NORTHEAST UTILITIES

THE CONNECTICUT LIGHT AND POWER COMPANY
WESTERN MASSACHUSETTS ELECTRIC COMPANY
HOLYOKE WATER POWER COMPANY
NORTHEAST UTILITIES SERVICE COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

General Offices • Selden Street, Berlin, Connecticut

P.O. BOX 270
HARTFORD, CONNECTICUT 06141-0270
(203) 666-6911

December 3, 1982

Docket No. 50-213
50-245
50-336
B10624

Director of Nuclear Reactor Regulation
Attn: Mr. Robert A. Clark, Chief
Operating Reactors Branch #3
Mr. Dennis M. Crutchfield
Operating Reactors Branch #5
U. S. Nuclear Regulatory Commission
Washington, DC 20555

Reference: (1) W. G. Counsil letter to NRC, dated October 4, 1980.

(2) R. A. Clark letter to W. G. Counsil, dated September 28, 1982.

Gentlemen:

Haddam Neck Plant
Millstone Nuclear Power Station, Unit Nos. 1 & 2
Request for Additional Information on IE Bulletin 80-11,
Masonry Wall Design

In Reference (2) the NRC Staff requested Connecticut Yankee Atomic Power Company (CYAPCO) and Northeast Nuclear Energy Company (NNECO) to supply additional information to our Reference (1) response to IE Bulletin 80-11 on Masonary Wall Design.

Accordingly, CYAPCO and NNECO hereby provide the NRC with information as documented in Enclosures 1, 2, and 3 for Haddam Neck, Millstone Unit 1, and Millstone Unit 2, respectively. Where drawings have been referred to for the Haddam Neck Plant and Millstone Unit 1 responses, three sets of drawings have been provided for Staff use.

We trust you will find this information responsive to your Reference (2) requests.

Very truly yours,

CONNECTICUT YANKEE ATOMIC POWER COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

W. G. Counsil
Senior Vice President

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Aperture Card
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Drawings To: (1 Set Rec'd)
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Docket Nos. 50-213
50-245
50-336
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Haddam Neck Plant
Millstone Nuclear Power Station, Unit Nos. 1 & 2
Request for Additional Information on IE Bulletin 80-11, Masonry Wall Design

December 3, 1982

Docket No. 50-213

ENCLOSURE I

HADDAM NECK PLANT

REQUEST FOR ADDITIONAL INFORMATION ON
IE BULLETIN 80-11 MASONRY WALL DESIGN

NOVEMBER, 1982

RESPONSE TO NRC QUESTIONS
MASONRY WALL RE-EVALUATION
(NRC IE BULLETIN 80-11)
HADDAM NECK PLANT

1. QUESTION:

Indicate how equipment loads were considered in the analysis.

RESPONSE:

Equipment loads were conservatively estimated in the field on the basis of their volume and contents. The details of how the significant components were attached to the masonry wall were obtained during the field inspection in order to determine how the loads would be transferred from the attachment. These conditions were represented in the model used to determine the masonry walls response.

2. QUESTION:

Indicate if testing for material properties was used in lieu of the minimum specified. If so, provide a description of the testing program and the test data.

RESPONSE:

All minimum values as outlined in the ACI 531-79 were used in the re-analysis of Haddam Neck.

3. QUESTION:

Indicate how variations in frequency due to uncertainties in mass, materials, and other parameters were considered in the analysis.

RESPONSE:

After making a preliminary review of the wall a decision was made determining the use of either the peak response from an appropriate response spectrum or a calculation made to determine the fundamental frequency of the wall. If the fundamental frequency of the wall was chosen, a broadened spectrum of 85 percent to 115 percent of the fundamental frequency of the wall and steel system was used to determine the corresponding maximum value of the assumed spectral acceleration.

4. QUESTION:

In calculating seismic loads, the Licensee multiplies the wall acceleration by factors of 1.3 for out-of-plane loads and 1.0 for in-plane loads. Justify this difference in factors used for in-plane and out-of-plane loads.

RESPONSE:

Since the wall is rigid (frequency $\geq$33 Hz) in response to in-plane loads no increase factor was needed in the evaluation of in-plane wall acceleration. A higher load factor was needed in the analysis of out-of-plane wall acceleration because the wall did not act rigid (frequency < 33 Hz). For these cases the criteria specifies that the value of the spectral acceleration be multiplied by 1.3 in order to get the total response of the wall which will include the contribution of higher modes.

5. QUESTION:

Indicate whether cracking of sections was given proper consideration in the analysis.

RESPONSE:

A field walkdown was conducted, and all cracking in the blockwalls was inspected and evaluated. Based on this analysis, the wall was modified so that only limited cracking would occur or that failure of the wall would not interfere with safety related equipment.

6. QUESTION:

Specify limits for the relative displacement in the shear stress equation (3) in Section 4.3.2 of Reference 2.

RESPONSE:

Considering in-plane loads the maximum deflection was based upon the maximum allowable shear stress of the wall without a specified relative displacement limitation.

7. QUESTION:

In Section 4.4.1 of Reference 2, the Licensee states that the response of in-filled panels to in-plane relative displacements may be analyzed by considering the non-linear composite action of the masonry wall and surrounding frame. The licensee is requested to identify all walls which were qualified by this technique. Discuss and provide the technical basis for the non-linear composite motion. The Licensee is also requested to provide the results of the analysis along with some sample calculations.

RESPONSE:

Although an outline of the non-linear composite action of masonry walls and corresponding frames was given, this method was not used in the re-analysis of masonry walls at the Haddam Neck Plant.

8. QUESTION:

In Reference 2, the Licensee indicated that arching action has been used to qualify some cf the masonry walls. The NRC, at present, does not accept the application of this method to masonry walls in nuclear power plants in the absence of conclusive evidence to justify this application. The Licensee is requested to indicate the number of walls which have been analyzed by this technique and provide a sample calculation. Before any conclusions can be made the following areas need technical verification:

- o Explain how arching action handles cyclic loading, especially when the load is reversed.

- o Provide justification and test data (if available) to validate the applicability of the arching theory to the masonry structures at Haddam Neck with respect to:

  a. nature of the load,
  b. boundary conditions,
  c. material strength,
  d. size of the test walls.

- o If hinges are formed in the walls, the capability of the structures to resist in-plane shear force would be diminished, and shear failure might take place. This in-plane shear force would also reduce the out-of-plane stiffness. Explain how the affect of this phenomenon can be accurately determined.

RESPONSE:

One masonry wall at Haddam Neck was analyzed considering arching action. Calculations outlining this method of analysis are included in Attachment 1.

Arching Theory

- o The arching theory of confined masonry walls is based on an extensive full-scale test program sponsored by the Defense Civil Preparedness Agency and the Veterans Administration (Ref. 1). The tests were performed in a "shock tunnel" in which a wall was "struck by a blast wave, the wave reflected from the wall returned to its source area, then re-reflected to strike the wall again, about 0.3 sec. after the first loading. Thus, these tests provided information on the ability of [arching] walls to withstand a number of reversing loading cycles."

- o a) At the Haddam Neck Plant, arching theory was used to determine the ability of masonry walls to resist uniform pressure loadings and earthquake loadings. The test data of Reference 1 provides justification for applying arching theory to masonry walls subjected to pressure loads. Reference 1 also cites field evidence which confirms the

increased resistance to out-of-plane earthquake loads of confined [arching] masonry walls over non-arching walls.

b) Boundary conditions required for the application of arching theory were validated through walkdown inspection.

c) Experiments on arching have been performed on walls that range in crushing strength from 1,000 psi to 3,000 psi (Ref. 2). The masonry walls at Haddam Neck fall within this range.

d) The tests of Reference 2 were performed on walls arching over an 8'-0" span while those of Reference 1 spanned 8'-6". We consider that application of arching theory to the of wall size encountered at Haddam Neck is appropriate.

o At its present stage of development, arching theory cannot adequately account for the effect of hinge formation on in-plane shear capacity. However, the wall at Haddam Neck analyzed by the arching method is not designed to act as a shear wall nor as a bearing wall and, therefore, is not subjected to significant in-plane load. This eliminates the need to determine how in-plane and out-of-plane loads interact.

9. QUESTION:

Indicate whether the construction practice for masonry walls at the Haddam Neck plant was in conformance with provisions specified for the special inspection category in ACI 531-79 [7]. If not, explain and justify the use of allowable stresses.

RESPONSE:

The practices used in the construction of the masonry walls at the Haddam Neck plant were set forth in the original specification for masonry work (Ref. 4). Field engineers did not find any deviations from these specification requirements. Higher stress allowables outlined in the special inspection category of ACI 531-79 were used.

10. QUESTION:

Justify by any existing test data or literature the use of 35 psi for allowable masonry diagonal tension.

RESPONSE:

The use of 35 psi for allowable masonry diagonal tension was chosen to correspond with non-linear composite action within masonry walls. This method of analysis was not used in the re-evaluation of masonry walls for the Haddem Neck Plant.

11. QUESTION:

Provide any increase factors that may have been used for allowable stresses under abnormal conditions. If they are higher than those listed in the SEB criteria [15], provide justification. The SEB factors are listed below by type of stress:

Axial or flexural compression	2.5
Bearing	2.5
Reinforcement stress except shear	2.0 but not to exceed f_y
Shear reinforcement and/or bolts	1.5
Masonry tension parallel to the bed joint	1.5
Shear carried by masonry	1.3
Masonry tension perpendicular to the bed joints	1.5
Reinforced masonry	0
Unreinforced masonry	1.3

RESPONSE:

SSE was the only factored load case considered. A 67% increase was used for all allowable SSE values. The allowable stresses were increased by 67% for the SSE because the probability of occurrence of the SSE is relatively low, and, if it does occur, the allowable stresses can reach the ultimate or yield stresses. For structural steel used in the modifications a 67% increase for the SSE will increase most of the allowable stresses to the yield stress. However, the maximum allowable steel stress with the 67% increase will always be less than the material yield stress to prevent permanent inelastic ductile deformation.

12. QUESTION:

Indicate if block pullout was considered in the analysis. If so, provide sample calculations of block pullout analysis.

RESPONSE:

Block pullout was considered in the analysis of masonry walls at Haddam Neck. Based on this analysis, plates were sized to transfer the pullout loads into the wall. Sample calculations demonstrating this analysis are included in Attachment 2.

A sample evaluation demonstrating this approach is included in attachment 2.

13. QUESTION:

Indicate whether the walls are stacked or running bond. If any stacked bond wall exists, provide sample calculations to obtain stresses of a typical wall.

RESPONSE:

All mortared masonry walls at the Haddam Neck Plant are running bond. Some unmortared masonry walls which are used only for shielding are stacked. These walls have been modified such that failure of the wall will not result in damage to safety-related equipment. Sample calculations demonstrating this type of design are included in Attachment 3.

14. QUESTION:

In Reference 4, the Licensee reports that all modifications have been completed except those for wall CYTB-1008 in the turbine building, which were scheduled to be completed by September 1, 1981. Verify the status of the modifications to this wall and other walls in inaccessible areas.

RESPONSE:

The modifications to all walls at the Haddam Neck Plant have been completed.

15. QUESTION:

In Reference 4, a final report was scheduled to be docketed upon completion of the modifications. Provide this report.

RESPONSE:

The final report outlining the masonry wall re-evaluation and design for the Haddam Neck Plant has been included in Attachment 5.

16. QUESTION:

Provide a description of the modifications and detailed drawings of some sample modifications. In addition, provide a sample calculation to show that the modified walls will be qualified under the working stress design method.

RESPONSE:

Modifications to the masonry walls included:

- o Detachment of the top and sides of the wall from the adjacent boundry supports due to large interstory displacements.

- o Insertion of angles at the wall edges to assure pinned boundary conditions.
- o Insertion of steel sections to transfer wall loads to the boundaries and limit wall spans.
- o Insertion of steel bumpers at the top of the wall to prevent overturning, and at the base of the wall to prevent sliding.
- o Shielding so that falling blocks would not hit safety related equipment.
- o Removal of the wall.
- o Rerouting of pipes.
- o Relocation of pipe supports.

Detailed drawings of some sample modifications have been included in this package. Sample calculations showing that the modified walls were qualified under the working stress design method are included in Attachments 3 and 4.

17. QUESTION:

Since the Haddam Neck plant is a part of the systematic evaluation program (SEP), the Licensee is requested to clarify whether the seismic analysis was performed based on SEP loadings. If not, provide justification.

RESPONSE:

The masonry walls at the Haddam Neck Plant were evaluated based on a set of response spectra generated specifically for the effort to respond to I&E Bullstin No. 80-11. The load combinations were those specified in the Plant's Final Design and Safety Analysis Report. The time constraints associated with the Bulletin did not afford the opportunity to use the response spectra which were being developed as part of the Systematic Evaluation Program (SEP). The response spectrum used to evaluate the masonry walls was the Interim Seismic Design Ground Spectrum. This criterion was considered to be conservative and the actual floor response spectra generated for the SEP will be compared to the assumed spectra used for I&E Bulletin No. 80-11. We do not expect the results of the concrete masonry wall re-evaluations to be affected by this review. Additional information is provided in the final report included as Attachment 5.

References:

1) Gabrielsen, B. L., Kaplan, K. "Arching in Masonry Walls Subjected to Out-of-Plane Forces," Earthquake Resistance of Masonry Construction, National Workshop, NBS 106, 1976.

2) McKee, K. E., Sevin, E., "Design of Masonry Walls for Blast Loading," ASCE Structural Journal, January, 1958.

3) Benjamin, R. J., and H. A. Williams, "The Behavior of One-Story Brick Shear Walls," Journal of the Structural Division, ASCE, July, 1958.

4) Stone & Webster Engineering Corporation, Specification for Masonry Work for Unit No. 1, Connecticut Yankee Atomic Power Plant, Connecticut Yankee Atomic Power Company, Haddam, Connecticut, Boston, Massachusetts, July 14, 1965.

ATTACHMENT 1

consists of a 21 page (sheets 2-20) example of arching action.

DERIVATION OF EQUATIONS

INITIAL STAGE

1. Midspan crack starts forming
2. One of the top corner just start touching top support.

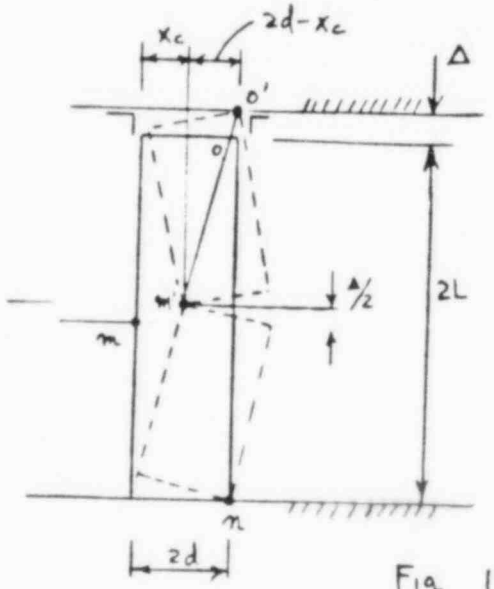


Fig 1a

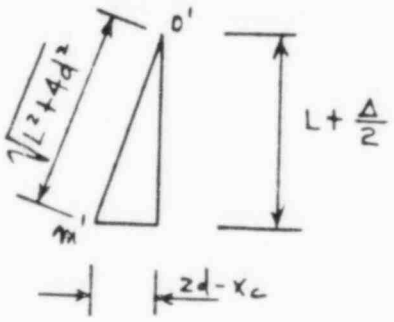


Fig 1b

Assumptions:

1. No horizontal and vertical forces developed
2. O' is directly above O
3. Crack developed at mid-section.
4. No deformation in the block wall.
5. Two sections of the wall behave like two rigid links with hinges at location O', m', and n

In Fig. 1b

$$\left(\sqrt{L^2+4d^2}\right)^2 = \left(L+\frac{\Delta}{2}\right)^2 + (2d-X_c)^2$$

$$L^2 + 4d^2 = X_c^2 + L\Delta + \frac{\Delta^2}{4} + 4d^2 - 4dX_c + X_c^2$$

$$\frac{\Delta}{4} + L\Delta + (X_c^2 - 4dX_c) = 0 \quad \cdots\cdots \text{EQN 1'}$$

$$\Delta = \frac{-L \pm \sqrt{L^2 - 4(\frac{1}{4})(X_c^2 - 4dX_c)}}{2 \cdot \frac{1}{4}}$$

$$= -2L \pm 2\sqrt{L^2 - (X_c^2 - 4dX_c)}$$

since Δ can not be negative

$$\boxed{\Delta = -2L + 2\sqrt{L^2 + (X_c^2 - 4dX_c)}} \qquad \text{EQN 1}$$

If we want X_c, then rearrange EQN 1'

$$X_c^2 + 4dX_c + (L\Delta + \frac{\Delta^2}{4}) = 0$$

$$X_c = \frac{-(-4d) \pm \sqrt{(-4d)^2 - 4(1)(L\Delta + \frac{\Delta^2}{4})}}{2}$$

$$= 2d \pm \sqrt{4d^2 - (L\Delta + \frac{\Delta^2}{4})}$$

since X_c can not be greater than $2d$

$$\boxed{X_c = 2d - \sqrt{4d^2 - (L\Delta + \frac{\Delta^2}{4})}} \qquad \text{EQN 2}$$

Equations for Various Block Sizes with 1/16" Gap

16 x 8 x 6 block

$2d = 5\frac{5}{8} = 5.625"$

$$X_c = (5.625) - \sqrt{(5.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 5.625 - \sqrt{31.6396 - 0.0625L}$$

16 x 8 x 8 block

$2d = 7\frac{5}{8} = 7.625"$

$$X_c = (7.625) - \sqrt{(7.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 7.625 - \sqrt{58.1396 - 0.0625L}$$

16 x 8 x 12 block

$2d = 11\frac{5}{8} = 11.625"$

$$X_c = (11.625) - \sqrt{(11.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 11.625 - \sqrt{135.1396 - 0.0625L}$$

JRS/BLUME
Jessie Street (at New Montgomery)
Francisco, California 94105

SHEET NO. 5

JOB NO. 8040-03 JOB CY BLOCK WALL BY LC DATE 10/24/80

CLIENT NUSCO SUBJECT ARCH THEORY FOR WALL WITH GAP CHK'D JSM DATE 10/24/80

16x8x6 BLOCK

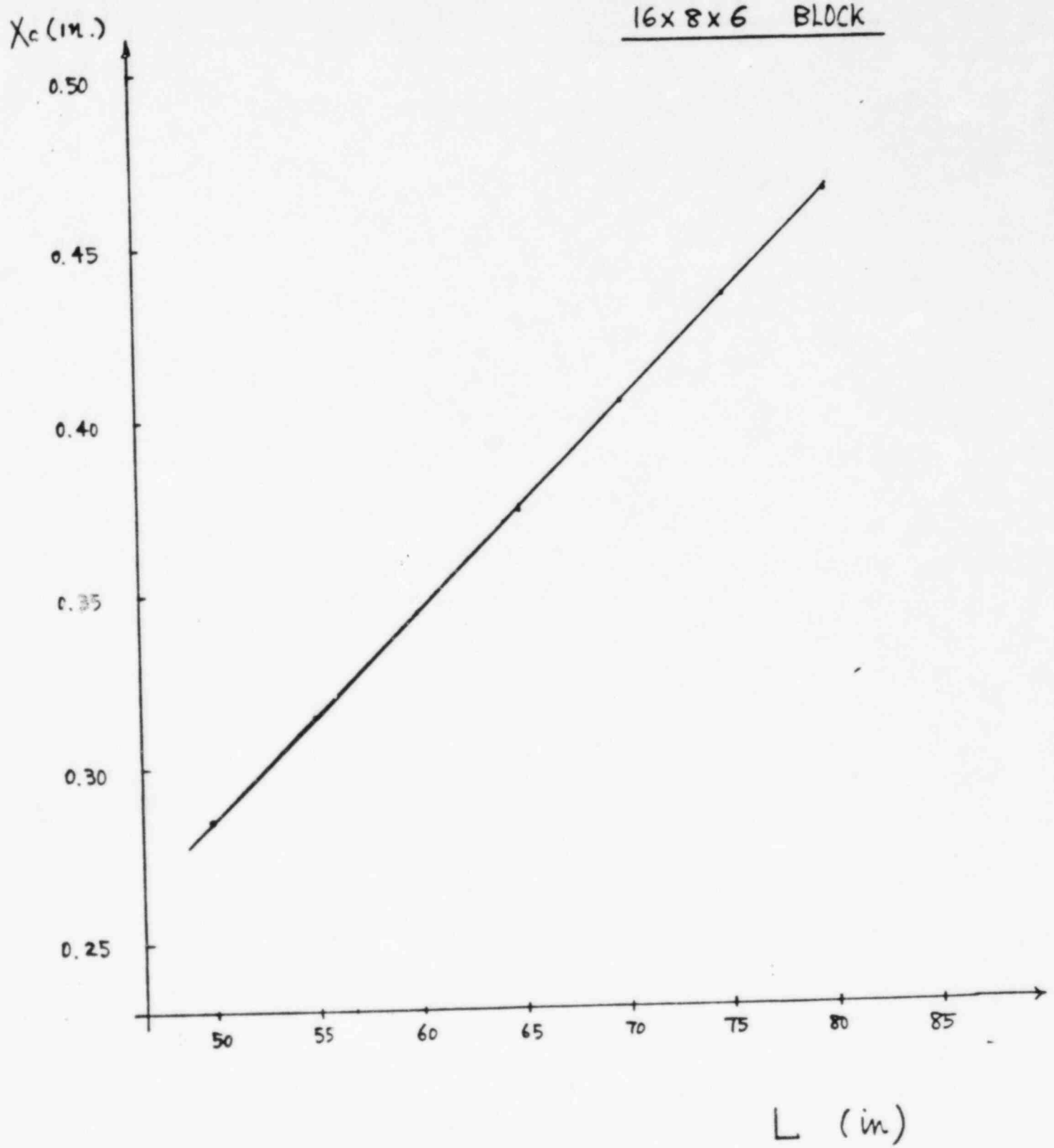


FIG. A

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SHEET NO. 5a
JOB NO. 8040-03 JOB CY BLOCK WALL BY LC DATE 10/24/80
CLIENT NUSCO SUBJECT ARCH THEORY FOR WALL WITH GAP CHK'D JSM DATE 10/24/80

16 x 8 x 8 BLOCK

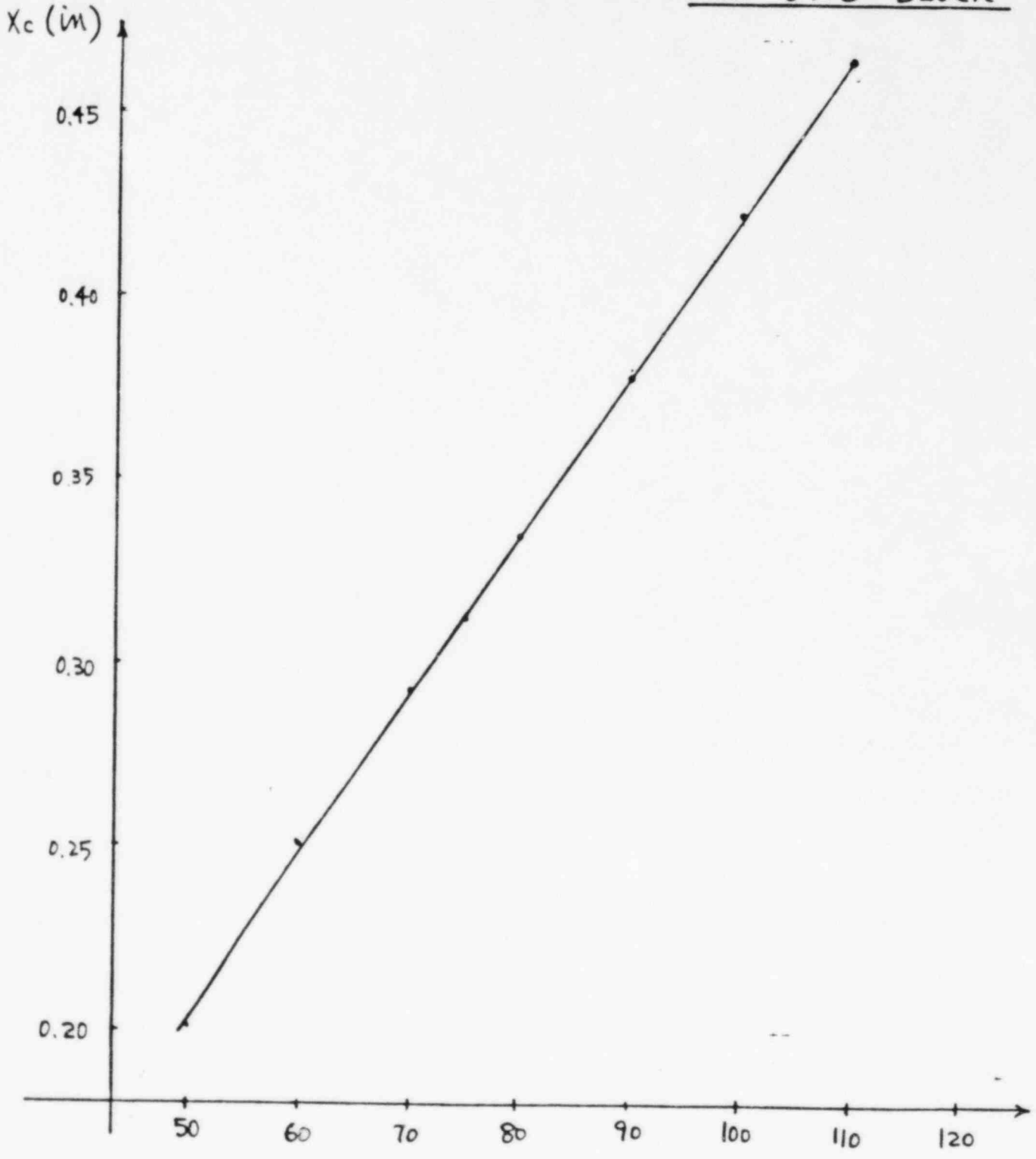


FIG. B

URS/BLUME

00 Jessie Street (at New Montgomery)

an Francisco, California 94105

SHEET NO. 5b

OB NO. 8040-03 JOB CT BLOCK WALLS BY LC DATE 10/24/80

LIENT NUSCO SUBJECT ARCH THEORY FOR WALL WITH GAP CHKD JSH DATE 10/24/80

16x8x12 BLOCK

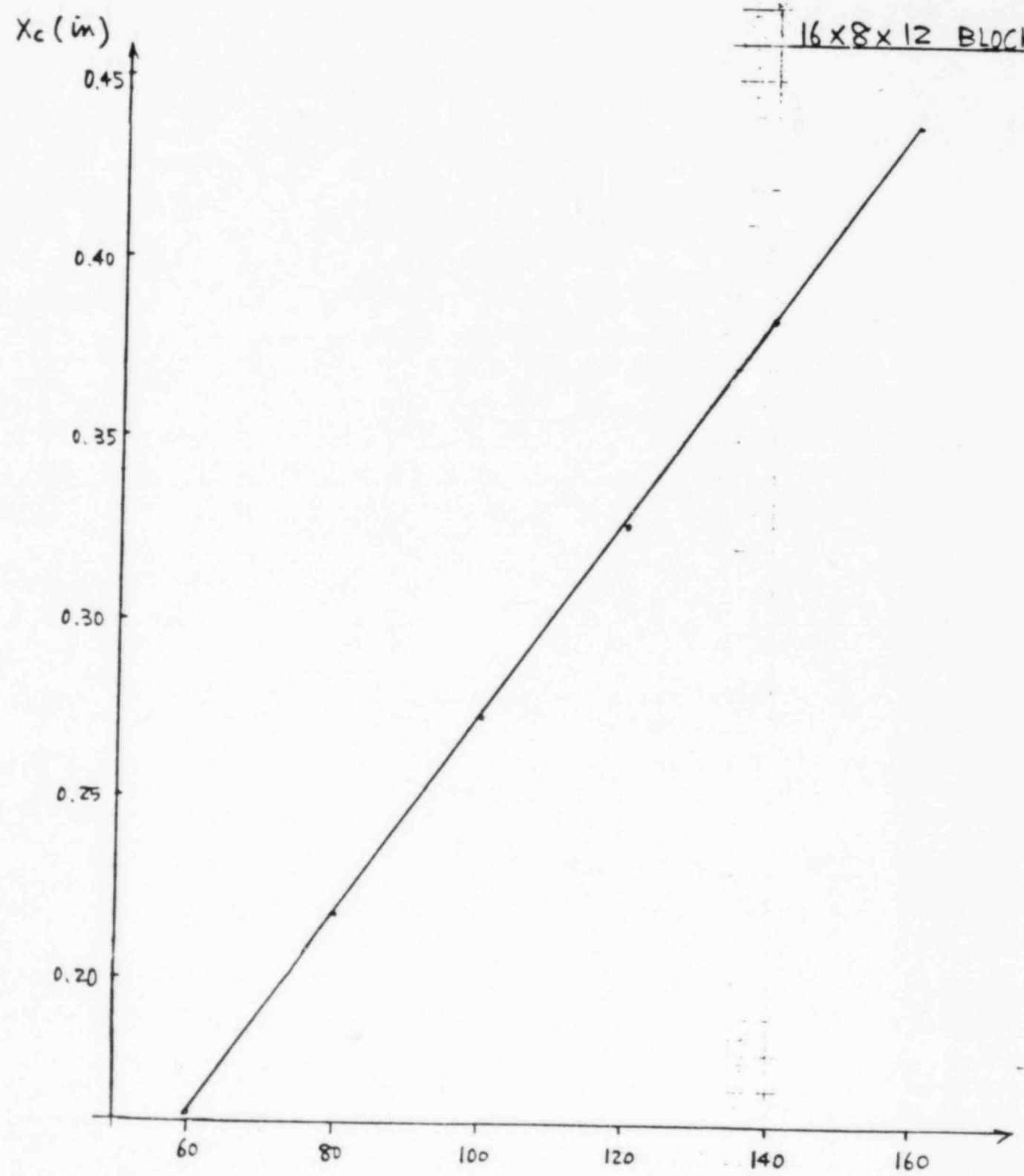
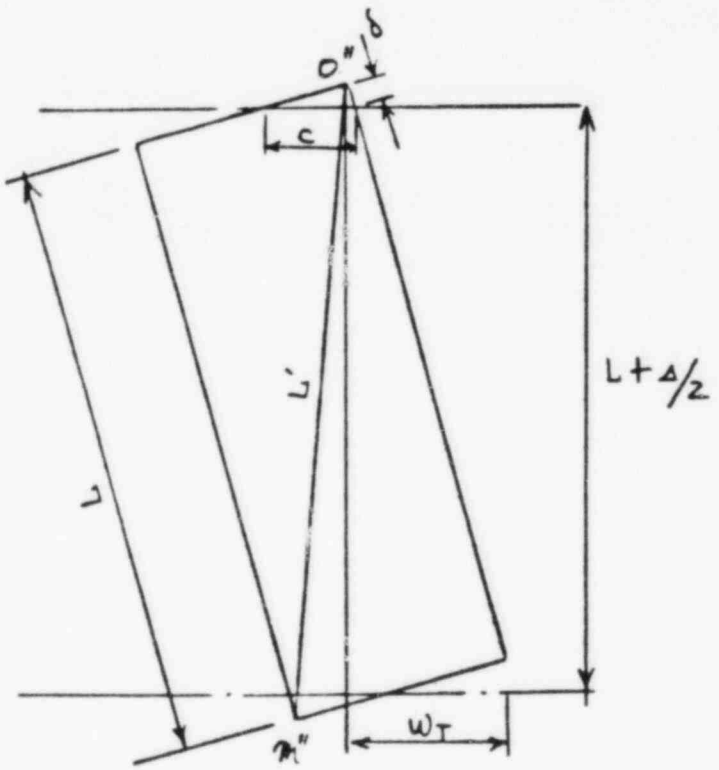


FIG C

AFTER CONTACT IS CREATED

1. Stress is developed in proportional to ϵ
2. The section rotate as rigid block.
3. The deformation in contact area is elastic.



where $L' = \sqrt{L'+4d^2}$

w_T = total mid-span deflection

Fig. 2a

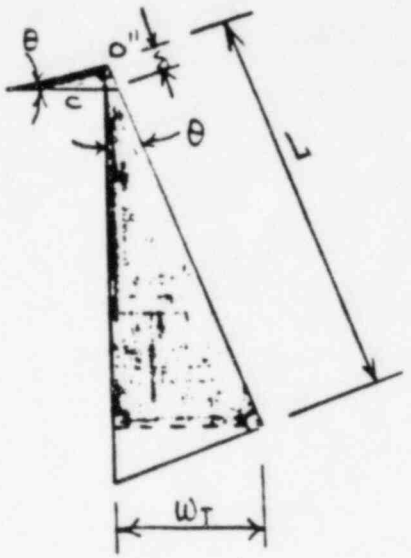


Fig 2b

In Fig 2b,

$$\sin\theta = \frac{\delta}{c} \quad , \quad \sin\theta = \frac{W_T}{L}$$

$$\frac{W_T}{L} = \frac{\delta}{c} \qquad \text{EQN a}$$

According to McDowell , $\epsilon = 2\epsilon_{avg} = \frac{2\delta}{L}$

$$\epsilon_m = \frac{\sigma_m}{E} = \frac{2\delta}{L}$$

$$\delta = \frac{\sigma_m L}{2E} \qquad \text{EQN b}$$

Put EQN. b into EQN. a

$$\frac{W_T}{L} = \frac{\sigma_m L}{2EC}$$

$$\boxed{C = \frac{\sigma_m L^2}{2E W_T}} \qquad \text{EQN. 3}$$

Consider a position where the diagonal, $\bar{L}'$, is perpendicular to support. In this position, total mid-span displacement will be 2d.

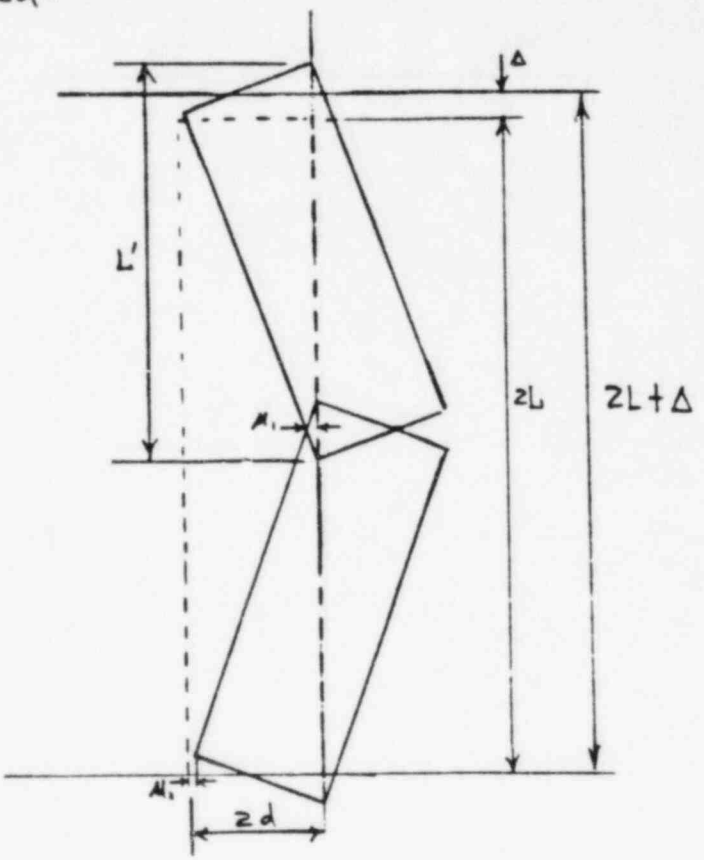


Fig. 3

Assumptions:

1. Effect of M_1 & M_2 is neglected due to small angle change and ratio of height to width of wall
2. The position shown in Fig. 3 is chosen because of the simplicity of geometry.
3. The position in Fig. 3 is used as reference position. In the reality, the stress may go beyond elastic range. However, we will assume it is still in the elastic range.
4. Due to small angle change, any strain perdendicular to support is same as strain parallel to the wall section

In Fig. 3

total shortening, ξ, in one segment will be

$$\xi_0 = L' - (2L+\Delta)/2$$

Since

$$\epsilon = \frac{2\xi}{L'}$$

$$\epsilon_0 = \frac{2L'-2L-\Delta}{L'}$$

Now, we assume there is a linear relationship between strain ϵ and midspan displacement w. For a wall without gap the following relationship will hold.

$$\frac{w}{w_0} = \frac{\epsilon}{\epsilon_0}$$

where w & ϵ are midspan displacement and strain respectively at any particular configuration. But ϵ should be below elastic limit

For wall with gap, the initial displacement X_c will not cause any strain to develop Therefore,

$$\frac{w-X_c}{w_0-X_c} = \frac{\epsilon}{\epsilon_0}$$

Since $E\epsilon = \sigma$, $\epsilon_0 = \frac{2L'-2L-\Delta}{L'}$ & $w_0 = 2d$,

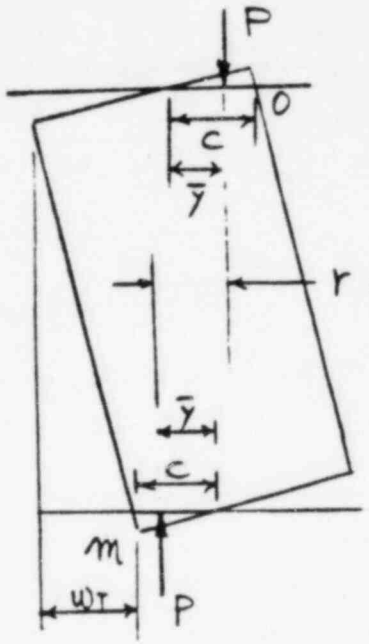
and we multiply the right hand side of above equation by $\frac{E}{E}$.

$$\frac{w_T - x_c}{2d - x_c} = \frac{\sigma}{E\left(\frac{2L' - 2L - \Delta}{L'}\right)}$$

$$\boxed{w_T = \frac{(2d - x_c)L'\sigma}{E(2L' - 2L - \Delta)} + x_c} \qquad \text{EQN 4}$$

where $\boxed{L' = \sqrt{4d^2 + L^2}}$ EQN 4A

w_T = total displacement including x_c



$\bar{y}$ = centroid of the force P
c = contact surface

$$r = 2d - W_T - 2(c - \bar{y})$$

Fig. 4

For hollow block, the compressive area is limited by the thickness of the hollow block flange.

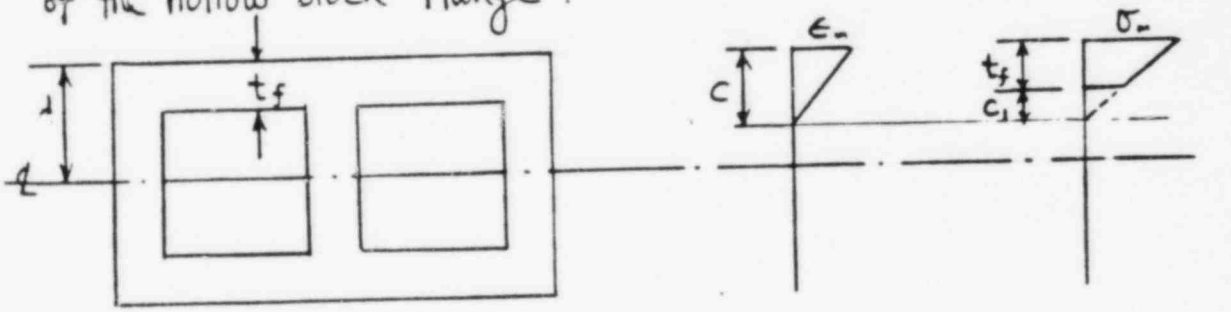


Fig. 5

In Fig 5

$$r = 2d - W_T - 2(c - \bar{y})$$

$$M = Pr$$

where $$P = \int_{c_1}^{c} \sigma_m \frac{y}{c}\, dA$$

c, c_1 refer to Fig. 5. For solid block $c_1 = 0$

$$P = \sigma_m \int_{c_1}^{c} \frac{y}{c}\, dA$$

define $$k_1 = \frac{\int_{c_1}^{c} y\, dA}{c}$$

$\bar{y}$ = distance between one end of stress block to centroid of the force, P. See Fig. 4.

$$= \frac{\int_{c_1}^{c} y^2\, dA}{\int_{c_1}^{c} y\, dA}$$

Therefore

$$M = \sigma_m k_1 [2d - W_T - 2(c - \bar{y})]$$

$$\boxed{M = \sigma_m k_1 [2(d-c) + 2\bar{y} - W_T]} \qquad \text{EQN 5}$$

where

$$K_1 = \frac{\int_{c_1}^{c} y\, dA}{C} \qquad \text{EQN 5A}$$

$$\bar{y} = \frac{\int_{c_1}^{c} y^2\, dA}{\int_{c_1}^{c} y\, dA} \qquad \text{EQN 5B}$$

For solid block or stress block fall within flange thickness

$$M = \frac{1}{2} \sigma_m C \left(2d - W_T - \frac{2}{3} C \right) \qquad \text{EQN 6}$$

References:

1. E.L. McDowell, K.E. McKee, and E. Sevin, "Arching Action Theory of Masonry Walls", ASCE Journal Structural Division, March 1956.

2. Edward Cohen and Edward Lairig, "Discussion of Arching Action Theory of Masonry Walls" ASCE Journal Structural Division, September 1956

PROCEDURE FOR ANALYZING WALL WITH GAP

DETERMINATION OF MOMENT CAPACITY

1. Use gap size equal to $1/16$". Determine L by $L = \frac{H-\Delta}{2}$.

2. Determine midspan displacement, X_c, by using the following equations or from Figs A, B, or C on p. 5, 5a, 5b respectively.

Block Size	EQUATION
16 x 8 x 6	$X_c = 5.625 - \sqrt{31.6396 - 0.0625L}$
16 x 8 x 8	$X_c = 7.625 - \sqrt{58.1396 - 0.0625L}$
16 x 8 x 12	$X_c = 11.625 - \sqrt{135.1396 - 0.0625L}$

Note X_c from either procedure should be less than 0.5". If X_c is greater than 0.5", the wall is not qualify for arch analysis

3. Use maximum allowable elastic stress, σ_m, and EQN 4 and 4A to calculate w_T.

$$w_T = \frac{(2d - X_c)L'\sigma_m}{E(2L' - 2L - \Delta)} + X_c \quad \text{------ EQN 4}$$

$$L' = \sqrt{4d^2 + L^2} \quad \text{------ EQN 4A}$$

If w_T is less than 0.5", skip step 3a

3a. Substitute 0.5 for w_T in EQN 4 and solve for σ_m

4. Use σ_m and w_T from step 2 or 2a to calculate C by Equation 3.

$$C = \frac{\sigma_m L^2}{2E w_T} \quad \text{------ EQN 3}$$

5. Use EQN 5, 5A and 5B for hollow block, or EQN 6 for solid block and hollow block with stress block contact surface C less than flange thickness to find M_{max}.

$$M_{max} = \sigma_m K_1 [2(d-C) + 2\bar{y} - W_T] \quad \text{EQN 5}$$

where $$K_1 = \frac{\int_{C_1}^{C} y\, dA}{C} \quad \text{EQN 5A}$$

$$\bar{y} = \frac{\int_{C_1}^{C} y^2\, dA}{\int_{C_1}^{C} y\, dA} \quad \text{EQN 5B}$$

$$M_{max} = \frac{1}{2}\sigma_m C \left[2d - W_T - \frac{2}{3}C\right] \quad \text{EQN 6}$$

6. By using the following equation to find maximum acceleration, Sa, which the wall can withstand.

$$Sa = \frac{8Mg}{1.3(2d)W(2L)^2} \quad *$$

where W = $\frac{\text{weight of block}}{\text{overall volume of the block}}$

* Derivation as the following

$$M = \frac{W(2L)^2}{8}$$

substitute $W = 1.3(2d)W$ into above equation and rearrange.

EXAMPLE (According to Procedure Listed on P. 14 & 15)

Data

$2L+\Delta = 88"$
$2d = 7.625"$ $\quad d = 3.8125"$
$\sigma_{max} = 386$ psi $\quad E = 4.2\times10^5$ psi
$W = 0.0434$ lb/in^3 $\quad t_f = 1.5"$

Step 1

Use $\Delta = 1/16" = 0.0625"$

$L = (88 - 1/16)/2 = 43.9688"$

Step 2

for $16\times8\times8$ block

$$X_c = 7.625 - \sqrt{58.1396 - 0.0625(43.9688)}$$
$$= 0.1825" < 0.5" \quad \text{O.K.}$$

Step 3

$$L' = \sqrt{(7.625)^2 + (43.9688)^2} = 44.6251"$$

$$W_T = \frac{(7.625-0.1825)(44.6251)(386)}{4.2\times10^5\,(2(44.6251)-2(43.9688)-0.0625)} + 0.1825$$

$$= 0.2442 + 0.1825 = 0.4267"$$

Step 4

$$C = \frac{386\,(43.9688)^2}{2\,(4.2\times10^5)(0.4267)} = 2.0820$$

Step 5

$t_f = 1.5 < 2.0820$

$c_1 = 2.0820 - 1.5 = 0.5820''$

$$K_1 = \frac{\int_{0.5820}^{2.0820} y\,dy}{2.0820} = 0.9597'' \quad \text{Assume 1" strip}$$

$$\bar{y} = \frac{\int_{0.5820}^{2.0820} y^2\,dy}{\int_{0.5820}^{2.0820} y\,dy} = 1.4728'' \quad \text{Assume 1" strip}$$

$$M_{max} = 386\,(0.9597)\,[2(3.8125 - 2.082) + 2(1.4728) - 0.4267]$$

$$= 2215 \text{ "-lb}$$

Step 6

$$S_a = \frac{8\,(2215)\,g}{1.3\,(7.625)\,(0.0434)\,(2 \times 43.9688)^2} = 5.3\,g$$

PROCEDURE II CALCULATE STRESS FOR A GIVEN MOMENT

1. Determine M_{max} per 1" strip

2. Calculate initial midspan displacement, X_c.

$$X_c = 2d - \sqrt{4d^2 - \left(L\Delta + \frac{\Delta^2}{4}\right)}$$

3. Calculate L'.

$$L' = \sqrt{4d^2 + L^2}$$

4. Calculate α.

$$\alpha = \frac{(2d - X_c)L'}{E(2L' - 2L - \Delta)}$$

5. Assume a value of σ. For first trial, use the value from the equation below.

$$\sigma = \frac{M_{max}}{S}$$

where S is the section modulus of block per 1" strip

6. Determine maximum midspan deflection, W_T.

$$W_T = \alpha \sigma_m + X_c$$

7. Determine contact surface, C.

$$C = \frac{\sigma_m L^2}{2E W_T}$$

8. If C is greater than flange thickness, go to step 8a. If C is less than or equal to flange thickness, go to step 8b.

8a. Determine M_{max} by the following equations.

$$M_{max} = \sigma_m K_1 [2(d-c) + 2\bar{y} - w_T]$$

$$K_1 = \frac{\int_{c_1}^{c} y\, dA}{C} \qquad \text{per 1" strip}$$

$$\bar{y} = \frac{\int_{c_1}^{c} y^2 dA}{\int_{c_1}^{c} y\, dA} \qquad \text{per 1" strip}$$

8b. Determine M_{max} by the following equation.

$$M'_{max} = \frac{1}{2}\sigma_m C \left[2d - w_T - \frac{2}{3}C\right]$$

9. If M'_{max} calculated from step 8a or 8b is same as M_{max} from step 1, values of σ_m, w_T are found. Otherwise, go to step 10.

