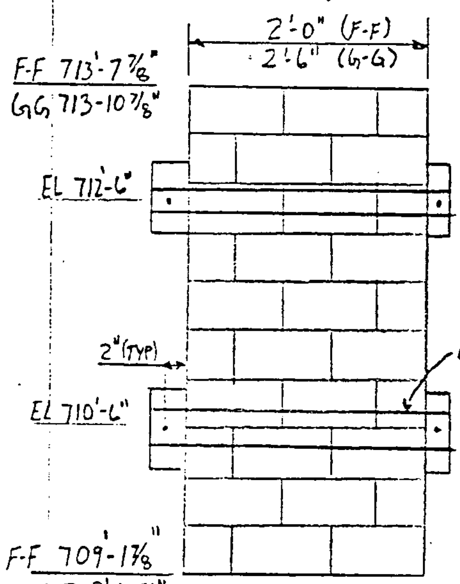
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR
TVA DWG'S 48N943, 41W732-2, 41W732-4

COMPUTED PLG DATE 12-11-81
CHECKED P.M. DATE 12-16-81

CHECK THE ADEQUACY OF THE RESTRAINTS FOR THE 6' THICK WALL ON THE NORTH SIDE OF THE FUEL TRANSFER TUBE @ ~EL 709

3.2.32 UNMORTARED BLOCK WALLS

SECTIONS F-F & G-G



STEP 1

RESTRAINT ~ ∠3"×3"×¼" w/ 2'-4⅞" TRIBUTARY WALL HEIGHT

STEP 2

$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{m}}$

$L = 34"$

$m = \frac{5 + 2.41' \times 6' \times 125\#/ft^3}{32.2\ ft/sec^2} \times \frac{1\ ft^2}{144\ in^2} = .421\ \frac{\#\text{-}sec^2}{in^2}$

$f_1 = \frac{\pi}{2 \times 34^2}\sqrt{\frac{29000000 \times 1.24}{.421}} = 12.56\ \frac{cyc}{sec}$

STEP 3

$1.1 f_1 = 13.81\ \frac{cyc}{sec}$ $T = .07$

$.9 f_1 = 11.30\ \frac{cyc}{sec}$ $T = .09$ ←

$f_1 = 12.56\ \frac{cyc}{sec}$ $T = .08$

FROM ROGER FIELD HORIZONTAL ACCELERATION IS .34

STEP 4

$w = E' = .34 \times [5 + 2.41 \times 6 \times 135] = .665\ \#/ft$ $M = \frac{.665 \times 2.83^2}{8} = .666^{K\text{-}1}$

$f_b = \frac{.666 \times 12}{.577} = 13.85^{ksi} < 32.4^{ksi}$ ∴ RESTRAINTS FOR F-F & G-G ARE ADEQUATE FOR STRESS

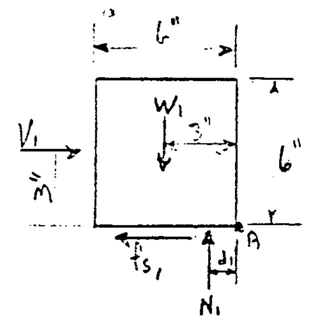
TVA 10180 (WR 7-75)

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

COMPUTED PCG DATE 12-8-81
CHECKED P.M. DATE 12-16-81

CHECK THE STABILITY OF THE TOP 2 BLOCKS OF THE WALL NORTH OF THE FUEL TRANSFER TUBE. THIS IS THE WORST CASE FOR EITHER WALL CONSIDERING UNRESTRAINED BLOCKS.

