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

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TVA

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						Thermal Power Design Project						Architectural, Hydro, & Special Projects Engineering & Design Branches							
MEB		EEB		CEB		Project Manager				R	A	AD		ME&D		CE&D		EE&D	
R	A	R	A	R	A					[illegible]	[illegible]	R	A	R	A	R	A	R	A
				NES	NES	MD		ED		CD									
				DC	[illegible]	R	A	R	A	R	A								
				JAE	JAE					PKL	PKL								
				RDG	RDG					[illegible]	[illegible]								
				[illegible]	[illegible]					[illegible]	[illegible]								
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				RJH	[illegible]					WDL	WDL								
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TABLE OF CONTENTS

		Page
1.0	SCOPE	1
2.0	PURPOSE	1
3.0	EVALUATION BASIS	1
3.1	Materials	1
3.1.1	Concrete Blocks	1
3.1.2	Mortar	1
3.2	Loads	2
3.2.1	Dead Loads (D)	2
3.2.2	Live Loads (L)	2
3.2.3	Seismic Loads (E, E')	2
3.2.4	Pipe Break Loads (P_a, Y_j, Y_m)	7
3.2.5	Tornado Loads (W_t)	7
3.2.6	Flood Loads (F)	7
3.3	Load Combinations	7
3.3.1	Service Loads	7
3.3.2	Extreme Environmental and Abnormal Loads	7
3.4	Allowable Stresses	8
3.4.1	Compressive Strength	8
3.4.2	Axial Stress	8
3.4.3	Flexure	9
3.4.4	Shear	9
3.4.5	Tensile Stress	9
3.4.6	Limitations of Stresses	9
3.4.7	Combined Stress	10
4.0	ANALYSIS	10
4.1	Mortared Block Walls	10
4.2	Unmortared Block Walls	10
5.0	REFERENCES	10

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 C55) 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. R,

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 nth 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:

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

where the parameters are defined in Step 1 of section 3.2.3.1.1. All frequencies ≤ 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.

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).

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

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.

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.

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, $1.1f_1$) 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 M 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

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

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

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

3.4.4 Shear

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

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

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

$v_m \leq f_s$

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

3.4.5 Tensile Stress

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

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

$F_t = 1.5\sqrt{m_o}$ $\leq$ 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^2 = Static frictional force at top face of block ($\mu = 0.7$)
f_{s2}^{s1} = 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 = Static frictional force at top face of block ($\mu = 0.7$)
f^{s1} = Static frictional force at bottom face of ($\mu = 0.7$)
N^{s2} = 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

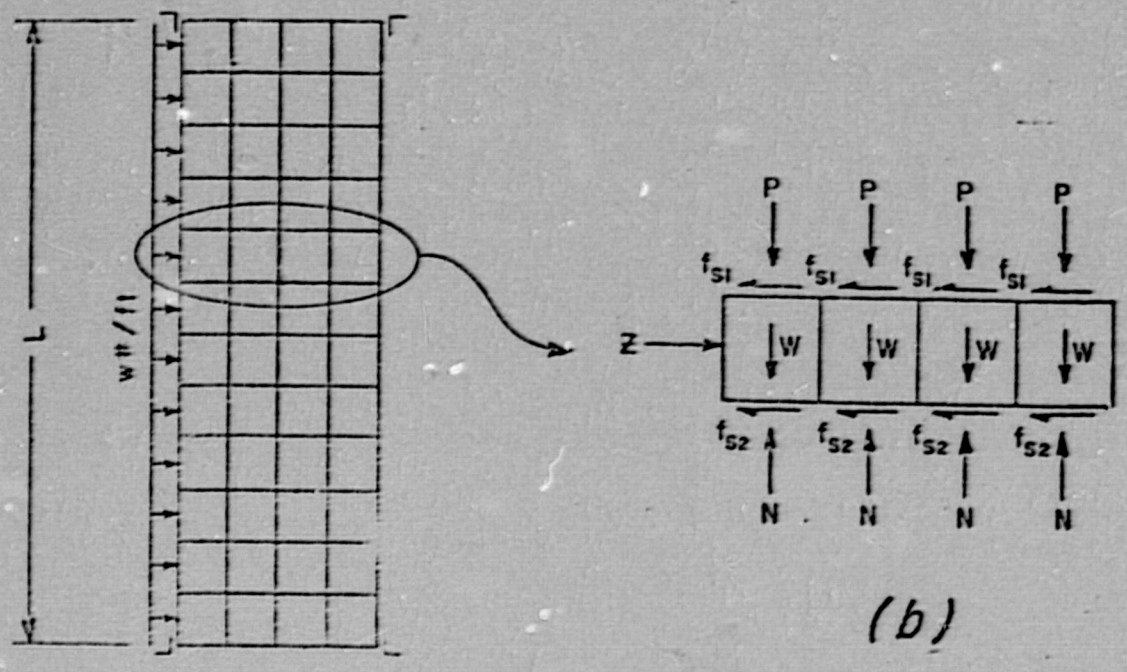
5.1 American Society for Testing and Materials (ASTM) "Solid Load-Bearing Concrete Masonry Units," Designation C145-71.