10. For 2nd trial, go to step 10a. For 3rd trial and beyond, go to step 10b.

10a. Assume σ_m by

$$\sigma_{m\ 2^{nd}\ trial} = \sigma_{m\ 1^{st}\ trial} \frac{M_{max}}{M'_{max\ from\ trial\ 1}}$$

10b Assume a linear relationship between σ and M. Use values of σ_m and M_{max} from previous trials and interpolation method to calculate new σ_m for next trial.

11 Go to step 6.

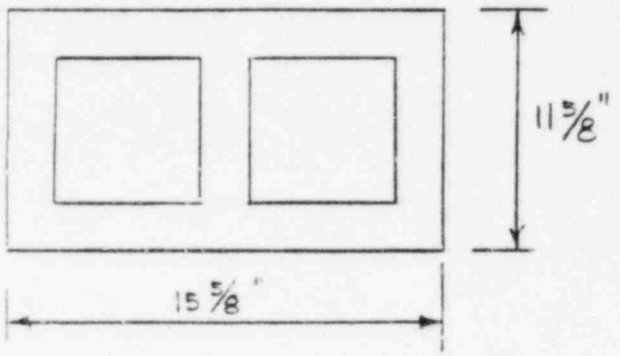
ATTACHMENT 2

consists of a page (sheets 188-189, CYPAB101) example of block pullout calculations.

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SHEET NO. 188
JOB NO. 8340-53 JOB CY BLOCK WALLS BY LC DATE 1/7/81
CLIENT NRCC SUBJECT CYFAE101 CHK'D BAS DATE 1-15-81

BLOCK PROPERTY



$$\text{Weight} = (62 - 3) \times \frac{150}{135} + 3 = 69\ \#/\square'$$

(3 = mortar)

↑ from Reinforced Masonry Engineering Handbook

$$I = 2\left[\frac{1}{12}(12)(1.25)^3\right] = 3.9$$

$$2[(12)(1.25)]\left[(11.625 - 1.25)/2\right]^2 = 807.3$$

$$811.2\ \text{in}^4 \text{ per } 12\text{" strip}$$

$$S = \frac{811.2}{\frac{1}{2} \times 11.625} = 139.6\ \text{in}^3 \quad \text{per } 12\text{" strip}$$

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SHEET NO. 189
JOB NO. 204-53 JOB CY BLOCK WALLS MODFICATION BY LC DATE 1/7/81
CLIENT NUSCO SUBJECT CYPAB101 CHK'D BAS DATE 1-15-81

Check Attachment Weight

Wall A: 330 lb / (192 × 124) (approx area of wall) = 0.014 psi
↑ P.11 of CYPAB101A evalution calc

Wall B: No major attachment weight

Wall C: 239 lb / [(22×12+8) × (12×10+6)] = 0.007 psi
↑ P.13 of CYPAE101E evaluation calc

Wall D: 1160 lb / (156 × 108) = 0.069 psi
↑ P.11 of CYPAE101D evaluation calc

Wall E: No major attachment weight

Wall F: (2367+400) / (174 × 156) = 0.10 psi
↑ P.11 of CYPAE101F evaluation calc

Use 0.10 psi for attachment weight on walls D & F.

Use 0.02 psi for attachment weight on walls A, B & E

By inspection, no problem with attachment pullout
Large duct/pipe penetration will be free of contact with wall

ATTACHMENT 3

consists of a 12-page (sheets 2-10) example of a masonry wall redesign such that failure will not result in damage to safety related equipment.

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SHEET NO. 2

JOB NO. 8040-53 JOB CONNECTICUT YANKEE BY PMN DATE 9-10-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D BAS DATE 9-19-80

WALLS # CYPAE 102 & 103 @ EL. 21'-6"

$\ell = 5'\text{-}6"$ $h = 4'\text{-}8"$ $t_w = 16"$

- SOLID SHIELD BLOCKS (BRICKS)
  ASSUME WT = 145 #/cft
- BOTTOM 4 COURSES MORTARED
- TOP 10 COURSES STACKED ON TOP OF BOTTOM COURSES

w = WT OF 16" BLOCK WALL = $145^{pcf} \times 1.33^{ft} = 193^{\#/sft}$

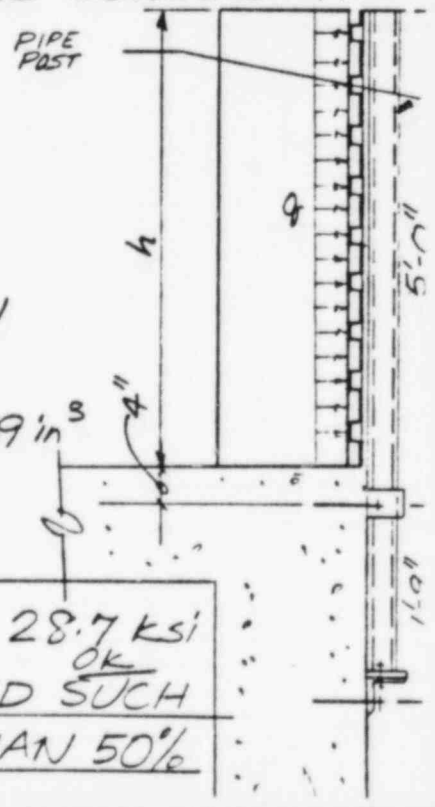
MAX. LATERAL FORCE (per criteria) $W_o = 1.3\, S_a \times w$

SINCE PEAK OF SPECTRUM $S_{a\,MAX.} = 0.42g$; USE $W_o = 1.3 \times 0.42 \times 193^{psf} = \underline{105^{psf}}$ AS A CONSERVATIVE VALUE.

USE 2 PIPE POSTS TO SUPPORT WALL NORMAL TO ITS PLANE.

FORCE ON EACH POST $q = W_o \times \ell/2$
$= 105 \times 5.67'/2 = 298^{\#/ft}$

NEGLECT FRICTIONAL RESISTANCE FOR CONSERVATIVE DESIGN OF SUPPORT SYSTEM.



DESIGN OF POSTS:

$q = 298^{\#/ft}$

$Q = qh = 298 \times 4.67 = 1392^{\#}$

$\therefore M_{max} = 1392\left(\frac{4.67' \times 12}{2} + 4"\right) = 44.55^{K\text{-}IN}$

ASSUME 3½ϕ STD PIPE W/ $S = 2.39\ in^3$

$\therefore f_b = \frac{M}{S} = \frac{44.55}{2.39} = 18.64\ ksi$

ALLOWABLE $F_b = 0.6 F_y \times 1.33 = 28.7\ ksi$ OK

$\therefore$ POST IS OK EVEN IF THEY ARE SPACED SUCH THAT ONE POST CARRIES MORE THAN 50% TOTAL LOAD

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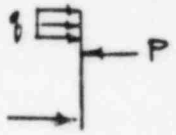
SHEET NO. 3

JOB NO. 2040-53 JOB CONNECTICUT YANKEE BY AMN DATE 9-10-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D BAS DATE 9-19-80

DESIGN OF STRAP PLATE ASSEMBLY:

MAX FORCE IN STRAP PL, $P = \frac{\text{END-MOMENT}}{\text{LEVER ARM OF RESISTING COUPL}}$



$= \frac{44.55}{1.75' \times 12''/ft} = 2.12^{k}$

$\therefore$ FORCE/BOLT $= 2.12/2 = \underline{1.06^{k}}$

FROM DESIGN CRITERIA ON HILTI KWIK-BOLT ULTIMATE VALUES (Ref. GCE-80-588 d/Oct 6, 1980), FOR 3000 psi CONCRETE & 3/4φX7"LG HILTI KWIK-BOLT,

ULTIMATE TENSILE STRENGTH = 18,400#

ALLOWABLE TENSILE STRENGTH, $f_t = \frac{18400}{4} = 4,600^{\#}$

(where 4 is the min. factor of safety)

$\therefore f_t = 4,600^{\#}$

$\therefore 4.60^{k} > 1.06^{k}$, 3/4φX7" KWIK BOLTS OK

BOLT SPACING = 10X0.75 = 7.5 in

or 2 x Req'd embedment depth.

FOR EMBEDMENT DEPTH OF 3¼", $f_t = \frac{9437}{4} = 2.36^{k} > 1.06^{k}$ OK

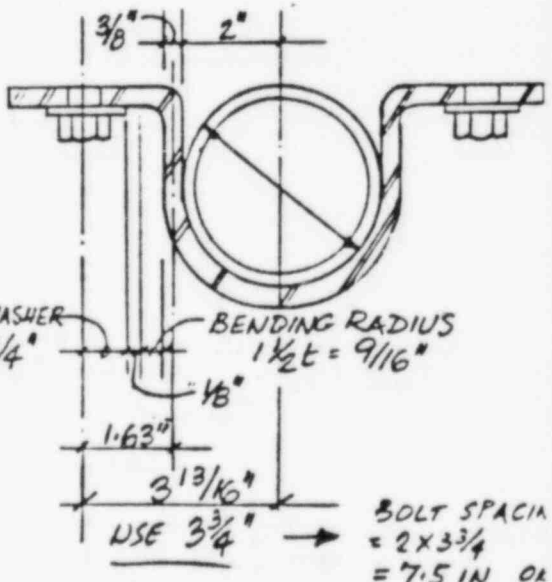
$\therefore$ 2 x 3¼ = 6.5 in

$\therefore$ BOLT SPACING = 7.5" PROVIDED.

SINCE ONLY TENSION & NO SHEAR IS PRESENT INTERACTION STRESSES OK BY OBSERVATION

MAX MOMENT IN STRAP PLATE

$= 1.06^{k} \times 1.63^{in} = 1.73^{k\text{-in}}$



ASSUME STRAP PL 3/8 X 3

$S_{STRAP} = \frac{3 \times (0.375)^2}{6} = 0.0703\ in^3$

$\therefore f_b = \frac{1.73}{0.0703} = 24.6\ ksi$

F_b allowable $= 0.75 F_y \times 1.33 < 0.9 F_y$

$= 0.9 \times 36\ ksi = 32.4\ ^{k}/_{in}$

STRAP PL 3/8 X 3 W/2-3/4φX7" KWIK-BOLTS

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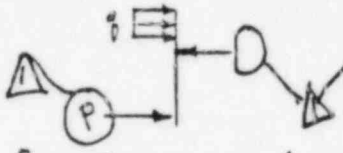
SHEET NO. 3A

JOB NO. 8040-52 JOB CONNECTICUT YANKEE BY PMN DATE 9-10-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D BAS DATE 9-19-80

DESIGN OF STRAP PLATE ASSEMBLY:

MAX FORCE IN BOTTOM ANGLE, P = $\frac{\text{END-MOMENT}}{\text{LEVER ARM OF RESISTING COUPLE}}$



$= \frac{44.55}{1.75' \times 12''/ft} = 2.12^k$

∴ FORCE/BOLT = 2.12/2 = **1.06^k**

FROM DESIGN CRITERIA ON HILTI KWIK-BOLT ULTIMATE VALUES (Ref. GCE-80-588 d/Oct 6, 1980), FOR 3000 psi CONCRETE & 3/4φ X 7" LG HILTI KWIK-BOLT

ULTIMATE TENSILE STRENGTH = 9,437#

ALLOWABLE TENSILE STRENGTH, $f_t = \frac{9,437\#}{4} = 2,36^k$

(where 4 is the min. factor of safety)

∴ $f_t = 4,600\#$

∴ 2.36^k > 1.06^k, **3/4φ X 7" KWIK BOLTS OK**

BOLT SPACING = 10 X 0.75 = 7.5 in

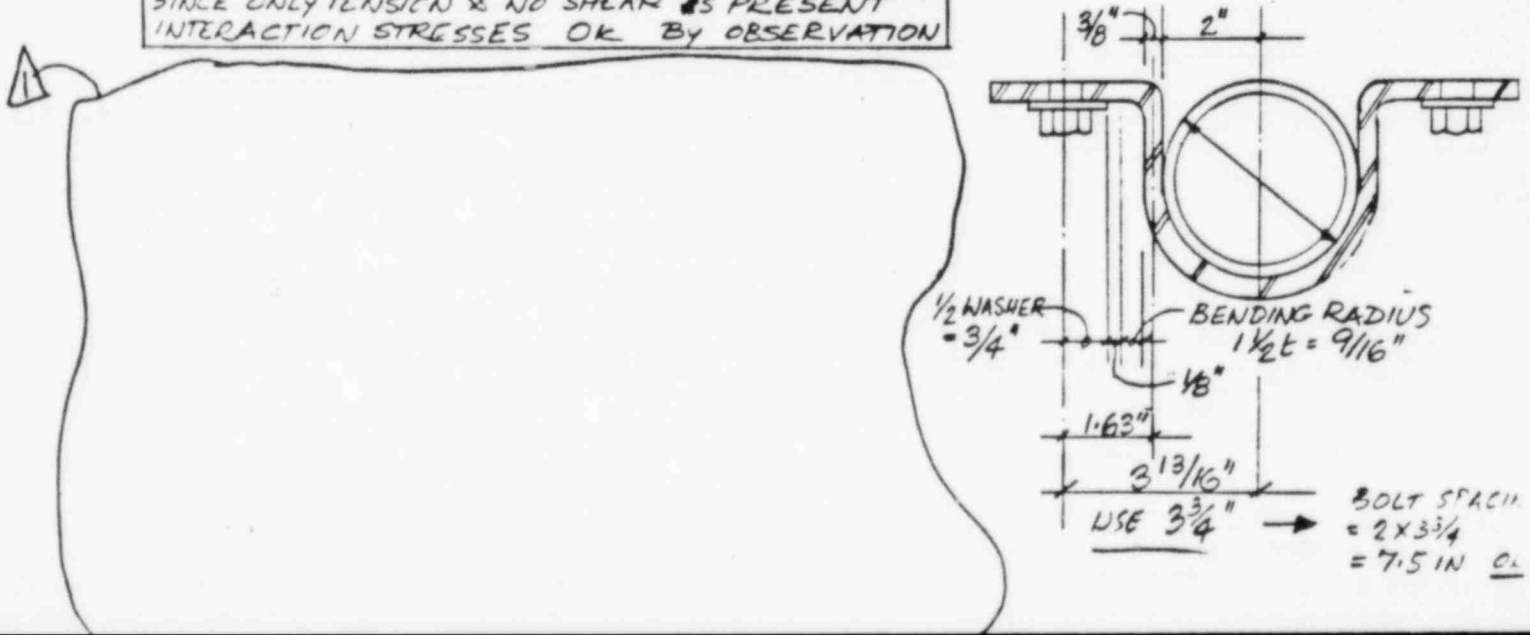
OR 2 X Req'd embedment depth.

FOR EMBEDMENT DEPTH OF 3¼", $f_t = \frac{9437}{4} = 2.36^k > 1.06^k$ OK

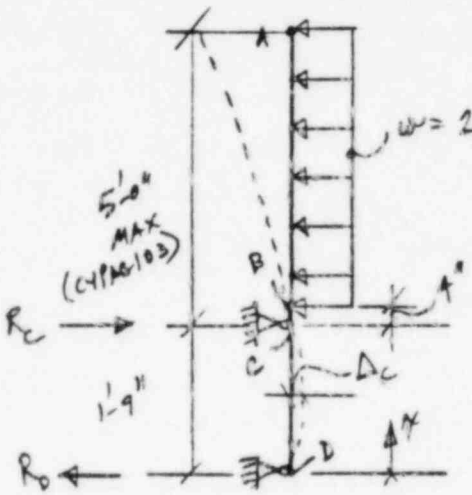
∴ 2 X 3¼ = 6.5 in

∴ BOLT SPACING = 7.5" PROVIDED.

SINCE ONLY TENSION & NO SHEAR IS PRESENT INTERACTION STRESSES OK BY OBSERVATION



A. Check deflection of 3½" ϕ standard pipe stiffeners in order to size the plate thickness to be added at support locations to concrete wall



ELEV. OF PIPE MODEL

$w = 298^{\#}/ft$ (pg. 2 of calcs.) — NOTE: WT. OF METAL DECK + PIPE IS NOT INCLUDED SINCE IT IS SMALL COMPARED TO w.

For 3½" STD. PIPE (see pg. 1-102 AISC)

O.D = 4"-0" $\quad I = 4.79\ in^4$
wt/ft = 9.11 $\quad S = 2.39\ in^3$
$A = 2.68\ in^2$

For deflection, assume w extends also from B to C (conservative assumption). Then Δ_C can be calculated as per case 25, pg. 2-206 AISC, where

$l = 21" \qquad a = 60"$

$$\Delta_C)_{max} @ x = \frac{l}{\sqrt{3}} = .03208\ \frac{wa^2l^2}{EI} = \frac{.298^{K}/_{ft} \times \frac{ft}{12"} \times (60")^2 (21")^2}{(29 \times 10^3\ ksi)(4.79\ in^4)} = .283"$$

Therefore, use a ½" thick plate between the two supports at C & D on the pipe and the concrete wall.

B. Check strap at C for reaction load R_C.

$$\Sigma M_D = 0 \Rightarrow R_C(21") = \underbrace{(.298^{K}/_{1})(4.67')}_{W = 1.392^{K}} \times \underbrace{(21" + 4" + \frac{56"}{2})}_{\bar{H} = 53"}$$

$$R_C = \frac{1.392^{K} \times 53"}{21"} = 3.51^{K} = R_C$$

Moment in strap (see pg. 3A of calcs.) = $M = \frac{3.51^{K}}{2} \times 1.63" = 2.86^{K\text{-}"}$

$M/S < F_b$ $\quad F_b = 1.67 \times .75 f_y \leq f_y$ $\quad \therefore F_b = 36$ ksi
($1.25 f_y$)

$S_{req} = M/F_b = \frac{2.86 \text{ k'}}{36 \text{ ksi}} = .079 \text{ in}^3$

$S_{strap} = \frac{bt^2}{6}$ assume $b = 4"$ then $.079 = \frac{4}{6} t^2 = .67 t^2$

$t^2 = \frac{.079}{.67} = .118$ $\quad t = .343$ $\quad \therefore$ use $t = 1/2" > .343"$

$\therefore$ use 4" x 1/2" strap.

c. Check HILTI-KWIK bolts for reaction load R_c.

For 3/4" bolts with 3 1/4" embedment, $T_{ALL} = \frac{9437\#}{4} = 2,359\#$

For 2 bolts, $T_{ALL} = 2 \times 2.36^k = 4.72^k$ (2 bolts per strap)

Shear load on bolt due to weight of decking + pipe + vertical seismic response load is negligible since bottom seat & support at D will take these loads (where no bolt tension is taken at D).

$\therefore \frac{R_c}{T_{ALL}} = \frac{3.51^k}{4.72^k} = .74 < 1.0$ OK.

$\therefore$ use 2-3/4"ϕ HILTI-KWIK BOLTS
W/ 3 1/4" EMBEDMENT FOR STRAPS

CHECK METAL DECK :

METAL DECK SPANS BETWEEN THE POSTS. ASSUMING MAX. SPACING OF 5'-8" AND PROVIDING ROBERTSON SEC.3 GAGE 20 METAL DECK

TOTAL ALLOWABLE UDL ON DECK = $\frac{3530^{*}}{(5.67)^2} \cdot 109$ psf

MAX LATERAL FORCE ON DECK = 105 psf < 109 psf Ok
(SEE SHT. 2)

∴ ROBERTSON SEC.3 GAGE 20 METAL DECK IS OK

OVERHANGS IN METAL DECK:
IN CASE OF ANY FIELD INTERFERENCE, THE PIPE POST LOCATIONS COULD BE ADJUSTED, LEAVING OVERHANGS BEYOND THE POSTS. ASSUME MAX OVERHANG = 1'-0"

$$M_{MAX}^{-VE} = \frac{105^{psf} \times (1.0)^2}{2} \times 12 = 0.63^{k\cdot in}$$

S_x^{-VE} FOR ROBERTSON SEC.3 GAGE 20 DECK = $0.273^{*} in^3$

$$\therefore f_b = \frac{0.63}{0.273} = 2.31 \text{ ksi} < 20 \text{ ksi}$$ Ok

NOTE : * THESE VALUES ARE OBTAINED FROM ROBERTSON Q-147TD-79(J)

NOTE : TO SOLVE THE PROBLEM OF POSSIBLE LOAD INCREASE CAUSED BY MOVEMENT OF SOLID BLOCKS WITHIN THE ½" GAP BETWEEN METAL DECKING & THE WALL, A BENT PLATE IS PROVIDED BETWEEN PIPE POST AND METAL DECK SO THAT METAL DECK RESTS SNUG AGAINST THE WALL FACE. SEE SHT. 6 FOR DETAILS

SEAT ANGLE : SEAT ANGLE RESISTS ONLY THE SELF WT OF PIPE POST WHICH IS 64# FOR A 3½ϕ x 7'-0" LONG PIPE. BY INSPECTION, NEW L4X4X3/8X0- PROVIDED IS OK.

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SHEET NO. 5

JOB NO. 9040-53 JOB CONNECTICUT YANKEE BY BAS DATE 9-16-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D PMJ DATE 9-19-80

WALL # CYPAB 102 & 103

FOR DETAILS NOT SHOWN, SEE SHT NOS. 12-16

DESIGN SUPPORT FOR WALL IN ITS OWN PLANE

A. CHECK VERTICAL BRACE (SEE SKETCH ON SHT. 12)

1. SEISMIC CRITERIA

$w_o = 0.546\ w$

2. WT. OF 16" CONCRETE BRICK WALL PER FOOT HEIGHT.

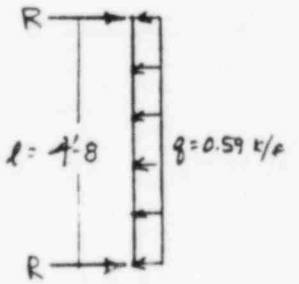
$w = 145\ pcf \times 1.33' \times 5.67'$ (wall length) $= 1.09\ k/ft$

3. LATERAL FORCE PER FOOT HEIGHT, q

$q = 0.546 \times 1.09\ k/ft = 0.59\ k/ft$

4. CHECK C8x11.5 BRACE

ASSUME SIMPLE SUPPORTS



$Q = ql = 0.59(4.67) = 2.76^k$

$R = Q/2 = 2.76^k/2 = 1.38^k$

$M_{MAX} = QL/8 = \frac{2.76^k \times 4.67' \times 12}{8} = 19.3^{k''}$

5 CHECK BENDING CAPACITY ABOUT MINOR AXIS,

$S_y = 0.78\ in^3$ $F_b = 1.33 \times 0.6 F_y = 29\ ksi$ (Seismic)

$f_b = \frac{M}{S_y} = \frac{19.3^{k''}}{0.78\ in^3} = 24.8\ ksi < 29\ ksi \longrightarrow$ OK

NOTE: C6x8.2 AND CONNECTIONS ARE SAME AS FOR WALL CYPAB 104, BUT STRESSES ARE SMALLER ∴ NO ADDITIONAL CALCS REQUIRED.

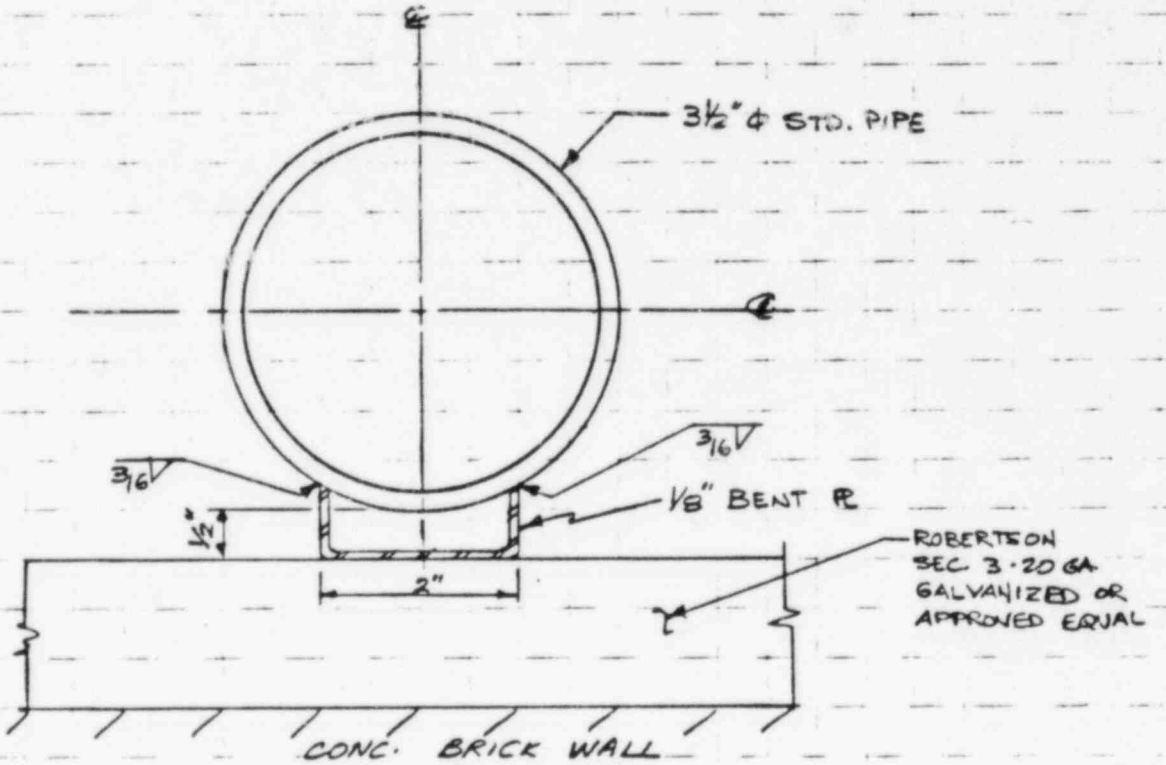
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SHEET NO. 6

JOB NO. 8240-03 JOB CONNECTICUT YANKEE BY BAS DATE 9-14-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D PMN DATE 9-19-80

AS A SOLUTION TO THE PROBLEM OF LOAD INCREASE CAUSED BY MOVEMENT WITHIN THE 1/2" GAP, PLACE A BENT PLATE BETWEEN PIPE AND METAL DECK AS SHOWN BELOW.



PLAN VIEW

SCALE: ONE-HALF FULL SIZE

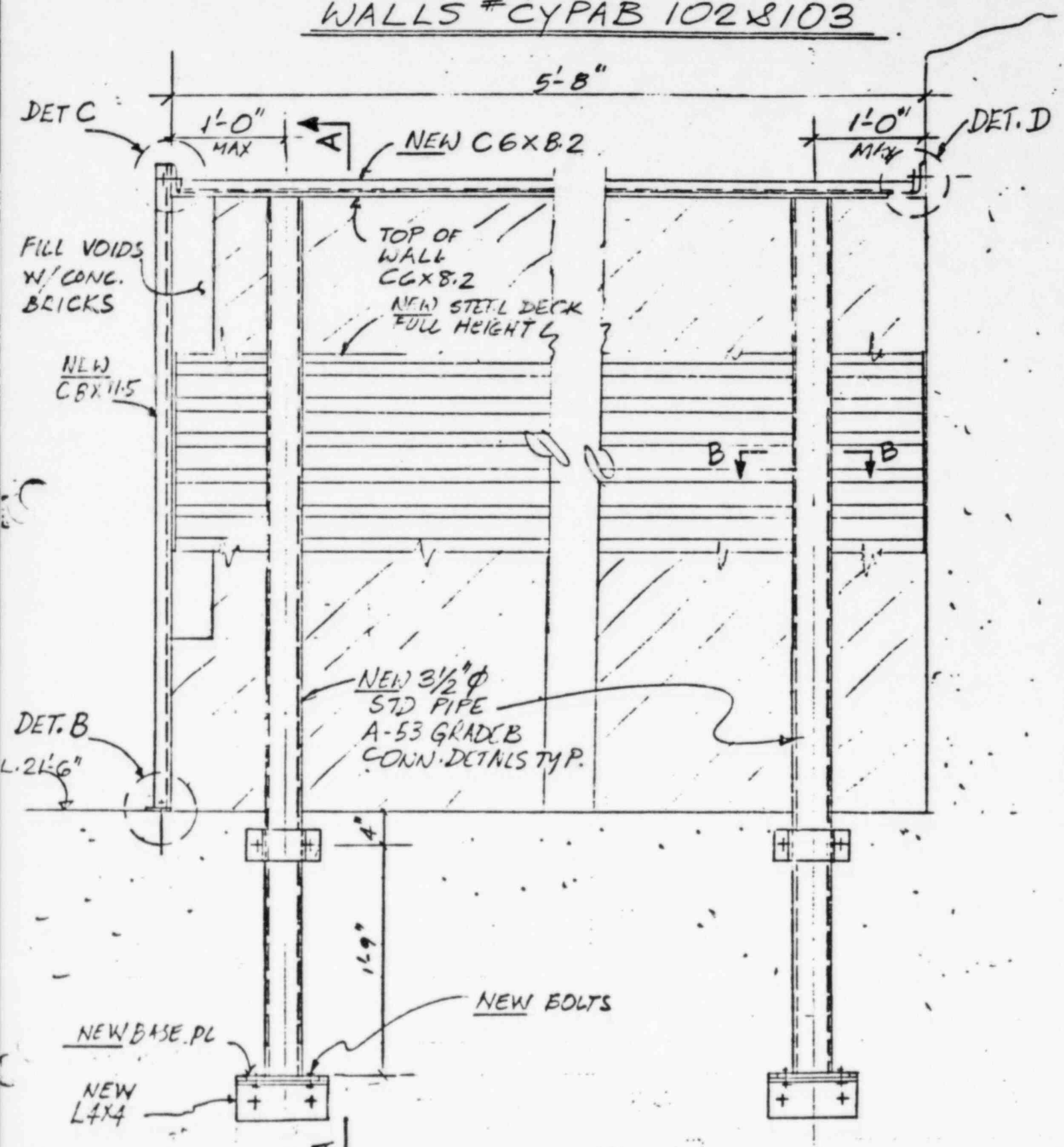
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SHEET NO. 7

JOB NO. 8044.05 JOB CONNECTICUT YANKEE BY PMN DATE 9-3-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D MH DATE 9-5-80

WALLS # CYPAB 102 & 103



ELEVATION OF NORTH FACE – CYPAB 102 AS SHOWN

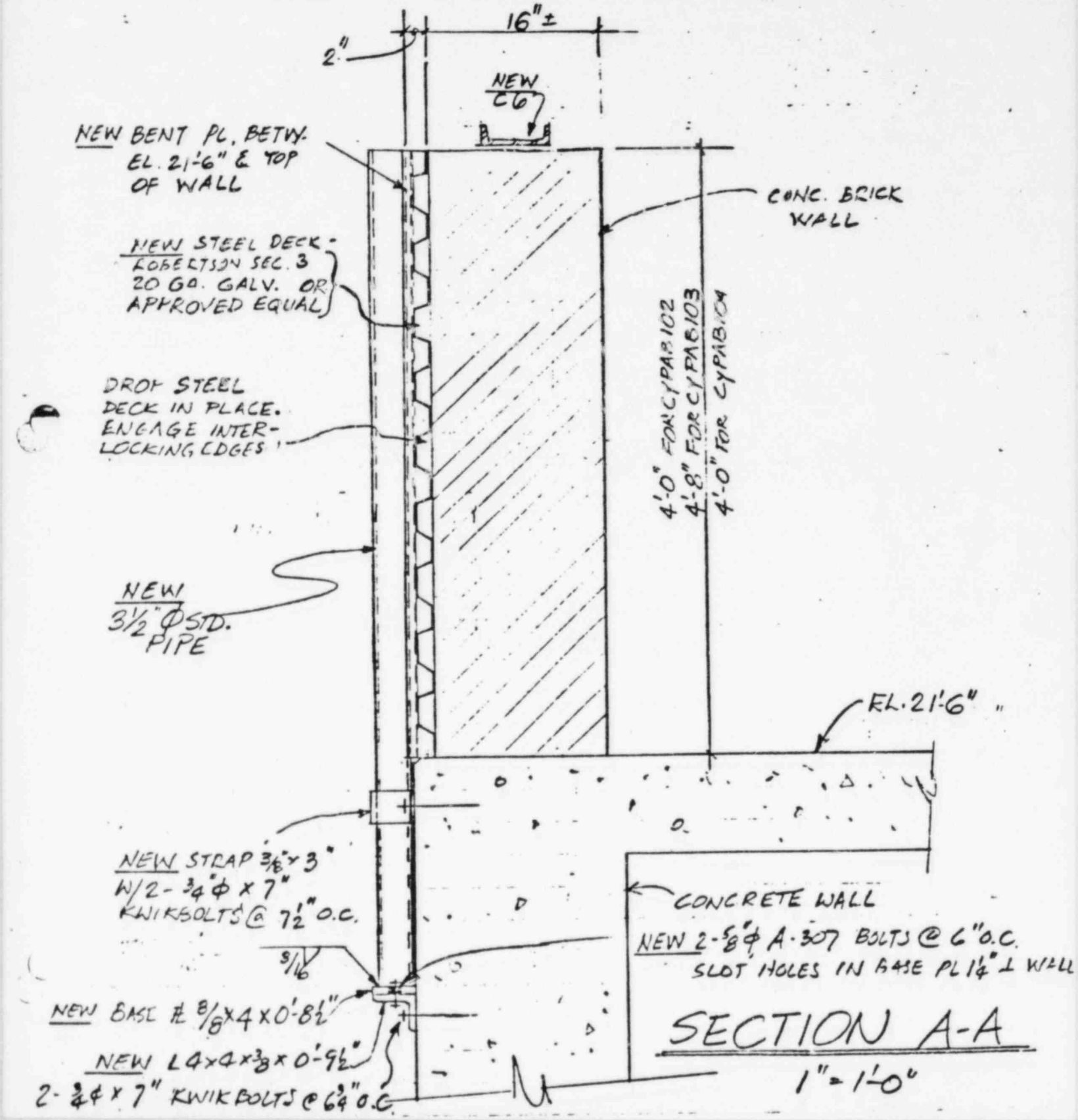
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SHEET NO. 8

JOB NO 8040-03 JOB CONNECTICUT YANKEE BY PMN DATE 9-2-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D MHL DATE 9-5-80

WALLS # CYPAB 102, 103 & 104



SECTION A-A

1" = 1'-0"

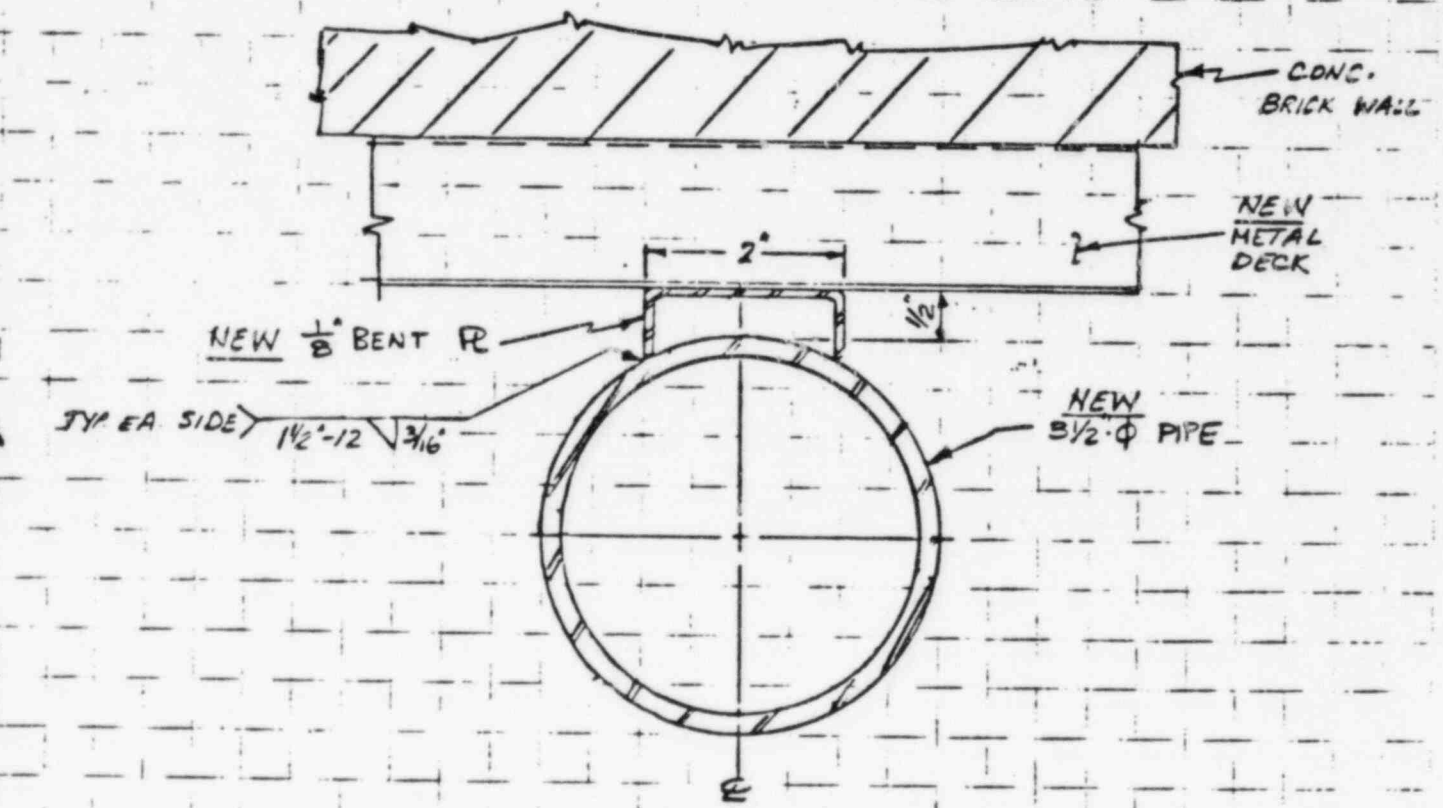
URS/BLUME
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

SHEET NO. 9

JOB NO. 6040-03 JOB CONNECTICUT YANKEE BY BAS DATE 9-14-80

CLIENT NUSCO SUBJECT BLOCK WALL MODIFICATIONS CHK'D [illegible] DATE 9-16-80

WALLS CYPAB 102, 103, & 104



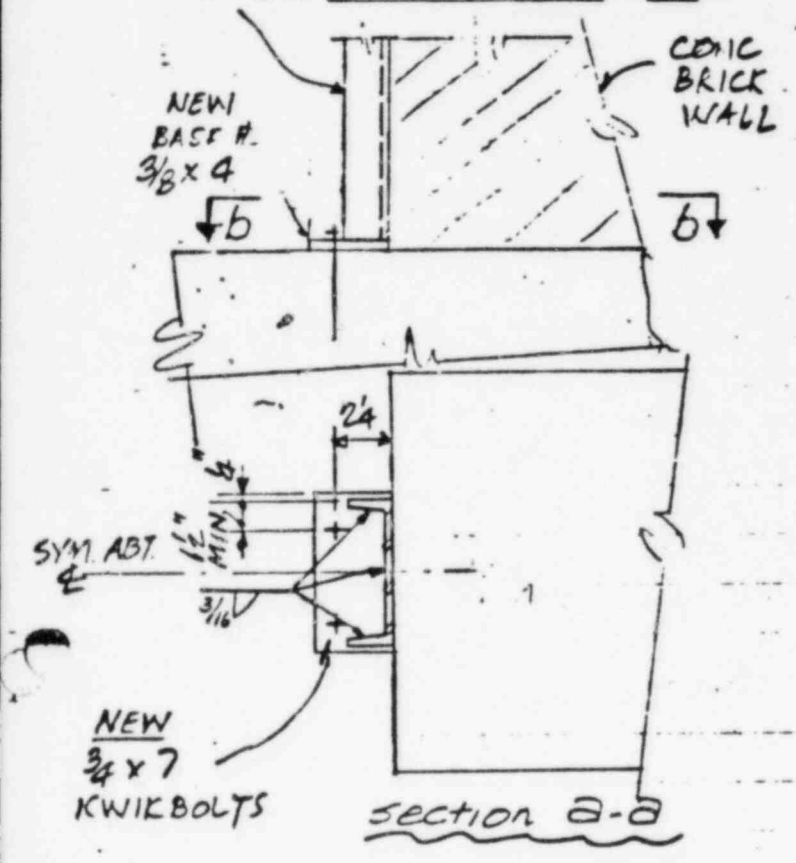
SECT. B-B

SCALE: ONE-HALF FULL SIZE

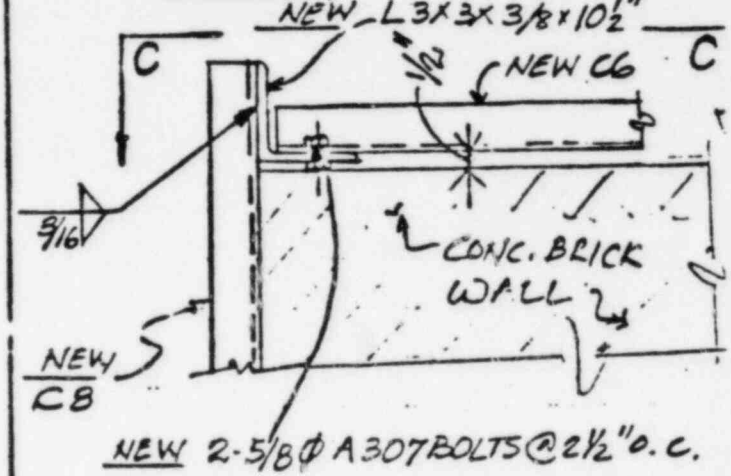
URS/BLUME
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

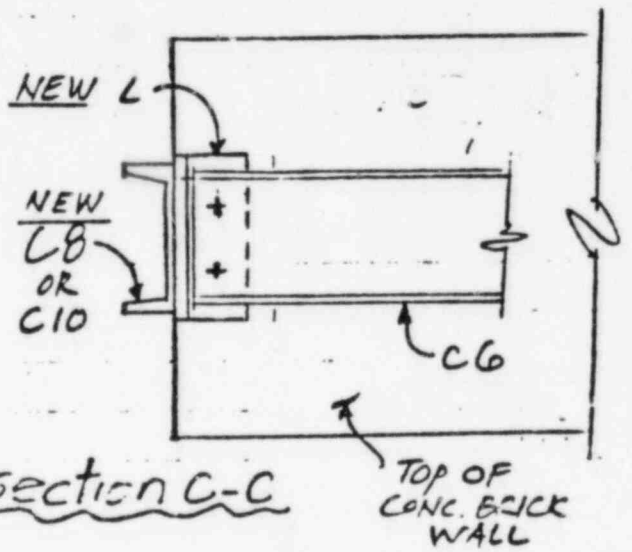
SHEET NO. 10
JOB NO. 8040-03 JOB CONN. YANKEE BY PMN DATE 9-3-80
CLIENT NUSCO SUBJECT CONC. BLOCK WALL MODIFICATIONS CHK'D [illegible] DATE 9-5-80

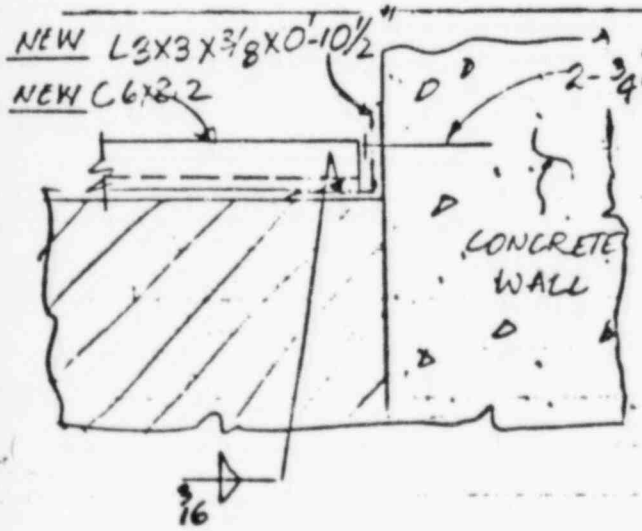
DETAIL (B)



DETAIL (C)







DETAIL (D)

SCALE 1½" = 1'-0"

DETAILS FOR WALLS CYPAB 102, 103, & 104

ATTACHMENT 4

consists of a 25-page (sheets 86-110) example of the use of the working stress design method used in the modification of masonry walls at the Haddam Neck Plant.

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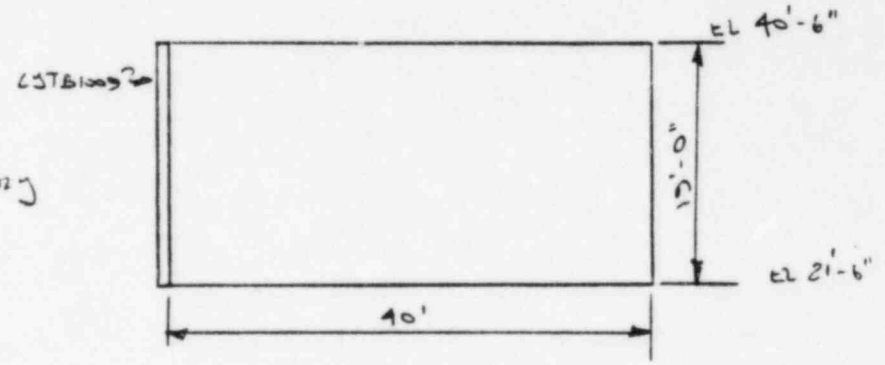
SHEET NO. B6

JOB NO. 8090.33 JOB CJ BLOCK WALL MODIFICATION BY JSM DATE 12-9-80

CLIENT NUSCO SUBJECT CJTB1010 CHK'D HLK DATE 12-22-80

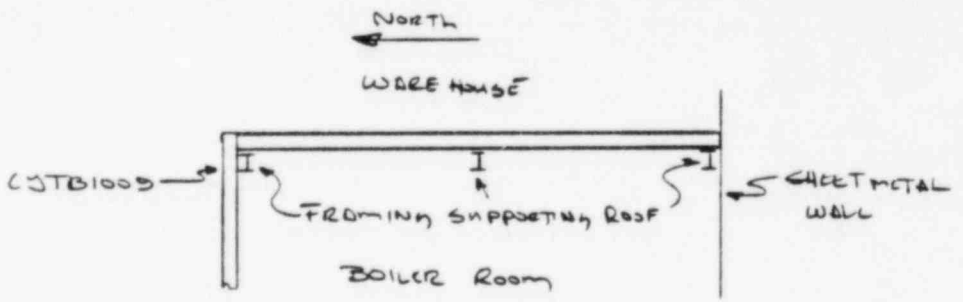
WALL DESCRIPTION

8" THICK HOLLOW MASONARY BLOCK



ELEVATION

LOOKING EAST



PLAN

NO SIGNIFICANT COMPONENTS ARE ATTACHED TO THE WALL. PIPES ARE ASSUMED TO HAVE GAPS AT THE WALL PENETRATIONS TO PREVENT LOADING.