TOP BLOCK ~ $a_v = .12$ (see sheet 8)



$$W_1 = (1-.12) \times \overbrace{(.5' \times .5' \times 135^{\#}/_{ft^3})}^{34} = 30^{\#}$$

$$V_1 = .34 \times 34^{\#} = 12^{\#}$$

$$N_1 = W_1 = 30^{\#}$$

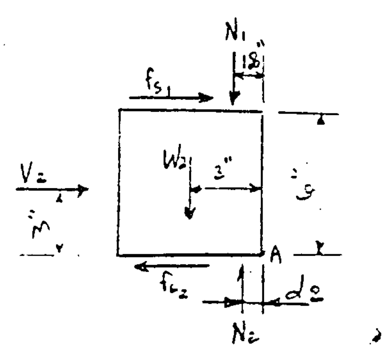
$$f_{s_{1,max}} = \mu N = .7 \times 30 = 21^{\#}$$

SINCE $12^{\#} < 21^{\#}$ BLOCK WILL NOT SLIDE

$$\Sigma M_A = 0 = 12 \times 3 + 30 d_1 - 30 \times 3$$

$d_1 = 1.8'' > 0''$ ∴ BLOCK WILL NOT ROTATE

SECOND BLOCK



$$N_2 = 30 + 30 = 60^{\#}$$

$$f_{s_2} = 12 + 12 = 24^{\#}$$

$$f_{s_{2max}} = .7 \times 60 = 42^{\#}$$

SINCE $24^{\#} < 42^{\#}$ BLOCK WILL NOT SLIDE

$$\Sigma M_A = 0 = 12 \times 3 + 12 \times 6 + 60 \times d_2 - 30 \times 1.8 - 30 \times 3$$

$d = .60'' > 0''$ ∴ BLOCK WILL NOT ROTATE.

TENNESSEE VALLEY AUTHORITY 11/01/73
WBNP INTERIOR CONCRETE STRUCTURE
LONGITUDINAL RESPONSE
G = 0.12 - REVISED MODEL (OCT..1973)
SAFE SHUTDOWN EARTHQUAKE
SPECTRAL EARTHQUAKE RESPONSE
5.0 PERCENT DAMPING
VERTICAL ACCELERATION

WCG-2-45
7
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
PCG 12-11-81
FOR INFO ONLY