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.

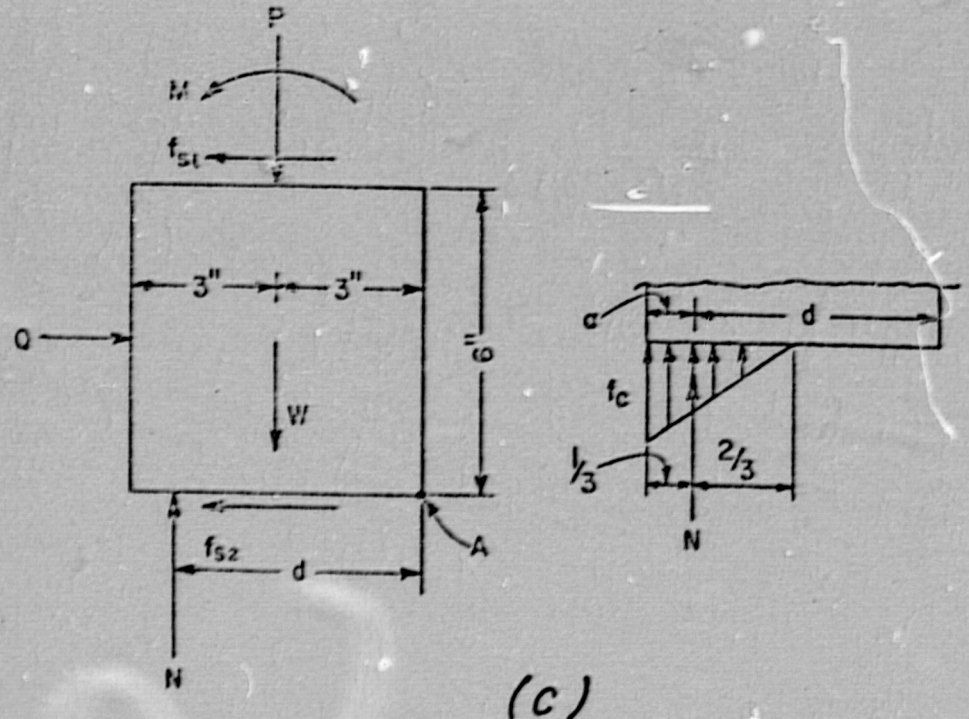
5.4 Introduction to Structural Dynamics, John M. Biggs, 1964, Chapter 4.

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



(a)

(b)



(c)

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

PCG 12-4-81
P.M. 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

PCG 12-10-81
P.M. 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.

TVA 11010 (WM 2-75)