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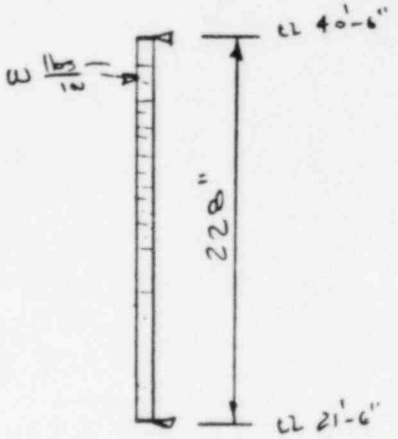
SHEET NO. 87

JOB NO. 8090-53 JOB CY BLOCK WALL MODIFICATION BY JSM DATE 12-5-80

CLIENT NUSCO SUBJECT CYTB1010 CHK'D HLC DATE 12-22-80

1. DETERMINE THE MAXIMUM ALLOWABLE DEFLECTION OF THE BLOCK WALL SPANNING VERTICALLY. (THE REINFORCING MUST BE RIGID ENOUGH TO PREVENT EXCESSIVE DEFLECTIONS.)

EVALUATE AN 8" WIDE VERTICAL STRIP



$\sigma_{ALL} = 1.67 \times 13.7 = 22.9$ psi

$\sigma_{ALL} = \frac{M}{S} = \frac{wL^2}{8S} = 22.9$ psi

SOLVING FOR w

$w = \frac{22.9 \times 8 \times 54}{228^2} = .1903 \frac{lbs}{IN}$

$\Delta = \frac{5wL^4}{384EI} = \frac{5 \times .1903 \times 228^4}{384 \times 420000 \times 206} = .0774"$ MAXIMUM ALLOWABLE MIDSPAN DEFLECTION OF BLOCK WALL SPANNING VERTICALLY

2. CALCULATE THE HORIZONTAL SPACING OF THE REINFORCING MEMBERS SUCH THAT THE FUNDAMINTAL MODE OF THE SPAN FALLS ON THE MINIMUM ACCELERATION VALUE OF THE RESPONSE SPECTRUM CURVE

INSPECTION OF THE RESPONSE SPECTRUM CURVES INDICATES THAT THE CONTROLLING SPECTRA IS THE E/W DIRECTION AT ELEV 37'-6".

$g_{MIN} = .5$ AT $.1$ sec

SINCE THE FREQUENCY CAN VARY BY $\pm 15\%$ THE MINIMUM PERIOD OF THE SPAN IS

$T = .1 \times \frac{1}{1.15} = .087$ sec

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San Francisco, California 94105

SHEET NO. 88

JOB NO. 8090-53 JOB LG BLOCK WALL MODIFICATION BY JSM DATE 12-9-80

CLIENT NUSCO SUBJECT CONDUIT CHK'D KHC DATE 12-22-80

FOR A SIMPLY SUPPORTED BEAM WITH A UNIFORM LOAD

$$\text{PERIOD } T_a = C\sqrt{\frac{wL^4}{EI}}$$

FOR AN 8" WIDE STRIP

$w = \frac{444}{16} = 2.775 \frac{\text{lbs}}{\text{IN}}$ $E = 420\,000 \text{ psi}$

$T_a = .085$ sec. $I = 205.8 \text{ IN}^4$

$C = .0324$

SOLVING FOR L

$$L = \sqrt[4]{\frac{EI\left(\frac{T_a}{C}\right)^2}{w}} = \sqrt[4]{\frac{420\,000 \times 205.8\left(\frac{.085}{.0324}\right)^2}{2.775}}$$

$L_{max} = 121"$ MAXIMUM ALLOWABLE SEISMIC SPAN LENGTH

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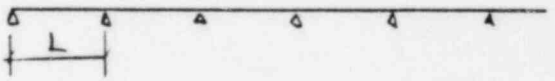
SHEET NO. 89

JOB NO. 8540-53 JOB CJ BLOCK WALL MODIFICATION BY JSM DATE 12-19-80

CLIENT NVSLO SUBJECT CJTB1010 CHK'D MJC DATE 12-23-80

DETERMINE MAXIMUM HORIZONTAL SPACING OF COLUMNS BASED ON ALLOWABLE BENDING STRESS

LET $M_{MAX} = \frac{wL^2}{8}$

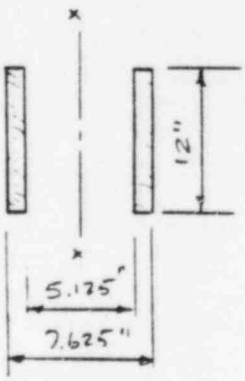


$\sigma_{ALL} = \frac{w_o L^2}{8S} = 1.67 \times 27 = 95 \text{ psi}$

FOR A 12" WIDE HORIZONTAL STRIP

$w_o = 1.3 \times .5 \times \frac{46}{12} = 2.49 \frac{\text{lbs}}{\text{IN}}$ UNIFORM LINEAR LOAD

$S = \frac{I}{C} = \frac{12(7.625^3 - 5.125^3)}{12 \times \frac{1}{2}(7.625)} = 80.97 \text{ IN}^3$



SOLVING FOR L

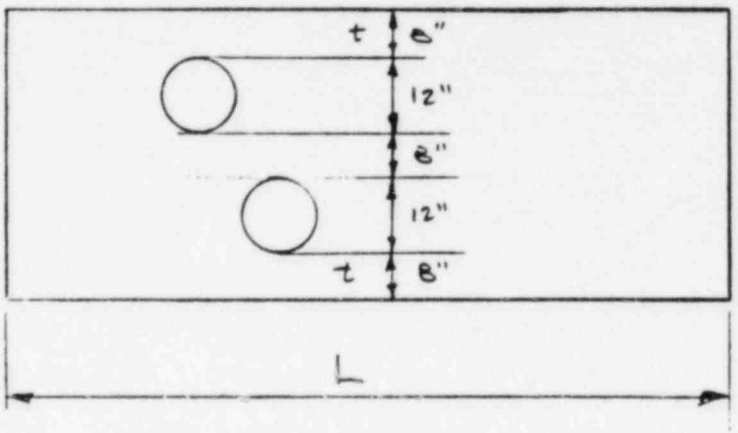
$L = \sqrt{\frac{\sigma_{ALL} \times 8 \times S}{w_o}} = \sqrt{\frac{45 \times 8 \times 80.97}{2.49}} = \underline{108"}$

CHECK MAXIMUM CANTILEVER SPAN ALONG SOUTH END OF WALL

$M_{MAX} = \frac{w_o L^2}{2}$

$L = \sqrt{\frac{45 \times 2 \times 80.97}{2.49}} = \underline{54"}$

CALCULATE THE MAXIMUM ALLOWABLE SPAN LENGTH ALONG THE NORTH EDGE OF THE WALL WHERE SEVERAL LARGE PIPE PENETRATIONS ARE LOCATED



THE WIDTH OF THE STRIP EXTENDS A DISTANCE OF 1t (8") PAST EACH PENETRATION.

WIDTH FOR WEIGHT CALCULATION = 48"
WIDTH FOR STRESS CALCULATION = 24"

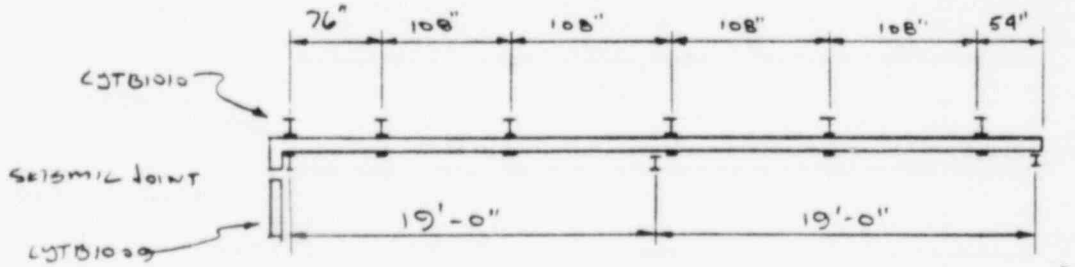
$W_o = \frac{48}{12} \times 2.49 = 9.96 \frac{lbs}{IN}$

$S = \frac{24}{12} \times 80.97 = 161.94\ IN^3$

$L = \sqrt{\frac{45 \times 8 \times 161.94}{9.96}} = 76.5"$

MAXIMUM CENTER TO CENTER COLUMN SPACING

SUMMARY



URS/BLUME
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

SHEET NO 5.1

JOB NO 5040-53 JOB CY BLOCK WALL MODIFICATION BY VSM DATE 12-19-80

CLIENT NUSCO SUBJECT CYTOTOXIC CHK'D HHC DATE 12-23-80

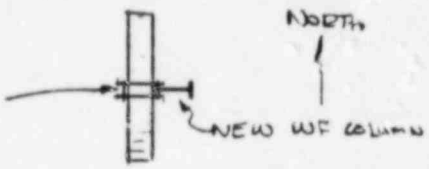
DETERMINE COLUMN SIZE

MAXIMUM TRIBUTARY WIDTH = 108" = 9'-0"

MAXIMUM ALLOWABLE MIDSPAN DEFLECTION = .0774"

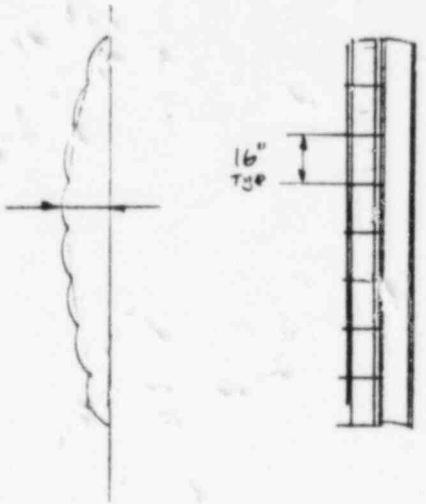
CONNECTION OF COLUMN TO WALL

NEW FLAT PLATE BOLTED THROUGH TO COLUMN



MAXIMUM DEFLECTION OF WALL INCLUDES DEFLECTION OF FLAT PLATE.

USE AN 8" x 3/8" FLAT PLATE
BOLT SPACING = 16"



FIND MAXIMUM DEFLECTION OF PLATE.

$$W_{o\ BLOCK} = 1.3 \times .5 \times \frac{46}{12} \times \frac{108}{12}$$

$$= 22.93 \frac{lbs}{IN} \quad \text{UNIFORM LINEAR LOAD.}$$

$$\Delta_{PLATE} = \frac{W_o L^4}{384EI} = \frac{22.93 \times 16^4}{384 \times 29 \times 10^6 \times \left(\frac{8 \times .375^3}{12}\right)} = .0038"$$

CHECK TO SEE IF PLATE IS STIFFER THAN THE BLOCK FOR A 12" WIDE VERTICAL BLOCK STRIP

$$\Delta_{BLOCK} = \frac{2.49 \times 16^4}{384 \times 420000 \times 308.7} = .0000033"$$

THE BLOCK IS MUCH STIFFER THAN THE PLATE CHECK TO SEE IF THE BLOCK CAN SPAN VERTICALLY BETWEEN THE BOLTS.

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SHEET NO. 92

JOB NO. 3040-53 JOB LG BLOCK WALL MODIFICATION BY JSM DATE 12-19-80

CLIENT PUSD SUBJECT LGT131070 CHK'D HLC DATE 12-23-80

BENDING STRESS IN BLOCK, VERTICAL STRIP BETWEEN BOLTS.

$$\sigma_{BLOCK} = \frac{M}{S} = \frac{w_o l^2}{12S} = \frac{2.49 \times \frac{108}{12} \times 16^2}{12 \times 80.97} = 5.9 \text{ psi} \qquad \sigma_{ALL} = 1.67 \times 13.7 = 22.9 \text{ psi}$$

OK

CHECK BENDING STRESS IN PLATE

$$f_b = \frac{M}{S} = \frac{w_o l^2}{12S}$$

$w_o = (2.49 \times \frac{108}{12}) + .55 = 23 \frac{\text{lbs}}{\text{IN}}$ (2.49: WEIGHT OF BLOCK; .55: WEIGHT OF PLATE) UNIFORM LINEAR LOAD ON PLATE

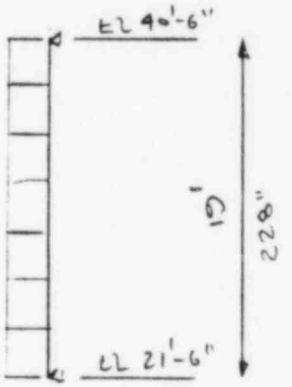
$$S = \frac{8 \times .375^2}{6} = .1875 \text{ IN}^3$$

$$f_b = \frac{23 \times 16^2}{12 \times .1875} = 2616 \text{ psi} < = 36 \text{ ksi} \qquad \text{OK}$$

CALCULATE THE REQUIRE I FOR THE COLUMN SECTION

MAXIMUM ALLOWABLE DEFLECTION OF COLUMN

$$\Delta_{COL} = \Delta_{TOTAL} - \Delta_{BLOCK} = .0774'' - .0000033 = .0774$$



UNIFORM LINEAR LOAD ON COLUMN

BLOCK = $46 \times 9 = 414 \frac{\text{lbs}}{\text{ft}}$

PLATE = $10.2 \frac{\text{lbs}}{\text{ft}}$

COLUMN = $40 \frac{\text{lbs}}{\text{ft}}$ EST.

$464 \frac{\text{lbs}}{\text{ft}}$

$$w_o = 1.3 \times .5 \times \frac{464}{12} = 25.14 \frac{\text{lbs}}{\text{IN}}$$

$$\Delta_{COL} = \frac{5 w_o L^4}{384 E I} = .0774"$$

SOLVING FOR I

$$I = \frac{5 w_o L^4}{384 E \Delta} = \frac{5 \times 25.14 \times 228^4}{384 \times 29 \times 10^6 \times .0774} = 394 \text{ IN}^4$$

USE A W18X35 LIGHTEST AVAILABLE SECTION

$I_{xx} = 513$ $S'_{xx} = 57.9 \text{ IN}^3$

CHECK STRESSES IN THIS MEMBER

$$f_b = \frac{M}{S'} = \frac{w_o L^2}{8 S'} = \frac{25.14 \times 228^2}{8 \times 57.9} = 2821 \text{ psi} \qquad F_b = 1.67 \times .6 \times 36 = 36 \text{ KSI}$$

OK

$$v = \frac{V}{A} = \frac{25.14 \times 228 \times 1/2}{17.71 \times .298} = 534 \text{ psi} \qquad F_v = 1.67 \times .4 \times 36 = 24 \text{ KSI}$$

OK

BECAUSE A BACKING PLATE CANNOT BE BOLTED TO THE COLUMN AT THE NORTH END OF THE WALL, SEE IF THE EXISTING 10WF39 COLUMN (ON THE WEST SIDE) CAN BE USED TO RESIST THE WALL MOVEMENT WHEN THE WALL MOVES WEST.

FOR A 10WF39 $I_{xx} = 210 \text{ IN}^4 < 436 \text{ IN}^4$ (I REQUIRED FOR A 9' WIDE SPAN)

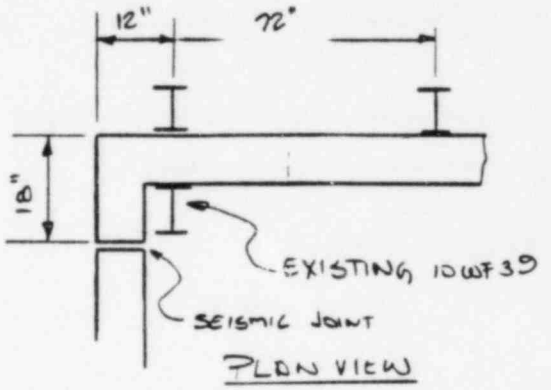
RECALCULATE THE REQUIRED I USING THE ACTUAL TRIBUTARY WIDTH SUPPORTED BY THE COLUMN.

URS/BLUME
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SHEET NO. 94

JOB NO. 8040-53 JOB CJ BLACK WALL MODIFICATION BY JSM DATE 12-15-80

CLIENT NUSCO SUBJECT CJT3101O CHK'D HWC DATE 12-23-80



PLAN VIEW

TRIBUTARY WIDTH = $\frac{72}{2} + 12 + 6 = 54"$

UNIFORM WEIGHT

$W_o = 1.3 \times .5 \times 46\left(\frac{54}{12}\right) + (1.3 \times .5 \times 39) = 161$ PLF

MAXIMUM ALLOWABLE MIDSPAN DEFLECTION = .0774"

$I = \frac{5 W_o L^4}{384 E \Delta_{MAX}} = \frac{5 \times \frac{161}{12} \times 228^4}{384 \times 29 \times 10^6 \times .0774} = 210 \text{ IN}^4$ OK THE EXISTING MEMBER IS ADEQUATE

EXISTING 10WF39

$I_{xx} = 210 \text{ IN}^4$ $S_{xx} = 42.2 \text{ IN}^3$

CHECK BENDING STRESS

$M_{MAX} = \frac{W_o L^2}{8} = \frac{\frac{161}{12} \times 228^2}{8} = 87,182$ IN-LBS

$f_b = \frac{M}{S} = \frac{87182}{42.2} = 2066$ PSI $F_b = 36$ KSI OK

CHECK SHEAR

$V_m = \frac{V}{A} = \frac{W_o L}{2A} = \frac{161 \times 19}{2} \times \frac{1}{(9.94 \times .315)} = 482$ PSI $V_{ALL} = 24$ KSI OK

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SHEET NO. 95

JOB NO. 8040-53 JOB CJ BLOCK WALL MODIFICATION BY NSM DATE 12-16-80

CLIENT NUSCO SUBJECT CYTB 1010 CHK'D HLC DATE 12-23-80

CONNECTION DETAILS

CHECK TENSILE FORCE ON BOLTS

BOLT SPACING = 16"

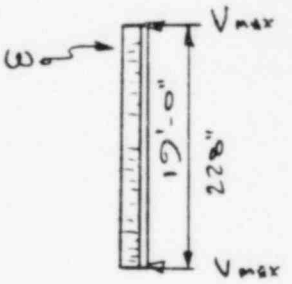
$W_o = 2514$ lbs/in

FORCE/BOLT = $25.14 \times 16 = 402 \frac{\text{lbs}}{\text{BOLT}}$ USE STANDARD 3/4" ϕ A307 BOLTS OK BY INSPECTION

CALCULATE MAXIMUM SHEAR FORCE AT COLUMN CONNECTIONS

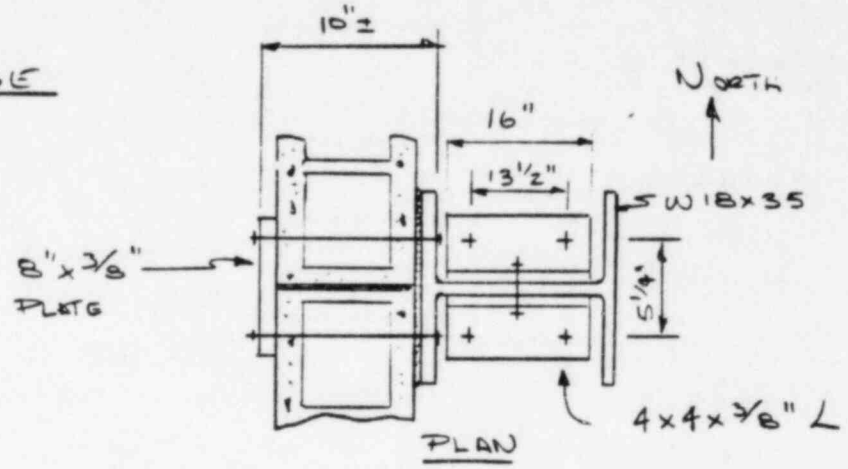
$$V_{max} = \frac{W_o L}{2}$$

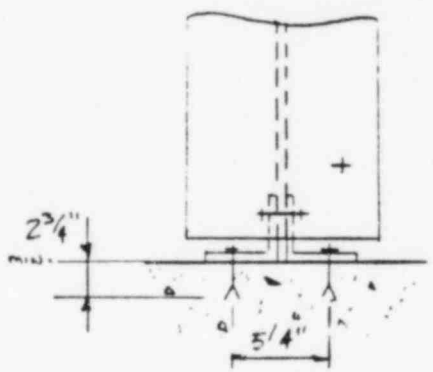
$W_o = 25.14 \frac{\text{lbs}}{\text{in}}$

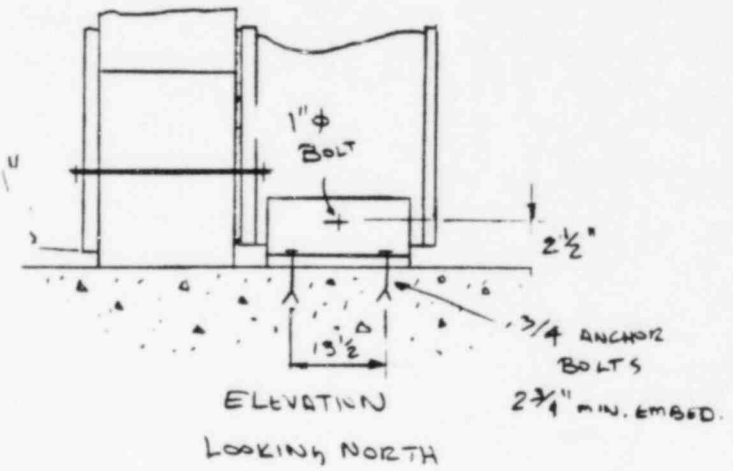


$V_{max} = \frac{W_o L}{2} = \frac{25.14 \times 228}{2} = 2866$ lbs

CHECK CONNECTION AT BASE







DETERMINE MAXIMUM SHEAR FORCE AT COLUMN TO BASE CONNECTION

HORIZONTAL FORCE = 2866 lbs

VERTICAL FORCE = WEIGHT OF COLUMN + VERT SEISMIC

$= (35 \times 19)(1 + \frac{2}{3} \times .5) = 886$ lbs

TOTAL SHEAR FORCE $= \sqrt{2866^2 + 886^2} = 3000$ lbs

THICKNESS OF COLUMN WEB .298

USING A307 BOLTS

TRY A 1" ϕ BOLT

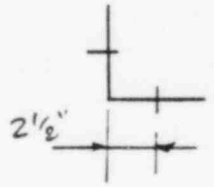
SHEAR CAPACITY = 15.71 KIPS DOUBLE SHEAR OK

BEARING CAPACITY = 12.2 KIPS 1/4" THICK MATERIAL OK

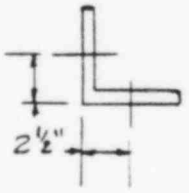
CHECK ANCHOR BOLTS

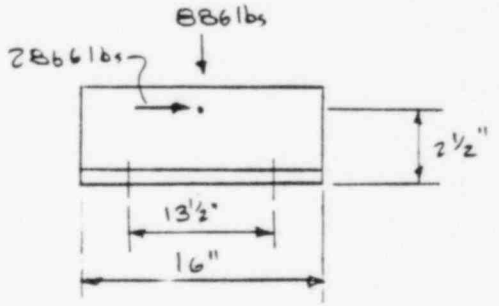
MINIMUM EDGE DISTANCE = 1 1/4" FOR A 3/4" BOLT

TRY A 4" x 4" x 3/8" L

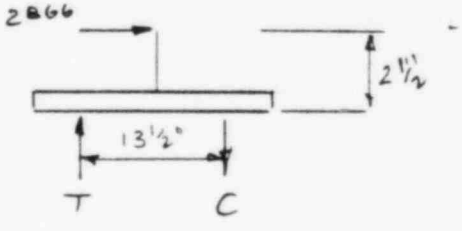


DETERMINE MAXIMUM TENSILE FORCE ON THE ANCHOR BOLT.





$$T = \frac{2866 \times 2\frac{1}{2}}{13\frac{1}{2}} \times \frac{1}{2} = 266 \frac{\text{lbs}}{\text{BOLT}}$$



MAXIMUM SHEAR = $\frac{2866}{4} = \frac{717 \text{ lbs}}{\text{BOLT}}$

Try a 3/4 ANCHOR BOLT 3 1/4 MINIMUM EMBEDMENT

MINIMUM SPACING = 7 1/2"

ACTUAL MINIMUM SPACING = 5 1/4"

RATIO $\frac{5\frac{1}{4}}{7\frac{1}{2}} = .70$

ALLOWABLE PULLOUT FORCE = $.7 \times \frac{9437}{4} = 1651 \text{ lbs}$

ALLOWABLE SHEAR FORCE $.7 \times \frac{19825}{4} = 3478 \text{ lbs}$

RATIO TO ALLOWABLE $\frac{266}{1651} + \frac{717}{3478} = .37 < 1.0$ OK

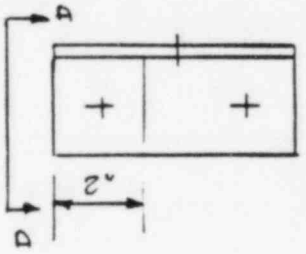
URS/BLUME
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

SHEET NO. 99

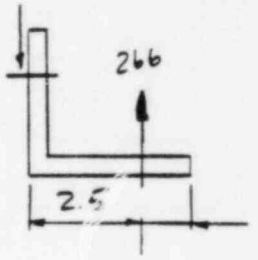
JOB NO. 8040-53 JOB CJ BLOCK WALL MODIFICATION BY JSM DATE 12-17-80

CLIENT NUSCO SUBJECT CJTB1010 CHK'D HHC DATE 12-23-80

DETERMINE THICKNESS OF ANGLE SECTION



PLAN VIEW



SECTION AA

ASSUME A 2" WIDE TRIBUTARY WIDTH RESISTS THE TENSILE FORCE OF THE ANCHOR BOLT.

$M_{max} = 2.5 \times 266 = 665$ IN-lbs

$F_b = 1.67\,(.6 \times 36) = 36$ KSI

$$S = \frac{b d^2}{6} = \frac{M_{max}}{F_b}$$

SOLVING FOR d

$$d = \sqrt{\frac{6 M_{max}}{F_b \times b}} = \sqrt{\frac{6 \times 665}{36000 \times 2}} = .24'' \sim 3/8''$$

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SHEET NO. 100

JOB NO. 804-53 JOB CE BLOCK WALL MODIFICATION BY JSM DATE 12-17-80

CLIENT NUSCO SUBJECT CE TIOGA CHK'D HHC DATE 12-23-80

CHECK BEARING STRESS ON CONCRETE

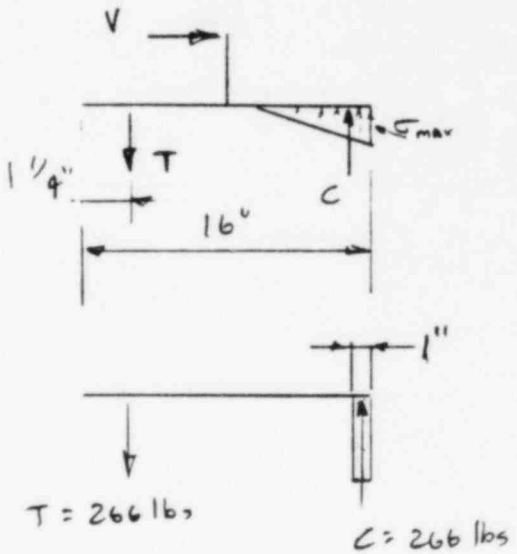
STRESS FROM VERTICAL LOAD

$$\sigma = \frac{P}{A} = \frac{886}{2(16 \times 4)} = 6.9 \text{ psi}$$

STRESS FROM HORIZONTAL LOAD

FIND σ_{max}

ASSUME A 1" WIDE CONTACT SURFACE ALONG THE EDGE OF THE STRIP.



$T = C$ BY EQUILIBRIUM

$$\sigma_{max} = \frac{C}{A} \quad \frac{266 \times 2}{1 \times 8} = 66 \text{ psi}$$

MAXIMUM STRESS = 6.9 + 66 = 72.9 psi

ALLOWABLE BEARING STRESS = 750 psi FOR $f'_c = 3000$ psi

OK

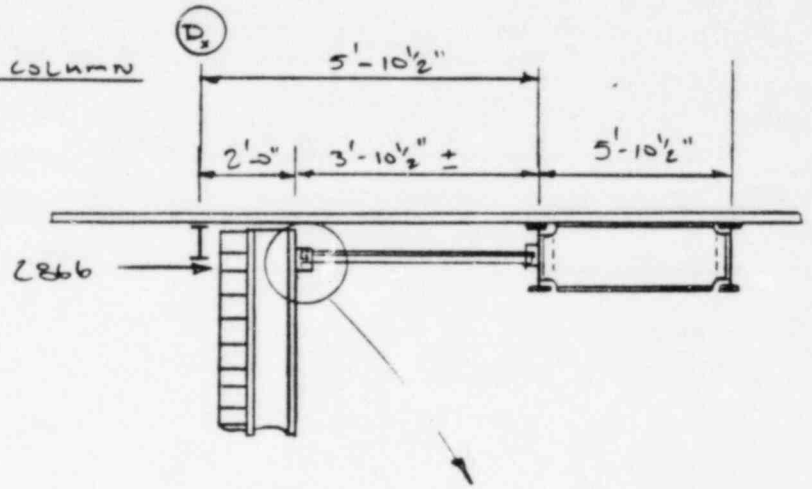
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SHEET NO. 101

JOB NO. 8540-53 JOB CJ BLOCK WALL MODIFICATION BY JSM DATE 12-17-80

CLIENT NVSCO SUBJECT CJT31010 CHK'D HLC DATE 12-23-80

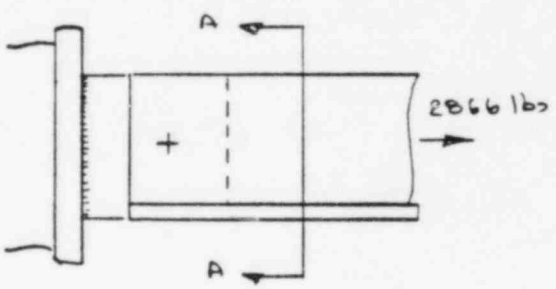
CHECK CONNECTION AT TOP OF COLUMN



SELECT A MEMBER TO TRANSFER THE LOAD FROM THE COLUMN INTO THE ROOF FRAME AND DECKING.

MAXIMUM FORCE = ±2866

ASSUME EACH END IS PINNED

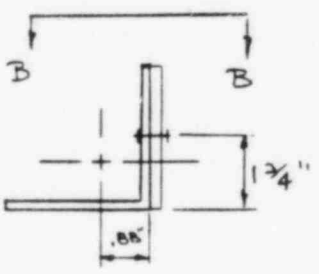


TRY A 3X3X3/8" L

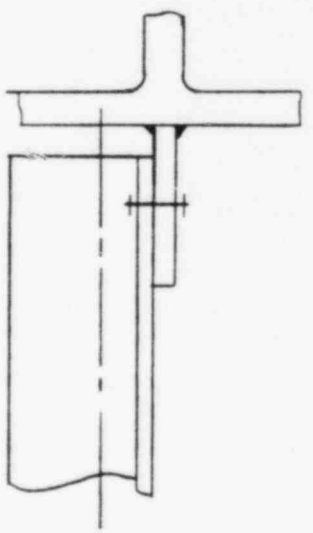
$r_{MIN} = .587$ $A = 2.11\ IN^2$

$S = .833\ IN^3$

$y = .888"$



SECTION AA



SECTION BB

CHECK AXIAL LOADING

$\frac{KL}{r} = \frac{1.0 \times 48}{.587} = 81.7$

$F_a = 15.16$ KSI

$f_a = \frac{2866}{(2.11 - .42)} = 1698\ PSI$

(NET AREA ASSUMING A 1" ϕ BOLT

CHECK BENDING STRESS

$M_x = 2866 \times (1.75 - .88) = 2493$ in-lbs

$M_y = 2866 \times .88 \quad 2522$ in-lbs

$f_{bx} = \frac{2493}{.833} = 2993$ psi

$f_{by} = \frac{2522}{.833} = 3028$ psi

$\frac{f_a}{F_A} = \frac{1698}{15130} = .112 < .15$

THE FOLLOWING IS APPLICABLE FOR COMBINED AXIAL & BENDING STRESS

$\frac{f_a}{F_A} + \frac{f_{bx}}{F_{Bx}} + \frac{f_{by}}{F_{By}} \leq 1.0$ $\quad F_{Dx} = F_{By} = 1.67 \times .6 \times 36 = 36$ ksi

$\frac{1698}{15180} + \frac{2993}{36000} + \frac{3028}{36000} = .28 < 1.0$ OK

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SHEET NO. 103

JOB NO. 8040-53 JOB CY BLOCK WALL MODIFICATION BY JSM DATE 12-17-80

CLIENT NUSCO SUBJECT CYTB1010 CHK'D HWC DATE 12-23-80

BOLT SELECTION

USING A307 BOLTS

TRY A 1" ϕ BOLT

SHEAR CAPACITY = 7.85 KIPS SINGLE SHEAR

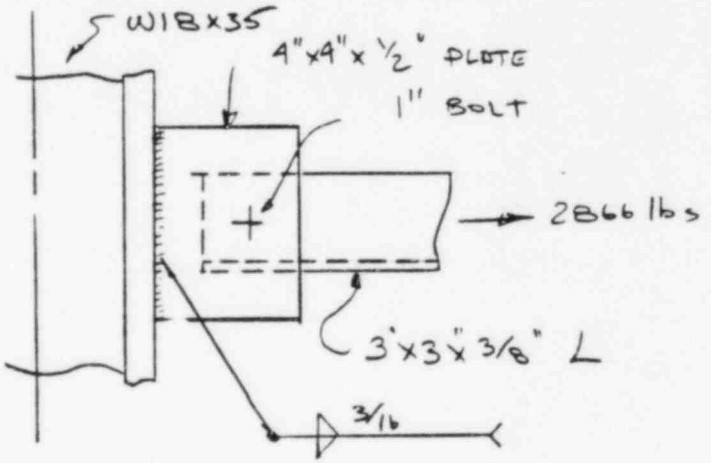
BEARING CAPACITY = 18.2 KIPS FOR 3/8" THICK MATERIAL

PLATE ATTACHMENT

USE A 3/8" THICK PLATE

WELD THE PLATE TO THE COLUMN AND THE ROOF BEAM

USE E70XX ELECTRODE AND 3/16" WELD



ALLOWABLE STRESS = 21 KSI

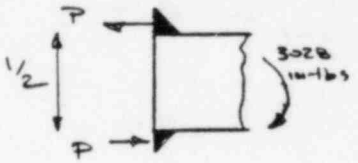
CAPACITY OF WELD

$$\frac{3}{16} \times \frac{1}{\sqrt{2}} \times 1 \times 21{,}000 = 2784 \ \frac{\text{lbs}}{\text{IN}}$$

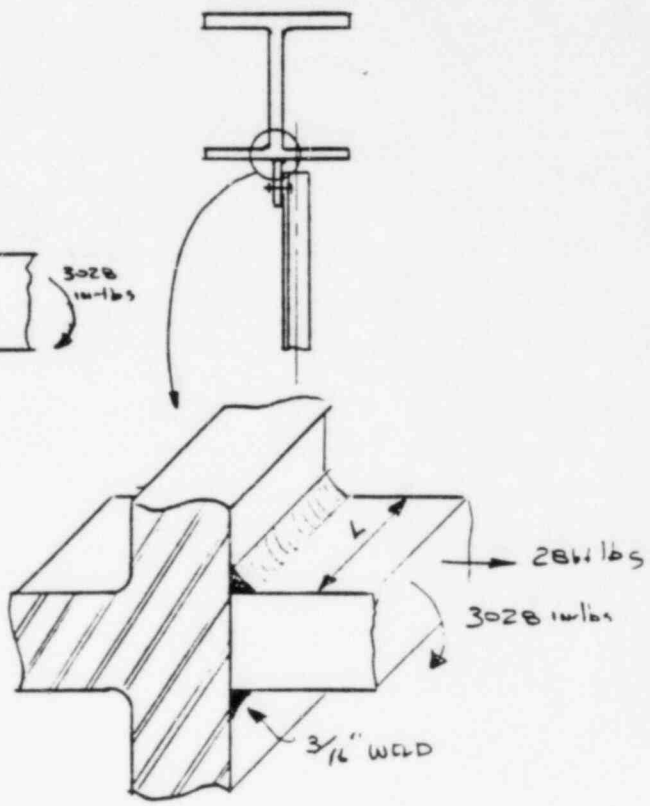
FIND WELD LENGTH REQUIRED TO RESIST BENDING MOMENT

$M_y = 3028$ in-lbs

$P = \frac{M}{d} = \frac{3028}{.5} = 6056$ lbs



$L = \frac{6056}{2784} = 2.18"$



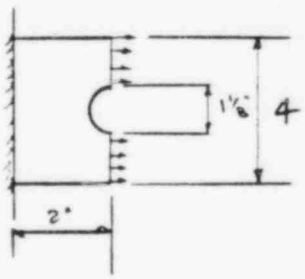
FIND WELD LENGTH REQUIRED TO RESIST AXIAL FORCE

$L = \frac{2866}{2784} \times \frac{L}{2} = \underline{.52"}$

$L_{TOTAL} = 2.18 + .52 = \underline{2.70}$

DETERMINE PLATE SIZE

TRY A 4" x 4" x 1/2" PLATE



$f_a = \frac{2866}{1/2\,(4 - 1\frac{1}{8})} = 1994$ psi

$f_b = \frac{M}{S} = \frac{3028}{\left(\frac{2.875 \times .5^2}{6}\right)} = 25277$ psi

$\frac{1994}{36000} + \frac{25277}{36000} = .76 < 1.0$ OK

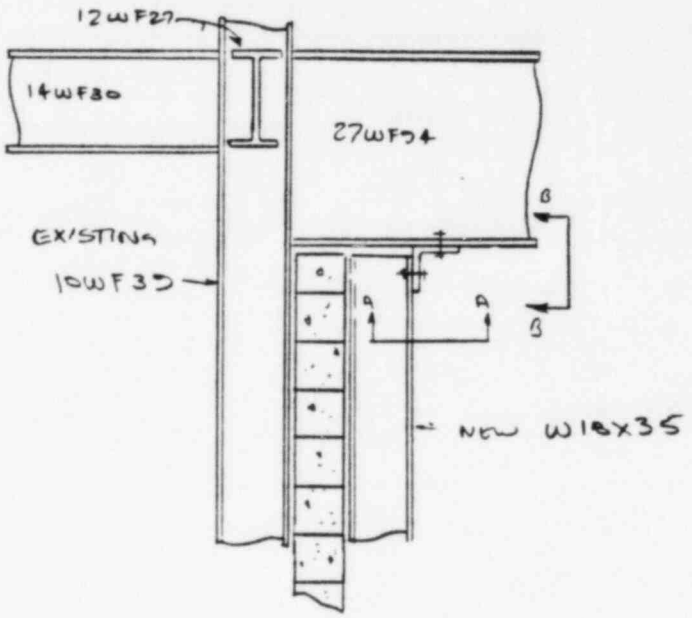
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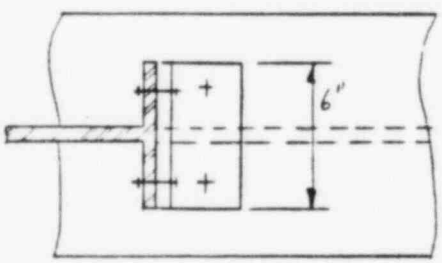
SHEET NO. 105

JOB NO. 8040-53 JOB CJ BLOCK WALL MODIFICATIONS BY VSM DATE 12-18-80

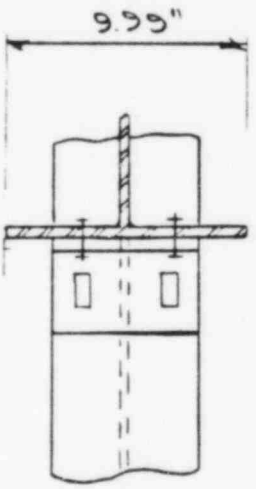
CLIENT NUSCO SUBJECT CJTB1010 CHK'D HJC DATE 12-23-80

CHECK CONNECTION TO ROOF GIRDER





SECTION AA

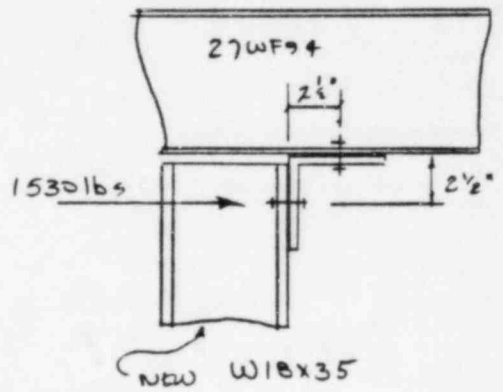


SECTION BB

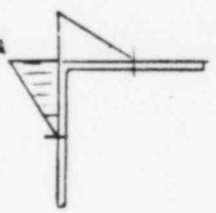
USE A 4x4x 3/8 L 6" LONG

MAXIMUM REACTION AT THE CONNECTION

$16 \times \frac{19}{2} = 1530$ lbs



$M_{MAX} = 2.5 \times 1530 = 3825$ IN-lbs



$f_b = \frac{M}{S} = \frac{3825}{\frac{6 \times (3/8)^2}{6}} = 27200$ psi

$f_a = \frac{1530}{3/8 \times 6} = 680$ psi

$\frac{680}{36000} + \frac{27200}{36000} = .77 < 1.0$ OK

CHECK FOR SLIDING

$$\frac{\text{HORIZONTAL FORCE}}{\text{VERTICAL FORCE}} \leq \frac{.8}{1.5} = .533$$

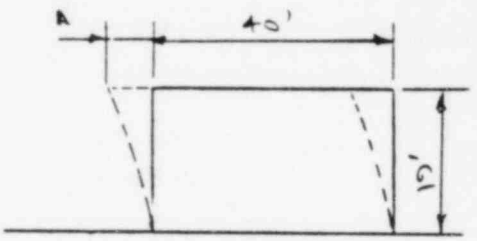
DETERMINE WALL FREQUENCY IN THE N/S PLANE

CALCULATE THE FORMULA $f = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}}$

WHERE Δ IS THE LATERAL DEFLECTION DUE TO A 1g LOAD.

BENDING SHEAR

$$\Delta = \frac{wL^4}{8EI} + \frac{1.2\,wL^2}{2GA}$$



$$w = \frac{46}{12}\frac{\text{psf}}{} \times 40 = 153.3\ \frac{\text{lbs}}{\text{in}}$$

$E = 420000$

$G = 168000$

$I = \frac{2.5\,(40\times12)^3}{12} = 23{,}040{,}000\ \text{in}^4$

$A = 2.5\times(40\times12) = 1200\ \text{in}^2$

$$\Delta = \frac{153.3 \times 228^4}{8\times420000\times23{,}040{,}000} + \frac{1.2\times153.3\times228^2}{2\times168000\times1200}$$

$= .0054 + .0237 = .0291"$

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SHEET NO. 10B

JOB NO. 8040-53 JOB CS BLOCK WALL MODIFICATIONS BY JSM DATE 12-18-80

CLIENT NUSCO SUBJECT CJTB1010 CHK'D [illegible] DATE 12-23-80

FOR A SINGLE DEGREE OF FREEDOM SYSTEM $f = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}}$

$f = \frac{1}{2\pi}\sqrt{\frac{386.4}{.0291}} = 18.4 \text{ Hz} \pm 15\% \quad 15.6 \text{ Hz} - 21.1 \text{ Hz}$

MAXIMUM PERIOD $= \frac{1}{15.6} = .064$ SEC.

THE WALL IS SUPPORTED ONLY BY THE FLOOR SLAB AT ELEV 21'-6"
FOR IN PLANE LOADING USE THE GROUND RESPONSE SPECTRA

$g_{MAX} = .26\,g$ SSE

VERTICAL ACCELERATION $= \frac{2}{3} \times .26 = .173\,g$

(WEIGHT OF WALL)

HORIZONTAL FORCE $= \frac{1.3 \times .26 \times W}{(1-.173) \times W} = .408 < .533$ OK
VERTICAL FORCE

CHECK FOR OVERTURNING

$\frac{\text{OVERTURNING MOMENT}}{\text{RESISTING MOMENT}} = \frac{1}{1.5} = .67$

$OTM = 1.3 \times .26 \times W \times 9.5 = \frac{3.211}{16.54} = .194 < .67$ OK
$RM = (1-.173) \times W \times 20$

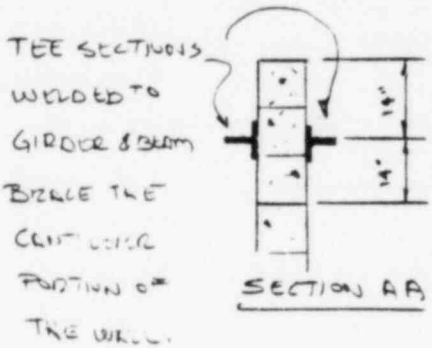
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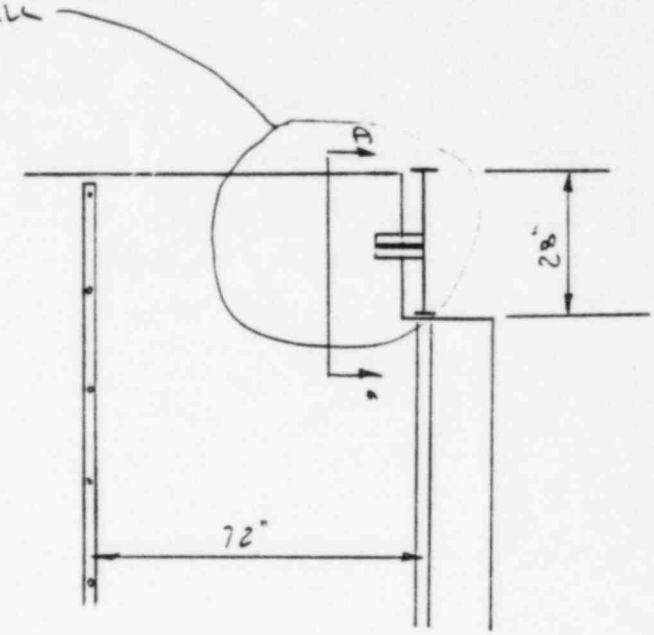
SHEET NO. 109

JOB NO. 8040-53 JOB (5) 12 BLOCK WALL MODIFICATIONS BY JSM DATE 12-23-80

CLIENT MUSCO SUBJECT LYTG 1010 CHK'D HLC DATE 12-23-80

CHECK LOADING OF WALL AROUND THE 27 WF 94 GIRDER AT THE NORTH END OF THE WALL



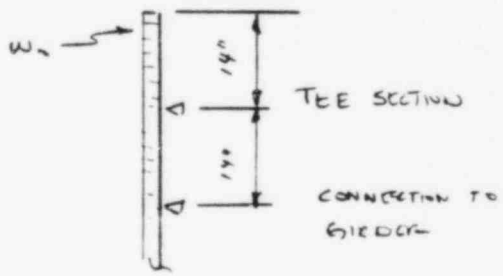


CHECK LOADING ON THE CANTILEVER SECTION

TRIBUTARY WIDTH = $\frac{72}{2} = 36"$

$W_o = 1.3 \times .5 \times 46 \times \frac{36}{12} \times \frac{1}{12} = 7.98 \frac{lbs}{IN}$

REACTION AT TEE = $7.98 \times (14+7) = 157 lbs$



CHECK BENDING STRESS AT THE TEE SUPPORT.