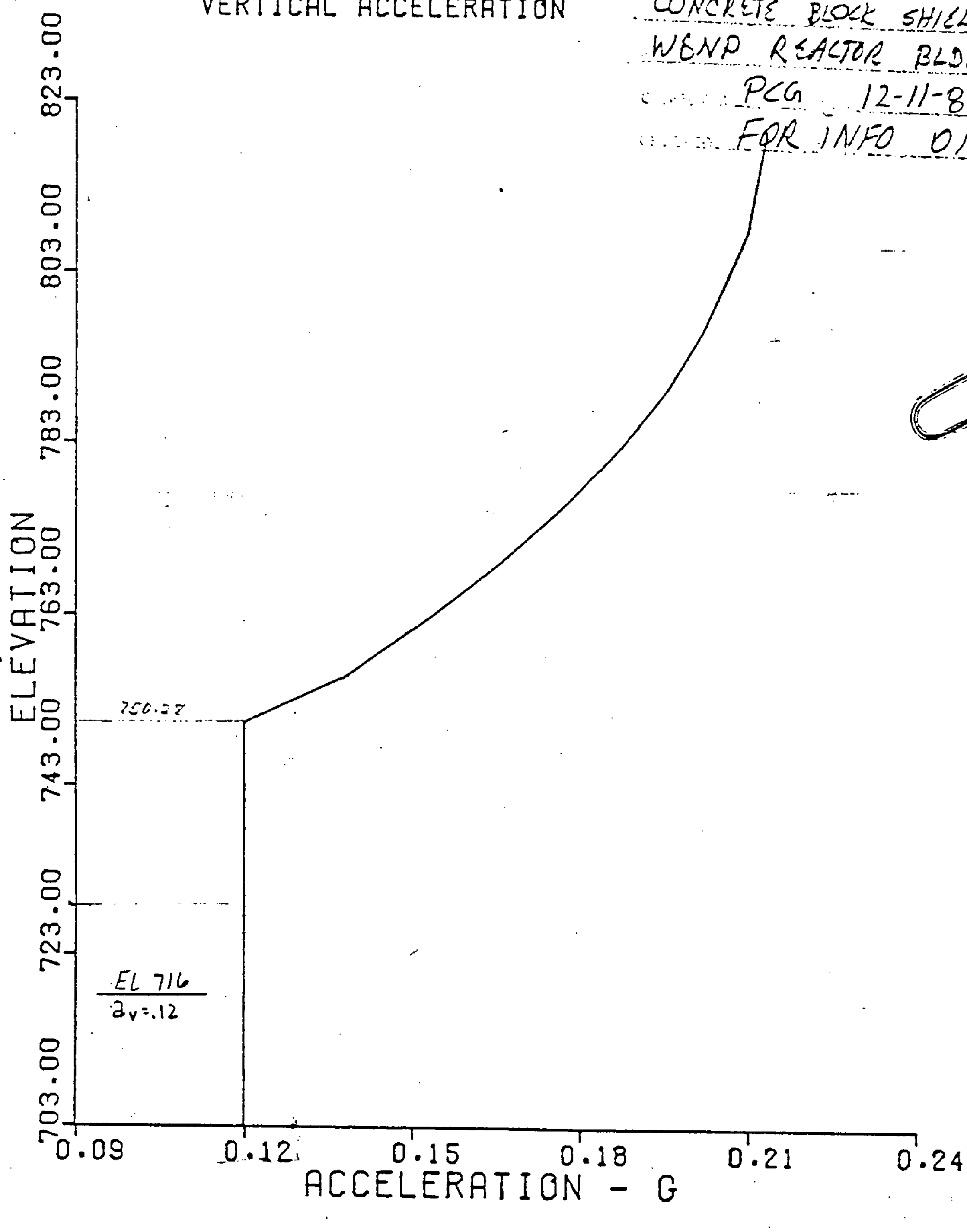
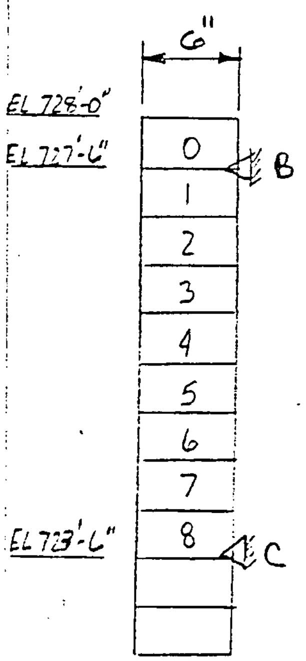
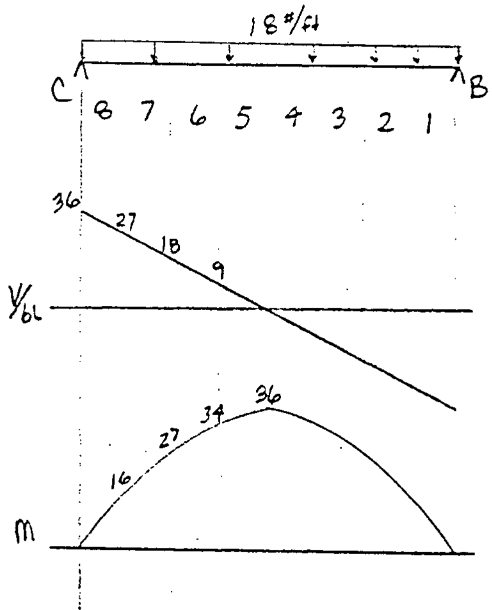


FIGURE B.12

CHECK THE STABILITY OF BLOCKS BETWEEN RESTRAINTS. THE WORST CASE IS SECT. AA @ AZ 106° BETWEEN RESTRAINTS AT ELEVATIONS 723'-6" AND 727'-6". ANALYZED AS A SIMPLE BEAM ACCORDING TO WB-DC-20-30 R1.