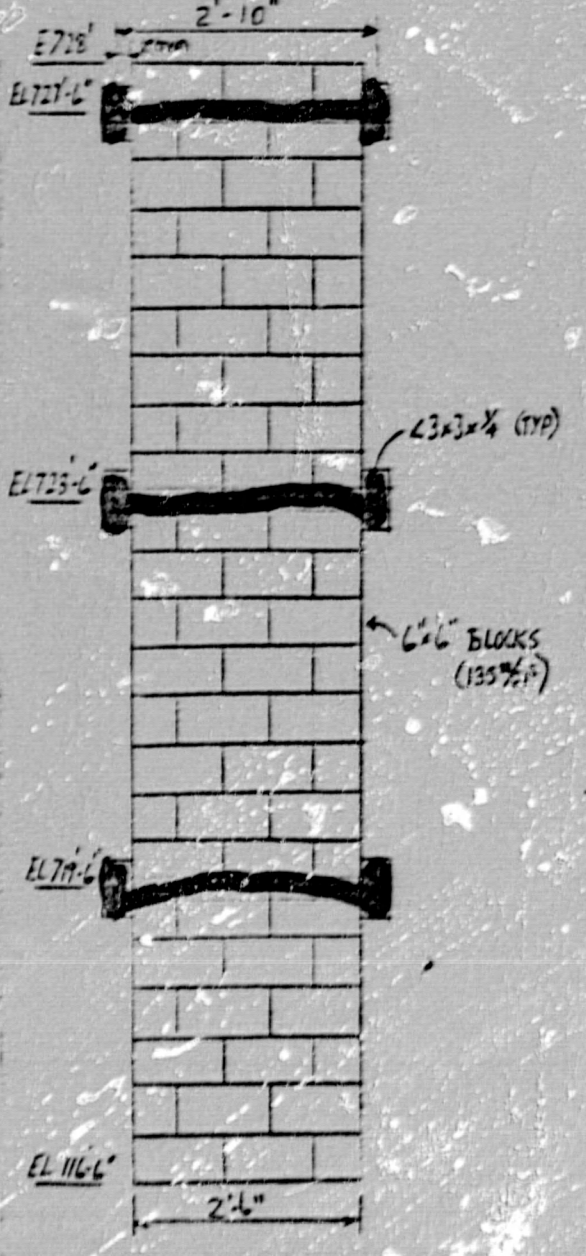
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

PCG 12-11-81
P.M. 12-16-81

CHECK THE ADEQUACY OF THE RESTRAINTS FOR THE WALL AT AZ 106°, EL 716'-6". SEE DWG'S 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

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

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

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

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

$m = \frac{5\#/ft + 4' \times 2.5' \times 175\#/ft^3}{322\ ft/sec^2} \times \frac{1ft^2}{144in^2} = .2922\ \frac{\#\text{-}sec^2}{in^2}$ (RESTRAINT & TRIBUTARY WALL MASS)

$$f_1 = \frac{\pi}{2(34)^2}\sqrt{\frac{29000000 \times 1.24}{.2922}} = 15.073\ \frac{cyc}{sec}$$

STEP 3

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

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

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

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

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWGS 48N743, 41W732-2, 41W732-4

PCG 12-11-81
P.M. 12-16-81

STEP 4

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

$M = \frac{wL^2}{8} = \frac{352 \times 2.83^2}{8} = 352^{\#\text{-ft}} = .352^{K\text{-}'}$