ASSUME A VERTICAL STRIP 2' (16") WIDE RESISTS THE MOMENT.

$M_{MAX} = \frac{W_o L^2}{2} = \frac{7.48 \times 14^2}{2} = 733 \text{ IN-lbs}$

$f = \frac{M}{S} = \frac{733}{108} = 6.8 \text{ PSI} \quad f_{ALL} = 1.67 \times 13.7 = 22.9 \text{ PSI}$ OK

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SHEET NO. 11 D

JOB NO. 8090-53 JOB LG BLOCK WALL MODIFICATIONS BY JSM DATE 12-23-80

CLIENT NUSCO SUBJECT CUT BEAM CHK'D HLC DATE 12-23-80

FABRICATE THE TEE FROM A 12" LONG W18X35 CUT IN HALF

DEPTH OF TEE = $\frac{17.71}{2} \approx 8\frac{3}{4}$"

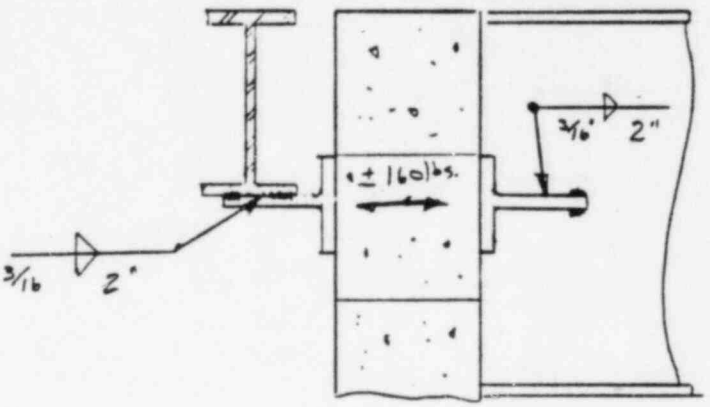
WIDTH OF TEE 6.00"

THICKNESS OF FLANGE .429"

THICKNESS OF WEB = .298"

AREA = 5.1 IN^2

WEIGHT = $18 \frac{lbs}{ft}$



WELD REQUIREMENTS

USE A 3/16" FILLET WELD

WELD CAPACITY = $\frac{3/16}{\sqrt{2}} \times 21000 = 2784 \frac{lbs}{IN}$

Attachment 5 consists of the Final Report on Safety-Related Masonry Walls at the Haddam Neck Plant.

FINAL REPORT ON SAFETY-RELATED MASONRY WALLS AT THE CONNECTICUT YANKEE ATOMIC POWER PLANT

prepared for

Northeast Utilities Service Company

URS/John A. Blume & Associates, Engineers

130 Jessie Street (at New Montgomery)

San Francisco, California 94105

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1. INTRODUCTION

1.1 Purpose

This report, prepared by URS/John A. Blume & Associates, Engineers (URS/Blume), for Northeast Utilities Service Company (NUSCO), summarizes the results of the analytical study of the structural adequacy of the safety-related masonry walls at the Connecticut Yankee Atomic Power Plant in response to the requirements of the U.S. Nuclear Regulatory Commission (NRC) IE Bulletin 80-11!

Connecticut Yankee is a 600-MW pressurized-water-reactor nuclear power plant located in East Haddam, Connecticut. The plant was designed by Stone & Webster Engineering, Boston, Massachusetts. It began commercial operation in 1968.

1.2 Objective and Scope of Study

The objective of the analysis of the safety-related masonry walls at Connecticut Yankee was to meet the requirements of NRC IE Bulletin 80-11. The bulletin specifically requires the following:

- A description of the safety-related masonry walls, including their function, material properties, physical characteristics, and locations
- A description of the practices employed during the construction of the safety-related walls
- Discussion of the development of the final design criteria used for analysis of the walls, along with detailed references to codes, reference material, and reports

- o A description of the types and magnitudes of loads that the walls are required to withstand
- o Discussion of the mechanism by which the loads are transferred from attached equipment to the safety-related walls

The masonry walls were analyzed for dead, live, seismic, and all other applied loads as specified in the FDSA (final design and safety analysis) report for Connecticut Yankee.

2. FIELD INVESTIGATIONS

2.1 Results of Field Inspection

The entire Connecticut Yankee plant was inspected to verify that all masonry walls in the plant were shown on the architectural drawings and that all walls on the drawings actually existed in the plant. The drawings were found to be correct.

All of the safety-related masonry walls at the Connecticut Yankee are categorized as non-load-bearing walls. These walls function as either partition walls, firewalls, or radition shielding walls. All of the walls are single-wythe concrete block walls, and all were built after construction of the main structural frame of the building in which they were located.

To determine which walls were to be selected for analysis, a masonry wall was defined as safety-related if it had safety-related systems, equipment, or components either attached to it or located in promixity to it in such a way that a wall failure could affect a safety-related system. Safety-related systems, equipment, and components were listed in the (QA) Category I Material Equipment and Parts List.[2]

In general, a safety-related system, item of equipment, or component was considered to be in proximity to the wall if the radial distance from the base of the wall to the edge of the item (i.e., the radial distance about an origin located at the bottom of either face of the wall) was less than

the height of the wall. This criterion was applied only to items directly in front of or behind the wall and in the same room or enclosed area as the wall. In special cases, where engineering judgment was used to define proximity, the reasons were justified and documented.

Table 2.1 is the final list of walls that were determined to be safety-related and shows the systems, equipment, or components that qualified each wall as safety-related.

2.2 Description of Masonry Walls

Table 2.2 lists the architectural drawings that show safety-related masonry walls at Connecticut Yankee. The drawings show the thickness of the walls and also whether they were grouted. The thickness and length of the walls were found to be correct as shown on the drawings.

Table 2.3 summarizes some of the required information obtained from the field inspection of each safety-related masonry wall. Additional information (e.g., block size, location of openings, weights and locations of attachments) needed for the analysis was also collected.

2.3 Construction Practices

The practices used in the construction of the masonry walls at Connecticut Yankee were set forth in the original specification for the masonry work[3] and are summarized in this section. The URS/Blume field engineers did not find any deviations from these specification requirements. Table 2.4

TABLE 2.1

SAFETY-RELATED MASONRY WALLS AT THE CONNECTICUT YANKEE ATOMIC POWER PLANT

Wall ID Number[a]	Location			Safety-Related Systems, Equipment, and Components in Proximity to the Wall
	Building	Floor Elevation	Column Lines	
CYCT1001	Containment	22'-0"	6	Wide-range level transmitter
CYSF1001 CYSF1002	New and Spent Fuel	21'-6"	R-S/104 S/104-105	Spent fuel pit heat exchanger pumps and motors (and associated piping and electrical cables)
CYSF2001		35'-0"	S/101-104	High-point suction pipe to spent fuel pit pump
CYDG1001 CYDG1002 CYDG1003 CYDG1004	Diesel Generator	21'-6"	D-E/12 D-E/12 D-E/12 D-E/13	Diesel generator switchgear panels, electrical cables, and air compressor
CYPAB101	Primary Auxiliary	21'-6"	F-H/10-11	Service water pipes, low-pressure safety-injection pumps and piping, and cable trays
CYPAB102			J-K/11-12	Charging pump B
CYPAB103			J-K/11-12	Charging pump A
CYPAB104			L/11	Charging metering pump
CYPAB105			L/12	Nonregenerative heat exchanger
CYPAB106			M-N/11-12	Service water, purification pump, piping, electrical conduits
CYPAB107			L-11	Charging metering pump
CYPAB108			K-11	Charging pump A
CYPAB201		35'-6"	F/11-12	Service water piping, batteries located in Service Building switchgear room
CYPAB202			F/10-11	Batteries located in Service Building
CYPAB203			F-H/11	Service water piping
CYPAB204			H/10-11	Service water piping
CYTB1007	Turbine	21'-6"	C-D/11	Service water piping
CYTB1008			D/1-2	Auxiliary feedwater flow transmitters
CYTB1009			C-E/1	Auxiliary feedwater piping
CYTB1010			E/0-1	Auxiliary feedwater piping
CYTB2004[b]		37'-6"	C-D/1	Feedwater piping
CYTB2005			C/1	Feedwater piping
CYSB1001[c] and CYSB1002	Service	41'-6"	F/10-12	Batteries, ventilation duct, switchgear
CYSB1003			D-F/8	Switchgear cable trays
CYSB2002		59'-6"	E/9.5	Control ventilation system (vent ducts, fans, switches, and air damper)
CYSB2003			E/8-9.5	Control ventilation system (vent ducts, fans, switches, and air damper)

a. The wall identification number is defined as follows: The first two letters stand for the plant (Connecticut Yankee). The next two or three letters indicate the building (e.g., YB = Turbine Building). The next number represents the floor level (e.g., 2 = second floor). The last two- or three-digit number is an arbitrary number assigned to the wall.

b. This is the top of CYTB1009.

c. These are the tops of CYPAB201 and CYPAB202.

TABLE 2.2

REFERENCE DRAWING LIST,

CONNECTICUT YANKEE MASONRY WALLS

Reference Drawing and Revision Number[a]	Wall ID Number
10899-FA-7C, Rev. 7	CYTB1009, 2004, 2005
10899-FA-11B, Rev. 8	CYSB1001, 1002, 1003, 2002, 2003
10899-FA-15A, Rev. 7	CYTB1007
10899-FA-16A, Rev. 6	CYTB1009, 2004, 2005, CYTB1010
10899-FA-20A, Rev. 8	CYPAB101, 102, 103, 104, 105, 106, 107, 108, 201, 202, 203, 204
10899-FA-22A, Rev. 6	CYSF1001, 1002, 2001
10899-FA-31A, Rev. 2	CYDG1001, 1002, 1003, 1004
10899-FC-38B, Rev. 4	CYCT1001
10899-FC-63A, Rev. 3	CYDG1001, 1002, 1003, 1004
10899-RA-3A, Rev. 9	CYSB1001, 1002, 1003, 2002, 2003, CYTB1007, 1008
10899-RA-11A, Rev. 10	CYTB1007, 1008

a. Drawings were completed by Stone & Webster Engineering, Boston, Massachusetts.

TABLE 2.3

IMPORTANT CHARACTERISTICS OF MASONRY WALLS AT CONNECTICUT YANKEE

Wall ID Number	Function	Dimensions (in.)			Type of Cell	Significant Attachments	Significant Openings	Boundary Conditions			
		L	h	t				Top	Bottom	Side 1	Side 2
CYCT1001	shield	65	42	18	solid blocks (stacked)	None	None	concrete beam	slab	concrete column	concrete column
CYSF1001	shield, firewall	332	153.5	11.625	filled (except top 2 courses)	5"-diameter pipe (to pump) anchored to wall	88"H x 40"W door passageway	concrete slab	slab	concrete wall	masonry wall
CYSF1002	shield, firewall	176	116	11.625	filled (except top 2 courses)	Fire hose, several 2"-diameter pipes	48"H x 40"W window opening	steel beam	slab	concrete wall	steel column
CYSF2001	shield, firewall	420	120	11.625	filled (except top course)	None	36"W x 85"H door	steel beam	slab	concrete wall[a]	masonry wall
CYDG1001	firewall	145.5	219	11.625	filled (except top course)	Miscellaneous conduits, 2 electrical boxes	36"W x 84"H door opening, many 5"-diameter penetrations grouped together	slab	slab	concrete wall, free along door jamb[a]	concrete wall[a]
CYDG1002	firewall	16	219	11.625	filled (except top course)	None	None	slab	slab	free along door jamb, continuous with (CYDG1001 and CYDG1003) elsewhere	same as side 1
CYDG1003	firewall	144	219	11.625	filled (except top course)	Miscellaneous conduits, 2 electrical boxes	36"W x 84"H door opening, many 5"-diameter penetrations grouped together	slab	slab	concrete wall, free along door jamb[a]	concrete wall
CYDG1004	firewall	83	146.5	11.625	filled (except area near bottom)	Miscellaneous conduits, 2 electrical boxes	None	concrete beam	slab	concrete wall[a]	concrete wall[a]
CYPAB101	partition, shield	1,002	132, 156	11.625	hollow	Several conduits	40"W x 86"H and 78"W x 110"H door openings	slab	slab	concrete column and wall[a]	concrete wall[a]
CYPAB102	shield	68	56	16	solid blocks (stacked)	None	None	free	slab	free	free
CYPAB103	shield	68	48	16	solid blocks (stacked)	None	None	free	slab	free	free
CYPAB104	shield	100	48	16	solid blocks	None	None	free	slab	concrete wall	free
CYPAB105	shield	170	222	18	solid blocks	None	None	slab	slab	concrete wall	free
CYPAB106	firewall	233	126	12	filled (except top course)	6 electrical boxes	16"-diameter opening	slab	slab	free	concrete wall
CYPAB107	shield	16	96	16	solid blocks	Fire extinguisher, electrical boxes	None	free	slab	concrete wall	free
CYPAB108	shield	16	96	16	solid blocks	None	None	free	slab	concrete wall	free
CYPAB201	firewall	418	202	11.625	solid (except top course)	Small conduits	None	steel beam	slab	masonry wall	steel column[a]

(continued)

TABLE 2.3 (Continued)

Wall ID Number	Function	Dimensions (in.)			Type of Cell	Significant Attachments	Significant Openings	Boundary Conditions			
		ℓ	h	t				Top	Bottom	Side 1	Side 2
CYPAB202 and CYSB1001	firewall	180	210	11.625	solid (except top course)	Hand ladder	None	steel beam	slab	masonry wall	steel column
CYPAB203	firewall	360	216	11.625	solid (except top course)	None	2 18" x 24" duct openings, 40"W x 84"H door opening	metal decking	slab	masonry wall	masonry wall
CYPAB204	firewall	400	210	11.625	solid (except top course)	Rail for double-wide door	Large duct, 158"H x 77"W door opening	steel beam	slab	masonry wall	free
CYTB1007	partition	348	184, 160	7.625, 11.625	hollow	Many attachments	30" x 30" opening for vent fan, numerous pipe penetrations in groups	slab	slab	masonry wall	masonry wall
CYTB1008	partition	2,400	228	7.625	hollow	Many attachments	Numerous openings	metal decking	slab	masonry wall	masonry wall
CYTB1009 and CYTB2004	partition	504	288	7.625	hollow	Numerous dial gauges and small conduits	42"W x 88"H door opening, miscellaneous pipe penetrations grouped together	slab	slab	masonry wall	masonry wall
CYTB1010	partition	480	228	7.625	hollow	Many attachments	A few pipe penetrations (D ≤ 12")	metal decking	slab	masonry wall	metal siding
CYTB2005	pipe chase column	--	--	--	hollow	None	None	free	slab	masonry wall, free at top	steel column
CYSB1002	partition	88	96	7.625	hollow	Metal siding mortared to one side	7" x 14" opening for vent duct	free	slab	steel column	free
CYSB1003	partition	488	210	11.625	hollow	Some heavy bulletin boards, small conduits	Some pipe penetrations and steel beam penetrations	metal decking	slab	free	free
CYSB2002	partition	773	210	7.625	hollow	Small conduits	Duct openings, 68"W x 86"H door opening	metal decking	slab	masonry wall	concrete wall
CYSB2003	partition	448	210	7.625	hollow	Small conduits	Numerous duct openings	metal decking	slab	free	masonry wall

a. Hairline crack (width ≤ 1/32") at boundary to masonry wall interface

TABLE 2.4

MATERIAL PROPERTIES OF CONNECTICUT YANKEE MASONRY WALLS

Material	Specification
Concrete Block Units:	
Exterior or Load-bearing Partitions	ASTM C90, Grade A
Non-Load-bearing Partitions	ASTM C129
Solid Block Units	ASTM C145, Grade B
Lightweight Block Units	ASTM C90, Grade B
Mortar	ATSM C270, Type N
Reinforcement	ATSM A305 and A15

lists the types of materials specified for construction of the masonry walls and the ASTM specification for each material.

The workmanship described in the specification was required to be of only the highest grade. Masonry blocks were to be kept dry while the walls were being constructed. The 3/8-in.-thick mortar joints were specified to have full mortar coverage on vertical and horizontal face shelves. Blocks that were to butt up solidly against the underside of the roof decking girders, and beams were to be mortared solidly in place along the joints. The mortar joints of all masonry walls were specified to be struck off flush against the wall surface. Bed joints were to be full and level and not furrowed. Vertical mortar joints were to be shoved tight.

Before starting the work, the contractor was to have furnished two samples of each type of concrete block for approval by Connecticut Yankee engineers. During the time of the wall construction, the engineers reserved the right to inspect the work.

The specifications also stated that the contractors were not to be relieved of any responsibility for quality, material, workmanship, correctness of details and dimensions, or any adjustments, changes, or repairs that may have been found necessary after delivery of the material. Furthermore, any work covered up without approval or consent of the engineering superintendent, or without necessary tests and inspection, could be required by the superintendent or public authorities to be uncovered for examination and testing.

In summary, it appears that the masonry walls at Connecticut Yankee were constructed in accordance with the masonry work specification and that they were constructed with the highest grade of workmanship and the assurance that the materials did not fall below the minimum specification requirements.

2.4 Priorities For Analysis

In accordance with the requirements of NRC IE Bulletin 80-11 (for the 60-day report), the safety-related masonry walls were assigned priorities for reanalysis of high, medium, or low. Within each of these three groups, individual priorities were established for each wall, and the walls were analyzed accordingly. The determination of the priorities for structural analysis included the following considerations:

- The importance of the safety-related equipment located either on a wall or near a wall. In general, equipment is safety-related if the equipment is necessary to shut the plant down safely during and after an earthquake or if the equipment can possibly fail and cause a release of radiation that could harm the public.

- The nature of the loading on each wall and the distribution of the loads. These loads included concentrated loads for equipment located on the wall, the distribution of loads from attachments on the wall, and inertial loads.

- Structural characteristics of the wall. Wall dimensions, wall thickness, whether the block cells were filled or hollow, whether the walls were stacked block walls or mortared block walls, the number of significant openings in the wall, and the conditions at the boundaries of the wall were the major considerations in this category.

3. DESIGN CRITERIA FOR ANALYSIS

Appendix A outlines in detail the design criteria used for analyzing the safety-related masonry walls at Connecticut Yankee and also contains a detailed commentary that provides justification for the use of some of the critical allowable stresses that are specified in the criteria.

Table 3.1 summarizes the important structural characteristics of the Connecticut Yankee masonry walls, and Table 3.2 gives the allowable stresses for the masonry walls according to the American Concrete Institute's code for concrete masonry structures (ACI 531-79), which are defined as the acceptance criteria of the design criteria document (see Appendix A).

Appendix B discusses the phenomenon of gapped arching action and its applicability to the safety-related masonry walls at Connecticut Yankee and presents the theoretical development of the actual equations used for the wall analysis.

Arching action can be considered in the analysis of the out-of-plane response of masonry walls provided the following conditions are met:

- The boundary elements are more rigid than the masonry wall.
- The structural boundaries around the masonry wall are able to withstand the loads imposed upon them by the arching action of the wall.
- There is adequate restraint against translational movement at the boundaries so that the wall ends do not move when the wall arches (e.g., wedging action or shear restraints allow the shear forces to be adequately transferred from the wall into the boundary support).

TABLE 3.1

STRUCTURAL CHARACTERISTICS OF MASONRY WALLS

AT CONNECTICUT YANKEE ATOMIC POWER PLANT

Item	Property
Blocks	Compressive Stress on Gross Area: ASTM C129: 350 lb/in.^{2} ASTM C90 Grade A: 1,000 lb/in.^{2}
ASTM C270 Type N Mortar	Compressive Stress m_o = 750 lb/in.^{2}
Masonry Walls	Weight Density = 150 lb/ft^3 Poisson's Ratio ν = 0.15 f'_m = 700 lb/in.^{2} Modulus of Elasticity E_m = $600f'_m$ = 420,000 lb/in.^{2} Modulus of Rigidity G = $0.4E_m$ = $240f'_m$ = 168,000 lb/in.^{2}

TABLE 3.2

ALLOWABLE STRESSES IN MASONRY WALLS AT CONNECTICUT YANKEE ATOMIC POWER PLANT ACCORDING TO ACI 531-79

Type of Stress	ACI Formula	Allowable Stress ($lb/in.^2$)
Flexural Compression	$0.33f'_m$	231
Bearing (on full area)	$0.25f'_m$	175
Bearing (on one-third area or less)	$0.375f'_m$	262.5
Flexural Tension Normal to Bed Joints -- Hollow Units	$0.5\sqrt{m_o}$	13.7
Flexural Tension Normal to Bed Joints -- Solid and Grouted Units	$1.0\sqrt{m_o}$	27.4
Flexural Tension Parallel to Bed Joints -- Hollow Units	$1.0\sqrt{m_o}$	27.4
Flexural Tension Parallel to Bed Joints -- Solid and Grouted Units	$1.5\sqrt{m_o}$	41.1
Shear for Flexural Members (no reinforcement)	$1.1\sqrt{f'_m}$	29.1
Shear for Shear Walls (no reinforcement)	$0.9\sqrt{f'_m}$	23.8

When a wall responds in an arching mode, a tension crack is assumed to have formed where the elastic moment and flexural tension stress are maximum. Blast tests, in which a wall cracks and responds by arching caused by out-of-plane loading, with several cycles of back-and-forth out-of-plane motion, have shown that the wall maintains its structural integrity throughout the entire arching response. Twenty-five years of theoretical development (see list of references in Appendix A) and testing indicate that arching action can be considered in the analysis of those masonry walls at the Connection Yankee plant that meet the conditions described above. Not all the Connecticut Yankee walls meet these conditions; however, when applicable, the arching action mode of response can be used to determine whether the wall will maintain its structural integrity with the boundary supports and not fall or impair the functional operation of any safety-related equipment.

4. PLANT MODIFICATIONS

Any safety-related wall which could not meet the analytical design criteria as outlined in Section 3, was modified by one of several different methodologies so that the resulting wall responses caused by the postulated loading conditions would not have an impact on nearby safety-related equipment. All of the necessary wall modifications were designed and drawn on structural drawings and all modifications have been completed.

REFERENCES

1. United States Nuclear Regulatory Commission, Masonry Wall Design, IE Bulletin 80-11, Washington, D.C., May 8, 1980.

2. Northeast Utilities Service Company, "QA Category I Material, Equipment and Parts List," Form ED3469, Prepared for Connecticut Yankee Atomic Power Plant by L. H. Levy, Rev. 1, June 26, 1980.

3. Stone & Webster Engineering Corporation, Specification for Masonry Work for Unit No. 1, Connecticut Yankee Atomic Power Plant, Connecticut Yankee Atomic Power Company, Haddam, Connecticut, Boston, Massachusetts, July 14, 1965.

APPENDIX A

CRITERIA TO REEVALUATE MASONRY WALLS

AT THE CONNECTICUT YANKEE ATOMIC POWER PLANT

CRITERIA TO REEVALUATE MASONRY WALLS AT THE CONNECTICUT YANKEE ATOMIC POWER PLANT

prepared for

Northeast Utilities Service Company
Under Purchase Order 706743

March 12, 1981

prepared by

URS/John A. Blume & Associates, Engineers
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

Rev. 1, 3/12/81

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1. INTRODUCTION

These criteria are applicable to the reevaluation of the masonry walls at the Connecticut Yankee Atomic Power Plant as required by the United States Nuclear Regulatory Commission (NRC) in IE Bulletin 80-11.

Characteristically, the masonry walls at this plant are constructed as non-structural partition and shield walls. None of these masonry walls have been incorporated into the lateral-load-resisting system or as major load-bearing walls. Furthermore, these masonry walls are predominantly composed of hollow, single-wythe construction with mortar joints between the masonry wall and adjoining structural elements.

All safety-related walls will be reevaluated for all loads and load combinations defined herein; the reevaluation will include the global response and the local transfer of the equipment loads to the walls. Revisions of these criteria to address special cases not covered in this document will be presented with the necessary supporting documentation to the NRC for review.

2. LOADS AND LOAD COMBINATIONS

2.1 Loads

The reevaluation will consider all relevant loads and load combinations specified in the FDSA for concrete design. The field survey of all safety-related masonry walls at the plant concluded that none of these walls is subjected to loads from tornados, winds, missiles, thermal gradients, pressure differentials, pipe whip, or jet impingement. Thus, only seismic loads must be considered in addition to dead and live loads. The following two procedures shall apply to seismic loads.

2.1.1 Wall Inertial Loads. Inertial loads due to earthquake loading will be based on the "Interim Seismic Design Ground Spectrum for CY" currently undergoing review by the NRC as part of the Systematic Evaluation Program (SEP). This site spectrum curve for the safe-shutdown earthquake (SSE) is only preliminary and utilizes a peak horizontal ground acceleration of 0.17g. Conservative estimates of amplifying the ground spectrum for various floor levels of different buildings will be made from this ground spectrum for analysis and design of the masonry walls. After the SEP detailed dynamic analysis is completed for the various buildings, the actual floor response spectrum values for the walls at different elevations in the various buildings will be compared to the assumed conservative estimates to ensure that the analysis and design of the masonry walls is adequate. Seismic input to the masonry walls will be as defined by the appropriate floor response spectra at the base and top of the wall; the maximum response from the use of either floor spectrum will be used in the analysis and design.

2.1.2 Equipment Inertial Loads. The equipment support loads imposed on the walls will be distributed on any wall in accordance with the method of equipment anchorage to the wall and the response of the attached equipment. Each significant load imposed on the wall from an attachment will be treated individually and will be determined on the basis of acceptable structural analysis methods. 1

2.2 Combination of Responses Due to Different Components of Ground Motion

Codirectional wall and equipment responses due to each horizontal component of ground motion will be combined on an absolute sum basis with responses due to the vertical component of ground motion.

2.3 Load Combinations

The reevaluation of masonry walls will consider the following load combinations:

Load Category	Load Factors			
	D	L	E	HE
Severe Environmental	1.0	1.0	1.0	
Extreme Environmental	1.0	1.0		1.0

where:

D = Gravity load of masonry wall and attachments

L = Live load

E = OBE inertial loads from masonry wall and attachments

HE = SSE inertial loads from masonry wall and attachments

3. MATERIAL PROPERTIES

For the determination of the strength and stiffness of the structures, the material properties will be taken as the minimum specified for the masonry, mortar, grout, and steel specified in the contract drawings and documents (see Table 1). Testing for material properties may be used in lieu of the minimum specified. Such testing will follow testing procedures defined in the American Concrete Institute (ACI) publication, *Building Code Requirements for Concrete Masonry Structures* (ACI 531-79). If testing other than that defined in ACI 531-79 is to be used, a description of the testing program and procedures for using test data will be submitted to the NRC for review and comment.

Damping in concrete masonry walls may be taken as 4% of critical for severe environmental load combinations, and 7% of critical may be used for extreme environmental load combinations.

TABLE 1

MATERIAL PROPERTIES - CY BLOCK WALLS

Item	Property
Concrete Block Units	
Exterior or Load-Bearing Partitions	ASTM C90, Grade A
Non-Load-Bearing Partitions	ASTM C129
Solid Block Units	ASTM C145, Grade B
Lightweight Block Units	ASTM C90, Grade B
Mortar	ASTM C270, Type N
Reinforcement	A305 and A15

4. ANALYSIS PROCEDURES

Masonry wall stresses shall be calculated on the basis of the following analysis procedures. Several acceptable procedures are defined herein ranging from the simplest and most conservative to the more rigorous and realistic. If more sophisticated analysis procedures are to be used, a description of such procedures will be presented to the NRC for review and comment.

4.1 Frequency Calculations

The fundamental frequencies of masonry walls will be calculated using values for modulus of elasticity as defined in Section 5; moments of inertia defined by the gross cross section, the mass of the wall, and all attachments; and with the appropriate boundary conditions. 1

4.1.1 Non-Load-Bearing Walls. Walls supported along all four edges will be assumed rigid (i.e., fundamental frequency greater than 33 Hz) in their own plane. In-plane frequency of all other walls will be calculated using standard dynamic analysis procedures. 1

4.1.2 Walls with Openings. The fundamental frequency of piers alongside openings in masonry walls will be calculated on the basis of a simply supported beam spanning the height of the opening and having the cross section of the pier.

4.1.3 Load-Bearing Walls. The fundamental frequency of load-bearing walls within the buildings will be calculated using standard dynamic analysis procedures, with adequate representation of any tributary masses.

4.2 Acceleration Response of the Walls

Given the seismic input to each wall as defined in Section 2 and the fundamental frequency of the wall, the applied loads for nonrigid walls (e.g., $f \leq$ Hz) on the wall shall be defined by a uniform load w_o, where: 1

$$w_o = 1.3 S_a m \qquad (1)$$

where:

w_o = Applied out-of-plane uniform load in the direction of ground motion

S_a = Spectral acceleration for the fundamental frequency of the wall. The value of S_a shall be from governing floor response spectra as defined in Section 2 and for wall damping as defined in Section 3.

m = Mass of wall

In the case where a fundamental frequency for the wall is not calculated, the value of S_a in Equation (1) shall be taken to be the peak of the floor response spectra for the appropriate damping value of the wall.

Applied loads, both horizontal and vertical, where the walls are rigid (e.g., $f \geq 33$ Hz) shall be defined by a uniform load w_p, where: △1

$$w_p = am \tag{2}$$

where:

w_p = Applied load in direction of ground motion

a = Zero-period acceleration from governing floor response spectra

m = Mass of wall

4.3 Level I Analyses

This level of analysis defines initial conservative procedures that may be used in the reevaluation of the masonry walls. More rigorous analytical techniques defined in this document may also be used to calculate the responses of the walls.

4.3.1 In-Plane Stresses. In-plane shear stresses in the walls and piers due to the wall and equipment loads will be calculated and compared to allowable stresses defined in Section 5.

4.3.2 In-Plane Relative Displacement. Masonry walls that are confined on the sides by lateral-force-resisting members (i.e., knock-out panels) shall be considered as infilled panels, and the response of these walls due to imposed in-plane relative displacements shall be calculated. A conservative estimate of the imposed shear stress τ is:

$$\tau = \frac{G\Delta}{h} \tag{3}$$

where:

τ = Shear stress in masonry panel

Δ = Relative displacement between top and bottom of wall

G = Shear modulus of masonry wall

h = Height of wall

4.3.3 Out-of-Plane. Out-of-plane moments on the walls shall be calculated for the wall and equipment loads. The moments shall be calculated considering the wall as a plate spanning in one direction only and simply supported at two ends. The elastic moment calculated must not impose extreme fiber stresses higher than the allowable tensile stress in mortar as defined in Section 5.

4.4 Level II Analyses

4.4.1 In-Plane Relative Displacements. An alternative and more accurate approach to the response of infilled panels than that described in Section 4.3.2 is to consider the composite action of the masonry wall and surrounding frame.

In general, this is a nonlinear problem since loads at the panel-frame interface can only be transmitted through compression, and the two components are free to separate from each other at this interface.

The diagonal tension in the panel due to the composite action of the panel and frame can be calculated with the use of a finite-element or finite-difference approach. This may be done by developing a composite frame-

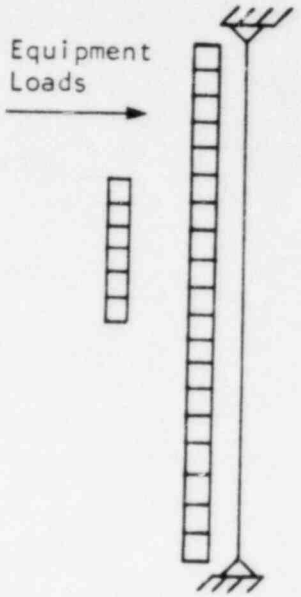
panel mathematical model with proper consideration of nonlinear interface conditions and then using the model to calculate the diagonal shear in the panel for the prescribed relative displacements.

4.4.2 Out-of-Plane. If the elastic out-of-plane moments calculated in Section 4.3.3 cause extreme fiber stresses higher than allowable, a crack perpendicular to the span of the wall will be assumed, and the capacity of the wall to resist the loads through arching action will be calculated as follows:

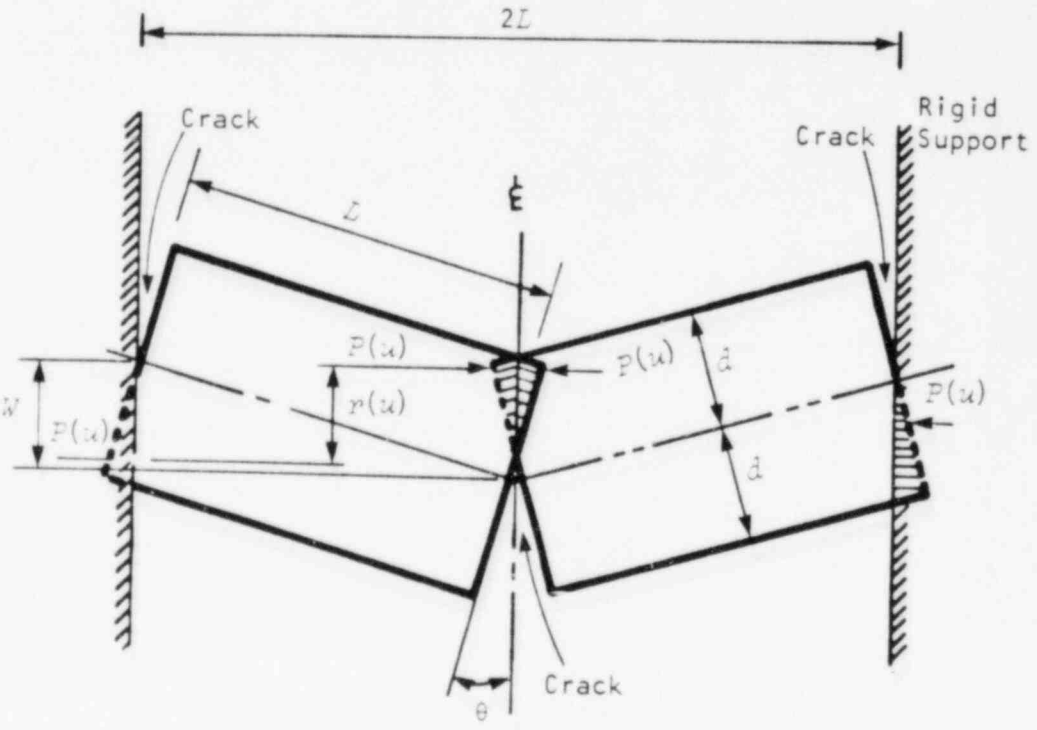
- The loading on the wall will remain as described in Sections 2 and 4.2 (see Figure 1a). This is conservative since a crack in the wall will render the wall more flexible and hence produce a smaller value of S_a in Equation (1) than that based on the elastic stiffness of the plate.
- A crack will be postulated perpendicular to the span of the wall (see Figure 1b) at the point of maximum elastic moment, and the wall capacity will be calculated on the basis of the moment $P(u)r(u)$, where:

  $P(u)$ = Compressive force developed at cracked boundaries (see Figure 1b)

  $r(u)$ = Moment arm between the two compressive forces (see Figure 1b)
- Maximum compressive stresses at the cracked sections and shear stresses in the wall shall be in accordance with the acceptance criteria defined in Section 5.
- All walls qualified on the basis of Section 4.4.2 shall have positive restraint against out-of-plane translation at the supports, as shown in Figure 2, or its equivalent.
- The maximum wall deflection (w) after initiation of cracking (see Figure 1b) shall not exceed 1/2 in.



a. Out-of-Plane Loads



b. Wall in Deflected Position

FIGURE 1 CRACKING FROM OUT-OF-PLANE STRESSES

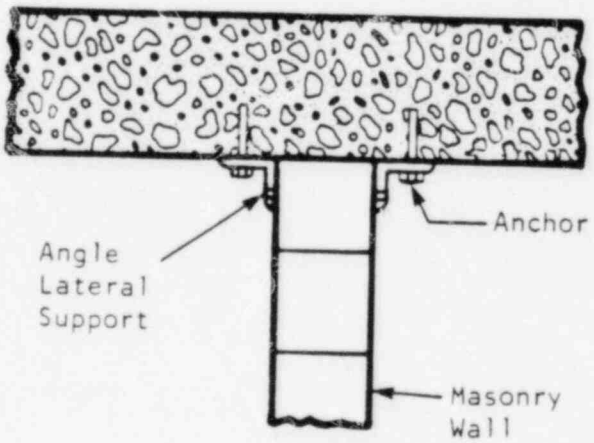


FIGURE 2 END SUPPORT FOR ARCHING ACTION

5. ACCEPTANCE CRITERIA

5.1 Masonry Construction

ACI 531-79 shall be used as the governing code for all allowables, except as follows:

- Modulus of Elasticity $E = 600f'_m$ but not greater than 3,000,000 psi
- Modulus of Rigidity $G = 240f'_m$ but not greater than 1,200,000 psi
- Minimum Factor of Safety (FS) to prevent sliding and overturning is 1.5.
- For OBE seismic loading, the allowable stresses can be increased by 33%.
- For SSE seismic loading, the allowable stresses can be increased by 67%.
- For pullout or punchout, the allowable punching shear stress is $2\sqrt{f'_m}$ for the masonry wall and $2\sqrt{m_o}$ for the mortar.

1

5.2 Steel Construction

All steel construction shall be in accordance with requirements of the American Institute of Steel Construction (AISC) in *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings*, 1979 edition.

6. ACCEPTANCE CRITERIA FOR MODIFICATIONS

All modifications to the masonry walls shall be in accordance with the acceptance criteria defined in Section 5.

Commentary on the Criteria to Reevaluate Masonry Walls at the Connecticut Yankee Atomic Power Plant

This document summarizes the results of various research studies performed in order to develop items discussed in Appendix A, specifically the ACI 531-79 allowable masonry stresses, the allowable stress increase, the modulus of elasticity, punching shear, sliding and overturning, and damping. The reference material reviewed as part of the research effort to help establish the reevaluation criteria is also listed and discussed.

ACI 531-79 Allowable Masonry Stresses

The allowable masonry stresses according to ACI 531-79 are specified as being the basis of the masonry acceptance criteria in Section 5.1 of the design criteria. Of the allowable stresses specified in the ACI code, the most significant stresses that controlled in determining the structural adequacy of the masonry walls were the stresses for flexural tension normal and parallel to mortar bed joints (for elastic analysis), flexural compression (for analysis of gapped arching action), and shear for shear walls and flexural members (for elastic analysis). Bearing, axial tension, and axial compression stresses were usually quite small in comparison with the values allowed by the ACI code.

A report by Jack R. Benjamin & Associates, Inc. (Attachment B), summarizes the results of a statistical analysis of all available test data to establish the safety margins inherent in the ACI allowable stresses. The statistical analysis focuses mainly on the allowable flexural tension, compression, and shear stresses, which are the critical stresses for determining the structural adequacy of the Connecticut Yankee masonry walls. The results of this study show that there are adequate margins of safety in the ACI allowable stresses when compared with the ultimate stresses that induce failure.

Allowable Stress Increase

Section 5.1 of the design criteria specifies that the allowable ACI normal working stresses are allowed to be increased by 33% for OBE loading and by 67% for SSE loading. The results of the statistical study by Benjamin show that these increases for the allowable stresses are reasonable and that the increased allowable stresses are still below the ultimate stresses. The

33% increase for the OBE in the allowable stresses is identical to the increase allowed in many structural codes (AISC, UBC, ACI) for short-term loading conditions. The allowable stresses can be increased by 67% for the SSE because the probability of occurrence of the SSE is relatively low, and, if it does occur, the allowable stresses can reach the ultimate or yield stresses. For structural steel, the 33% and 67% increase factors are also permissible (the AISC allows the 33% increase for wind or seismic loading); the 67% increase for the SSE will increase most of the allowable stresses to the yield stress (e.g., the normal allowable bending stress is $0.6F_y$, and $1.67 \times 0.6F_y = F_y$). However, the maximum allowable steel stress with the 67% increase will always be less than the material yield stress to prevent permanent inelastic ductile deformation.

Modulus of Elasticity

There is a wide variation in the available test data on the modulus of elasticity of masonry block (E_m). The Applied Technology Council's code value $E_m = 600f'_m$ is closer to the middle of the range than the ACI's value $E_m = 1{,}000f'_m$. The ACI and UBC are considering changing to the ATC value. Therefore, the design criteria specified the use of $E_m = 600f'_m$. A lower value of E_m yields higher calculated values of strains and deflections and lower values for frequency in masonry walls; however, the seismic loading could be smaller or larger when using a lower value of E_m, depending on where the calculated frequencies fall on a response spectrum.

Punching Shear

For pullout or punchout through masonry walls, the allowable punching shear stress is specified in Section 5.1 of the design criteria as $2\sqrt{f'_m}$ for the masonry block and $2\sqrt{m_c}$ for the mortar. There is no criterion for punchout in any of the available masonry codes; however, for ultimate strength design of concrete, the ultimate punching shear stress is specified as $4\sqrt{f'_c}$ (where f'_c is the compressive strength of the concrete). In converting this ultimate value to an equivalent allowable value, it is necessary to divide the ultimate value by an adequate factor of safety. Using a factor of safety of 2.0 gives an allowable value of $2\sqrt{f'_c}$ for concrete. Substituting the compressive strength of the masonry wall (f'_m) for f'_c gives an allowable value of $2\sqrt{f'_m}$ for normal loads. The 33% and 67% increase factors apply for increasing the

allowable stresses for the OBE and SSE loading combinations, respectively. The actual shear stress due to punchout or pullout is calculated as the actual load divided by the surface area of the pullout pyramid or cone (a procedure similar to checking for proper embedment of anchor bolts or finding the required thickness of a concrete footing supporting a column load).

Sliding and Overturning

As stated in Section 5.1 of the design criteria, the minimum factor of safety to prevent sliding and overturning of the masonry walls is specified as 1.5. This minimum factor of safety is to be used for both OBE and SSE loading. When normal design loads are resisted by structural members (retaining walls, buried structures, etc.) a factor of safety of 2.0 is normally used. However, since the OBE and SSE loadings are applied only for short periods of time, use of a factor of safety of 1.5 is reasonable to ensure that a masonry wall does not slide or overturn. Moreover, the NRC indicates that factors of safety for overturning and sliding of 1.5 for the OBE and 1.1 for the SSE are acceptable (*Standard Review Plan*, NUREG-75/087, Section 3.85, Subsection II.5). A coefficient of friction (u) of 0.8 for masonry against steel or concrete can be used for calculating the resisting friction force to prevent sliding.

Damping

Damping values ranging from 8% to 10% were determined from tests of double-wythe masonry walls built with solid unreinforced brick (see Reference 7), in which walls were loaded out of plane until the first cracks were formed. The results suggest that a damping value larger than 7% for the SSE could possibly be justified for concrete masonry walls. No test data were found that discussed damping values for single-wythe masonry walls.

Section 3.0 of the design criteria specifies that 4% and 7% of critical damping can be used for the OBE and SSE, respectively. These values were selected on the basis of the acceptable values for reinforced concrete structures specified in NRC Regulatory Guide 1.61, *Damping Values for Seismic Design of Nuclear Power Plants*.

Reference Material

Following this section is a list of the reference material reviewed as part of the research effort to help establish the reevaluation criteria. The specific information obtained from the references and its application are summarized in this section.

Benjamin and Williams[1] investigated the effectiveness of the unreinforced brick masonry walls, both without bounding frames and with reinforced concrete or structural steel frames, in resisting horizontal racking loads. Smith and Carter[2] established a procedure to predict the ultimate racking load that can be applied to a masonry wall and checked the effectiveness of this approach by using Simms's[3] racking test results. Williams[4] performed several tests to determine the racking strength of reinforced hollow concrete block walls and brick walls. He also studied the influence of precompression on the ultimate strength of the masonry walls. Mainstone and Weeks[5] have conducted several racking tests on solid brick walls bounded by concrete-encased steel frames to investigate the influence on the composite stiffness of the stiffness and modes of deformation of the frames relative to that of the walls. Schrivener[6] did a series of tests on hollow unreinforced concrete masonry walls subjected to racking loads. The main purpose of his tests was to study the effect on the strength of walls of varying the reinforcement ratio and the spacing of reinforcing bars. These references provided valuable experimental data useful to the study of the racking of masonry panels.

Using elasticity theory, Blume and Proulx[7] derived a formula to determine the maximum diagonal tension and nominal shear stress for the center element of a square panel subjected to a diagonal compressive load. This derivation was helpful in the determination of allowable diagonal tension in masonry panels.

Smith et al.[8] did finite-element studies of bricks in diagonal tension that were useful in providing insight into the stress patterns that occur in racked masonry panels. Smith[9] also derived a method for calculating an equivalent diagonal strut to model the behavior of masonry panels. Although

it was based on limited data, this method could provide a useful modeling technique if further research is done.

References 10 through 15 gave useful background information to the studies of the racking of masonry panels.

McDowell et al.[16] developed an arching action theory for masonry walls subjected to transverse bending and correlated this theoretical work to experimental results that were unexplainable by conventional analysis. His theory formed the basis of the analysis of the out-of-plane arching of masonry wall panels. Cohan and Laing[17] presented a simplified theory in their discussion of McDowell's paper using a stress block rather than an elasto-plastic model. Eremin,[18] Cohan and Laing,[19] and McKee and Sevin,[20] investigated some of the design aspects of the arching of walls in their work.

Gabrielsen et al.[21-24] did shock tunnel testing of arching walls, providing data on their cyclic performance and ultimate strengths.

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Approvals

Principal in Charge
John W. Reed

Signature: John W. Reed Date 4/6/81

Project Manager
John W. Reed

Signature: John W. Reed Date 4/6/81

Director of Quality Assurance
Fred A. Webster

Signature: Fred A. Webster Date 4/6/81

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SUMMARY AND CONCLUSIONS

Review of available literature and statistical analyses for unreinforced block wall test data was performed for the following types of allowable stresses given in the ACI 531-79 Code:

- Compression
- Tension normal to bed joint and parallel to bed joint
- In-plane shear

Because of the charateristics of block walls being analyzed by URS/Blume in response to the requirements of NRC IE Bulletin No. 80-11, only unreinforced block walls constructed with running bond were considered.

A statistical analysis of data for each stress type produced a factor of safety and data dispersion statistic. The data dispersion statistic was defined to be equal to the number of standard deviations between the mean and the allowable stress.

Factors of safety and dispersion statistic values greater than two provide an adequate basis for defending the ACI 531-79 allowable values for existing block walls. Based on this criterion and engineering judgment, it is concluded that the following ACI 531-79 Code allowable stress values can be safely used:

Compression:	$0.33f'_m$
Tension parallel to bed joint	
Hollow units:	$1.0\sqrt{m_o}$
Solid or grouted units:	$1.5\sqrt{m_o}$
In-plane shear:	$0.9\sqrt{f'_m} \leq 34$ for $M/Vd_r \geq 1.0$

where f'_m is the block prism compressive strength, and m_o is the mortar cube compressive strength.