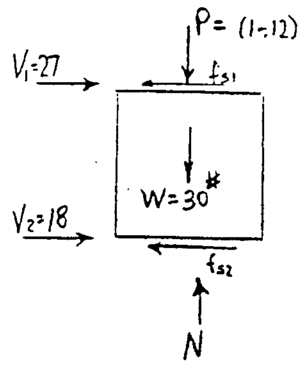
$w = .26 \times 2.66/ft \times 34\#/bl = 18\#/ft$



$V_{max} = \frac{w\ell}{2} = \frac{18 \times 4}{2} = 36\#$

$M_{max} = \frac{w\ell^2}{8} = \frac{18 \times 4^2}{8} = 36\#\text{-}1$

SHEAR ~ BLOCK 2



$fs_{1\,max} = \mu P = .7 \times 60 = 42\# > 27\#$

$N = P + W = 90\#$

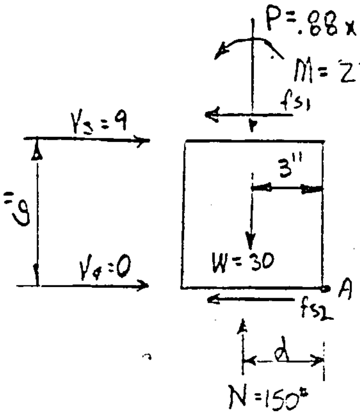
$fs_{2\,max} = \mu N = .7 \times 90 = 63\# > 18\#$

∴ BLOCK WILL NOT SLIDE

WCG-2-45

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4 — PCG 12-11-81
P.M. 12-16-81

MOMENT ~ BLOCK 4



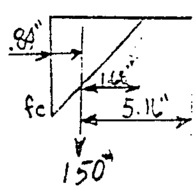
$f_{s_1 \, max} = \mu P = 120 \times .7 = 84^{\#} > 9^{\#}$

$f_{s_2 \, max} = \mu(N) = .7 \times 150 = 105^{\#} > 0^{\#}$

$\Sigma M_A = 0 = 27 - Nd + (120 + 30) \times .25$

$d = .43' = 5.16'' < 6''$ ∴ NO ROTATION

CHECK CONCRETE STRESS:

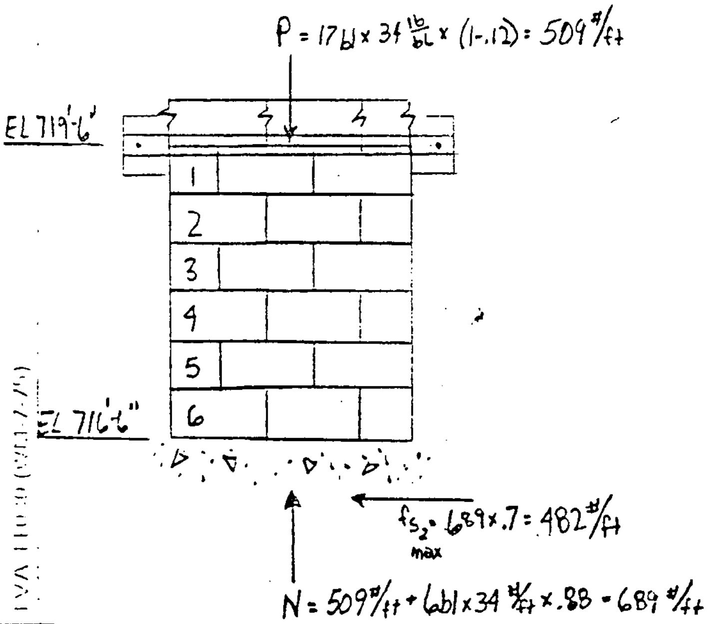


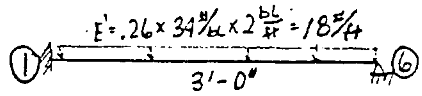
$\frac{f_c \times 2.52''}{2} = 150^{\#}$

$f_c = \frac{150 \times 2}{2.52} = 119^{\#}/_{in^2} < 1350^{\#}/_{in^2}$ (3.1.3) ∴ OK

CHECK STABILITY OF WALL BETWEEN EL 719'-6" & 716'-6": (SECT. A-A)

TREAT WALL AS A PROP CANTILEVER





SHEAR @ 6 = $\frac{18 \times 3}{2} = 27^{\#}$

SINCE $27^{\#} < 482^{\#}$ NO SLIPPAGE OF BLOCKS @ BASE