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

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

SINCE $7.32^{KSI} < 32.4^{KSI}$ RESTRAINT IS OK IN BENDING

CHECK TENSION IN ½"Ø BOLTS

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

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

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

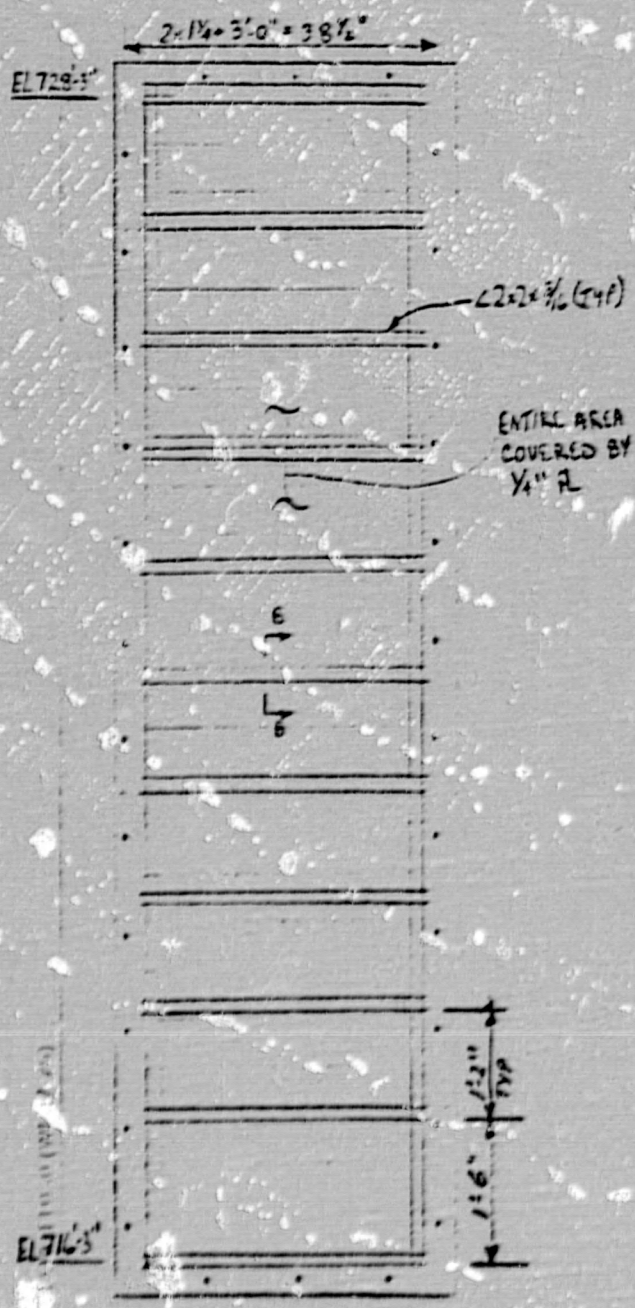
PCG 12-11-81
P.M. 12-16-81

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

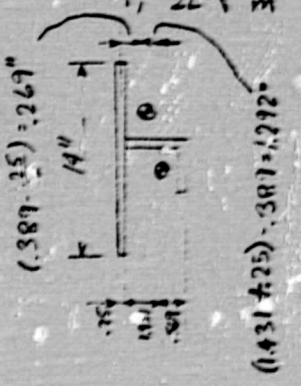


STEP 1

RESTRAINT ~ ¼" PL w/ ∠2×2×3/16 STIFFENERS w/ $\frac{1'-6"+1'-2"}{2}$ = 1'-4" MAX. TRIBUTARY WALL HEIGHT

STEP 2

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



B-B (TYP. SECTION)

$\Sigma M_{BOT_{PL}} = (.715)(1.431+.25) + (.25)(14)(\frac{.25}{2}) = 1.639$

$\text{CENTROID} = \frac{1.639 \text{ in}^2\text{-in}}{(.715+.25)} = .389"$

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

$m = \frac{(1.33)(2.5)(.35 \text{ lb/ft}^3) + 2.44 + (14)(\frac{1}{4})(.28 \text{ lb/in}^3) \times \frac{12 \text{ in}}{\text{ft}}}{32.2 \text{ ft/sec}^2} \times \frac{1 \text{ ft}^2}{144 \text{ in}^2} = .0999 \frac{\text{lb-s}^2}{\text{in}^2}$

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

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG 48N993, 41W732-2, 41W732-4

PCG 12-11-81
P.M. 12-16-81

STEP 3

$f_1 = 28.02 \frac{cyc}{sec}$ T = .04

$.9f_1 = 25.22 \frac{cyc}{sec}$ T = .04 ←

$1.1f_1 = 30.82 \frac{cyc}{sec}$ T = .05

HORIZONTAL ACCELERATION IS .26 (SEE NOTE ON p4)

STEP 4

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

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

$f_b = \frac{Mc}{I} = \frac{.151 \times 12 \times 1.861}{2,408} = 1.40^{ksi} < 32.4^{ksi}$ ∴ RESTRAINT IS ADEQUATE FOR STRESS

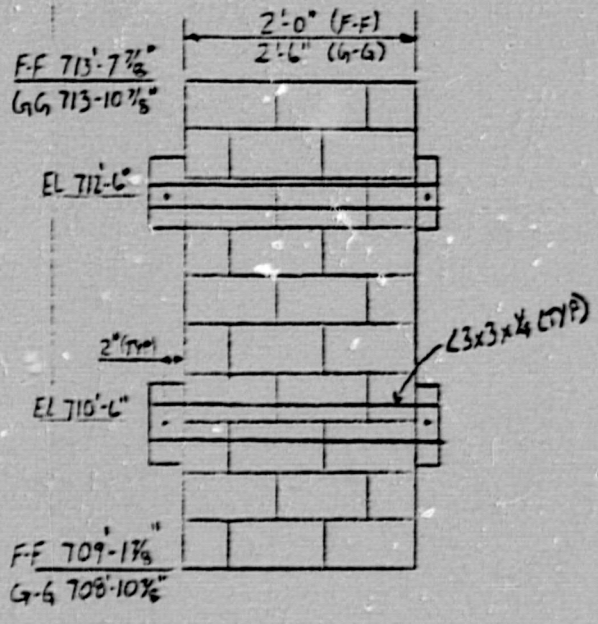
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR
TVA DWG'S 48N943, 41W732-2, 41W732-4