The test data for tension normal to the bed joint produced mixed results. Is is, therefore, concluded that the allowable values given in the ACI 531-79 for this stress type may not be reliably conservative. However, since arch action can be used in the event that computed stresses for tension normal to the bed joint exceed allowable values, this may not be a serious limitation. Finally, it is recommended that the allowable in-plane shear stress for M/Vd_r ratios less than 1.0 also be based on the formula, $V_m = 0.9\sqrt{f'_m} \leq 34$.

INTRODUCTION

Review of available literature and statistical analyses for unreinforced block wall test data were performed to develop a technical justification for using the allowable stresses given in the American Concrete Institute Code: Building Code Requirements for Concrete Masonry Structures (ACI 531-79) (Ref. 1). This project supports work currently being conducted by URS/John A. Blume & Associates, Engineers (URS/Blume) in response to the requirements of USNRC IE Bulletin No. 80-11 (Ref. 2), which concerns the structural adequacy of existing safety-related masonry walls in nuclear power plants.

Based on discussions with URS/Blume engineers, it was decided that the following stress categories would be reviewed since they directly affect the structural analyses currently being conducted for block walls in the Connecticut Yankee Atomic Power Plant (Haddam Neck):

- Compression
- Tension normal to bed joints and parallel to bed joints
- In-plane shear

Because of the characteristics of the block walls in this plant, only allowable stresses for unreinforced block walls constructed with running bond were considered.

Procedure

Reports and technical papers available in the open literature were reviewed, and data obtained from testing block walls and masonry components (i.e., block units and mortar) were statistically analyzed to provide a rational basis for justifying the allowable stresses given in ACI 531-79.

Although some references classify allowable stresses based on the type of block or mortar, it was found to be more meaningful to perform the analyses by first normalizing the data with respect to the appropriate allowable stress. An example of the statistical analysis procedure is shown in Figure 1. For the example data set shown, a statistical analysis of the test values (Case I) produces a mean and standard deviation (i.e., 159 psi and 29 psi, respectively). These values in turn are used to obtain two measures of conservatism for the assumed allowable value of 60 psi. The first measure is the factor of safety, which is equal to the mean value divided by the allowable value. This is a traditional parameter which has been used in the past to describe the level of conservatism. The second measure of conservatism, which is the number of standard deviations between the mean and allowable value, is obtained by subtracting the latter two values and dividing the result by the computed standard deviation. This is a meaningful measure since it is directly related to the probability of strength values being less than the allowable value. While the factor of safety measures the average relationship, the dispersion statistic reflects both the average trend and the standard deviation of the data.

A second analysis case is also shown in Figure 1 where the ratios of test values to the allowable value are statistically analyzed. (This is the normalizing procedure referred to above.) Note that the allowable value corresponds to R equal to 1; thus the mean and factor of safety are the same for this case. Both analysis procedures give the same results for the example shown in Figure 1; however, the approach demonstrated by Case II is used when the allowable values vary from test to test, since the Case I approach cannot be applied. Because this is the general situation for the block wall test data which were encountered, the method of analyzing ratios was used in the statistical analyses presented in this report.

In summary, the procedure used to analyze the block wall test data found in the various references was to first divide each test value by

the corresponding allowable value based on the ACI 531-79 Code. This required for each test specimen that a corresponding block or mortar strength value be available, since the allowable ACI Code stresses are functions of either the block compressive strength, f'_m, or the mortar compressive strength, m_o. Once these ratios were obtained, a statistical analysis was performed to obtain the factor of safety and dispersion statistic as described above.

Report Organization

The statistical analyses and results of the study for compression, tension, and shear are presented in the subsequent sections of this report. Summary and conclusions are given in the first report section.

EXAMPLE DATA

Test Value	Allowable Value	Ratio, R
125 psi	60 psi	2.08
150	60	2.50
175	60	2.92
145	60	2.42
200	60	3.33

Case I - Analysis of Test Values

Mean	= 159 psi	
Standard Deviation	= 29 psi	
Factor of Safety	= 2.7	$(\frac{159}{60})$
Dispersion Statistic (Number of Standard Deviations to Allowable Value)	= 3.4	$(\frac{159-60}{29})$

Case II - Analysis of Ratios

Mean	= 2.7	
Standard Deviation	= .484	
Factor of Safety (i.e., mean)	= 2.7	
Dispersion Statistic (Number of Standard Deviations to Allowable Value) (i.e., R = 1)	= 3.4	$(\frac{2.7-1}{.484})$

Figure 1. Example Statistical Analysis

COMPRESSION

Test data for compression of unreinforced masonry walls are summarized in Reference 3 which was prepared by the National Concrete Masonry Association (NCMA). This document provides failure compression stress test data and associated compression values from block prism tests (i.e., f'_m) for data reported in the open literature. Both pure axial compression and eccentric load test data are given.

Reference 4 gives a summary of data produced for wall arch-action tests. Data for line loading compression tests of prisms (to simulate the interface between two sections of the same wall at a hinge point), two-point wall static tests, and wall blast load tests are given. These data are particularly pertinent since the structural computations being performed use allowable compression to determine the capacity of walls in arch action. Since only unit block capacities were available in Reference 4, f'_m was estimated based on assumed strength for the mortar used in the arch-action tests.

Table 1 summarizes the results of the statistical analysis of the compression stress test data. Both the axial compression stress allowable value, F_a, and the flexural compressive stress allowable value, F_m, given in ACI 531-79 were investigated in the analysis. Thus, two sets of factors of safety and dispersion statistics are given in Table 1. The results for the "Combined Data" shown at the bottom of Table 1 are based on data from all the referenced sources combined in a single statistical analysis for each of the two allowable values.

It was found for the large data set corresponding to the pure compressive tests (i.e., 159 data points) that the distribution of ratios (i.e., test value to allowable value) is skewed toward higher values. Using a lognormal distribution it was found that the number of geometric standard deviations for F_a and F_m is equal to 5.0 and 3.4,

respectively. These values are higher than the values given in Table 1 (i.e., 2.8 and 2.3, respectively), which are independent of any distribution, and indicate that the degree of conservatism is probably greater than reflected in Table 1.

It should be noted that the smallest test value in all the tests exceeds the F_a and F_m allowable values by factors of 2.2 and 1.5, respectively, which provides additional assurance that the ACI allowable values are conservative.

Based on comparing the results of the statistical analysis with the criteria set forth in the summary and conclusions, it is concluded that the F_m allowable value given in the ACI 531-79 Code can be safely used in determining the capacities of unreinforced block walls for arch action.

Table 1. Summary of Compression Stress Statistical Analysis

Data Source	Number of Test Values	Allowable Value $F_a = 0.225 f'_m [1-(\frac{h}{40t})^3]$ Factor of Safety	Allowable Value $F_a = 0.225 f'_m [1-(\frac{h}{40t})^3]$ Number of Standard Deviations	Allowable Value $F = 0.33 f'_m$ Factor of Safety	Allowable Value $F = 0.33 f'_m$ Number of Standard Deviations
NCMA Data (Ref. 3)					
Pure Compression Tests	159	3.7	2.8	2.5	2.3
Eccentric Load Tests	37	4.6	5.1	3.1	4.4
URS/Arch Wall Data (Ref. 4)					
Line Loading Static Tests	4	8.2	6.0	5.6	5.6
Two-Point Static Tests	2	2.2	4.1	1.5	2.5
Blast Tests	1	3.9	--	2.7	--
Combined Data	203	3.9	2.9	2.6	2.4

TENSION

Allowable stresses for tension perpendicular to the bed joint (i.e., for walls spanning vertically) and tension parallel to the bed joint (i.e., for walls spanning horizontally) were compared with rupture modulii obtained from flexural wall tests. Although summaries of test data are given in Reference 3, no corresponding values for mortar cube compressive stress, m_o, are given. The various references cited in Reference 3 were obtained and reviewed. Several discrepancies were found as discussed below.

Tension Perpendicular to Bed Joint

Modulii of rupture values for hollow units are given in Reference 3 for tests conducted by Hedstrom (Ref. 6), Copeland (Ref. 7) and the NCMA. Based on a telephone conversation with R. E. Van Langingham, Technical Services Engineer at NCMA, it was learned that supporting documentation for the NCMA test data given in Reference 3 is not available. Since m_o values cannot be provided, these data could not be analyzed using the procedures adopted for this study.

In Reference 3, the data produced by Hedstrom were classified as either N or O type mortar. However, the original paper (Ref. 6) lists the mortar as M or S. Although the mix properties satisfy the ASTM C270 requirements for mortar type based on material proportions, the cube compressive strengths also satisfy the ASTM C270 property specifications which consist of minimum stress values. In the statistical analyses performed for this study the measured m_o strength values from the tests were used to compute the corresponding allowable values based on the following formulas given in ACI 531-79:

Tension normal to bed joints

Hollow units: $F_t = 0.5\sqrt{m_o}$

Solid or grouted units: $F_t = 1.0\sqrt{m_o}$

Thus, the mortar type (i.e., M, S, N, or O) is not pertinent to the analysis.

Conspicuously absent from the data given in Reference 3 are test results obtained by Fishburn (Ref. 8). Although Fishburn's results are used for other types of stresses discussed in Reference 3, no reference is made to Fishburn's test results for tension normal to the bed joint, which are low relative to other test data. One possibility for discounting Fishburn's results is that all tests were made at 15 days rather than the usual 28 days. But since both the flexure tests and mortar cube strengths were conducted at 15 days, both the strength and allowable values are comparable. It can be argued that since tension is proportional to the square root of m_o, testing at 15 days may produce unconservative results; however, the opposite situation occurred since Fishburn's data give relatively low tension values compared to the allowable stresses. Fishburn's data were included in the analysis.

Yokel (Ref. 5) tested walls in flexure which included compression loads. However, six of the tests had no compression loads and corresponded to the configuration being analyzed in this study. Thus, these data were also included in the analysis.

Table 2 lists the results of the statistical analyses for tension stresses normal to the bed joint. The results are listed by individual data source, and statistics are given for the case where all sources were combined in a single statistical analysis. Fishburn's data (27 data points) tend to dominate the results. Both Fishburn's and Hedstrom's data lead to low results, which suggest caution. In contrast, the results based on Yokel and Copeland produce acceptably high statistical values.

It is concluded that the allowable values given in the ACI 531-79 may not be reliably conservative. However, since arch action can be used in the event that computed stresses for tension normal to the bed joint exceed allowable values, this may not be a serious limitation.

Tension Parallel to Bed Joint

For block walls laid in running bond, bending in the horizontal direction is not as critical as in the vertical direction. Reference 3 implies that allowable tension stresses parallel to bed joints traditionally have been made equal to twice the allowable values for tension normal to bed joints for hollow units. Available data support the conclusion that this practice is conservative. The ACI 531-79 Code gives the following allowable values for tension parallel to bed joints:

Hollow units:	$F_t = 1.0\sqrt{m_o}$
Solid or grouted units:	$F_t = 1.5\sqrt{m_o}$

Note that the value for solid units is less than twice the allowable value for tension normal to bed joints.

Table 3 gives a summary of the results of the statistical analysis for tension stresses parallel to bed joints. Although the Fishburn data result in a factor of safety slightly less than 2, it is judged that these results are acceptably conservative. Livingston's data reinforce the viewpoint of many researchers that the strength of walls with running bond in the horizontal direction is a function of the block strength rather than the mortar strength. This is probably true since failure occurs not only along joints, but also through the blocks themselves. Because of this result the data for tension stresses parallel to the bed joint were not combined into a single analysis.

Based on the criteria given in the summary and conclusions section, it is concluded that the allowable values for tension stresses parallel to bed joints given in ACI 531-79 can be safely used.

Table 2. Summary of Statistical Analysis for Tension Stresses Normal to Bed Joint

Date Source	Number of Test Values	Factor of Safety	Number of Standard Deviations
Hedstrom (Ref. 6)	11	1.7	1.5
Copeland (Ref. 7)	8	3.0	3.1
Yokel (Ref. 5)	6	2.8	1.7
Fishburn (Ref. 8)	27	1.4	0.9
Combined Data	52	1.9	0.9

Table 3. Summary of Statistical Analysis for Tension Stresses Parallel to Bed Joint

Date Source	Number of Test Values	Factor of Safety	Number of Standard Deviations
Hedstrom (Ref. 6)	7	2.5	2.7
Fishburn (Ref. 8)	6	1.9	2.1
Livingston (Ref. 9)	24	9.4	2.0

SHEAR

Test data for racking tests of unreinforced, ungrouted concrete block masonry wall construction are very limited. From Whittemore, et al. (Ref. 10), Schneider (Ref. 11), and Fishburn (Ref. 8), shear test data were extracted for a total of 18 racking tests. For each racking test the allowable stress was compared to the shear stress at first crack and to the shear stress at the ultimate load. Although first crack was considered to be the important stress type in this study, the analysis of the ultimate shear strength was included to indicate that the shear capacity is significantly greater than the first crack capacity.

In the above comparisons, allowable stresses were defined by two methods. The first method defined the allowable stress, v_m, according to the formulas given in Table 10.1 of ACI 531-79. These formulas are:

$$v_m = 0.9\sqrt{f'_m} \leq 34 \qquad M/Vd_r \geq 1 \qquad (1)$$

$$v_m = 2.0\sqrt{f'_m} \leq 40(1.85 - M/Vd_r) \qquad M/Vd_r < 1 \qquad (2)$$

Based on a review of the racking test data, the ratio, M/VD_r, in all cases was less than or equal to 1.0. Thus, only formula (2) above was used to determine allowable stress. In the second method of determining the allowable shear stress, only the first formula above is used, regardless of the M/Vd_r ratio.

The Whittemore, et al. tests included compression tests in addition to racking tests for each type of wall. To determine the allowable shear stress, the value of f'_m was assumed to be equal to the average compressive strength of the walls, i.e., 922 psi. This value of f'_m is compatible with the compressive strength of the concrete block used in the tests (approximately 2,000 psi on the net area) and the mortar type (i.e., type N). Based on this value of f'_m, and using the second

method of determining the allowable shear stress, 27.3 psi was computed to be the maximum value permitted. Similarly, an allowable value was computed using the ACI 531-79 equations. In determining the shear stress on the walls, the loads given were divided by the net area of the block which was assumed to be twice the thickness of the face shell (i.e., 2 x 1.08 inches) times the wall length.

In the Schneider tests, the f'_m value could not be taken directly from the data since these tests did not include compression tests for full wall units. However, data on the average compressive strength of the block (i.e., 2,962 psi) were provided, and with the mortar type (i.e., type S) Table 4.3 of ACI 531-79 was used to determine an f'_m value of 1,687 psi. Based on this value of f'_m, and using the second method of determining the allowable shear stress, 34 psi is the maximum value permitted. Since the Schneider tests had different M/Vd_r ratios (i.e., M/Vd_r = .2, .5, or 1.0), the allowable stresses based on the ACI 531-79 equations ranged from 34 psi to 66 psi.

Fishburn provided the most extensive racking test data for unreinforced, ungrouted masonry walls. These tests were concerned with mortar properties and their effects on masonry strength. Both type S and type N mortar were used in these racking tests, and each mortar type was mixed so that some tests had a mortar type of moderate strength and high water retention while an equal number of tests used the same mortar type but were mixed to have higher compressive strength but less water retention and air content. Thus, the effects of the variability of mortar strength/water content for a given mortar type was included in the results of these tests. The Fishburn tests included 15-day compression tests of each type of wall. The value of f'_m used to determine the allowable stress was taken from the average compressive strength of walls of common mortar type/water content. The compressive strength for the type N mortar walls ranged from 1,030 to 1,130 psi and for the type S mortar walls from 1,160 to 1,240 psi. The allowable stresses using the second method ranged from 28.9 to 31.7 psi. Similarly, an allowable

range of values was computed using the ACI 531-79 equations. In determining the shear stress on the walls, the load given was divided by the net area of the block wall which was stated in the Fishburn report to be 38% of the gross area of the block. The Fishburn report does not provide first crack data but does state that cracks first appear at about 60% of the maximum load. Thus, first crack data were assumed to be 60% of the ultimate for each test.

Table 4 lists the results of the statistical analysis for shear stress. The results are listed by individual data source, and results for walls of mortar types S and N are separated for the Fishburn data. Statistics are given for each data source and mortar type, although the number of data values (i.e., 3 or 6) makes the estimate of the dispersion statistic unreliable.

Statistical parameters are also given for the "Combined Data" analysis. The results of this analysis indicate that the allowable shear values computed by the ACI 531-79 formula for M/Vd_r ratios less than 1.0 may not be reliably conservative in relationship to determining the strength at first crack (factor of safety = 1.1). In contrast, the use of the second method of determining allowable shear stresses produces a factor of safety of 2.4 in determining the first crack. The relatively low dispersion statistic (i.e., 1.1) corresponding to the 2.4 factor of safety is a result of mixing the Schneider data with the Whittemore et al. and Fishburn data. The Schneider data have significantly higher shear values at first crack than either the Whittemore, et al. or Fishburn data. This results in greatly increasing the standard deviation or uncertainty for the "Combined Data," while only moderately increasing the combined factor of safety.

Based on the results of analyzing the shear stress test data, it is recommended that the second method of determining allowable shear stresses, namely $V_m = 0.9\sqrt{f'_m} \leq 34$, be used in analyzing the in-plane behavior of unreinforced block walls.

Table 4. Summary of Shear Stress Statistical Analysis

Data Source	Number of Test Values	Allowable Value: ACI 531-79 First Crack Factor of Safety	ACI 531-79 First Crack Number of Standard Deviations	ACI 531-79 Ultimate Factor of Safety	ACI 531-79 Ultimate Number of Standard Deviations	$0.9\sqrt{f'_m} \leq 34$ First Crack Factor of Safety	$0.9\sqrt{f'_m} \leq 34$ First Crack Number of Standard Deviations	$0.9\sqrt{f'_m} \leq 34$ Ultimate Factor of Safety	$0.9\sqrt{f'_m} \leq 34$ Ultimate Number of Standard Deviations
Whittemore, et [illegible] (Ref. 10)	3	0.92	-.68	1.1	1.1	2.0	4.0	2.5	5.2
Schneider (Re[illegible]	3[1]	2.6	1.7	3.2	3.5	4.6	1.6	4.7	2.8
Fishburn (Ref. 8)									
Type N Mortar	6	0.79	-1.2	1.3	1.2	1.8	2.1	2.9	3.1
Type S Mortar	6	1.1	0.94	1.8	5.5	2.4	7.1	4.0	8.8
Combined Data	18[1]	1.1	0.20	1.8	1.0	2.4	1.1	3.5	2.5

[1] Only two test values given for first crack by Schneider (Ref. 11).

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APPENDIX B

Gapped Arching Action: Theory and Procedure

The hand calculations that make up most of this appendix were performed to show the derivation of the theoretical equations used in the structural analysis of walls for the out-of-plane gapped arching action mode of response.

The assumptions made for the development of the equations are:

- The rigidity of the boundary supports is greater than the out-of-plane rigidity of the wall.
- The boundary supports can adequately withstand the reaction loads (shear and compressive) transferred from the ends of the wall.
- Structural steel angles (or the equivalent) will be provided as simple pinned supports for the ends of the wall so that the wall ends cannot translate and so that the shear reaction at the wall ends can be adequately transferred to the adjacent boundary supports.
- a 1/16-in.-wide crack at the top of any wall (1/16-in.-wide crack for any wall height) will account for the possible shrinkage of the top mortar joint of the wall.

For walls where an observed crack at the top of a wall exceeds 1/16 in. in width, or where the boundary supports are not rigid enough, equivalent gaps larger than 1/16 in. can be substituted for the 1/16 in. used in the equations presented to develop the specific equations for any particular case.

The version of gapped arching action presented in this appendix is slightly different from that presented in typical reference papers (e.g., Reference 24 of Appendix D, Attachment A) in that the wall ends are prevented from translating before the wall end supports make contact with the boundary supports.

A search of existing literature containing test results and theoretical explanations for arching and gapped arching action response indicated that rigid arching walls can resist six to eight times the loads that gapped arching walls can withstand but that gapped arching walls are still considerably stronger than walls mounted as simple elastic beams. Reports of results of blast tests, in which a wall cracks and responds by arching (caused by out-of-plane loading) with many cycles of back-and-forth motion, show that the wall maintains its structural integrity throughout the entire arching response.

Therefore, there is sufficient documentation that analysis of walls for rigid and gapped arching action modes of response is justified and that the test results can be explained by analytical structural theories when the appropriate requirements for arching action are satisfied.

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SHEET NO. 1

JOB NO. 8540-03 JOB CY BLOCK WALLS BY LC DATE 10/14/80

CLIENT NUSCO SUBJECT SHRINKAGE EFFECT CHK'D JSM DATE 10-29-80

Range of wall height in CY : 10 ft - 18 ft

Shrinkage strain = 0.001 in/in
(Ref: Winter & Nilson, "Design of Concrete Structures" 9th Ed, P. 26)

Say the creep strain is in the same range

Total shrinkage = 2 x 0.001 = 0.002 in/in

All CY blocks are 7 5/8" and block joints are 3/8" thick.

$$\Delta = \frac{3/8}{7\,5/8} \times (.002) \times 12 \times H$$

$$= 0.05 \times .002 \times 12 \times H$$

$$\Delta = 0.0012\ H$$

where Δ = gap
H = wall height in ft

for H = 25 ft

$$\Delta = 0.0012\,(25) = 0.03" = \frac{1}{33}"$$

For the uncertainty involved, we use the conservative value of 1/16" for gap size.

DERIVATION OF EQUATIONS

INITIAL STAGE

1. Midspan crack starts forming
2. One of the top corner just start touching top support.

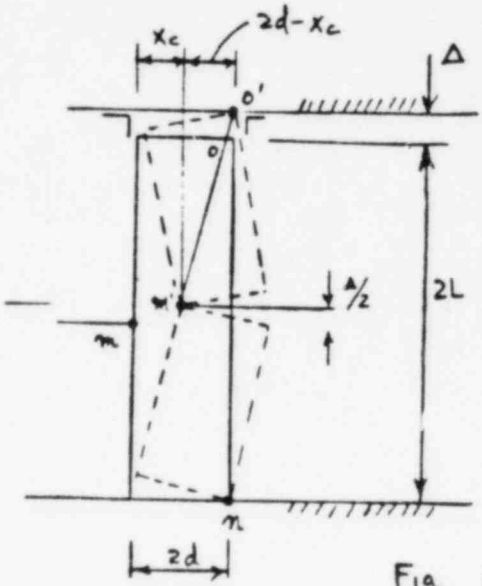


Fig 1a

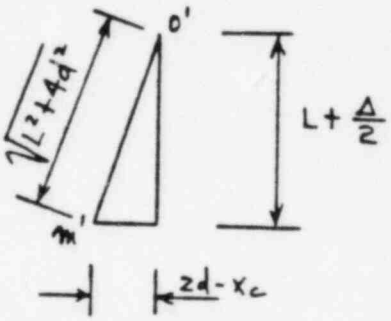


Fig 1b

Assumptions :

1. No horizontal and vertical forces developed
2. o' is directly above o
3. Crack developed at mid-section.
4. No deformation in the block wall.
5. Two sections of the wall behave like two rigid links with hinges at location o', m', and n

In Fig. 1b

$$\left(\sqrt{L^2+4d^2}\right)^2 = \left(L+\frac{\Delta}{2}\right)^2 + \left(2d-X_c\right)^2$$

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SHEET NO. 3

JOB NO. 8040-03 JOB CY BLOCK WALLS BY LC DATE 10/14/80

CLIENT NUSCO SUBJECT ARCH THEORY FOR WALLS WITH GAP CHKD DATE 10/14/80

$$L^2 + 4d^2 = X_c^2 + L\Delta + \frac{\Delta^2}{4} + 4d^2 - 4dX_c + X_c^2$$

$$\frac{\Delta}{4} + L\Delta + (X_c^2 - 4dX_c) = 0 \quad \cdots\cdots \text{EQN 1'}$$

$$\Delta = \frac{-L \pm \sqrt{L^2 - 4(\frac{1}{4})(X_c^2 - 4dX_c)}}{2 \cdot \frac{1}{4}}$$

$$= -2L \pm 2\sqrt{L^2 - (X_c^2 - 4dX_c)}$$

since Δ can not be negative

$$\boxed{\Delta = -2L + 2\sqrt{L^2 + (X_c^2 - 4dX_c)}} \qquad \text{EQN 1}$$

If we want X_c, then rearrange EQN 1'

$$X_c^2 + 4dX_c + (L\Delta + \frac{\Delta^2}{4}) = 0$$

$$X_c = \frac{-(-4d) \pm \sqrt{(-4d)^2 - 4(1)(L\Delta + \frac{\Delta^2}{4})}}{2}$$

$$= 2d \pm \sqrt{4d^2 - (L\Delta + \frac{\Delta^2}{4})}$$

since X_c can not be greater than $2d$

$$\boxed{X_c = 2d - \sqrt{4d^2 - (L\Delta + \frac{\Delta^2}{4})}} \qquad \text{EQN 2}$$

Equations for Various Block Sizes with 1/16" Gap

16 x 8 x 6 block

$2d = 5\frac{5}{8} = 5.625"$

$$X_c = (5.625) - \sqrt{(5.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 5.625 - \sqrt{31.6396 - 0.0625L}$$

16 x 8 x 8 block

$2d = 7\frac{5}{8} = 7.625"$

$$X_c = (7.625) - \sqrt{(7.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 7.625 - \sqrt{58.1396 - 0.0625L}$$

16 x 8 x 12 block

$2d = 11\frac{5}{8} = 11.625"$

$$X_c = (11.625) - \sqrt{(11.625)^2 - \left(0.0625L + \frac{(0.0625)^2}{4}\right)}$$

$$= 11.625 - \sqrt{135.1396 - 0.0625L}$$

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SHEET NO. 5

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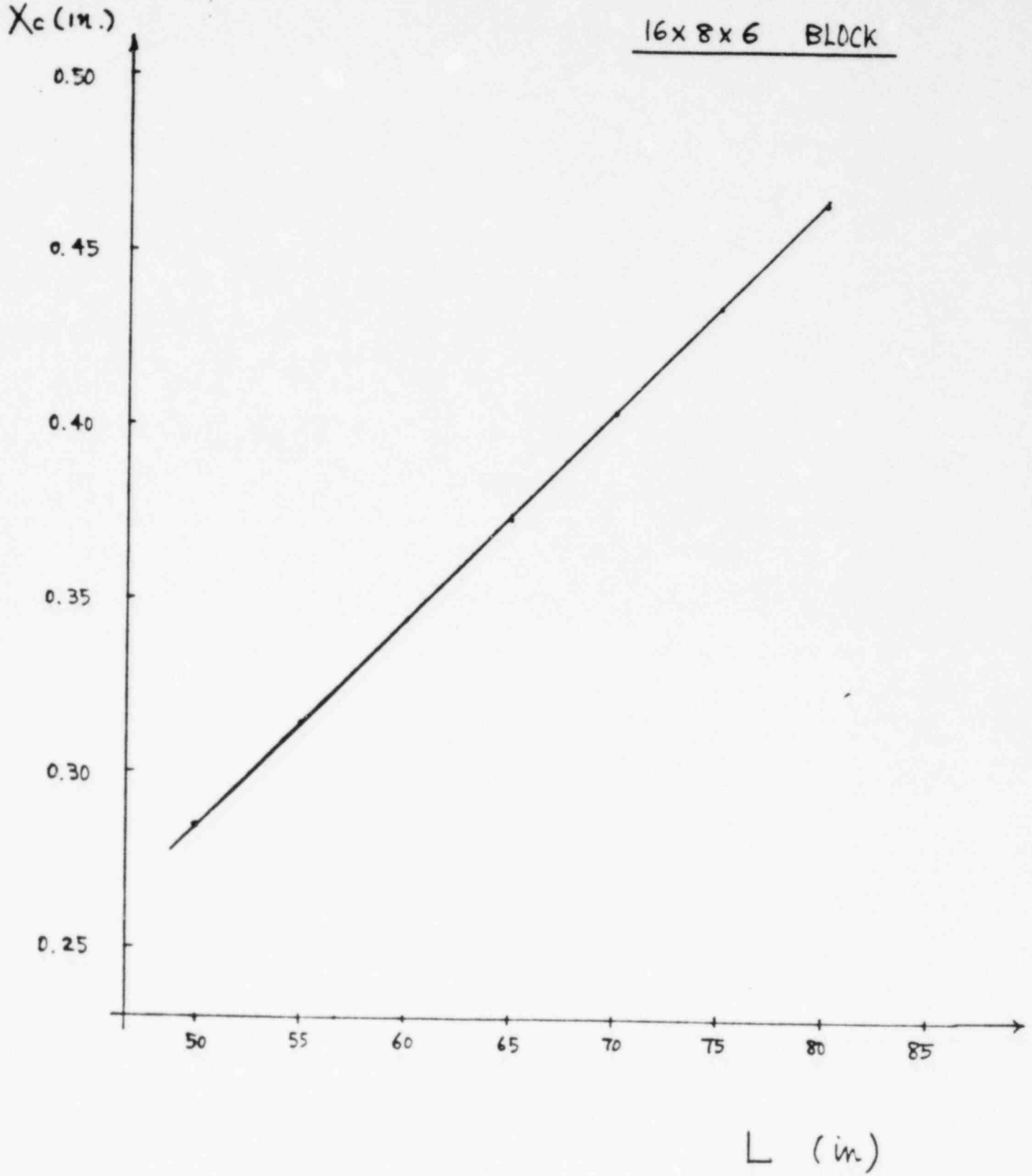


FIG. A

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SHEET NO. 5a

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16 x 8 x 8 BLOCK

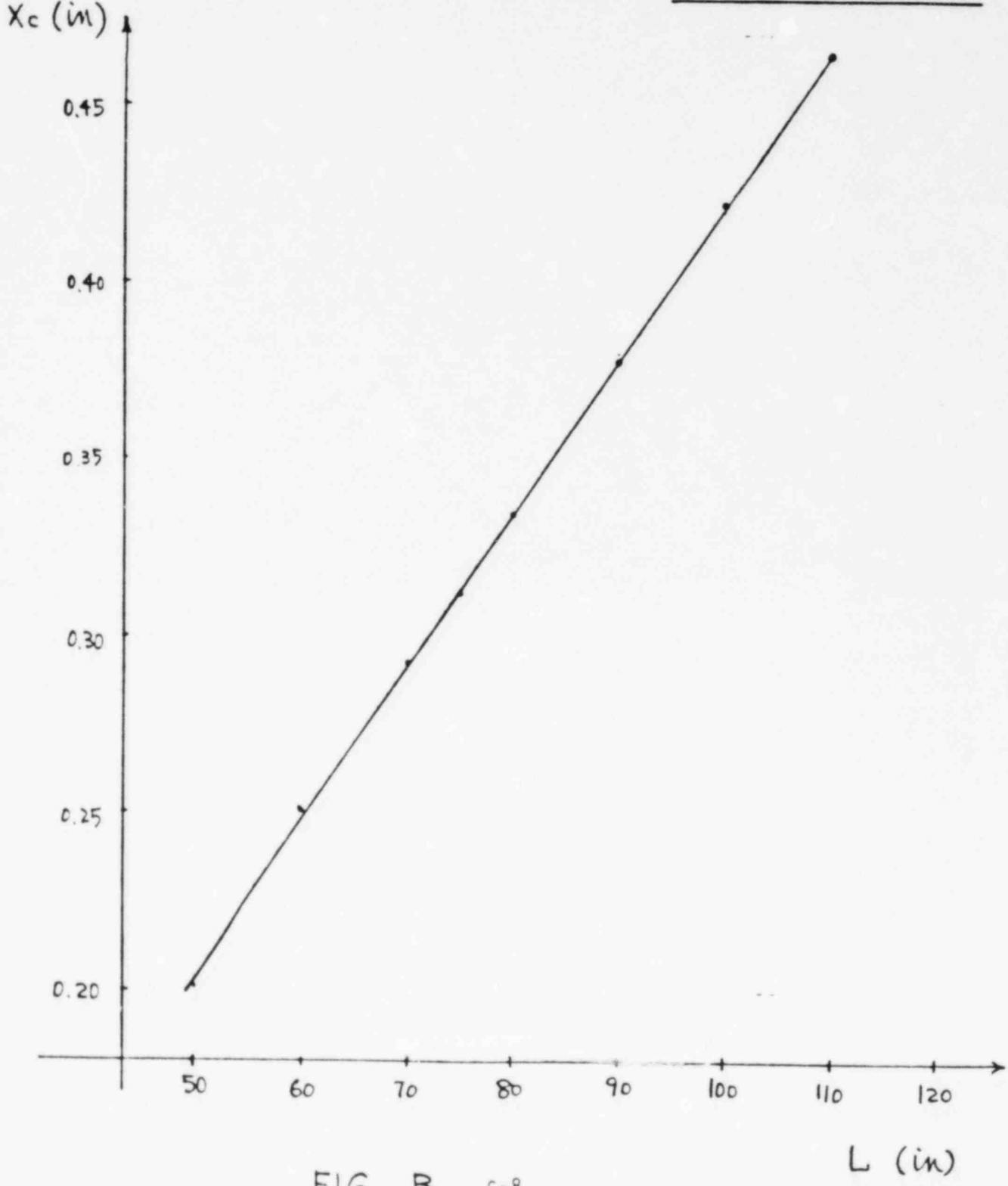


FIG. B

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SHEET NO. 5b

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CLIENT NUSCO SUBJECT ARCH THEORY FOR WALL WITH GAP CHKD JSH DATE 12/24/80

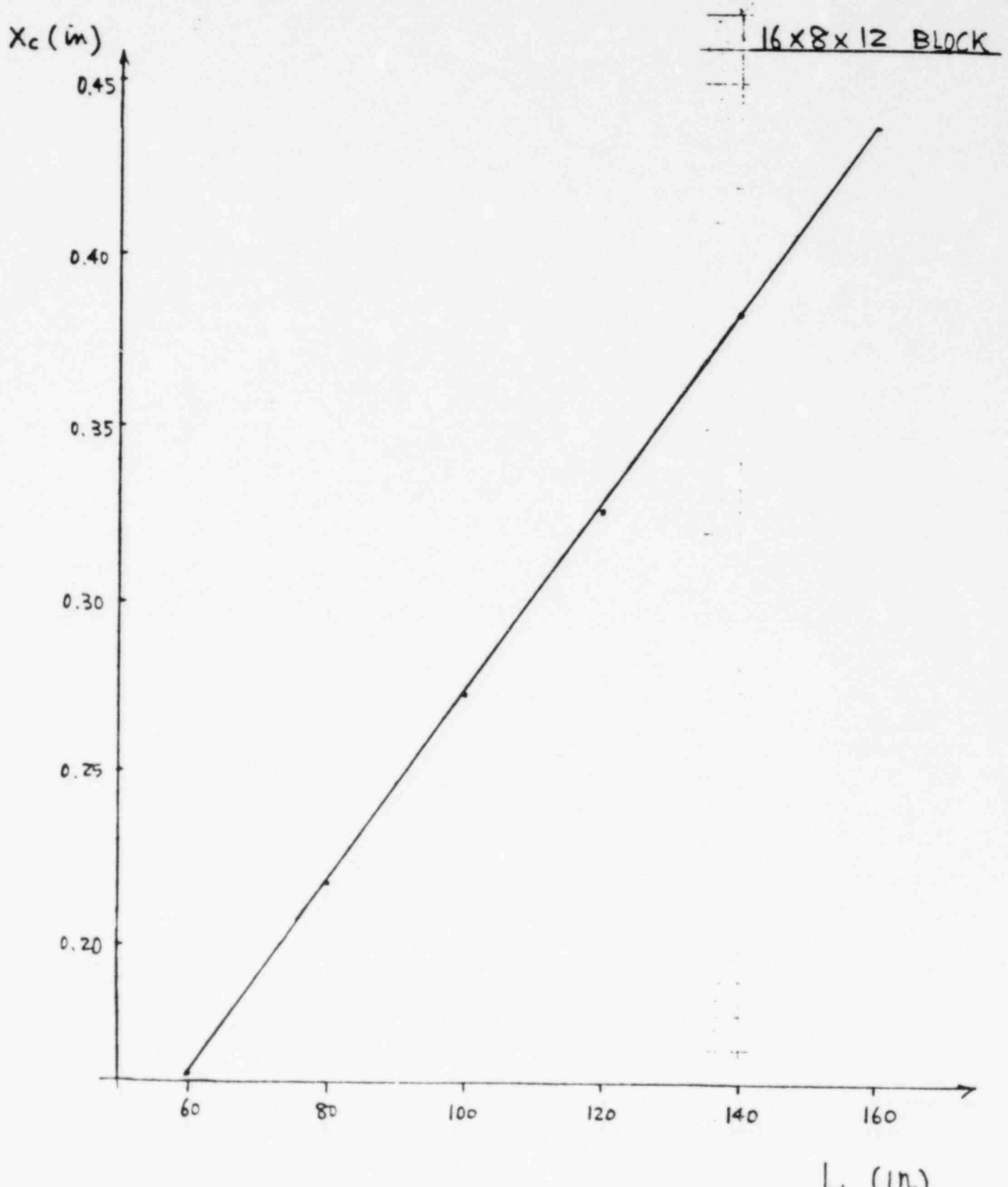
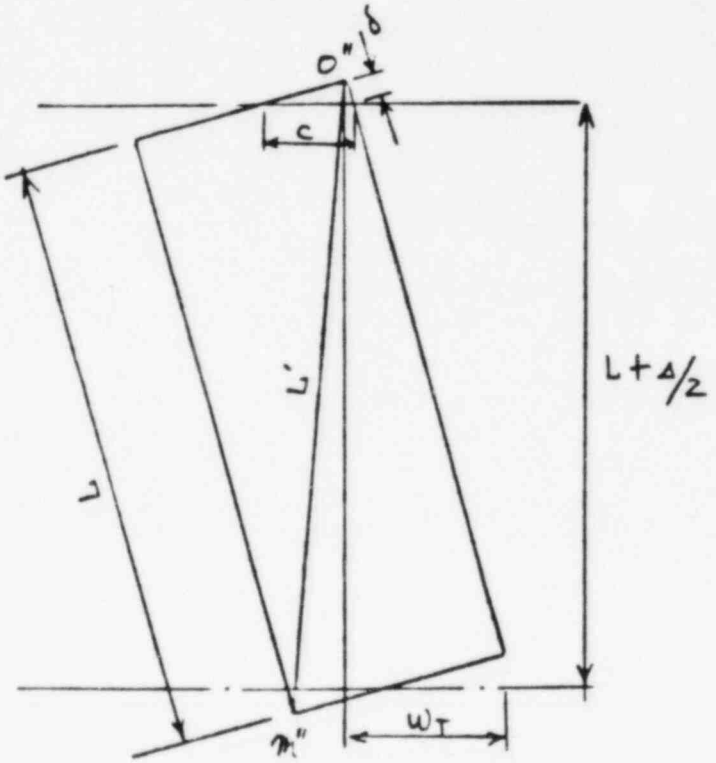


FIG C

AFTER CONTACT IS CREATED

1. Stress is developed in proportional to ϵ
2. The section rotate as rigid block.
3. The deformation in contact area is elastic.



where $L' = \sqrt{L'+4d^2}$

w_T = total mid-span deflection

Fig. 2a

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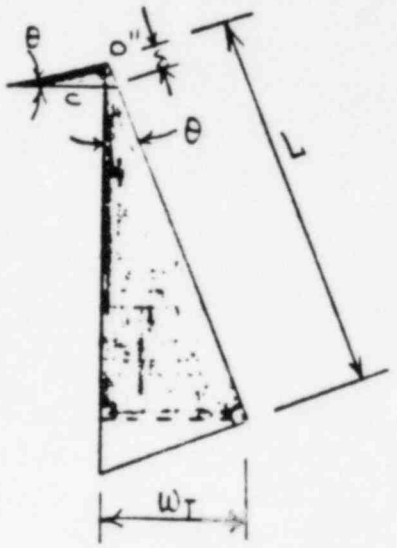


Fig 2b

In Fig 2b,

$$\sin\theta = \frac{\delta}{c} \quad , \quad \sin\theta = \frac{W_T}{L}$$

$$\frac{W_T}{L} = \frac{\delta}{c} \qquad \text{EQN a}$$

According to McDowell , $\epsilon = 2\epsilon_{avg} = \frac{2\delta}{L}$

$$\epsilon = \frac{\sigma_m}{E} = \frac{2\delta}{L}$$

$$\delta = \frac{\sigma_m L}{2E} \qquad \text{EQN b}$$

Put EQN. b into EQN. a

$$\frac{W_T}{L} = \frac{\sigma_m L}{2EC}$$

$$\boxed{C = \frac{\sigma_m L^2}{2E W_T}} \qquad \text{EQN. 3}$$

Consider a position where the diagonal, L', is perpendicular to support. In this position, total midspan displacement will be 2d.

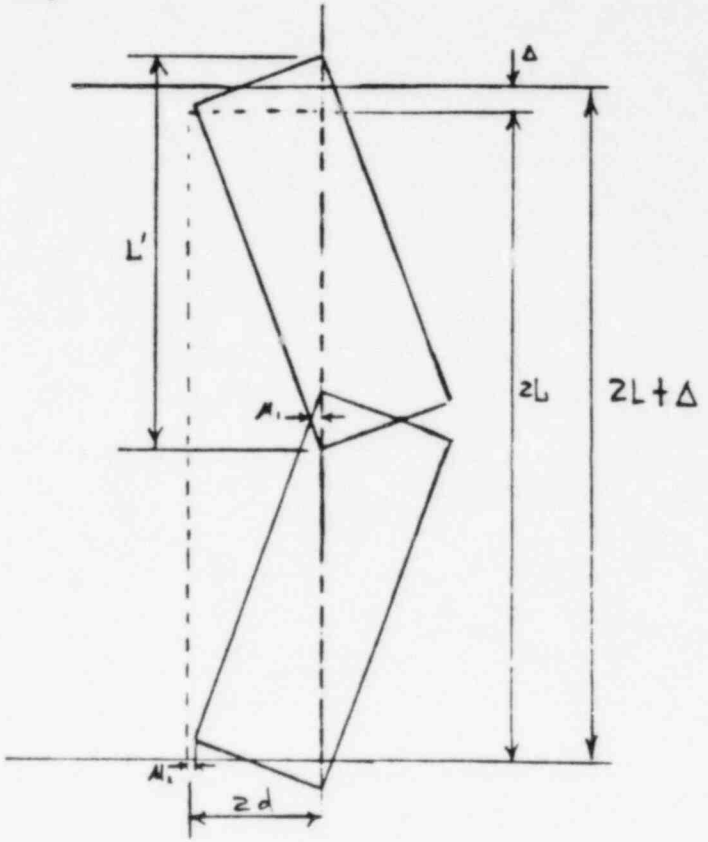


Fig. 3

Assumptions:

1. Effect of M_1 & M_2 is neglected due to small angle change and ratio of height to width of wall
2. The position shown in Fig. 3 is chosen because of the simplicity of geometry.
3. The position in Fig. 3 is used as reference position. In the reality, the stress may go beyond elastic range. However, we will assume it is still in the elastic range.
4. Due to small angle change, any strain perdendicular to support is same as strain parallel to the wall section

In Fig. 3

total shortening, ξ, in one segment will be

$$\xi_0 = L' - (2L+\Delta)/2$$

Since

$$\epsilon = \frac{2\xi}{L'}$$

$$\epsilon_0 = \frac{2L'-2L-\Delta}{L'}$$

Now, we assume there is a linear relationship between strain ϵ and midspan displacement w. For a wall without gap the following relationship will hold.

$$\frac{w}{w_0} = \frac{\epsilon}{\epsilon_0}$$

where w & ϵ are midspan displacement and strain respectively at any particular configuration. But ϵ should be below elastic limit

For wall with gap, the initial displacement x_c will not cause any strain to develop Therefore,

$$\frac{w-x_c}{w_0-x_c} = \frac{\epsilon}{\epsilon_0}$$

Since $E\epsilon = \sigma$, $\epsilon_0 = \frac{2L'-2L-\Delta}{L'}$ & $w_0 = 2d$,

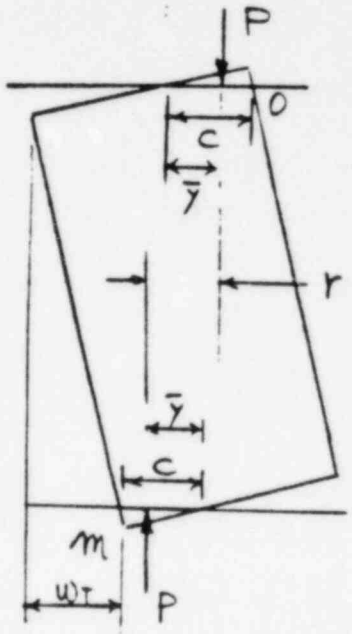
and we multiply the right hand side of above equation by $\frac{E}{E}$.

$$\frac{w_T - x_c}{2d - x_c} = \frac{\sigma}{E\left(\frac{2L' - 2L - \Delta}{L'}\right)}$$

$$\boxed{w_T = \frac{(2d - x_c)L'\sigma}{E(2L' - 2L - \Delta)} + x_c} \qquad \text{EQN 4}$$

where $\boxed{L' = \sqrt{4d^2 + L^2}}$ EQN 4A

w_T = total displacement including x_c

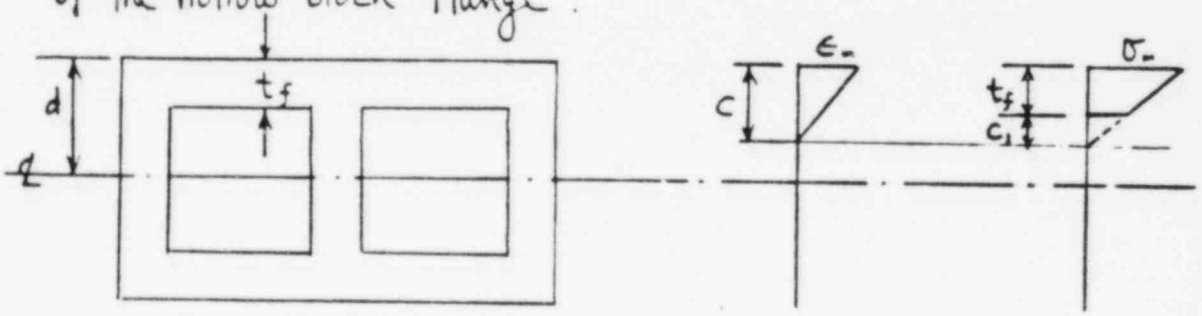


$\bar{y}$ = centroid of the force P
c = contact surface

$$r = 2d - W_T - 2(c - \bar{y})$$

Fig. 4

For hollow block, the compressive area is limited by the thickness of the hollow block flange.



strain stress

Fig. 5

In Fig 5

$$r = 2d - W_T - 2(c - \bar{y})$$

$$M = Pr$$

where $$P = \int_{c_1}^{c} \sigma_m \frac{y}{c}\, dA$$

c, c_1 refer to Fig. 5. For solid block $c_1 = 0$

$$P = \sigma_m \int_{c_1}^{c} \frac{y}{c}\, dA$$

define $$k_1 = \frac{\int_{c_1}^{c} y\, dA}{c}$$

$\bar{y}$ = distance between one end of stress block to centroid of the force, P. See Fig. 4.

$$= \frac{\int_{c_1}^{c} y^2\, dA}{\int_{c_1}^{c} y\, dA}$$

Therefore

$$M = \sigma_m k_1 [2d - W_T - 2(c - \bar{y})]$$

$$\boxed{M = \sigma_m k_1 [2(d-c) + 2\bar{y} - W_T]} \qquad \text{EQN 5}$$

where

$$K_1 = \frac{\int_{c_1}^{c} y\, dA}{C} \qquad \text{EQN 5A}$$

$$\bar{y} = \frac{\int_{c_1}^{c} y^2\, dA}{\int_{c_1}^{c} y\, dA} \qquad \text{EQN 5B}$$

For solid block or stress block fall within flange thickness

$$M = \frac{1}{2} \sigma_m C \left(2d - W_T - \frac{2}{3} c \right) \qquad \text{EQN 6}$$

References:

1. E.L. McDowell, K.E. Mckee, and E. Sevin, "Arching Action Theory of Masonry Walls", ASCE Journal Structural Division, March 1956

2. Edward Cohen and Edward Laing, "Discussion of Arching Action Theory of Masonry Walls" ASCE Journal Structural Division, September 1956

PROCEDURE FOR ANALYZING WALL WITH GAP

DETERMINATION OF MOMENT CAPACITY

1. Use gap size equal to 1/16". Determine L by $L = \frac{H-\Delta}{2}$.

2. Determine midspan displacement, X_c, by using the following equations or from Figs A, B, or C on p. 5, 5a, 5b respectively.

Block size	EQUATION
16 x 8 x 6	$X_c = 5.625 - \sqrt{31.6396 - 0.0625L}$
16 x 8 x 8	$X_c = 7.625 - \sqrt{58.1396 - 0.0625L}$
16 x 8 x 12	$X_c = 11.625 - \sqrt{135.1396 - 0.0625L}$

Note X_c from either procedure should be less than 0.5". If X_c is greater than 0.5", the wall is not qualify for arch analysis.

3. Use maximum allowable elastic stress, σ_m, and EQN 4 and 4A to calculate w_T.

$$w_T = \frac{(2d - X_c)L'\sigma_m}{E(2L' - 2L - \Delta)} + X_c \quad \text{------ EQN 4}$$

$$L' = \sqrt{4d^2 + L^2} \quad \text{------ EQN 4A}$$

If w_T is less than 0.5", skip step 3a

3a. Substitute 0.5 for w_T in EQN 4 and solve for σ_m

4. Use σ_m and w_T from step 2 or 2a to calculate C by Equation 3.

$$C = \frac{\sigma_m L^2}{2E w_T} \quad \text{------ EQN 3}$$

5. Use EQN 5, 5A and 5B for hollow block, or EQN 6 for solid block and hollow block with stress block contact surface c less than flange thickness to find M_{max}.

$$M_{max} = \sigma_m K_1 [2(d-c) + 2\bar{y} - w_T] \quad \text{EQN 5}$$

where $$K_1 = \frac{\int_{c_1}^{c} y\, dA}{c} \quad \text{EQN 5A}$$

$$\bar{y} = \frac{\int_{c_1}^{c} y^2\, dA}{\int_{c_1}^{c} y\, dA} \quad \text{EQN 5B}$$

$$M_{max} = \frac{1}{2} \sigma_m c \left[2d - w_T - \frac{2}{3}c\right] \quad \text{EQN 6}$$

6. By using the following equation to find maximum acceleration, Sa, which the wall can withstand.

$$S_a = \frac{8M g}{1.3(2d)W(2L)^2} \quad *$$

where W = $\frac{\text{weight of block}}{\text{overall volume of the block}}$

* Derivation as the following

$$M = \frac{W(2L)^2}{8}$$

substitute $W = 1.3(2d)W$ into above equation and rearrange.

EXAMPLE (According to Procedure Listed on P. 14 & 15)

Data

$2L+\Delta = 88"$
$2d = 7.625"$ $\quad d = 3.8125"$
$\sigma_{max} = 386$ psi $\quad E = 4.2\times10^5$ psi
$W = 0.0434$ lb/in^3 $\quad t_f = 1.5"$

Step 1

Use $\Delta = 1/16" = 0.0625"$

$L = (88 - 1/16)/2 = 43.9688"$

Step 2

for 16 x 8 x 8 block

$$X_c = 7.625 - \sqrt{58.1396 - 0.0625(43.9688)}$$
$$= 0.1825" < 0.5" \quad \text{O.K.}$$

Step 3

$$L' = \sqrt{(7.625)^2 + (43.9688)^2} = 44.6251"$$

$$W_T = \frac{(7.625 - 0.1825)(44.6251)(386)}{4.2\times10^5\,(2(44.6251) - 2(43.9688) - 0.0625)} + 0.1825$$

$$= 0.2442 + 0.1825 = 0.4267"$$

Step 4

$$C = \frac{386\,(43.9688)^2}{2\,(4.2\times10^5)(0.4267)} = 2.0820$$

Step 5

$t_f = 1.5 < 2.0820$

$c_1 = 2.0820 - 1.5 = 0.5820''$

$$K_1 = \frac{\int_{0.5820}^{2.0820} y\,dy}{2.0820} = 0.9597'' \quad \text{Assume 1" strip}$$

$$\bar{y} = \frac{\int_{0.5820}^{2.0820} y^2\,dy}{\int_{0.5820}^{2.0820} y\,dy} = 1.4728'' \quad \text{Assume 1" strip}$$

$$M_{max} = 386\,(0.9597)\,[2(3.8125 - 2.082) + 2(1.4728) - 0.4267]$$

$$= 2215''\text{-lb}$$

Step 6

$$S_a = \frac{8\,(2215)\,g}{1.3\,(7.625)(0.0434)(2 \times 43.9688)^2} = 5.3\,g$$

PROCEDURE II CALCULATE STRESS FOR A GIVEN MOMENT

1. Determine M_{max} per 1" strip

2. Calculate initial midspan displacement, X_c.

$$X_c = 2d - \sqrt{4d^2 - \left(L\Delta + \frac{\Delta^2}{4}\right)}$$

3. Calculate L'.

$$L' = \sqrt{4d^2 + L^2}$$

4. Calculate α.

$$\alpha = \frac{(2d - X_c)L'}{E(2L' - 2L - \Delta)}$$

5. Assume a value of σ. For first trial, use the value from the equation below.

$$\sigma = \frac{M_{max}}{S}$$

where S is the section modulus of block per 1" strip

6. Determine maximum midspan deflection, W_T.

$$W_T = \alpha \sigma_m + X_c$$

7. Determine contact surface, C.

$$C = \frac{\sigma_m L^2}{2E W_T}$$

8. If C is greater than flange thickness, go to step 8a. If C is less than or equal to flange thickness, go to step 8b.

8a. Determine M_{max} by the following equations.

$$M_{max} = \sigma_m K_1 [2(d-c) + 2\bar{y} - w_T]$$

$$K_1 = \frac{\int_{c_1}^{c} y\, dA}{C} \qquad \text{per 1" strip}$$

$$\bar{y} = \frac{\int_{c_1}^{c} y^2 dA}{\int_{c_1}^{c} y\, dA} \qquad \text{per 1" strip}$$

8b. Determine M_{max} by the following equation.

$$M'_{max} = \frac{1}{2}\sigma_m C [2d - w_T - \frac{2}{3}C]$$

9. If M'_{max} calculated from step 8a or 8b is same as M_{max} from step 1, values of σ_m, w_T are found. Otherwise, go to step 10.

10. For 2nd trial, go to step 10a. For 3rd trial and beyond, go to step 10b.

10a. Assume σ_m by

$$\sigma_{m\ 2^{nd}\ trial} = \sigma_{m\ 1^{st}\ trial} \frac{M_{max}}{M'_{max\ from\ trial\ 1}}$$

10b Assume a linear relationship between σ and M. Use values of σ_i and M'_{max} from previous trials and interpolation method to calculate new σ_m for next trial.

11 Go to step 6.

Docket No. 50-245

ENCLOSURE II

MILLSTONE NUCLEAR POWER STATION, UNIT NO. 1

REQUEST FOR ADDITIONAL INFORMATIONON
IE BULLETIN 80-11, MASONRY WALL DESIGN

NOVEMBER, 1982

RESPONSE TO NRC QUESTIONS
MASONRY WALL REEVALUATION
(NRC IE BULLETIN 80-11)
MILLSTONE PLANT #1

1. QUESTION:

Indicate whether walls are stacked or running bond. If any stack bond wall exists, provide sample calculations of the stresses for a typical wall.

RESPONSE:

All masonry walls at Millstone Unit #1 are running bond.

2. QUESTION:

Explain and justify the difference (if any) between the load combinations provided in the plant FSAR and the load combinations used in the re-evaluation of the masonry walls.

RESPONSE:

In the initial review tornado loads were not addressed. A final review including all loads and load combinations based on the requirements for Class 1 structures as outlined in the Millstone Unit #1 FSAR has since been completed as described in Reference 5.

3. QUESTION:

Indicate how earthquake forces in three directions were considered in the analysis.

RESPONSE:

The maximum stress was assumed to be the absolute sum of the effects of the vertical seismic load in combination with the horizontal seismic load acting in the most unfavorable direction. This corresponds to the method utilized in the Millstone Unit #1 FSAR which was used as the basis for our licensing document.

4. QUESTION:

Justify the use of an increase factor of 1.5 for allowable masonry shear and tension normal to the bed joint (Page 1, Exhibit C-1, Reference 2). The SEB criteria (4) allow an increase of only 1.3 for shear and tension normal to the bed joint. If any existing test data will be used to justify this increase factor, the Licensee is required to discuss the applicability of these tests to the masonry walls at Millstone Unit 1 with particular emphasis on the following:

- Boundary conditions
- Nature of loads
- Size of test walls
- Type of masonry construction (block and mortar type, grouted or ungrouted)

The Licensee is also requested to indicate, if SEB criteria were to be used, how many walls could not be qualified and to identify these walls.

RESPONSE:

Out-of-Plane Shear

The equation for allowable flexural shear given in the masonry codes is the same as that for concrete in ACI 318-63. However, this is intended for beams rather than walls and thus is not directly applicable to walls in out-of-plane flexure. For unreinforced walls the limits on tensile stress preclude any significant flexural shear stress. However, peripheral shear at the boundaries is still evaluated.

Since no code value for peripheral shear exists, it is reasonable to make use of the criteria for plain concrete; i.e., $2 (f'c)^{\frac{1}{2}}$. The ACI 318-63 value for allowable flexural compressive stress is $0.45f'c$ while the allowable for masonry for normal loads is $0.33f'm$. Making a similar ratio for the peripheral shear yields a masonry allowable of $1.5 (f'm)^{\frac{1}{2}}$. Since the failure mode is similar to that for plain concrete, and concrete structures use factored/normal allowable ratios in excess of 1.5, the 1.5 ratio for masonry is conservative.

The unreinforced masonry walls at Millstone 1 were generally qualified based on allowable horizontal spans which were governed by allowable tension parallel to the bed joint. A review of the allowable span calculations shows that maximum shear stress for these span lengths are on the order of 25% of the allowable. Thus, the walls will also meet the SEB criteria.

In-Plane Shear

The masonry walls at Millstone Unit #1 are not part of the structural load resisting system and do not carry seismic shear loads. The walls, therefore, need not satisfy the allowable stress criteria for in-plane shear and the increase factor of 1.5 was not used. Thus, the use of the SEB criteria would not effect the results.

Tension Normal to the Bed Joint

Reference 3 reported seven tests of hollow unit construction with Type S mortar. Conservatively selecting the lowest modulus of rupture reported gives a value of 60 psi ultimate, which is approximately 2 times greater than the Millstone Unit 1 factored load allowable of 32 psi.

Additionally, Omote, et. al. (Reference 4) reported tests for hollow brick masonry showing variation of modulus of rupture versus mortar compressive strength. A graph of his results was compared to a curve showing twice the Millstone Unit #1 allowable stress for hollow unit masonry. All values used were found to be acceptable.

It should be noted that tension normal to the bed joints was not used as a basis for overall qualification of walls. Local tension normal to the bed joints was permitted in isolated cases where this mechanism was not the primary load path for the overall wall system. Thus, use of the SEB criteria would not affect the results.

5. QUESTION:

Provide sample calculations for block pullout of attachments analysis and tornado analysis.

RESPONSE:

Block pullout capacity was not an important issue at Millstone Unit #1 since, in general, safety related items of significant weight are not supported by the masonry walls. The types of loads on the masonry walls due to attachments are much smaller than the allowable pullout values.

Sample calculations for block pullout of attachments analysis and tornado analysis are included in Attachments 1 and 2 respectively.

6. QUESTION:

Regulatory Guide 1.61 allows 4% damping for operating basis earthquake (OBE) and 7% damping for safe shutdown earthquake (SSE). Provide the damping values used in the analysis and justify them if they are higher than those allowed by Regulatory Guide 1.61.

RESPONSE:

All Millstone Unit #1 safety related unreinforced masonry walls were designed such that the frequency of the system was in the rigid range.

Safety related reinforced masonry walls were not required to be in the rigid range because wall capacities were large and modifications to satisfy rigid range criteria would have been excessive. The walls were qualified using a minimum amplification factor of 5 applied to the maximum floor acceleration.

7. QUESTION:

In Reference 2, it was indicated that arching action analysis may have been used to qualify some masonry walls. The NRC, at present, does not accept the application of this technique to masonry walls in nuclear power plants in the absence of conclusive evidence to justify this application. The Licensee is requested to indicate the

number of walls which have been analyzed by this technique and to provide sample calculations.

In addition, the following areas need technical verification before any conclusion can be made about arching analysis:

- o Explain how the arching action theory deals with cyclic loading, especially when the load is reversed.

- o Provide justification and test data (if available) to validate the applicability of arching action theory to the masonry structures at Millstone Unit 1, with particular emphasis on the following areas:

  a. nature of the load
  b. boundary conditions
  c. material strength
  d. size of test walls.

- o If hinges are formed in the walls, the capability of the structures to resist an in-plane shear force will be diminished, and shear failure might take place. This in-plane shear force would also reduce the out-of-plane stiffness. Explain how the effect of this phenomenon can be accurately determined.

RESPONSE:

Twenty-four "blockouts" (blockouts are masonry walls which are constructed within, and completely fill a formed opening in a reinforced concrete wall) at Millstone Unit #1 have been analyzed considering arching action. Calculations outlining this method of analysis are included in Attachment 3.

- o The arching theory of confined masonry walls is based on an extensive full-scale test program sponsored by the Defense Civil Preparedness Agency and the Veterans Administration (Reference 1). The tests were performed in a "shock tunnel" in which a wall was "struck by a blast wave, the wave reflected from the wall returned to its source area, then re-reflected to strike the wall again, about 0.3 seconds after the first loading. Thus, these tests provided information on the ability of (arching) walls to withstand a number of reversing loading cycles."

  (a) At the Millstone Unit #1 Plant, arching theory was used primarily to determine the ability of blockouts to resist uniform pressure loadings and earthquake loadings. The test data of Reference 1 provides justification for applying arching theory to masonry walls subjected to pressure loads. Reference 1 also cites field evidence which confirms the increased resistance to out-of-plane earthquake loads of confined (arching) masonry walls over nonarching walls.

(b) Boundary conditions required for the application of arching theory were validated by walkdown inspection of the blockouts to be analyzed.

(c) Experiments on arching have been performed on walls that range in crushing strength from 1,000 psi to 3,000 psi (Reference 2). The blockouts at Millstone Unit #1 fall within this range.

(d) The tests of Reference 2 were performed on walls arching over an 8'-0" span while those of Reference 1 spanned 8'-6". We consider that application of arching theory to the range of blockout sizes encountered at Millstone Unit #1 is appropriate.

- At its present stage of development, arching theory cannot adequately account for the effect of hinge formation on in-plane shear capacity. However, the blockouts at Millstone Unit #1 analyzed by the arching method are not designed to act a shear walls or as bearing walls and, therefore, are not subjected to significant in-plane load. This eliminates the need to determine how in-plane and out-of-plane loads interact.

8. QUESTION:

Reference 3 indicated that a final report on the modifications was being prepared and would be submitted upon completion. Provide this final report.

RESPONSE:

The final report on the re-evaluation and modifications to masonry walls has been included in Attachment 6.

9. QUESTION:

Provide a description of the required modifications and explain by sample calculation how they rectify wall deficiencies. In addition, provide detailed drawings of some sample modifications.

RESPONSE:

Modifications to the masonry walls included:

- The insertion of angles at the wall edges to assure pinned boundary conditions.

- The insertion of steel sections to transfer wall loads to the boundaries and limit wall spans.

- Shielding so that falling blocks would not hit safety related equipment.

- Rerouting of pipes.

o Relocation of pipe supports.

Sample calculations outlining an analysis made to rectify wall deficiencies are included in Attachments 2 and 4. Detailed drawings of some sample modifications have been included in this package.

10. QUESTION:

Since Millstone Unit #1 is a part of the systematic evaluation program (SEP), the Licensee is requested to clarify whether SEP loadings for seismic analysis have been used. If not, provide justification.

RESPONSE:

The masonary walls at Millstone Unit 1 were evaluated based upon building floor accelerations generated in the original design. These accelerations were used to compute the seismic inertia forces for rigid systems supported by those floors. The SEP review conducted by Lawrence Livermore Laboratory for the NRC staff produced actual floor response spectra for Category 1 structures. This report concluded that overall the original seismic design basis for the plant was acceptable. The floor response spectra generated by the SEP analysis will be compared to the design values used in response to I&E Bulletin 80-11 to insure that the approach used was adequate. We do not expect this review to adversely effect the results of the concrete wall evaluation. Additional information on the seismic analysis used to qualify masonry walls is contained within the final report included as Attachment 6.

11. QUESTION:

Page 2 of Section 3.0 of Reference 2 is missing. Provide this page.

RESPONSE:

This page has been included in Attachment 5.

REFERENCES

1. Gabrielsen, B. L., Kaplan, K., "Arching in Masonry Walls Subjected to Out-of-Plane Forces," Earthquake Resistance of Masonry Construction, National Workshop, NBS 106, 1976.

2. McKee, K. E., Sevin, E., "Design of Masonry Walls for Blast Loading," ASCE Structural Journal, January, 1958.

3. "Recommended Guidelines for the Reassessment of Safety Related Concrete Masonry Walls", prepared by Owners and Engineering Firms Informal Group on Concrete Masonry Walls, October 6, 1980.

4. Omote, Y. Mayes, R. L. Chen, S. W. J., Clough, R. W., "A Literature Survey - Transverse Strength of Masonry Walls", EERC-77/07, March 1977.

5. W. G. Counsil letter to R. C. Haynes, dated September 24, 1982.

ATTACHMENT 1

consists of a 4 page (Sheet No. M000 Sec. 5-1 to M000 Sec. 5-4) example of block pullout calculations.

EES

CALCULATION SHEET

PROJECT MILLSTONE UNIT #1		PREPARED BY B. Davey	DATE 1-7-81
SUBJECT N.R.C. I & E BULLETIN 80-11		CHECKED BY B. Woodhouse	DATE 1-27-81
SYSTEM		JOB NO. 8014	FILE NO. 1/F
ANALYSIS NO.	REV. NO.	SHEET NO. M000 SEC 5-1	

SECTION 5

Summary

CALCULATION OF MAXIMUM PULL-OUT CAPACITY OF A SINGLE CONC. BLOCK BY USING THE SHEAR STRESS ALLOWABLES FOR A SHEAR WALL

DETERMINING ACTUAL VERTICAL LOAD FOR EACH ELEVATION IN TURBINE AND REACTOR BUILDING THAT WILL GENERATE THE PULLOUT CAPACITY

PULLOUT CAPACITY (P) = ACTUAL LOAD (W) X Bldg ACCEL. (g)

$$P = W \cdot g$$

$$\therefore W = P/g$$

EES
EARTHQUAKE NGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT MILLSTONE UNIT #1

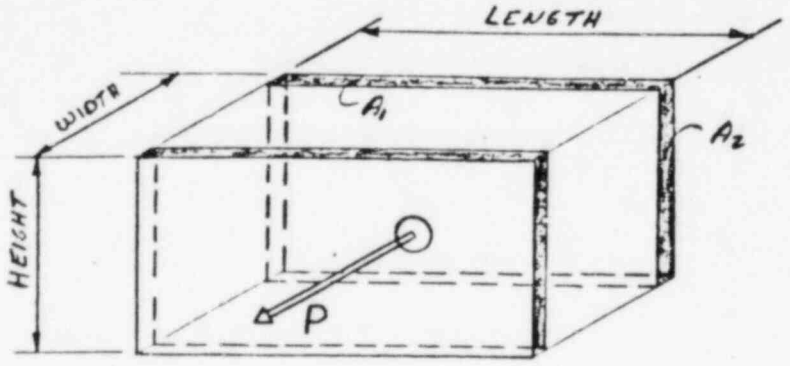
SUBJECT NRC-IE-BULLETIN 80-11 - BLOCK WALL PULL-OUT CAPACITY

PREPARED BY	DATE
DIMITRIOS ANTYPOPOULOS	9/9/80
CHECKED BY	DATE
RJ Richard	9/10/80
JOB NO. 80014	FILE NO. 1-F
SHEET NO. MDDO SEC 5-2	

MAX PULL-OUT CAPACITY CALCULATION

(FOR UNREINFORCED, HOLLOW UNIT MASONRY)

FROM MILLSTONE MASONRY JOB SPEC NO. NPC-M1-A3 ALL CONC. BLOCKS HAVE FACE SHELL MORTAR EXCE THE BASE COURSE. (Ref design input document M1-80014-01-06



MORTAR AREA CALCULATION

WALL THICKNESS (IN)	BLOCK HEIGHT (IN)	BLOCK LENGTH (IN)	BLOCK FACE SHELL TKN (IN)	AREA A_1 (IN^2)	AREA A_2 (IN^2)	TOTAL AREA $4(A_1+A_2)$ (IN^2)
4	7.625	15.625	.75	11.72	5.72	69.76
6	↓	↓	1.0	15.63	7.63	93.04
8			1.25	19.53	9.53	116.24
12			1.5	23.44	11.44	139.52

1

EES
EARTHQUAKE ENGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT MILLSTONE UNIT #1

SUBJECT NRC-IE-BULLETIN 80-11, REVISED BLOCK WALL PULL-OUT CAPACITY

PREPARED BY	DATE
S. Dowis	10/2/8
CHECKED BY	DATE
C Fiorelli	10/2/8
JOB NO.	FILE NO.
80014	1/F

SHEET NO. M000 SEC 5-3

MAX. PULL-OUT CAPACITY CALCULATION

FROM MORTAR AREA CALCULATIONS:

WALL THICKNESS (in)	TOTAL AREA (in^2)
4	69.76
6	93.04
8	116.24
12	139.52

FROM DESIGN CRITERIA, EXHIBIT C-1, ALLOWABLE SHEAR WALL STRESSES (PSI)

	C90	C126	C145
DESIGN EARTHQUAKE	33	23	30
MAX. EARTHQUAKE	50	34	44

NOTE: THE SHEAR AREA USED FOR ALL BLOCK TYPES IS THAT OF C129 (FACE SHELL AREAS ONLY). THIS IS VERY CONSERVATIVE FOR REINFORCED (C-90) AND SOLID (C145) BLOCK.

EARTHQUAKE ENGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT MILLSTONE UNIT #1

SUBJECT NRC-IE-BULLETIN 80-11-REVISED BLOCK WALL PULL-OUT CAPACITY

PREPARED BY	DATE
L. Dowrie	10/2/8
CHECKED BY	DATE
C Fiorelli	10/2/8
JOB NO. 80014	FILE NO. 1/F
SHEET NO. MOOD SEC 5.4	

MAX. PULL-OUT CAPACITY CALCULATION COND. ⚠1

WALL THICKNESS (IN)	SHEAR AREA (IN^2)	DESIGN EARTHQUAKE		MAXIMUM EARTHQUAKE	
		ALL SHEAR STRESS (PSI)	PULL-OUT CAPACITY (lbs)	ALL SHEAR STRESS (PSI)	PULL-OUT CAPACITY (lbs)
4	69.76	33	2302	50	3488
	"	23	1604	34	2372
	"	30	2093	44	3069
6	93.04	33	3070	50	4652
	"	23	2140	34	3163
	"	30	2791	44	4094
8	116.24	33	3836	50	5812
	"	23	2674	34	3952
	"	30	3487	44	5115
12	139.52	33	4604	50	6976
	"	23	3209	34	4744
	"	30	4186	44	6139

ATTACHMENT 2

consists of a 45-page (F.M.R. No. 1 and M4 0-43) example of an analysis made for tornado loadings including modifications made to rectify wall deficiencies.

CYGNA ENERGY SERVICES

Field Modification Request

PROJECT: Millstone 1 NRC 80-11 Effort

CLIENT: Northeast Utilities Service Co.

F.M.R. No. 1

Job No.: 82028

Page 1 of 1

Analysis Problem No.: 0004
Wall I.D. No.: T-32C&D

Location Bldg. TURBINE
Elev. 34'-6"

Description

MODIFICATIONS ARE REQUIRED TO QUALIFY WALLS T-32C&D FOR TORNADO EFFECTS PER PROJECT MEMO #4

Prepared by: Joe Chrzan 4/30/82 Date

Approved by: Donald E. Johnson 4/30/82 Date

Desposition :

INSTALL MODIFICATIONS PER DRAWING No 82028-MWD-1 REV 0

REF. CALC. SET 82028-M4

CYGNA
Joe Chrzan 4/30/82 Prepared by Date
Donald E. Johnson 4/30/82 Approved by Date
Charles T. Otto 4/30/82 PE Date

NUSCO
Engineering Approval Date
Construction Approval Date

Millstone 1 Plant Engineer
Approval by Date

Modification Complete

Contractor Date

QC Approval Date

Remarks :

ATTACHED DRAWING:

82028-MWD-1 REV 0

CYGNA Closeout Review

PE : _____

PQAE : _____

CYGNA

UNCONTROLLED COPY

Calculation Cover Sheet

Project: MILLSTONE UNIT NO. 1	Job No. 82028
	File No. 1/F
Client: NUSCO	Calc. Set No. M4-0
	No. of Sheets 43

Subject

NRC I & E BULLETIN 80-11 MASONRY WALL TORNADO EVALUTION FOR WALLS T-32C & T-32D

Statement of Problem

DETERMINE THE ALLOWABLE CAPACITY OF THE MASONRY, BOUNDARY MODIFICATIONS, & EXISTING STEEL FOR WALL T-32C & T-32D AND COMPARE TO THE NEW TORNADO LOAD'S. SHOULD CAPACITIES PROVE TO BE TOO SMALL, DESIGN NEW MODS TO SUSTAIN NEW LOADS.

Sources of Data

- REF EES CALCULATIONS M107-BINDER BOOK 7 F & M000-BINDER BOOK 1 F
- FIELD WALKDOWN DATA - DESIGN INPUT # 82028-DI-4
- REFERENCE NUSCO DWG # 25202-54100 REV. 0 SHT 2 OF 2
- REFERENCE GENERIC CALCULATION # 82028-M1-BINDER 1/F
- MILLSTONE UNIT-1 - MASONRY DESIGN CRITERIA (80014-DC-1-REV 3) IN CONJUCTION WITH PROJECT MEMOS 3 & 4
- REF EBASCO DWGS G197925-REV1 & G197945 REV 6

Sources of Formulae & References

1. AISC MANUAL OF STEEL CONSTRUCTION - 7TH EDITION
2. REINFORCED MASONRY DESIGN - SCHNEIDER & DICLEY - FIRST ED (1980)
3. ASCE PAPER 3269 "WIND FORCES ON STRUCTURES"

Remarks

FOR SUMMARY SEE PAGE 3 OF 43
FOR MODIFICATIONS SEE FMR-1

Originators	Checkers	Distribution	Revision No. 0
J. CHIASSON	D. Johnson		Supersedes Calculation Set No. N/A
			Approved By: Charles T. Pitts Date: 4/29/82

1005.00

Calculation Sheet	Prepared By: Joe Chiarra	Date 4/13/82
	Checked By: Donald E. Ware	Date 4/29/82
Project MILLSTONE UNIT NO. 1	Job No. 82028	File No. 1/F
Subject MASONRY WALL REANALYSIS		
System WALLS T-32C & T-32D	Sheet No.	M4 - 1
Analysis No. M4 Rev. No. 0		

INDEX

		PAGE
1.	INTRODUCTION	2
2.	SUMMARY (INCLUDING MOD DETAILS)	3-6
3.	RESULTS OF INVESTIGATION	7
4.	ANALYSIS OF EXISTING MASONRY	8-10
5.	ANALYSIS OF PREVIOUS MODIFICATIONS	11-15
6.	ANALYSIS OF EXISISTING BUILDING STEEL	16-26
7.	DESIGN OF NEW MODIFICATIONS	27-43

EES

CALCULATION SHEET

PREPARED BY: Joe Chiasson	DATE: 3/31/82
CHECKED BY: Donald E Hinson	DATE: 4/29/82
PROJECT: MILLSTONE UNIT NO. 1	JOB NO.: 82028 ✓
SUBJECT: MASONRY WALL ANALYSIS	FILE NO.: 1/F
SYSTEM: WALLS T-32C & T-32D	
ANALYSIS NO.: M4 REV. NO.: 0	SHEET NO.: M4-2

1. INTRODUCTION

THE PURPOSE OF THIS CALCULATION IS TO DETERMINE THE ALLOWABLE CAPACITY OF THE MASONRY, THE BOUNDARY MODIFICATIONS, AND THE EXISTING STEEL FOR WALLS T-32C & T-32D AND COMPARE THESE CAPACITIES TO A 2.5 PSI POSITIVE TORNADO PRESSURE DROP LOAD AND AN INCREASED TORNADO WIND VELOCITY OF 360 MPH (REFERENCE MILLSTONE 1 FSAR & SECTION 12 OF AMENDMENT 15).

ADDITIONAL MODIFICATIONS WILL BE DESIGNED SHOULD THE CAPACITIES PROVE TO BE LESS THAN THE APPLIED LOADS.

A 360 MPH TORNADO WIND VELOCITY TRANSLATES AS FOLLOWS:

— FOR WINDWARD PRESSURE —

REFERENCE ASCE PAPER 3269 - "WIND FORCES ON STRUCTURES" AND PROJECT MEMO # 9

$$P = (.8)\,.00256\,V^2$$

$$= (.8)\,.00256\,(360)^2 = \underline{265\ psf}$$ - CONTROLLING POSITIVE PRESSURE

PREVIOUS PRESSURE = 195 PSF

— FOR LEEWARD PRESSURE —

$$P = (.5)\,.00256\,V^2$$

$$= (.5)\,.00256\,(360)^2 = \underline{166\ psf}$$

FOR 2.5 PSI PRESSURE:

$$P = 2.5\ \frac{\#}{IN^2} \times \frac{144\ IN^2}{1\ FT^2} = \underline{360\ psf}$$ - CONTROLLING NEGATIVE PRESSURE

PREVIOUS PRESSURE = 115 PSF

131 Rev 0

2. SUMMARY

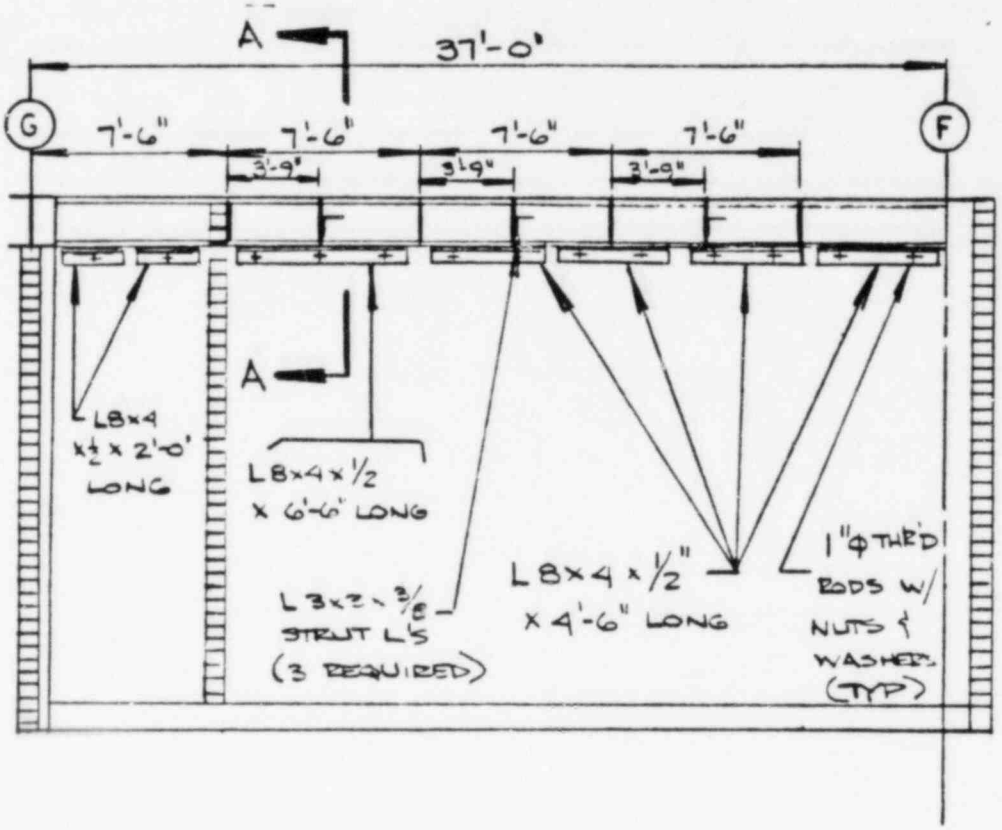
BASED ON EXTENSIVE EXAMINATION AND ANALYSIS OF WALLS T-32C & T-32D, IT HAS BEEN DETERMINED THAT ADDITIONAL MODIFICATIONS WILL NEED TO BE INSTALLED IN ORDER TO SATISFY THE MOST RECENT TORNADO PRESSURE DROP AND TORNADO WIND VELOCITY LOADS.

RESULTS TABULATED ON PAGE 7 OF THIS CALCULATION SHOW 1) MODIFICATION WILL BE NEEDED TO DECREASE THE UN-SUPPORTED SPAN OF THE WALLS TO DECREASE BENDING IN THE MASONRY AND 2) MODIFICATION WILL BE NEEDED TO STRENGTHEN EXISTING STEEL IN ORDER TO PREVENT OVERSTRESS DUE TO COMBINED STRONG AND WEAK AXIS BENDING OF EXISTING W21'S.

L 8x4's ARE TO BE WELDED TO THE BOTTOM FLANGE OF THE EXISTING W21 AND THRU-BOLTED TO WALLS T-32C & D IN ORDER TO REDUCE THE BENDING IN THE MASONRY. THREE ADDITIONAL L 3x3x3/8 STRUTS WILL BE ADDED IN ORDER TO STRENGTHEN THE EXISTING W21 IN WEAK AXIS BENDING. DETAIL DESIGN SKETCHES CAN BE FOUND ON PAGES 4, 5, & 6. OF THIS CALCULATION

Calculation Sheet

Project	MILLSTONE UNIT NO. 1
Subject	MASONRY WALL REANALYSIS
System	WALLS T-32C & T-32D
Analysis No.	M4
Rev. No.	0
Prepared By:	Joe Chavez
Date	4/9/92
Checked By:	Donald C. Jackson
Date	4/29/92
Job No.	82028
File No.	1/F
Sheet No.	M4 – 4



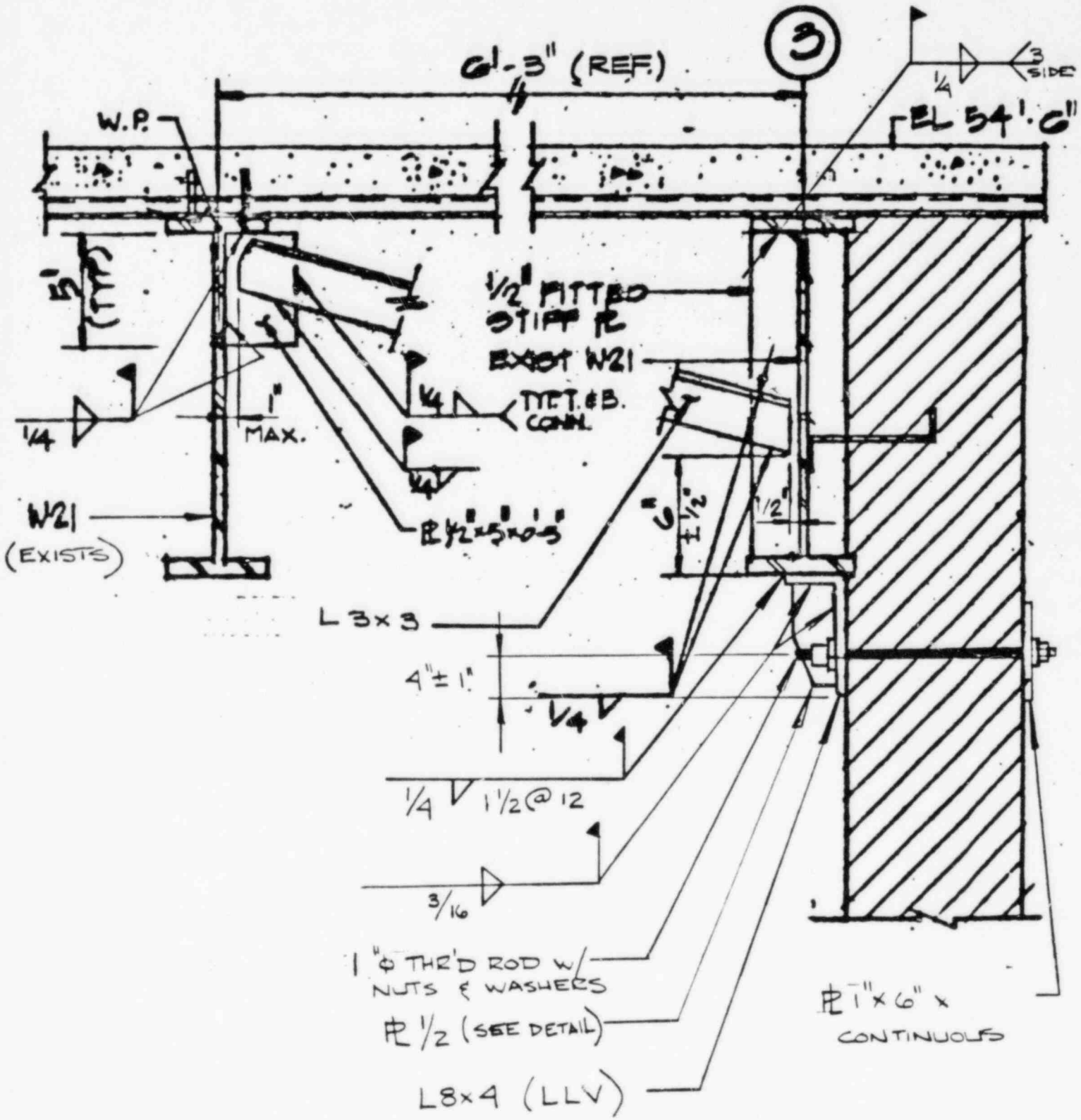
ELEVATION LOOKING SOUTH

T-32C & T-32D

NOTE :- LOCATION OF ADDITIONAL L 3x3x⅜ STRUTS ARE ± 6"
- LOCATION OF THR'D RODS ARE ± 6"
- ANGLE LENGTHS ARE ± 1½"

Calculation Sheet

Project	MILLSTONE UNIT NO. 1
Subject	MASONRY WALL REANALYSIS
System	WALLS T-32C & T-32D
Analysis No.	M4
Rev. No.	0
Prepared By:	Joe Chiarra
Date	4/9/92
Checked By:	Donald C Flaherty
Date	4/29/92
Job No.	82028
File No.	1/F
Sheet No.	M4-5

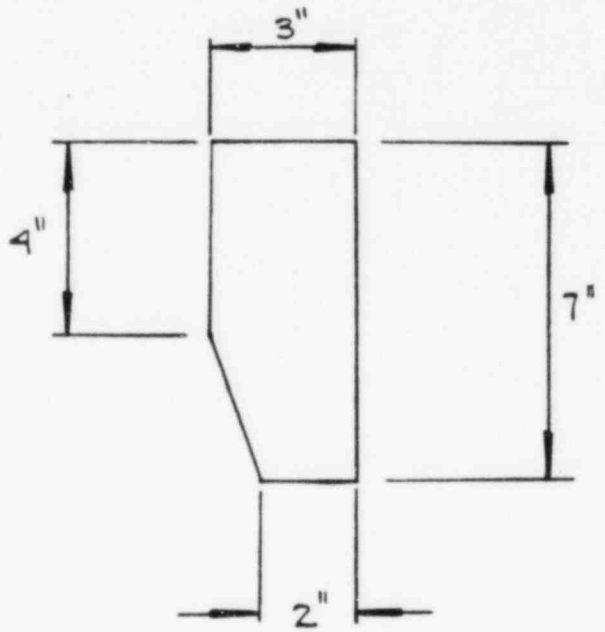


SECTION A-A

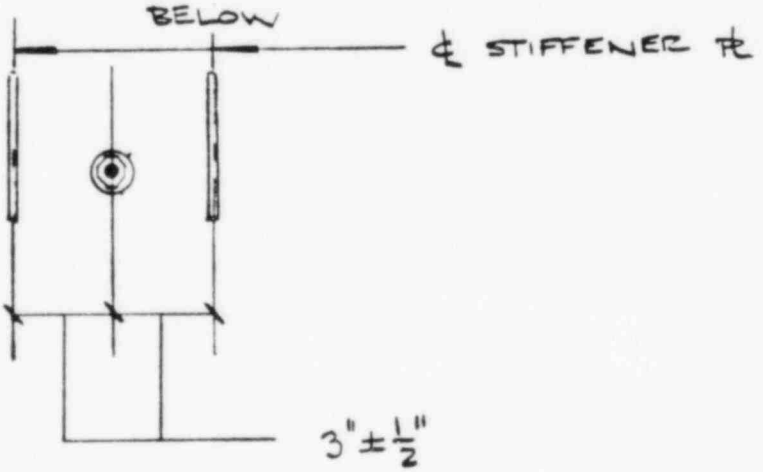
Calculation Sheet

Project	MILLSTONE UNIT NO. 1
Subject	MASONRY WALL REANALYSIS
System	WALLS T-32C & T-32D
Analysis No.	M4
Rev. No.	0
Prepared By:	Joe Chiasson
Date	4/9/82
Checked By:	Donald E Johnson
Date	4/28/82
Job No.	82029
File No.	1/F
Sheet No.	M4 - 6

STIFFENER ℄ DETAIL



NOTE : LOCATE A SET OF ℄ 1/2" STIFFENERS AT EACH 1" ϕ THR'D ROD AS SHOWN BELOW



3. RESULTS OF INVESTIGATION

THE FOLLOWING IS A TABLE OF THE EXISTING ALLOWABLE LOADS FOR COMPARISON WITH THE DESIGN LOADS

WALLS T-32C & T-32D			
	WALL COMPONENT	LIMITING PRESSURES	
		MAXIMUM POSITIVE PRESSURE (PSF) COMPARE TO 265 PSF	MAXIMUM NEGATIVE PRESSURE (PSF) COMPARE TO 360 PSF
SECTION #4	MASONRY IN BENDING	337	337
	MASONRY IN OUT-OF-PLANE SHEAR	398	398
SECTION #5	L 3x3 STRUT	354	857
	WELD GROUPS	485	485
SECTION #6	EXISTING W 21 STRONG & WEAK AXIS BENDING	265	N.A. - SEE NOTE ON PAGE #19
	BOLTED CONNECTIONS	265	> 360
	SHEAR CONNECTORS	603	603

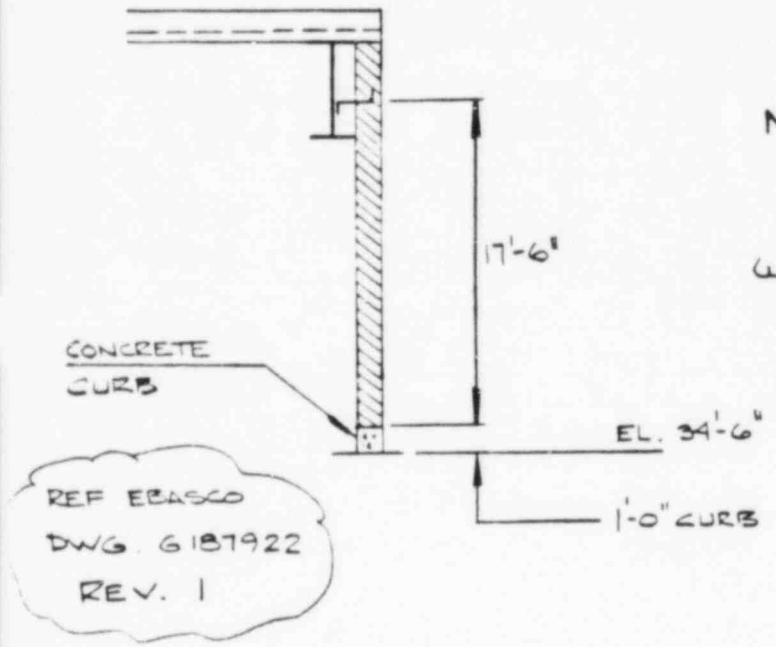
EES

CALCULATION SHEET

PROJECT MILLSTONE UNIT NO. 1	PREPARED BY Joe Chiarenza	DATE 4/1/92
SUBJECT MASONRY WALL ANALYSIS	CHECKED BY David C. Johnson	DATE 4/20/92
SYSTEM WALL T-32C & T-32D	JOB NO. 82028	FILE NO. 1/F
ANALYSIS NO. M4 REV. NO. 0	SHEET NO. M4-8	

SECTION 4.