PCG 12-11-81
P.M. 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 UNREINFORCED BLOCK WALLS

SECTIONS F-F & G-G



STEP 1

RESTRAINT ~ L 3" x 3" x 1/4" @ 2'-4 7/8" TRIBUTARY WALL HEIGHT

STEP 2

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

$L = 34"$

$m = \frac{5 \times 2.41' \times 6' \times 12 \frac{\#}{ft^3}}{32.2 \frac{ft}{sec^2}} \times \frac{ft^2}{144 in^2} = .421 \frac{\#\text{-sec}^2}{in^2}$

$f_1 = \frac{\pi}{2 \times 34^2}\sqrt{\frac{29{,}000{,}000 \times 1.24}{.421}} = 12.56 \frac{cycl}{sec}$

STEP 3

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

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

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

FROM ROGER FIELD HORIZONTAL ACCELERATION IS .34

STEP 4

$w = E' = .34 \times [5 \times 2.41 \times 6 \times 1.25] = 6.65 \#/ft$ $M = \frac{.665 \times 2.83^2}{8} = .666^{k\text{-}i}$

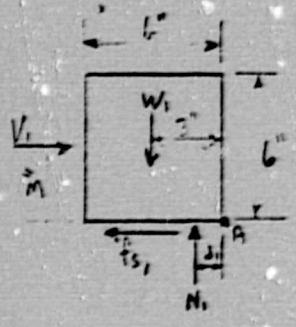
$f_b = \frac{.666 \times 12}{.577} = 13.85^{ksi} < 37.4^{ksi}$ ∴ RESTRAINTS FOR F-F & G-G ARE ADEQUATE FOR STRESS

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

PCG 12-8-81
P.M. 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)



$V_1' = (1 - .12) \times (.5 \times .5 \times 135\#/ft^2) = 30\#$ (34)

$V_1 = .34 \times 34\# = 12\#$

$N_1 = W_1 = 30\#$

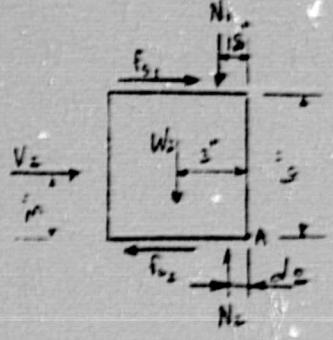
$f_{s_{1max}} = \mu N = .7 \times 30 = 21\#$

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

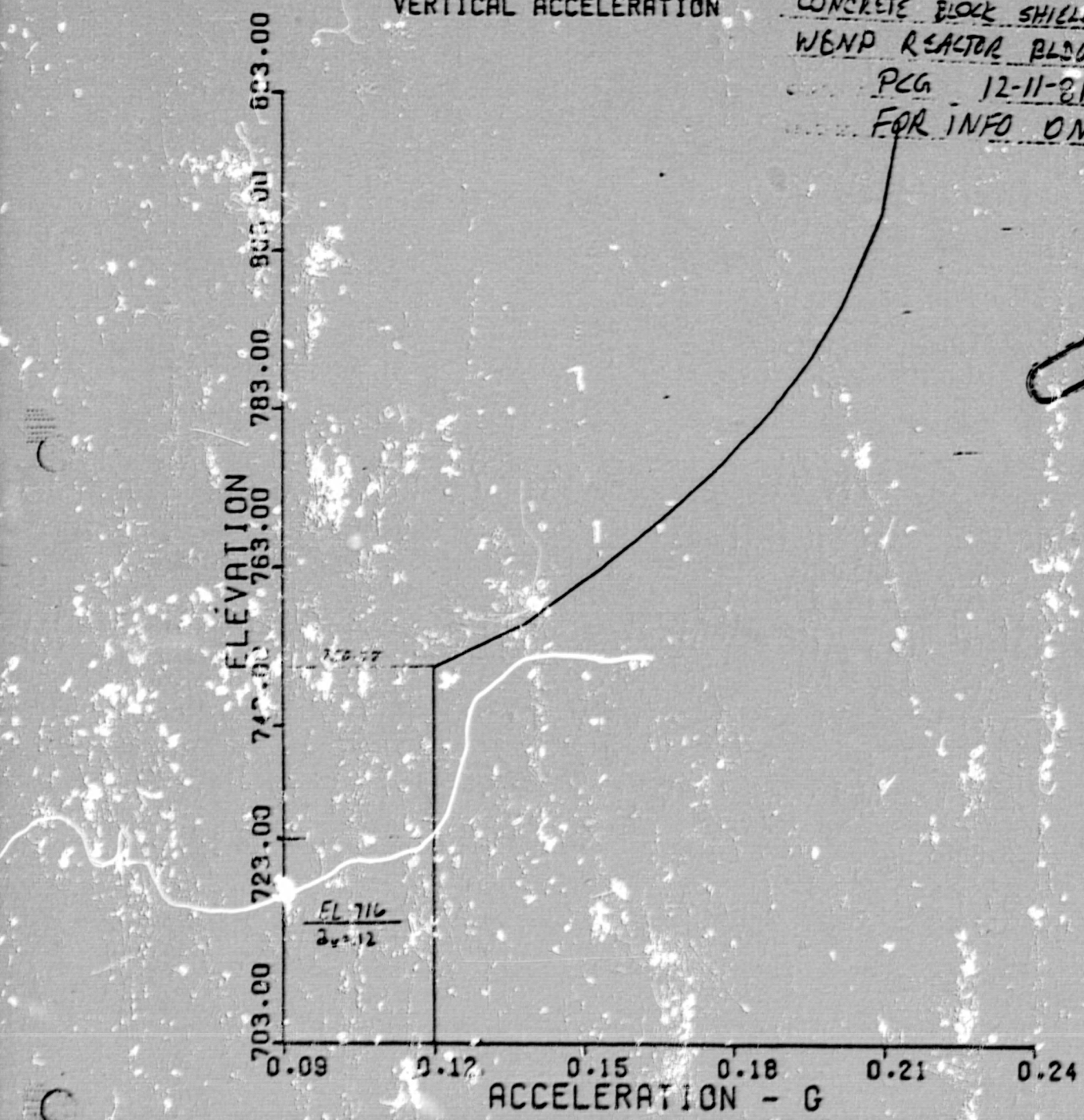
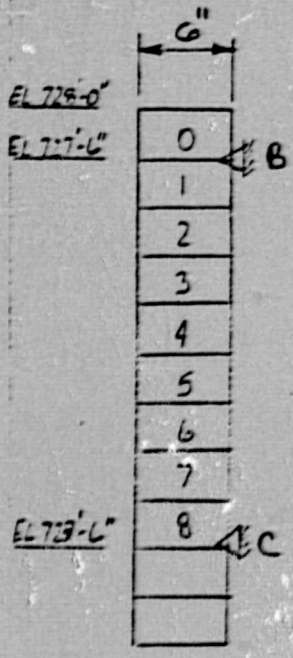


FIGURE B.12

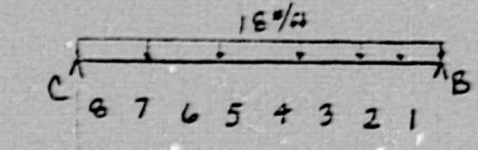
CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 48N943, 41W732-2, 41W732-4

PCG 12-11-81
P.M 12-16-81

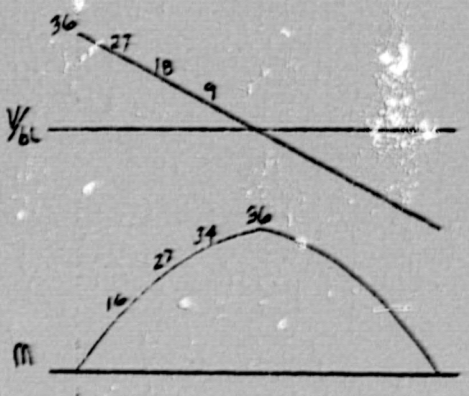
CHECK THE STABILITY OF BLOCKS BETWEEN RESTRAINTS. THE WORST CASE IS SECT. AA @ AZ 106° BETWEEN RESTRAINTS AT ELEVATIONS 723'-6" AND 727'-6". ANALYZED AS A SIMPLE BEAM ACCORDING TO WB-DC-20-30 R1.



$W = .26 \times 264/ft \times 34''/12 = 18''/in$

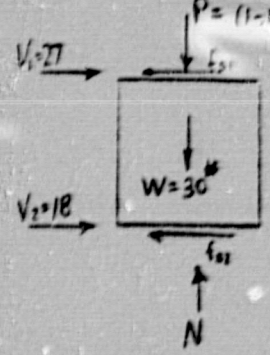


$V_{max} = \frac{wl}{2} = \frac{18 \times 4}{2} = 36''$



$M_{max} = \frac{wl^2}{8} = \frac{18 \times 4^2}{8} = 36''-1$

SHEAR ~ BLOCK 2



$f_{s_1 max} = \mu P = .7 \times 60 = 42'' > 27''$

$N = P + W = 90''$

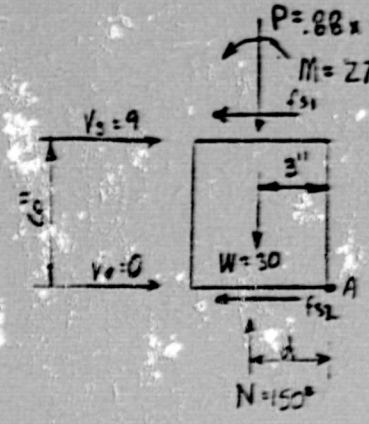
$f_{s_2 max} = \mu N = .7 \times 90 = 63'' > 18''$

∴ BLOCK WILL NOT SLIDE

CONCRETE BLOCK SHIELDING WALLS
WBNP REACTOR BLDG
TVA DWG'S 9BN943, 41W932-2, 41W732-4

PCG 12-11-81
P.M. 12-16-81

MOMENT - BLOCK 4



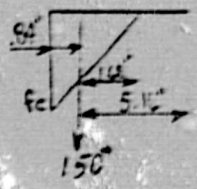
f_{s_1} max $= MP = 120 \times .7 = 84\# > 9\#$

f_{s_2} max $= M(.1) = .7 \times 150 = 105\# > 0\#$

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

$d = .43' = 6.16" < 6"$ ∴ NO ROTATION

CHECK CONCRETE STRESS:

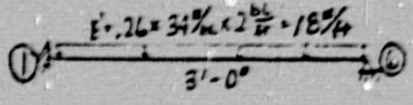


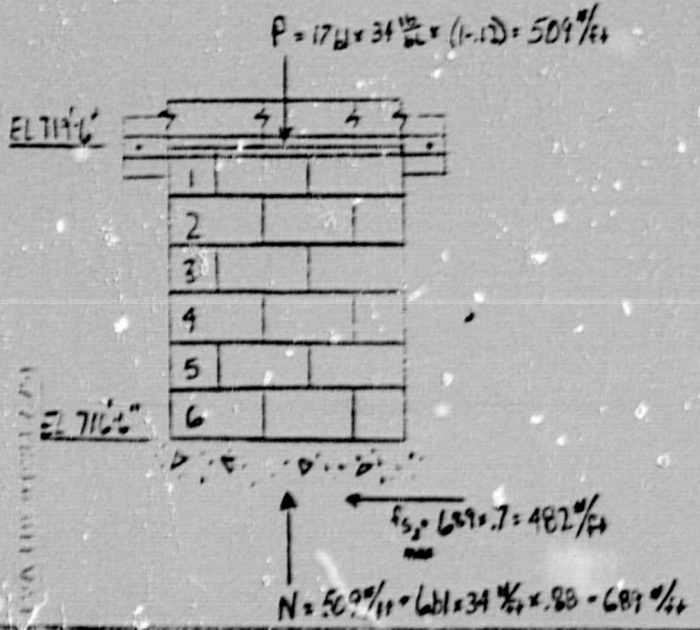
$\frac{f_c \times 2.52"}{2} = 150\#$

$f_c = \frac{150 \times 2}{2.52} = 119\#/in^2 < 1350\#/in^2$ (3.4.3) ∴ 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