DETERMINE THE ALLOWABLE LOAD CAPACITY OF THE MASONRY IN BENDING AND OUT OF PLANE SHEAR



REF EBASCO DWG. G187922 REV. 1

FOR BENDING $M'_{ALLOWABLE\ TO\ CMUDS} = 12886 \frac{FT-\#}{FT}$

REF CALC + 82028-M1 PAGE 1/7

$$M_{MAX} = \frac{W_{MAX}\, l^2}{8}$$

$$W_{MAX} = \frac{M_{MAX}\,(8)}{l^2} = \frac{12886\,(8)}{(17.5)^2}$$

$$= \boxed{337\ psf < 360\ psf}$$

MODIFICATION REQUIRED

DETERMINE THE MAXIMUM SPAN BASED ON THE ALLOWABLE BENDING MOMENT

$W = 360\ PSF$

$M_{MAX} = 12886 \frac{FT-\#}{FT}$

$$l_{MAX} = \sqrt{\frac{M_{MAX}\,(8)}{W_{MAX}}} = \sqrt{\frac{12886 \frac{FT-\#}{FT}\,(8)}{360\ \#/FT^2}}$$

$$= \boxed{16.92'\ \text{MAXIMUM}}$$

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL ANALYSIS
SYSTEM	WALL T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiacchia
DATE	4/1/82
CHECKED BY	Donald C. Gibson
DATE	4/20/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-9

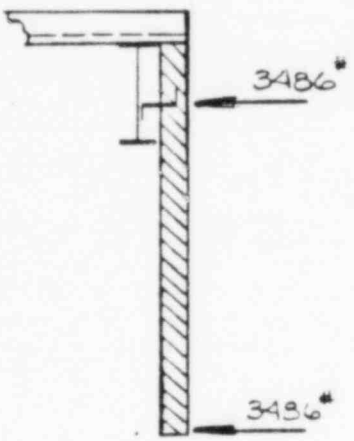
SECTION 4. (CONTINUED)

$V_{ALLOWABLE\ FOR\ OUT\text{-}OF\text{-}PLANE\text{-}SHEAR} = 83$ psi

REF CALC M000
SECTION 14-9
BINDER 80014/1/F

FOR 12" MASONRY BLOCK 42 IN^2 OF SHEAR AREA CAN BE USED FOR EVERY 1 FT OF WALL LENGTH

$$V_{MAX} = 42 \frac{IN^2}{FT} \times 83 \frac{\#}{IN^2} = 3486\ \#/FOOT$$



$$V_{MAX} = \tfrac{1}{2} W_{MAX}\, l$$

$$W_{MAX} = \frac{V_{MAX} \times (2)}{l} = \frac{3486\ \#/FT \times 2}{17.5\ FT}$$

$$= \boxed{398\ psf > 360\ psf}$$

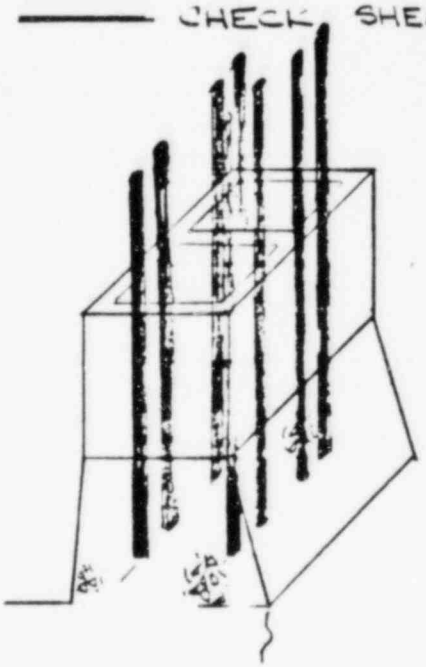
WALL IS ACCEPTABLE IN OUT-OF-PLANE SHEAR

Calculation Sheet	Prepared By: Joe Chiaia	Date: 4/23/82
	Checked By: D.C. Lee	Date: 4/28/82
Project: MILLSTONE UNIT NO. 1	Job No.: 82028	File No.: 1/F
Subject: MASONRY WALL EEANALYSIS		
System: WALL T-32C & T-32D	Sheet No.: M4 - 10	
Analysis No: M4 Rev No: 0		

SECTION 4. (CONTINUED)

CHECK THE TRANSFER OF LOAD FROM MASONRY BLOCK WALL TO 1'-0" CONCRETE CURB.

CHECK SHEARING FAILURE OF 8- #6 BARS



AREA OF 1-#6 BAR = .44 IN²

TOTAL AREA = 8(.44) = 3.52 IN²

ALLOWABLE STRESS = .4 Fy = .4(40 KSI)

TOTAL P = 3.52 IN² × .4(40 KSI) 1.33 = 74.9 K

(1/3 INCREASE REF DC-1 REV. 3)

$\frac{74.9^{K}}{3'-4''} = 22.5\ K/FT$

$\omega = \frac{V(2)}{\ell} = \frac{22.5^{K/FT}(2)}{17.5\ FT} =$ 2571 PSF > 360 PSF > 265 PSF → O.K.

CHECK BEARING OF REBAR AGAINST MASONRY

(#6 BAR)

BEARING AREA = 8 BARS × 3/4" ϕ × 5(3/4)" = 22.5 IN²

(5 BAR DIAMETERS)

TOTAL P = 22.5 IN² × 675 PSI = 15.2 K / 3'-4"

(REF DC-1 REV. 3 EXHIBIT C - PG 3 OF 7)

15.2 K / 3'-4" = 4.56 K/FT

$\omega = \frac{V(2)}{\ell} = \frac{(4.56^{K/FT})(2)}{17.5} =$ 522 PSF

522 PSF > 360 PSF > 265 PSF → O.K.

(FOR DETAILS REFERENCE EBASCO DWG G-187922 REV. 1)

1006.00

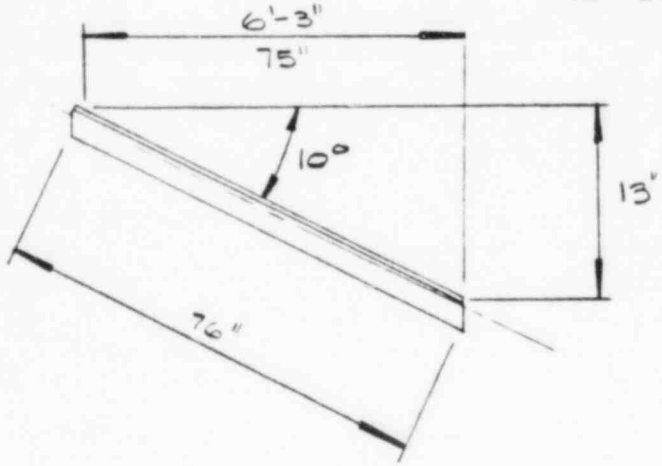
SECTION 5.

DETERMINE THE ALLOWABLE LOAD CAPACITY OF THE EXISTING MODIFICATIONS INCLUDING :

A. L 3x3 STRUT (TENSION & COMPRESSION)
B. WELD OF L 3x3 STRUT TO ℄ 1/2" AND 1/2" FITTED STIFFENER ℄

NOTE : FOR DETAIL OF EXISTING MODS SEE NUSCO DWG. # 25202-54100 SHT 2 OF 2

A. 1. FOR L 3x3x3/8 STRUT IN COMPRESSION - COMPARE TO 265 PSF POSITIVE PRESSURE



$l = 76"$ $A_s = 2.11\ in^2$
$k = 1.0$
$r_z = .587"$

$$\frac{kl}{r_z} = \frac{1.0(76)}{.587} = 129.5$$

$F_a = 8.91$ ksi — REF AISC 7TH EDITION PAGE 5-84

$P_{ALLOWABLE\ COMPRESSION} = 1.33(A_s)(F_a) = 1.33(2.11)\,8.91$

1/3 INCREASE FOR WIND - DC-1 REV 3

$= 25.0$ KIP

HORIZONTAL COMPONENT OF PULTIMATE $= \cos 10^\circ (25.0)$
$= 24.6$ KIP

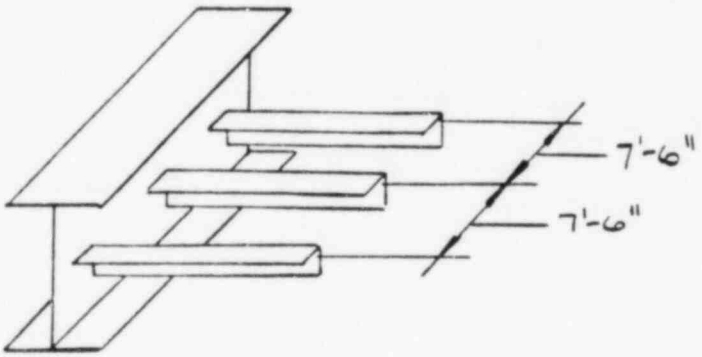
CHECK WIDTH - THICKNESS RATIO : REF AISC 1.9.1

$$\frac{w}{t} = \frac{3}{.375} = 8 < 76/\sqrt{36} = 12.67$$

CONSIDER L3x3 STRUT AS FULLY EFFECTIVE - OK

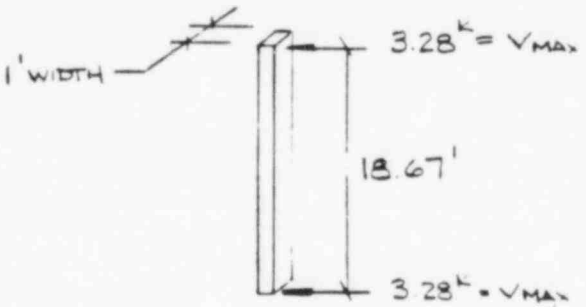
© Bentley Systems, Incorporated

SECTION 5. (CONTINUED)



$$\frac{24.6^{KIPS}}{7'-6"} = 3.28^{KIP/FT}$$

$$V_{MAX} = \tfrac{1}{2}(w_{MAX})\,l$$



$$w_{MAX} = \frac{V_{MAX}\,(2)}{l} = \frac{3.28^{K/FT} \times (2)}{18.67'}$$

$$= \boxed{354\ PSF > 265\ PSF} \quad O.K.$$

A. 2 L 3x3x 3/8 STRUT IN TENSION - COMPARE TO 360 PSF NEGATIVE PRESSURE

$l = 76"$
$r_z = .587"$
$A_s = 2.11\ IN^2$

SLENDERNESS RATIO : (REF AISC 1.8.4)

$$\frac{l}{r} = \frac{76}{.587} = 129.5 < 240 \quad - O.K.$$

(REF AISC 1.5.1.1)

$$F_t = .60\,F_y = .6(36) = 22\ KSI$$

(1/3 INCREASE FOR WIND - DC-1 REV 3)

$$P_{ULTIMATE} = A_S(F_b) = 2.11(1.33)(22) = 61^{KIP}$$

HORIZONTAL COMPONENT OF $P_{ULTIMATE} = \cos 10^{\circ}(61)$

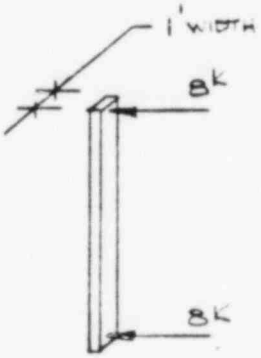
$= 60\ KIP$

EES 101 Rev 0

SECTION 5. (CONTINUED)

DETERMINE ALLOWABLE PRESSURE:

$$V_{MAX} = \frac{60^{KIP}}{7'-6''} = 8\ KIP/FT$$



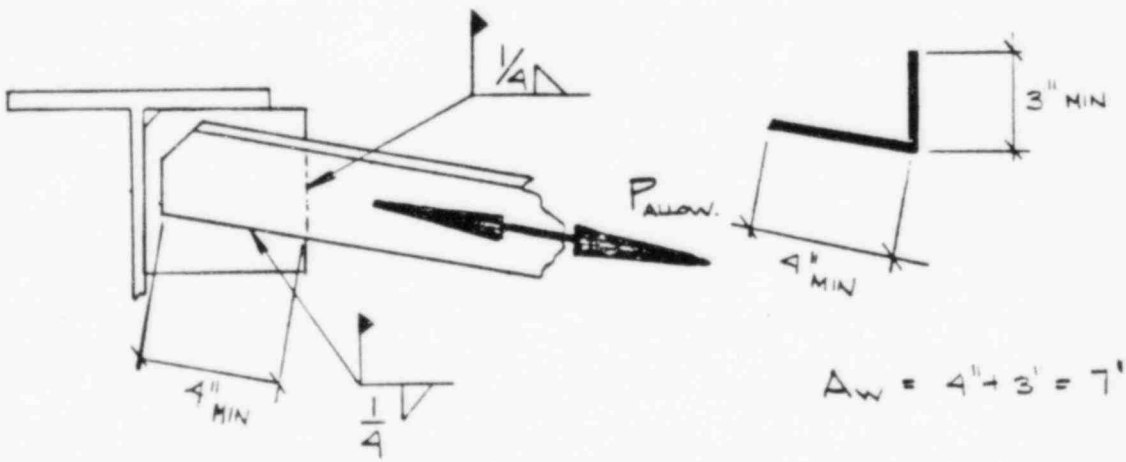
$$V_{MAX} = 1/2(w_{MAX})\,l$$

$$w_{MAX} = \frac{V_{MAX}(2)}{l} = \frac{8^{K}/FT \times (2)}{18.67'} = 857\ PSF$$

857 PSF > 360 PSF OK.

B. CHECK WELD OF L 3x3 STRUT TO PL 1/2" AND TO 1/2" FITTED STIFFENED PL

— WELD OF L 3x3 TO PL 1/2 —

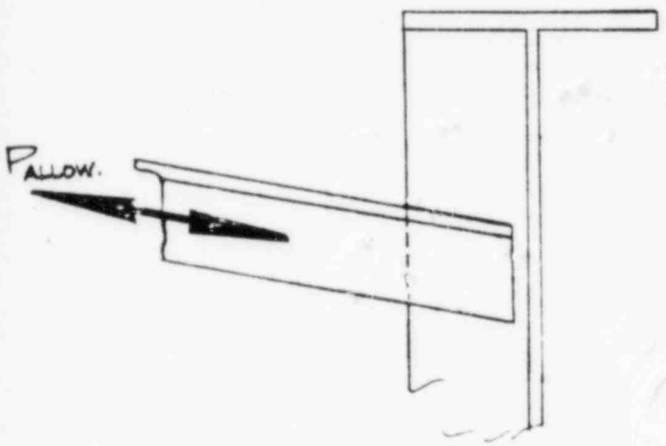


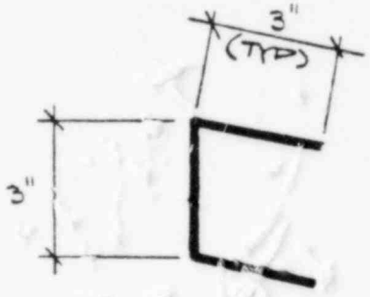
$A_W = 4'' + 3'' = 7''$

EES	CALCULATION SHEET	PREPARED BY	DATE
		Joe Chiarenza	4/2/82
		CHECKED BY	DATE
		David F. Hanna	4/20/82
	PROJECT MILLSTONE UNIT NO. 1	JOB NO. 82028	FILE NO. 1/F
	SUBJECT MASONRY WALL ANALYSIS		
	SYSTEM WALLS T-32C & T-32D		
	ANALYSIS NO. M4 REV. NO. 0	SHEET NO.	M4 - 14

SECTION 5. (CONTINUED)

— WELD OF L 3x3 TO FITTED STIFFENER PL —





$A_w = 3" \times 3 = 9"$

NOTE : ANALYZE THE WELD OF L 3x3 TO PL 1/2 TO PROVIDE ANALYSIS OF THE MOST CRITICAL WELD (7" < 9")

$$P_{ALLOWABLE} = 1.33\,(21\ KSI)(.707)(7")\,1/4 = 34.6^{K}$$

(21 KSI FOR E70-XX ELECTRODE REF DC-1 REV 3 PG 7/10)

(1/3 INCREASE FOR WIND - DC-1 REV. 3)

HORIZONTAL COMPONENT OF $P_{ALLOWABLE} = \cos 10^{\circ}\,(34.6^{K}) = 34^{KIP}$

DETERMINE ALLOWABLE PRESSURE :

$$V_{MAX} = \frac{34^{KIP}}{7'-6"} = 4.53^{KIP/FT} = 1/2\ W_{MAX}\ \ell$$

$$W_{MAX} = \frac{V_{MAX}(2)}{\ell} = \frac{(4.53^{K/FT})(2)}{7.5} = 485\ PSF$$

EES 101 Rev 0

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL ANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4 REV. NO. 0

PREPARED BY	Joe Chiarra
DATE	4/2/8:
CHECKED BY	Dinakar P Johnson
DATE	4/28/87
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-15

SECTION 5. (CONTINUED)

485 psf > 360 psf OK

NOTE: THE WELD OF THE PL 1/2 TO THE EXISTING W 21 x 55 AND THE WELD OF THE 1/2" FITTED STIFFENER PL TO THE EXISTING W 21 x 55 ARE DEEMED ACCEPTABLE BY COMPARISON OF THEIR RESPECTIVE WELD AREAS WITH THE 7" WELD AREA OF THE WELD GROUP JOINING THE L 3x3 STRUT TO THE PL 1/2

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL ANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiarenza
DATE	4/2/92
CHECKED BY	David C. Stinson
DATE	4/29/92
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-16

SECTION 6.

DETERMINE THE ALLOWABLE LOAD CAPACITY OF THE EXISTING STEEL - REFERENCE EBASCO DWG. G197995 Rev 6

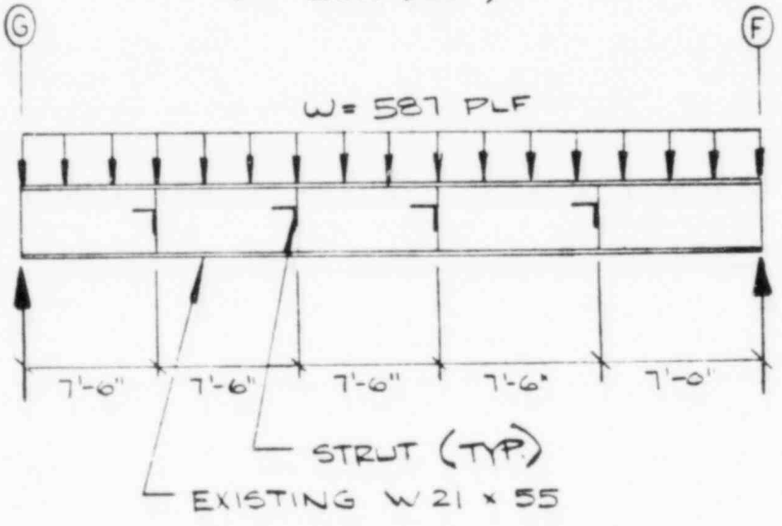
A. CHECK W21 x 55 FOR COMBINED STRONG AND WEAK AXIS BENDING

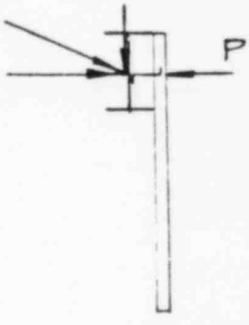
B. CHECK BOLTED CONNECTIONS FOR W21 x 55

C. CHECK SHEAR IN SHEAR CONNECTORS

D. CHECK ADJACENT W21 FOR LOADING INDUCED BY STRUT ANGLES

A. 1. STRONG AND WEAK AXIS BENDING DUE TO POSITIVE PRESSURE (COMPARE TO 265 PSF)





DETERMINE UNIFORM LOAD IN DIRECTION OF STRONG AXIS

REF EBASCO DWG G197995 Rev 6

LL = 100 PSF

SPAN BETWEEN ROOF FRAMING = 6'-3"

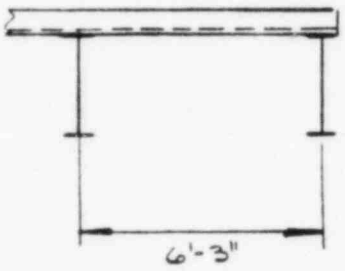
EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL ANALYSIS
SYSTEM	WALL T-32C & T-32D
ANALYSIS NO.	M4 REV. NO. 0

PREPARED BY	DATE 4/2/92
CHECKED BY	DATE 4/2/92
JOB NO. 82028	FILE NO. 1/F
SHEET NO.	M4-17

SECTION 6. (CONTINUED)



$$W_{LL.} = \frac{6'-3''}{2} \times 100\ PSF = 313\ PLF$$

$W_{D.L}$ = 1.) SLAB $\frac{4''}{12''/1} \times 150\ ^{\#}/_{FT^3} \times \frac{6'-3''}{2}$ = 156

2.) CONC. FILL - ASSUME $20^{PLF} \times \frac{6'-3'}{2}$ = 63

3.) BEAM WEIGHT - = 55

274 PLF

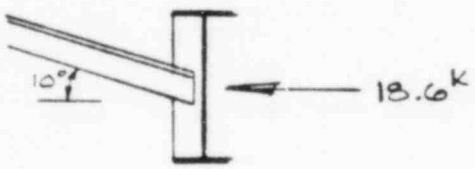
TOTAL w = 313 + 274 = 587 PLF

CONSIDER THE HORIZONTAL PRESSURE = 265 PSF

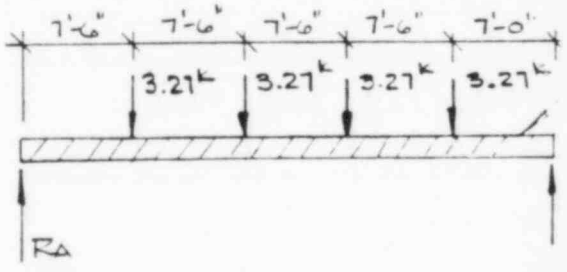
REACTION AT TOP OF WALL = $\frac{265\ (18.67)}{2}$ = 2.47 K/FT

HORIZONTAL REACTION AT STRUT ANGLES = 2.47 K/FT × 7.5' = 18.6 K

VERTICAL REACTION AT STRUT ANGLES:



18.6 K × TAN 10° = 3.27 KIP



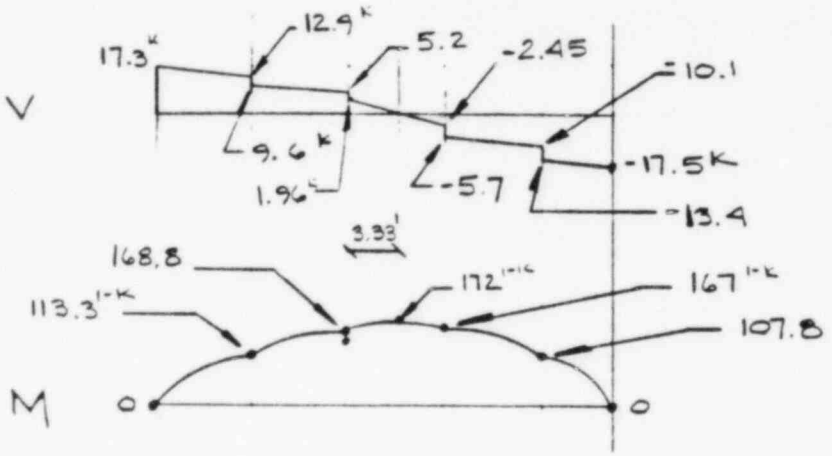
—DETERMINE f_{bx}

$$R_A = \frac{.587\ (37)}{2} + \frac{3.27}{37}(7 + 14.5 + 22 + 29.5)$$

= 17.3 K

$$R_B = \frac{.587(37)}{2} + \frac{3.27}{37}(7.5 + 15 + 22.5 + 30) = 17.5\ K$$

SECTION 6. (CONTINUED)



$M_{MAX} = 172'\text{-K}$

FOR W 21 x 55 : $S_x = 110\ IN^3$ $L_c = 8.7'$

$S_y = 11.8\ IN^3$ $L_u = 9.5'$

$$f_{bx} = \frac{M}{S_x} = \frac{172'\text{-K} \times 12''/'}{110\ IN^3} = 18.76$$

$F_{bx} = 1.33(24\ KSI)$ SINCE 1.) CONTINUOUS LATERAL SUPPORT

2.) $F_y' = 49.0\ KSI$, $F_y'' = 55.2\ KSI$ } FULLY COMPACT

(1/3 INCREASE FOR WIND - DC-1 REV. 3)

$F_{by} = .9(F_Y) = .9(36)$ ← SINCE $1.33(.75)F_y = 35.9\ KSI > .9(F_y)$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \rightarrow \frac{18.76}{1.33(24)} + \frac{f_{by}}{.9(36)} = 1.0$$

$f_{by} = 13.35\ KSI$

$$M_y\ \text{ALLOWABLE} = f_{by}(S_y) = \frac{13.35(11.8)}{12} = 13.13\ K\text{-FT}$$

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL REANALYSIS
SYSTEM	WALL T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0
PREPARED BY	Joe Diaso
DATE	4/2/87
CHECKED BY	Mark C. Dixon
DATE	4/28/87
JOB NO.	8029
FILE NO.	1/F
SHEET NO.	M4-19

SECTION 6. (CONTINUED)

DETERMINE THE ACTUAL My

SPAN BETWEEN STRUTS = 7.5'

$$My_{MAX} = \frac{wl^2}{12} = \frac{2.474\,(7.5)^2}{12} = 11.6^{FT-K}$$

$11.6^{FT-K} < 13.13^{FT-K}$ ← COMBINED STRONG AND WEAK AXIS BENDING IS OK FOR A POSITIVE PRESSURE = 265 PSF

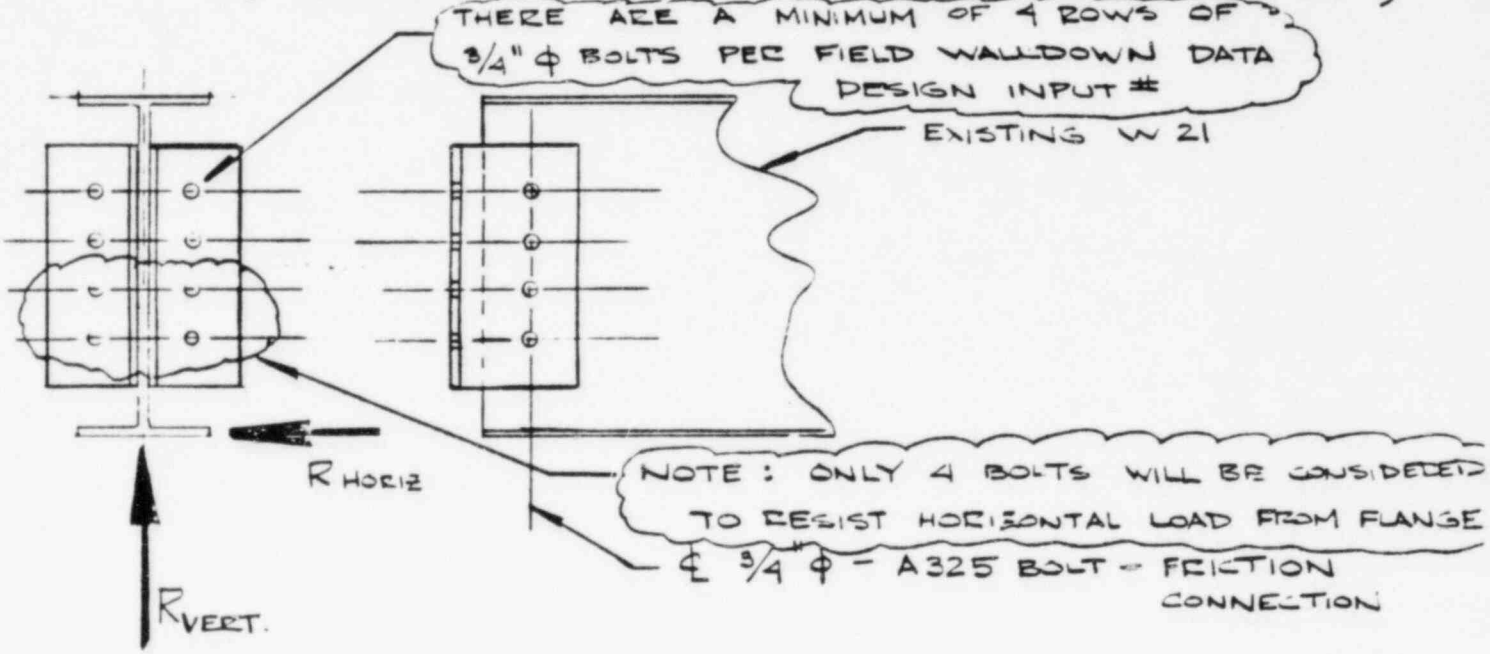
NOTE : IT IS NOT NECESSARY AT THIS TIME TO CHECK STRONG & WEAK AXIS BENDING IN THE EXISTING W21 FOR A NEGATIVE PRESSURE OF 300 PSF SINCE MODIFICATIONS ADDED TO STRENGTHEN THE MASONRY WILL GREATLY ALTER THE RESULTS OBTAINED FOR A STRONG & WEAK AXIS BENDING ANALYSIS.

ADDITION OF L 8×4's TO THE BOTTOM FLANGE OF THE EXISTING W21 WILL CAUSE ALL WEAK AXIS BENDING TO BE RESISTED BY THE BOTTOM FLANGE RATHER THAN THE ENTIRE CROSS-SECTION, THEREBY OVERSTRESSING f_{by}.

ADDITIONAL STRUT ANGLES WILL HAVE TO BE ADDED. FURTHER ANALYSIS WILL BE DONE FOR COMBINED STRONG AND WEAK AXIS BENDING WITH THE ADDITIONAL STRUTS IN PLACE. THIS WILL BE DONE FOR BOTH POSITIVE & NEGATIVE PRESSURE. SEE SECTION 4. – MODIFICATION DESIGN

SECTION 6. (CONTINUED)

B. CHECK BOLTED CONNECTION FOR EXISTING W 21 ASSUMING 3 ADDITIONAL STRUTS ARE IN PLACE. (SEE PAGE 34)



FOR 3/4" ϕ - A325 - REFERENCE AISC 7TH EDITION - MANUAL OF STEEL CONSTRUCTION - PAGE 4-4 HIGH STRENGTH BOLTS IN FRICTION TYPE CONNECTIONS.

ALLOWABLE SHEAR = 6.63 KIP/BOLT × 1.33 = 8.82 K/BOLT

FOR A POSITIVE PRESSURE OF 265 PSF

REF P 29 OF THIS CALC

$R_{VERT} = 17.96^{K}$ — SAY 18^{K}

REF P 31 OF SECTION 4 THIS CALCULATION

$R_{HORIZ} = 2.71^{KLF} \times \frac{7.5'}{2} = 10.2^{K}$

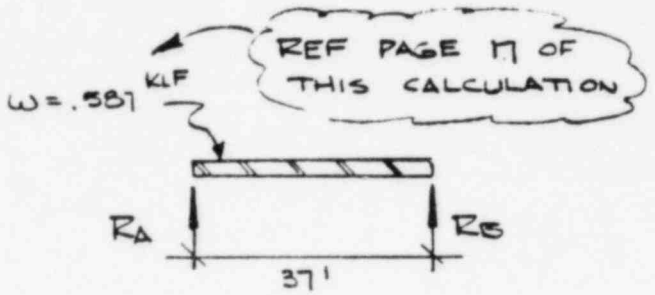
$$R = \sqrt{\left[\frac{18^{K}}{8\ \text{BOLTS}}\right]^2 + \left[\frac{10.2^{K}}{4\ \text{BOLTS}}\right]^2} = 3.4^{K/\text{BOLT}} < 8.82^{K/\text{BOLT}}$$

O.K.

Calculation Sheet	Prepared By: Joe Chiaia	Date 4/6/82
	Checked By: [illegible]	Date 4/29/82
Project MILLSTONE UNIT NO. 1	Job No. 82028	File No. 1/F
Subject MASONRY WALL REANALYSIS		
System WALL T-32C & T-32D	Sheet No.	M4 - 21
Analysis No. M4 Rev. No. 0		

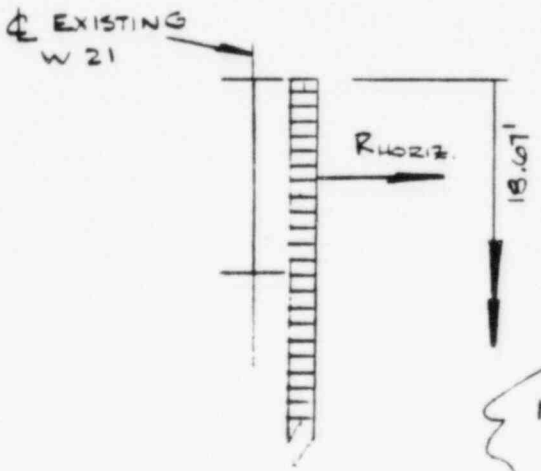
SECTION 6. (CONTINUED)

FOR A NEGATIVE PRESSURE OF 360 PSF



$$R_A = R_B = \frac{wl}{2} = \frac{.587(37)}{2} = 10.9^K$$

$$R_A = R_Z = R_{VERT}$$



DETERMINE $R_{HORIZONTAL}$:

$$360 \text{ PSF} \times \frac{18.67'}{2} = 3361 \text{ PLF}$$

$$R_{HORIZ} = 3361 \text{ PLF} \times \frac{7.5'}{2} = 12.6^K$$

NOTE : AGAIN, ONLY 4 BOLTS WILL BE CONSIDERED TO RESIST THE HORIZONTAL LOAD FROM THE FLANGE

$$R_{RESULTANT\ SHEAR} = \sqrt{\left[\frac{R_{VERT}}{8}\right]^2 + \left[\frac{R_{HORIZ}}{4}\right]^2}$$

$$= \sqrt{\left[\frac{10.9^K}{8 \text{ BOLTS}}\right]^2 + \left[\frac{12.6^K}{4 \text{ BOLTS}}\right]^2}$$

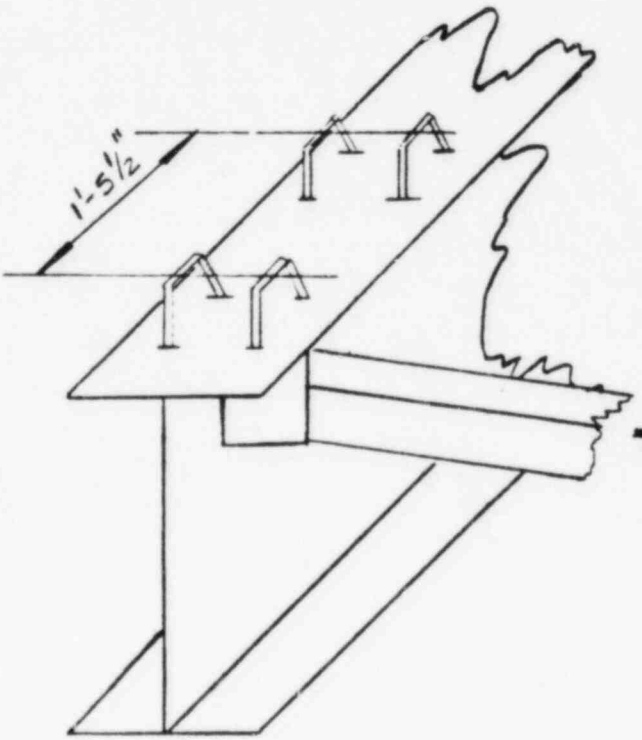
$$= \boxed{3.43^K/\text{BOLT} < 8.82^K/\text{BOLT ALLOWABLE}}$$

→ CONNECTIONS ARE O.K.

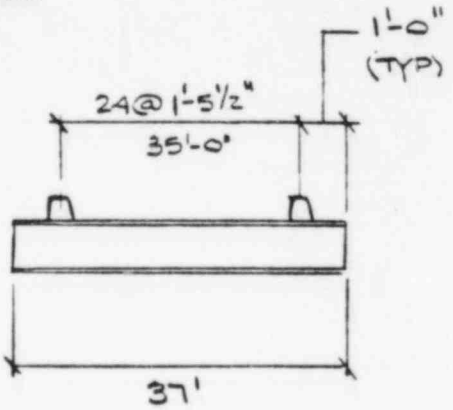
Calculation Sheet	Prepared By:	Date
		4/9/82
Project MILLSTONE UNIT NO. 1	Checked By:	Date
Subject MASONRY WALL REANALYSIS	Job No. 82028	File No. 1/F
System WALLS T-22C & T-22D	Sheet No.	M4-22
Analysis No. M4 Rev. No. 0		

SECTION 6. (CONTINUED)

C. CHECK THE SHEAR IN THE SHEAR CONNECTORS



NOTE: 50 SHEAR CONNECTORS ARE ATTACHED TO A 37' SPAN OF BEAM. CONNECTOR ARE ATTACHED IN 2 EQUAL ROWS (REFERENCE EBASCO DWG. G187995 REV. 6)



STRUTS ARE SPACED AT 7'-6" = 90"

$$\text{SHEAR CONNECTORS / STRUT} = \frac{7'\text{-}6\text{"} \text{ / STRUT}}{1'\text{-}5\tfrac{1}{2}\text{" / CONNECTOR}} \times 2$$

2 ROWS OF 25 CONNECTORS

= 10 CONNECTORS / STRUT

USE 8 CONNECTORS / STRUT TO BE CONSERVATIVE

NOTE: SHEAR CONNECTORS WILL BE SUBJECT TO TWO INDEPENDENT LOADS SIMULTANEOUSLY. 1) SHEAR LOADS FROM L 3x3 STUTS AND 2) SHEAR LOADS IMPOSED BY BENDING OF THE ROOF SYSTEM SINCE COMPOSITE ACTION WAS USED IN THE ROOF DESIGN

SECTION 6. (CONTINUED)

SHEAR LOAD IMPOSED BY BENDING OF ROOF SYSTEM:

$V_H = 5.72^{k}$ — REF CALC # 80014 - M107 - 22

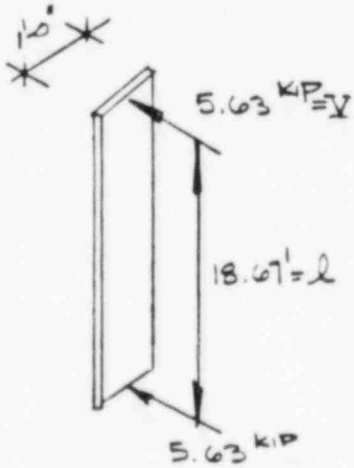
ALLOWABLE LOAD PER CONNECTOR:

$V_{ALLOWABLE} = 11.0^{k}$ — REFERENCE COFAR CATALOG

$V_{STRUTS} = V_{ALLOWABLE} - V_H = 11.0^{k} - 5.72^{k} = 5.28^{KIP}$

$5.28^{KIP}/_{CONNECTOR} \times 8\ \frac{CONNECTOR}{STRUT} = 42.2^{KIP}/_{STRUT}$

$\frac{42.2^{KIP}/_{STRUT}}{7'-6''/_{STRUT}} = 5.63^{KIP}/_{FT}$ — LINE LOAD ALONG WALL



ALLOWABLE PRESSURE

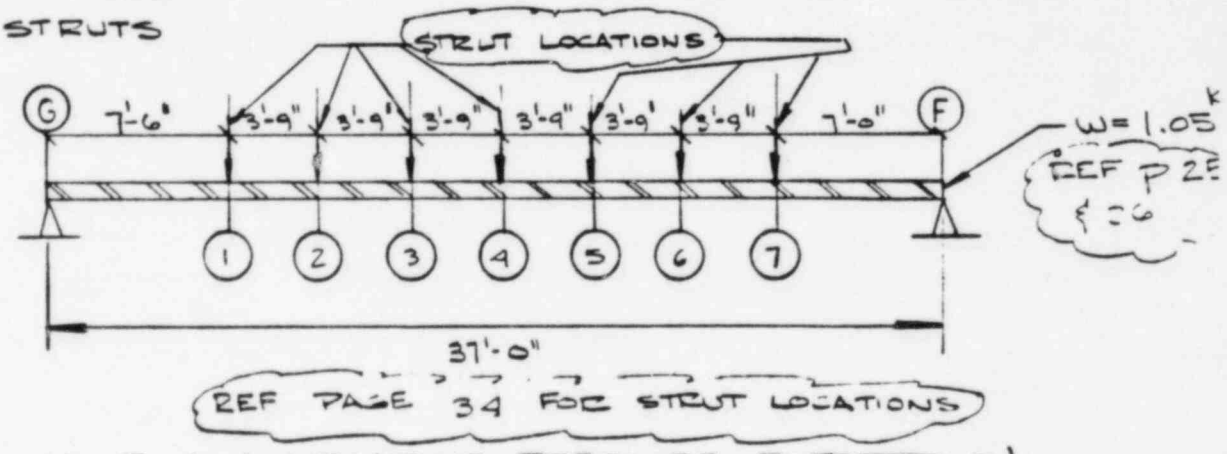
$$P_{ALL} = \frac{V \times 2}{l} = \frac{5.63\ \frac{KIP}{FT} \times 2}{18.67\ FT}$$

$$= \boxed{603\ PSF}$$

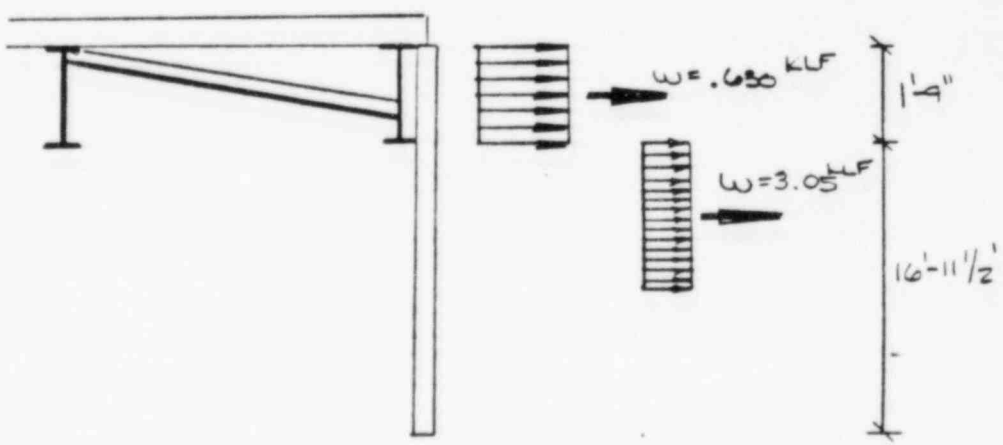
Calculation Sheet	Prepared By: Joe Chasser	Date: 4/28/82
	Checked By: Donald E Johnson	Date: 4/28/82
Project: MILLSTONE UNIT NO. 1	Job No: 82029	File No: 1/F
Subject: MASONRY WALL REANALYSIS		
System: WALLS T-32C & T-32D	Sheet No:	M4 - 24
Analysis No: M4 Rev. No: 0		

SECTION 6. (CONTINUED)

CHECK THE ADJACENT W 21 FOR LOADS IMPOSED BY BOTH EXISTING AND ADDITIONAL STRUTS



NOTE : A NEGATIVE PRESSURE EXERTED ON WALLS T-32C & D WILL CAUSE TENSION IN THE STRUTS AND AN ADDITIONAL DOWNWARD LOAD ON THE ADJACENT W 21 x 55



W_{TOP} = 1'-9" x 360 PSF = 630 PLF

W_{BOTTOM} = 16'-11½"/2 x 360 PSF = 3053 PLF

W_{TOTAL} = 3.05 KLF + .630 KLF = 3.68 KLF

Calculation Sheet	Prepared By: Joe Chiarenza	Date 4/23/87
	Checked By: Donald E. Johnson	Date 4/28/87
Project MILLSTONE UNIT NO. 1	Job No. 8202B	File No. 1/F
Subject MASONRY WALL REANALYSIS		
System WALL T-32C & T-32D	Sheet No.	M4 – 25
Analysis No M4 Rev. No 0		

SECTION 6. (CONTINUED)

LOAD IN STRUTS 2, 3, 4, 5, & 6

$3'-9'' \times 3.68^{KLF} = 13.8^{K}$

DOWNWARD LOAD ON ADJACENT W21

$= 13.8^{K} \times \sin 10^{\circ} = 2.40^{K}$

LOAD IN STRUT 1

$7'-6''/2 + 3'-9''/2 \times 3.68 KLF = 20.7^{K}$

DOWNWARD LOAD ON ADJACENT W21

$= 20.7^{K} \times \sin 10^{\circ} = 3.59^{K}$

LOAD IN STRUT 7

$7'-0''/2 + 3'-9''/2 \times 3.68 KLF = 19.8^{K}$

DOWNWARD LOAD ON ADJACENT W21

$= 19.8^{K} \times \sin 10^{\circ} = 3.43^{K}$

UNIFORM LOAD ON ADJACENT W21

LL = 100 PSF ← REF EBASCO DWG 187955 REV 6

DL : SLAB = $\frac{4''}{12''} \times 150$ PCF = 50 PSF

MISC = ASSUME = 10 PSF

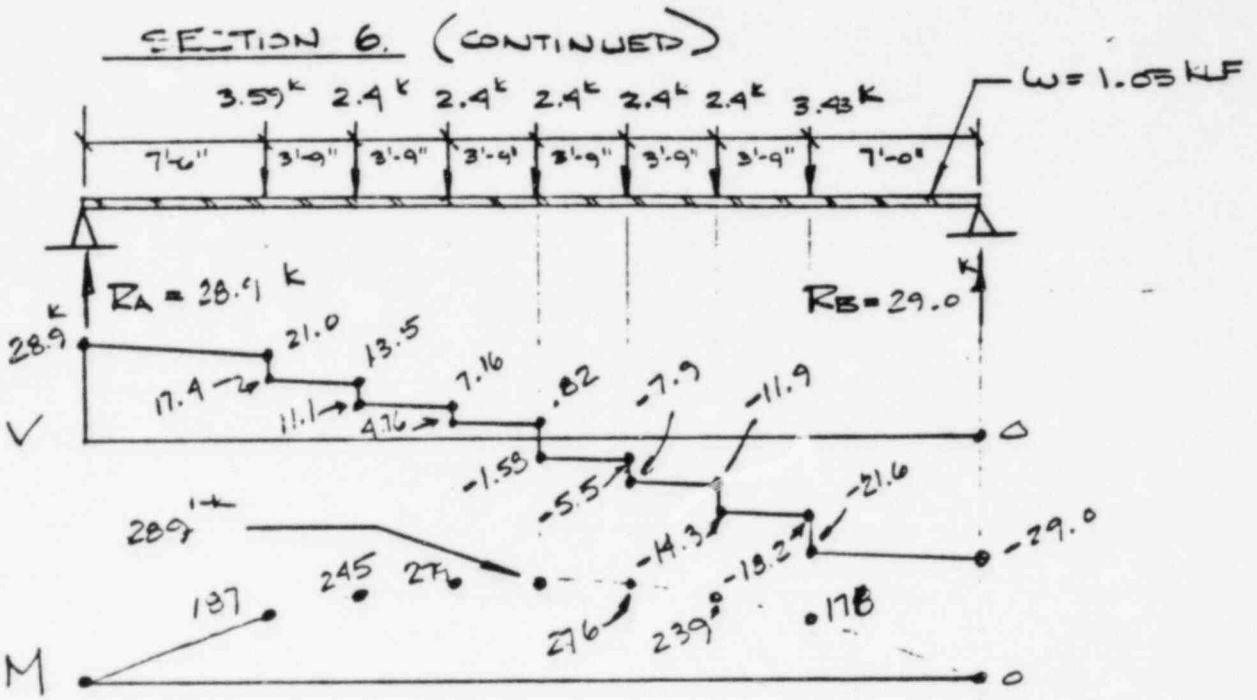
60 PSF

TOTAL DL + LL = 160 PSF

SPAN BETWEEN ADJACENT W21 → $W = 160 \text{ PSF} \times (6'-3'') + 55 \text{ PLF} = 1.05^{KLF}$ ← WEIGHT OF BEAM

Calculation Sheet	Prepared By: Joe Dimeo	Date: 4/28/82
	Checked By: Mark E. Johnson	Date: 4/28/82
Project: MILLSTONE UNIT NO. 1	Job No.: 8079	File No.: 1/F
Subject: MASONRY WALL REANALYSIS		
System: WALLS T-32C & D	Sheet No.:	M4-26
Analysis No: M4 Rev No: 0		

SECTION 6. (CONTINUED)



$$R_A = \frac{1.05(37)}{2} + 3.43\left(\frac{7}{37}\right) + \frac{2.4}{37}\left(10.75' + 14.5' + 18.25' + 22 + 25.75\right) + 3.59\left(\frac{7.5}{2}\right)$$

$$= 28.9^{\,k}$$

$$R_B = \frac{1.05(37)}{2} + 3.59\left(\frac{7.5}{37}\right) + \frac{2.4}{37}\left(11.25 + 15 + 18.75 + 22.5 + 26.25\right) + 3.43\left(\frac{7}{2}\right)$$

$$= 29.0^{\,k}$$

REF AISC 7TH EDITION P 2-140

FULL LATERAL SUPPORT OF COMPRESSION FLANGE ∴ $F_y' = 44$ ksi & $F_y'' = 55.2$ ksi

REFERENCE CALC # 8004-M107-22

$M_{MAX} = 289^{\,1\text{-k}}$

$F_b = 1.33(24) = 32$ ksi

$S_{b\ \text{COMPOSITE}}^{\text{STEEL}} = 143.7\ \text{IN}^3$

$S_{t\ \text{COMPOSITE}} = 351\ \text{IN}^3$

$$f_{b\ \text{STEEL}} = \frac{M}{S} = \frac{289^{\,1\text{-k}} \times 12^{\,\text{IN/FT}}}{143.7} = 24.1^{\,\text{ksi}} < 32\ \text{ksi}$$ OK

1/3 INCREASE FOR WIND

$$f_{b\ \text{CONCRETE}} = \frac{12M}{n S_t} = \frac{12(289)}{(9)351} = 1.10\ \text{ksi} < 1.33(.45 f_c') = 1.7$$

OK

REF AISC 7TH

EES

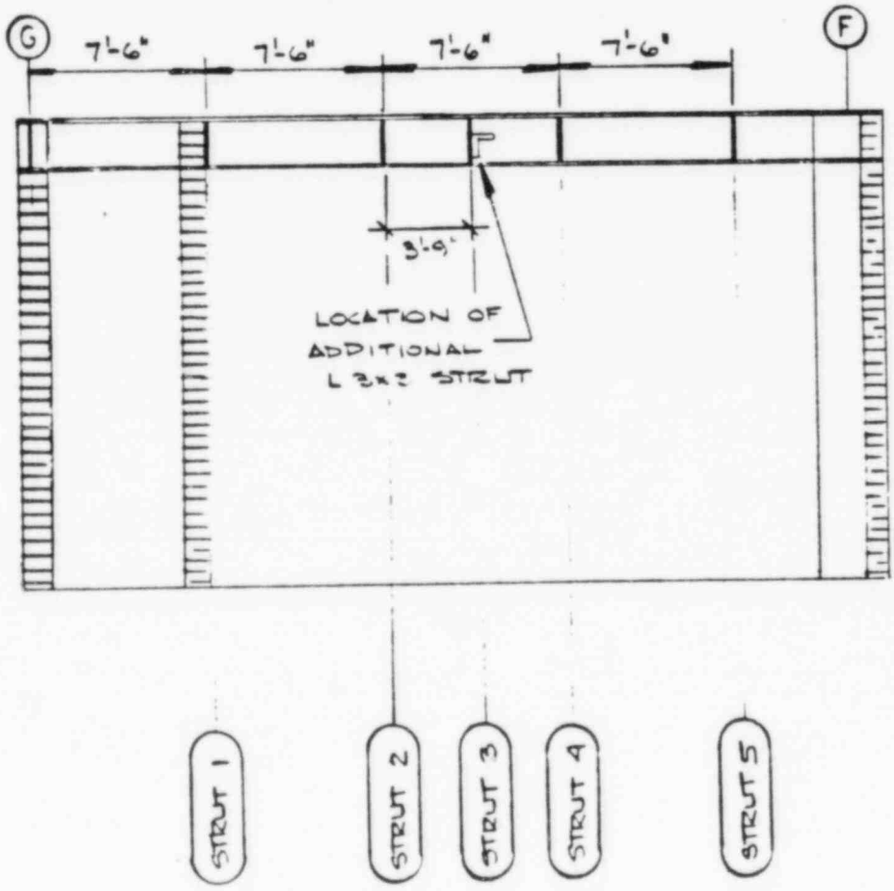
CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL REANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiarot
DATE	4/6/82
CHECKED BY	Donald C. Johnson
DATE	4/28/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-27

SECTION 7. - MODIFICATION DESIGN

MODIFICATIONS WILL INCLUDE L8x4's WELDED TO THE BOTTOM FLANGE OF THE EXISTING W 21 AND THRU-BOLTED TO THE WALL. THE ADDITION OF ANOTHER L 3x3 STRUT WILL BE NECESSARY IN ORDER TO SATISY WEAK AXIS BENDING.



EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL REANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

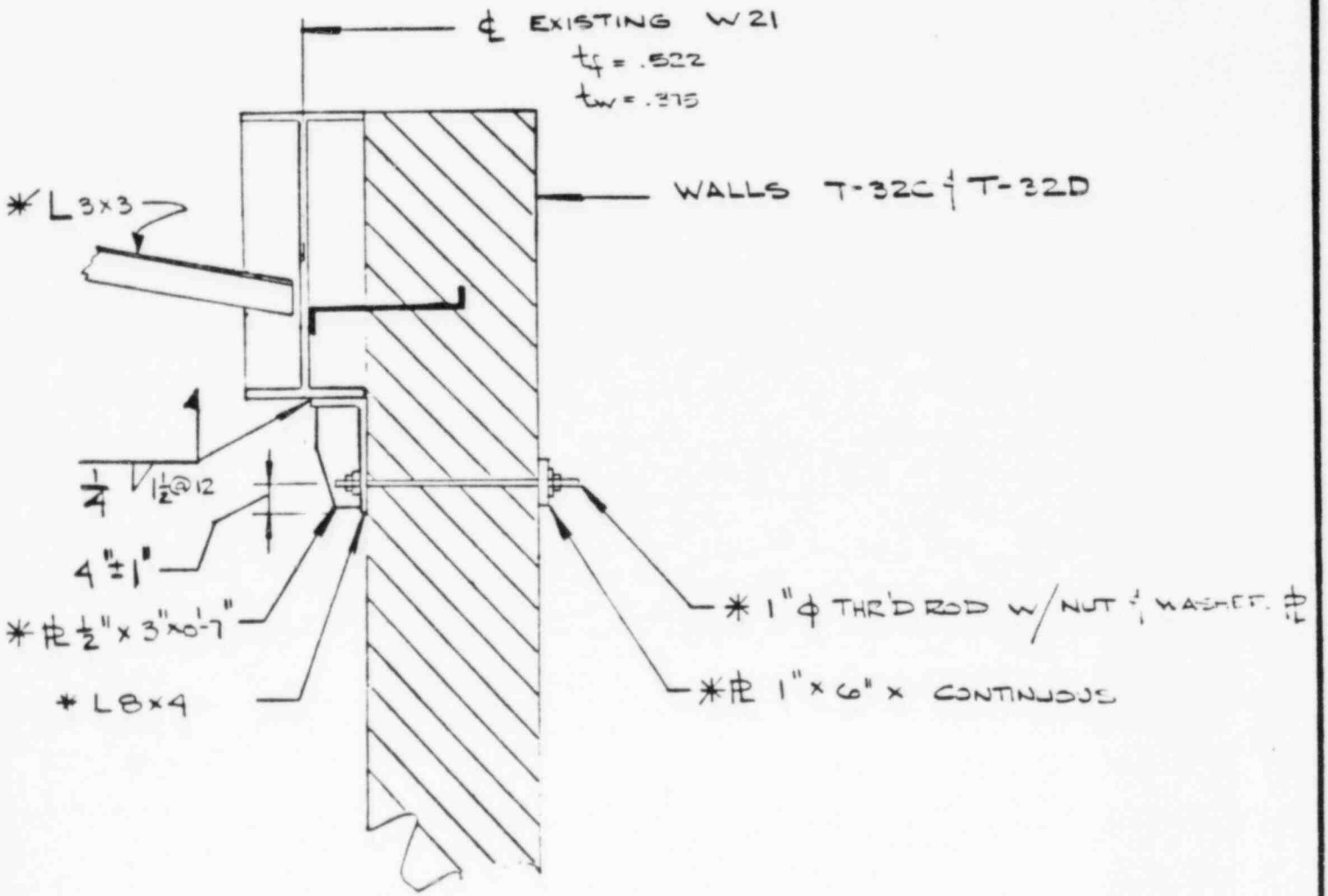
PREPARED BY	Joe Chaves
DATE	4/5/82
CHECKED BY	Donald E Johnson
DATE	4/28/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-28

SECTION 7.

DESIGN MODIFICATIONS TO GIVE ADDITIONAL CAPACITY TO WALLS T-32C & T-32D IN ORDER TO MEET THE FOLLOWING :

TORNADO WIND POSITIVE PRESSURE = 265 PSF

TORNADO DEPRESSIZATION NEG. PRESSURE = 360 PSF



* - DENOTES NEW MATERIAL

EES

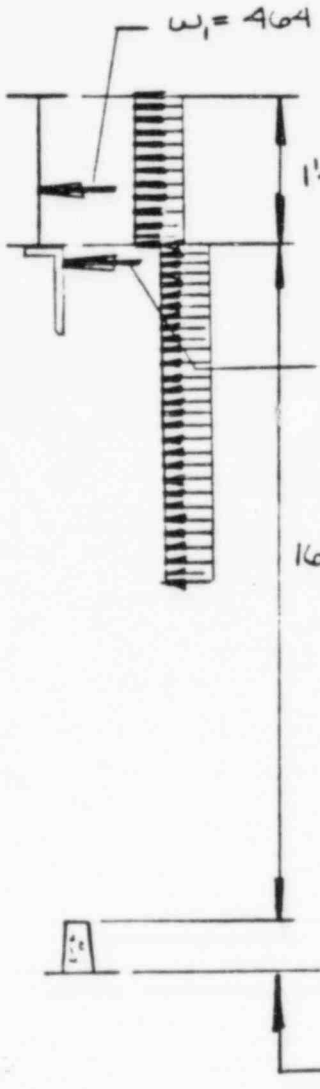
CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL ANALYSIS
SYSTEM	WALLS T-32 & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiaia
DATE	4/7/82
CHECKED BY	Donald C Johnson
DATE	4/28/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4 -29

SECTION 7. - MODIFICATION DESIGN

CHECK COMBINED STRONG & WEAK AXIS BENDING IN EXISTING W 21 FOR A POSITIVE PRESSURE OF 265 PSF (360 MPH TORNADO WIND VELOCITY)



LOAD RESISTED BY THE ENTIRE W 21 CROSS-SECTION

w_1 = 1'-9" x 265 PSF = 464 PLF

LOAD RESISTED BY ONLY THE BOTTOM FLANGE OF THE EXISTING W 21

$w_2 = \frac{16'-11\frac{1}{2}"}{2} \times 265 \text{ PSF} = 2247 \text{ PLF}$

NOTE: TO CALCULATE STRUT L 3x3 LOAD, $W = w_1 + w_2$

SO $W = .464^{KLF} + 2.247^{KLF} = 2.71^{KLF}$

EES	CALCULATION SHEET	PREPARED BY Joe Chacon	DATE 4/6/82
	PROJECT MILLSTONE UNIT NO. 1	CHECKED BY Donald E Johnson	DATE 4/28/82
	SUBJECT MASONRY WALL REANALYSIS	JOB NO. 82028	FILE NO. 1/F
	SYSTEM WALL T-32C & T-32D		
	ANALYSIS NO. M4 REV. NO. 0	SHEET NO. M4-30	

SECTION 7. (CONTINUED)

— LOAD ON STRUTS 1 & 5 —

HORIZONTAL LOAD = 7.5' x 2.71 KLF = 20.3 KIP

VERTICAL LOAD = 20.3 K x TAN 10° = 3.58 KIP

— LOAD ON STRUTS 2 & 4 —

HORIZONTAL LOAD = $\left(\frac{7.5'}{2} + \frac{3.75'}{2}\right)$ x 2.71 KLF = 15.2 KIP

VERTICAL LOAD = 15.2 KIP x TAN 10° = 2.68 KIP

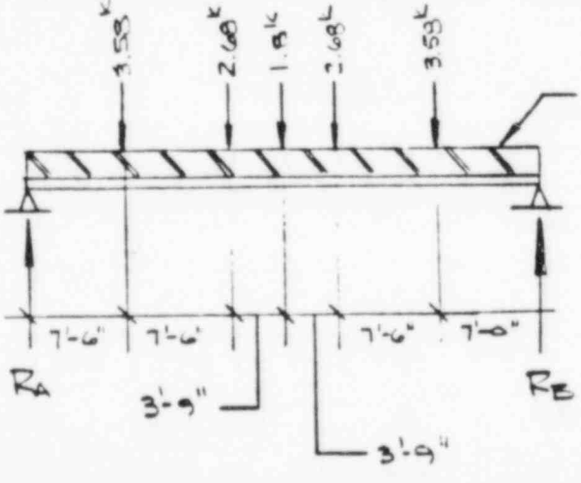
— LOAD ON STRUT 3 - ADDITIONAL STRUT

HORIZONTAL LOAD = 3.75' x 2.71 KLF = 10.2 K

VERTICAL LOAD = 10.2 K x TAN 10° = 1.80 K

DOWNWARD UNIFORM LOAD = 587 PLF

REF P. 17 OF THIS CALCULATION



$$R_A = \frac{.587(37)}{2} + \frac{3.58}{37}(7 + 29.5) + \frac{2.68}{37}(14.5 + 22) + \frac{1.8}{37}(15.5)$$

$$= 17.92^{K}$$

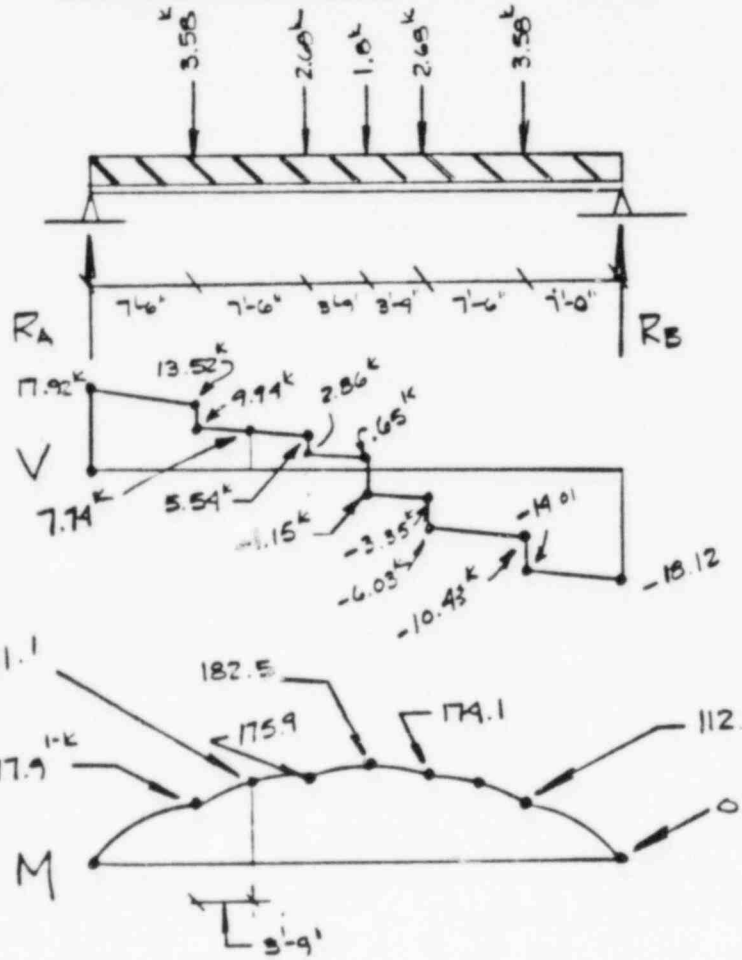
EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL REANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiaia — DATE 4/6/82
CHECKED BY	Donald E Johnson — DATE 4/28/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-31

SECTION 7. (CONTINUED)



$$R_B = \frac{.587(37)}{2} + \frac{3.58}{37}(7.5+30) + \frac{2.68}{37}(15+22.5) + \frac{1.8}{37}(19.75)$$

$$= 18.12^{K/F}$$

$$M_{MAX} = 182.5^{1-K}$$

$$f_{bx} = \frac{M_x}{S_x} = \frac{182.5 \times 12}{110\ in^3}$$

REF P18

$$= 19.91\ ksi$$

$$F_{bx} = 1.33(24)$$ ← REF F18

DETERMINE F_{by} FOR BOTTOM FLANGE ONLY

$$\frac{b_f}{t_f} = \frac{8.215"}{.522"} = 15.7$$

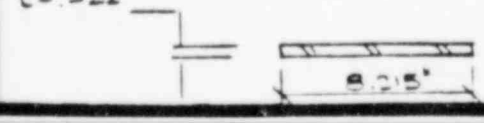
USE AISC 1.5-59 FROM 8TH EDITION SINCE →

$$\frac{65}{\sqrt{36}} < 15.7 < \frac{95}{\sqrt{36}}$$

$$F_{by_{F.O.}} = F_y\left[.79 - .002\left(\frac{b_f}{t_f}\right)\sqrt{F_y}\right]$$

$$= 36\left(.79 - .002(15.7)(6)\right) = 21.66\ ksi$$

DETERMINE S_y:



$$S_y = \frac{bd^2}{6} = \frac{.522(8.215)^2}{6}$$

$$= 5.87\ in^3$$

SECTION 7. (CONTINUED)

DETERMINE $F_{b_{YT}}$ FOR ENTICE SECTION $= 1.33(.75)F_y = 35.91$ KSI

$35.91 \text{ KSI} > (.9)F_y = 32.4 \text{ KSI}$

$\therefore$ USE $.9(F_y) = 32.4$ KSI

DETERMINE $My_{F.O.}$: (FLANGE ONLY)

$$My_{F.O} = \frac{2.247^{KLF}(3.75)^2}{12} = 2.63 \text{ K-FT}$$

USE 12 FOR A FIXED-FIXED CONDITION $\left(\frac{wl^2}{12}\right)$

$$f_{by_{F.O}} = \frac{2.63 \text{ K-FT} \times 12 \frac{in}{ft}}{5.87 \text{ in}^3}$$

$$= 5.38 \text{ KSI}$$

DETERMINE My_T:

$$My_T = \frac{.469(3.75)^2}{12} = .544^{K-FT}$$

$$f_{by_T} = \frac{.544(12)}{11.8} = .553 \text{ KSI}$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by_{FO}}}{F_{by_{FO}}} + \frac{f_{b_{YT}}}{F_{b_{YT}}} = 1 \rightarrow \frac{19.91}{1.33(24)} + \frac{5.38}{1.33(21.6)} + \frac{.553}{(.9)F_y} = .83$$

$.83 < 1$ → OK

EES

CALCULATION SHEET

PROJECT MILLSTONE UNIT NO. 1
SUBJECT MASONRY WALL ANALYSIS
SYSTEM WALLS T-32C & T-32D
ANALYSIS NO. M4 REV. NO. 0

PREPARED BY Joe Thiesen DATE 4/6/92
CHECKED BY Donald E Johnson DATE 4/28/92
JOB NO. 82028 FILE NO. 1/F
SHEET NO. M4-33

SECTION 7. (CONTINUED)

CHECK A SECTION BETWEEN STRUTS 1 & 2

$M_x = 151.1^{I-K}$

$$f_{bx} = \frac{151.1 \times 12}{110} = 16.48 \text{ KSI}$$

$$My_{F.O.} = \frac{wl^2}{12} = \frac{2.247\,(7.5)^2}{12} = 10.53 \text{ K-FT}$$

$$f_{by_{F.O}} = \frac{M_{yFO}}{S_{yFO}} = \frac{10.53 \text{ K-FT}\,(12 \frac{IN}{FT})}{5.87 \text{ IN}^3} = 21.5 \text{ KSI}$$

$$My_T = \frac{wl^2}{12} = \frac{.464\,(7.5)^2}{12} = 2.175 \text{ K-FT}$$

$$f_{by_T} = \frac{M_{yI}}{S_{yT}} = \frac{2.175\,(12)}{11.8} = 2.21 \text{ KSI}$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by\,F.O.}}{F_{by\,F.O.}} + \frac{f_{byI}}{F_{byT}} = 1 \rightarrow \frac{16.48}{1.33(24)} + \frac{21.5}{1.33(21.6)} + \frac{2.21}{.9(F_y)} = 1.33$$

1.33 > 1.0

N.G. - MORE STRUTS ARE NEEDED

EES

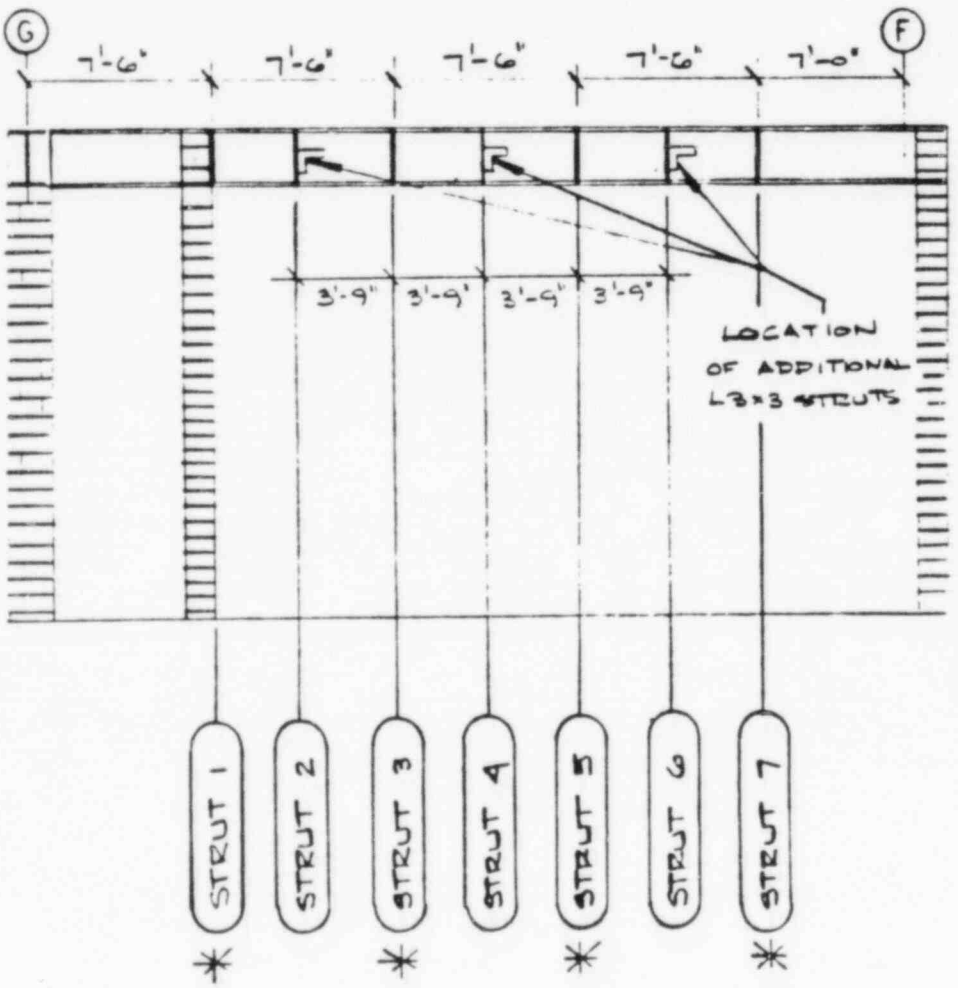
CALCULATION SHEET

PROJECT	MILLSTONE UNIT NO. 1
SUBJECT	MASONRY WALL REANALYSIS
SYSTEM	WALLS T-32C & T-32D
ANALYSIS NO.	M4
REV. NO.	0

PREPARED BY	Joe Chiesa
DATE	4/6/82
CHECKED BY	Donald E. Johnson
DATE	4/28/82
JOB NO.	82028
FILE NO.	1/F
SHEET NO.	M4-34

SECTION 7. MODIFICATION DESIGN (CONTINUED)

SINCE THE 1 ADDITIONAL L3x3 STRUT WAS NOT ENOUGH TO QUALIFY THE EXISTING W21 IN WEAK AXIS BENDING, A TOTAL OF 3 ADDITIONAL STRUTS MUST BE USED.



* DENOTES EXISTING STRUT

SECTION 7. (CONTINUED)

— LOAD ON STRUTS 1 & 7 —

HORIZONTAL LOADS = $\left(\frac{7.5'}{2} + \frac{3.75'}{2}\right) 2.71^{KLF} = 15.2^{KIP}$

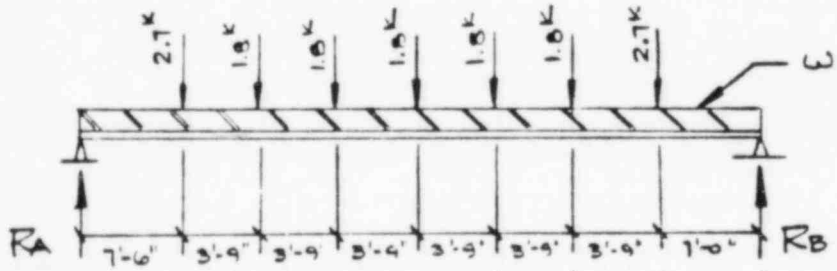
VERTICAL LOAD = $15.2^{K} \times \tan 10^\circ = 2.68^{KIP} \approx 2.7^{K}$

— LOAD ON STRUTS 2 THRU 6 —

HORIZONTAL LOADS = $3.75' (2.71^{KLF}) = 10.2^{K}$

VERTICAL LOAD = $10.2^{K} (\tan 10^\circ) = 1.8^{K}$

DOWNWARD UNIFORM LOAD = 587 PLF (REF P 17 OF THIS CALC)

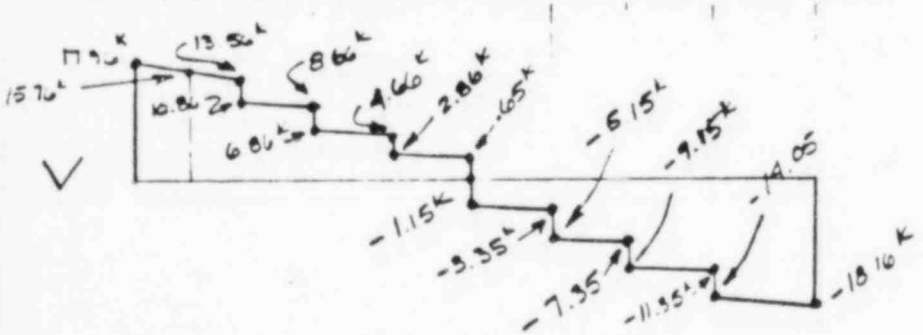


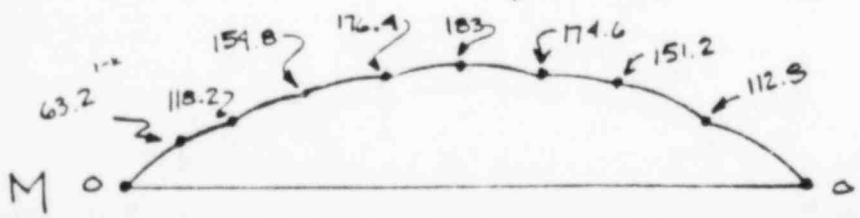
$$R_A = \frac{.587(37)}{2} + \frac{2.7}{37}(29.5 + 7) + \frac{1.8}{37}(10.75 + 14.5 + 18.25 + 22 + 25.75)$$

$$= 17.96^{KIP}$$

$$R_B = \frac{.587(37)}{2} + \frac{2.7}{37}(7.5 + 30) + \frac{1.8}{37}(11.25 + 15 + 18.75 + 22.5 + 26.25)$$

$$= 18.16^{K}$$





$M_{x\,MAX} = 183^{I-K}$

SECTION 7. (CONTINUED)

$$f_{bx} = \frac{M_{MAX}}{S_x} = \frac{183\,(12)}{110} = 19.96 \text{ KSI}$$

$$My_{\substack{\text{FLANGE}\\\text{ONLY}}} = \frac{wl^2}{12} = \frac{2.247\,(3.75)^2}{12} = 2.63 \text{ K-FT}$$

$$My_{\substack{\text{TOTAL}\\\text{SECTION}}} = \frac{wl^2}{12} = \frac{.464\,(3.75)^2}{12} = .544 \text{ K-FT}$$

$$f_{by\ F.O.} = \frac{M_{x\,FO}}{S_{y\,FO}} = \frac{2.63\,(12)}{5.87} = 5.38 \text{ KSI}$$

$$f_{by\,T} = \frac{M_{y\,T}}{S_{yT}} = \frac{.544\,(12)}{11.8} = .553 \text{ KSI}$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by\ F.O.}}{F_{by\ F.O.}} + \frac{f_{by\,T}}{F_{by\,T}} = 1 \longleftarrow \frac{19.96}{1.33\,(24)} + \frac{5.38}{1.33\,(21.66)} + \frac{.553}{.9\,(Fy)}$$

$$= \boxed{.830 < 1.0} \longleftarrow \text{OK}$$

— TRY A SECTION BETWEEN STEUT 1 & COLUMN LINE (G)

AT MIDSPAN $M_x = 63.2$ K

$$f_{bx} = \frac{M_x}{S_x} = \frac{63.2\,(12)}{110} = 6.89 \text{ KSI}$$

$$My_{F.O.} = \frac{wl^2}{12} = \frac{2.247\,(7.5)^2}{12} = 10.53 \text{ K-FT}$$

$$f_{by\ F.O.} = \frac{M_{y\,FO}}{S_{y\,F.}} = \frac{10.53\,(12)}{5.87} = 21.5 \text{ KSI}$$

SECTION 7. (CONTINUED)

$$M_{yT} = \frac{wl^2}{12} = \frac{.464(7.5)^2}{12} = 2.175 \text{ K-FT}$$

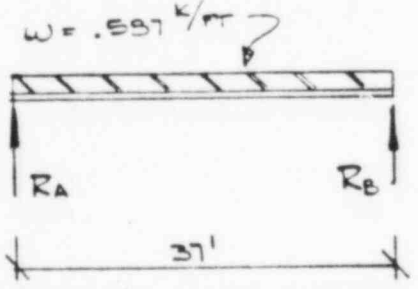
$$f_{byT} = \frac{M_{yT}}{S_{yT}} = \frac{2.175(12)}{11.8} = 2.21 \text{ KSI}$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by\,F.O.}}{F_{by\,F.O.}} + \frac{f_{byT}}{F_{byT}} = 1 \longrightarrow \frac{6.89}{1.33(24)} + \frac{21.5}{1.33(21.66)} + \frac{2.21}{.9(36)} = 1.0$$

→ OK

CHECK COMBINED STRONG AND WEAK AXIS BENDING OF EXISTING W21 FOR A NEGATIVE PRESSURE OF 360 PSF (TORNADO PRESSURE DROP LOAD)

NOTE : THERE IS NO ADDITIONAL LOAD DUE TO COMPRESSION IN THE L 2x2 STRUTS FOR A NEGATIVE PRESSURE.



$$M_{MAX\ MIDSPAN} = \frac{wl^2}{8} = \frac{.597(37)^2}{8} = 100.5 \text{ K-FT}$$

FOR W 21 x 55

$S_x = 110 \text{ IN}^3$
$L_c = 8.7'$
$L_u = 9.5'$

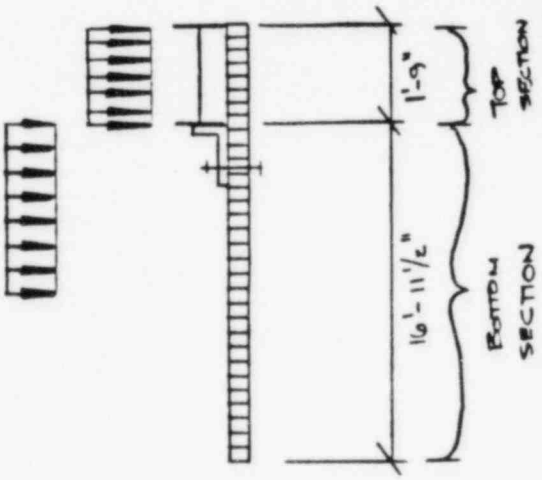
COMPRESSION FLANGE IS CONTINUOUSLY SUPPORTED → $F_{bx} = 24$ KSI

$$f_{bx} = \frac{M}{S} = \frac{100.5 \times 12}{110} = 10.96 \text{ KSI}$$

$F_y' = 44.0$ KSI
$F_y'' = 55.2$ KSI
} FULLY COMPACT

SECTION 7. (CONTINUED)

HORIZONTAL PRESSURE = 360 PSF



NOTE: FOR THE SECTION OF WALL BELOW THE EXISTING W21 (16'-11 1/2" SECTION) ONLY THE BOTTOM FLANGE WILL RESIST WEAK AXIS BENDING. FOR THE UPPER 1'-9", THE ENTICE CROSS-SECTION WILL RESIST WEAK AXIS BENDING.

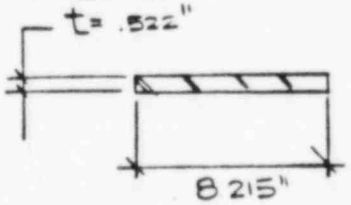
FOR BOTTOM SECTION:

$$360 \text{ PSF} \times \left(\frac{16'-11\frac{1}{2}}{2}\right) = 3053 \text{ \#/FT} = 3.05 \text{ K/FT}$$

FOR 3'-9" SECTION BETWEEN STRUTS

LOAD FROM THR'D ROD WILL BE DISTRIBUTED ALONG THE BOTTOM FLANGE

$$M_y = \frac{wl^2}{12} = \frac{3.05(3.75)^2}{12} = 3.57 \text{ K-FT}$$



$$S_y = \frac{bd^2}{6} = \frac{.522(8.215)^2}{6} = 5.87 \text{ IN}^3$$

$$f_{by} = \frac{M_y}{S_y} = \frac{3.57(12)}{5.87} = 7.30 \text{ IN}^3$$

$F_{by} = 21.66$ ← REF P31 OF THIS CALC

FOR TOP SECTION:

$$360 \text{ PSF} \times 1'-9" = 630 \text{ \#/FT} = .63 \text{ K/FT}$$

$$M_y' = \frac{wl^2}{12} = \frac{.63(3.75)^2}{12} = .74 \text{ K-FT}$$

SECTION 7. (CONTINUED)

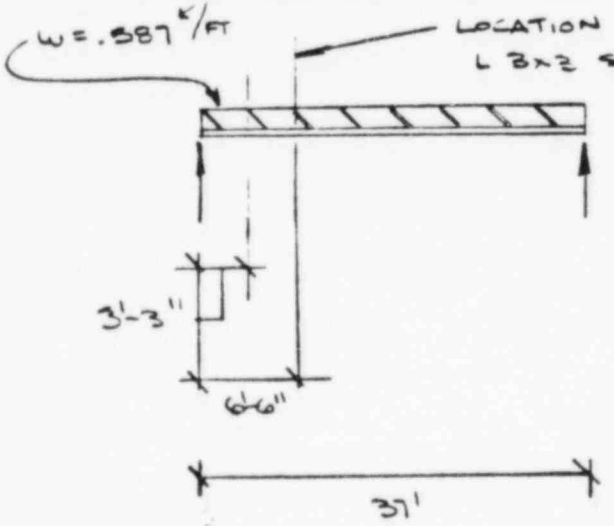
$S_y' = 11.8$ IN ← FOR ENTIRE CROSS-SECTION

$$f_{by}' = \frac{M_y'}{S_y'} = \frac{.74 \text{ K-FT } 12 \text{ IN/FT}}{11.8 \text{ IN}^3} = .75 \text{ ksi}$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} + \frac{f_{by}'}{F_{by}'} \leq 1.0$$

$$\frac{10.96}{1.33(24)} + \frac{7.30}{1.33(21.66)} + \frac{.75}{(.9)(36)} = .62 < 1 \leftarrow \text{OK}$$

— TRY THE SECTION WITH THE LONGEST LENGTH UNSUPPORTED IN THE WEAK AXIS ≅ 6'-6"



$$M_x = \frac{wx}{2}(l-x)$$

$$= \frac{.587(3.25)}{2}(37-3.25)$$

$$= 32.2 \text{ FT-K}$$

$$f_{bx} = \frac{M_x}{S_x} = \frac{32.2(12)}{110} = 3.51 \text{ ksi}$$

FOR BOTTOM SECTION

$$M_y = \frac{wl^2}{12} = \frac{3.05(6.5)^2}{12} = 10.74$$

$$f_{by} = \frac{M_y}{S_y} = \frac{10.74(12)}{5.87} = 21.95 \text{ ksi}$$

FOR TOP SECTION

$$M_y' = \frac{wl^2}{12} = \frac{.63(6.5)^2}{12} = 2.22 \text{ K-FT}$$

$$f_{by}' = \frac{M_y'}{S_y'} = \frac{2.22(12)}{11.8} = 2.26 \text{ ksi}$$

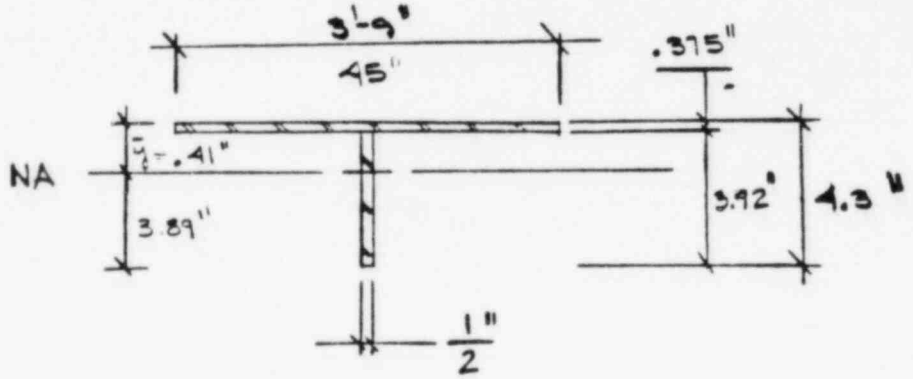
Calculation Sheet	Prepared By: Joe Chiarenza	Date 4/8/82
	Checked By: Donald E. Hillman	Date 4/28/82
Project MILLSTONE UNIT NO. 1	Job No. 82028	File No. 1/F
Subject MASONRY WALL REANALYSIS	Sheet No.	M4-40
System WALLS T-32C & T-32D		
Analysis No. M4 Rev. No. 0		

SECTION 7. (CONTINUED)

$$\frac{3.51}{1.33(24)} + \frac{21.95}{1.33(21.66)} + \frac{2.26}{.9(36)} = .94 < 1$$

OK FOR NEGATIV PRESSURE

—CHECK LOCAL BENDING OF EXISTING W21 AT STRUT LOCATIONS



FIND NA. :

$$\bar{y} = \frac{16.875\left(\frac{.375}{2}\right) + 1.96\left(.375 + 3.92/2\right)}{16.875 + 1.96} = .41"$$

$$I = \frac{(.375)^3(45)}{12} + 16.875\left(.41 - .375/2\right)^2 + \frac{(3.92)^3(.25)}{12}$$

$$+ 1.96(1.93)^2 = 9.59$$

$$S = \frac{I}{C} = \frac{9.59\ IN^4}{3.89\ IN} = 2.46\ IN^3$$

MAX LOAD PER STRUT :

$$360\ PSF \times \frac{16'-11\frac{1}{2}"}{2} = 3.05\ K/FT \times 3.75' = 11.4^{K}$$

$$M = 11.4^{K} \times 5' = 57^{1-K}$$

Calculation Sheet	Prepared By: Joe Chiarito	Date 4/9/82
	Checked By: David B. Weiman	Date 4/28/82
Project MILLSTONE UNIT NO. 1	Job No. 82028	File No. 1/F
Subject MASONRY WALL REANALYSIS		
System WALLS T-32C & T-32D	Sheet No.	M4-41
Analysis No. M4 Rev. No. 0		

SECTION 7. (CONTINUED)

$$f_b = \frac{M}{S} = \frac{5.72^{1\text{-k}}(12^{\text{in}}/_{\text{ft}})}{2.46\ \text{in}^3} = 27.9\ \text{ksi}$$

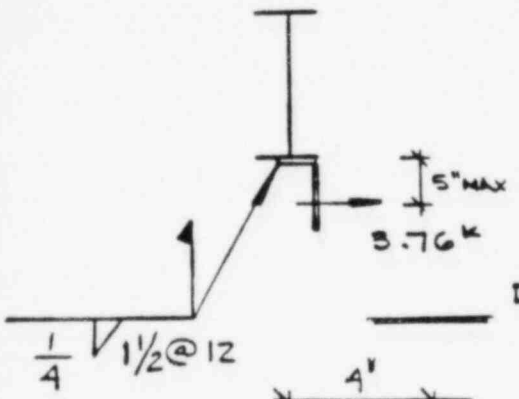
$$27.9\ \text{ksi} < 1.33(24\ \text{ksi}) = 32\ \text{ksi}$$

DESIGN WELD OF L 8x4 TO EXISTING W 21

FOR P_{MAX} = 360 PSF LL = 3.05 K/FT FOR 37'

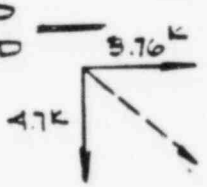
ACTUAL LENGTH OF MOD = 30'

MODIFIED LINE LOAD = $3.05 \times \frac{37'}{30'} = 3.76^{K}/_{FT}$



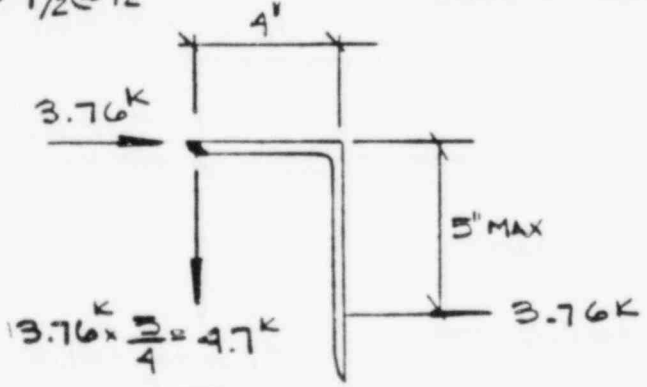
DETERMINE WELD LENGTH REQUIRED

$R_{RESULTANT} = \sqrt{(3.76^K)^2 + (4.7)^2} = 6.0^K$



1/3 INCREASE FOR WIND & SEISMIC - REF DC-1 REV. 3

$f_{ALLOWABLE} = 1.33(21^{ksi}) = 27.93^{ksi}$



$$\frac{6.0^K}{l^{\text{in/ft}} \times 1/4 \times .707(1.33)} = 21$$

ASSUMING 1/4" WELD

$$l = \frac{6^K}{1/4" \times .707(21)(1.33)} = 1.2"$$

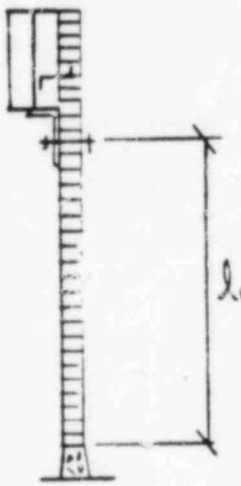
SAY 1 1/2" EVERY FOOT (MINIMUM)

NOTE: THE WELD HAS BEEN DESIGNED FOR A NEGATIVE PRESSURE = 360 PSF. A POSITIVE P = 265 PSF DOES NOT NEED A DESIGN SINCE THE LOADS WILL BE MUCH SMALLER.

Calculation Sheet	Prepared By: Joe Chiaro	Date 4/28/87
	Checked By: Donald E Brown	Date 4/28/87
Project: MILLSTONE UNIT NO. 1	Job No. 8228	File No. 1/F
Subject: MASONRY WALL REANALYSIS		
System: WALLS T-32 & T-32D	Sheet No.	M4 -42
Analysis No. M4 Rev. No. 0		

SECTION 7 MODIFICATION DESIGN (CONTINUED)

CHECK BENDING IN MASONRY



$M_{ALLOWABLE} = 1288 \frac{FT-\#}{FT}$

$M_{MAX} = \frac{w_{MAX} l^2}{8}$

$w_{MAX} = \frac{M_{MAX}(8)}{l^2} = \frac{1288(8)}{(16.71)^2}$

$= 369$ PSF > 360 PSF

MASONRY IS OK IN BENDING FOR MAX PRESSURE

DETERMINE THE NUMBER OF BOLTS REQUIRED

$P_{MAX} = 360$ PSF

LINE LOAD AT TOP = $\frac{360(16'-11\frac{1}{2}")}{2} = 3.05^K/FT$

$3.05^K/FT \times 37' = 113^K$

MINIMUM NUMBER OF BOLTS = $\frac{113^K}{13.33(1.33)} = 6.4$ BOLTS

REF AISC 7TH EDITION P4-3 FOR A36-1"Φ

1/3 INCREASE FOR WIND & SEISMIC DC-1 REV 3

13 BOLTS ARE IN THE DESIGN > 6.4 O.K.

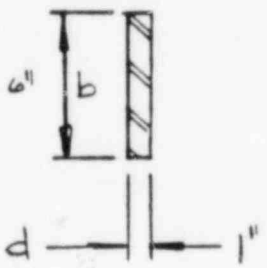
Calculation Sheet	Prepared By: Joe Chiasso	Date: 4/9/92
	Checked By: Donald E Johnson	Date: 4/28/92
Project: MILLSTONE UNIT NO. 1	Job No: 8203B	File No: 1/F
Subject: MASONRY WALL REANALYSIS		
System: WALL T-32C & T-32D	Sheet No: M4 - 43	
Analysis No: M4 Rev. No: 0		

SECTION 7. MODIFICATION DESIGN (CONTINUED)

— LOCAL BENDING OF L8x4

NOTE: SINCE STIFFENER PLATES HAVE BEEN PLACED AT THE LOCATION OF EACH .THRU BOLT, IT IS NOT NECESSARY TO ANALYZE LOCAL BENDING OF THE L8x4

— WEAK AXIS BENDING OF PL 1"



$$S = \frac{bd^2}{6} = \frac{6(1)^2}{6} = 1 \text{ IN}^3$$

SPACING BETWEEN BOLTS = 2'-9" = 33"

MALLOWABLE = Fb(S) (1/3 INCREASE FOR WIND, SEC. 1 REF 3; AISC 1.5.1.4.3 7TH EDITION)

$Fb = (1.33)(.66)(F_y) = 1.33(.75)(36) = 35.91$ KSI

$35.91 \leq .9(F_y) \Rightarrow$ USE $.9(F_y) = 32.4$

MALLOWABLE = 32.4 KSI (1 IN³) = 32.4 IN-KIP

= 3[illegible] IN-#

$$M = \frac{wl^2}{10}$$

$$W_{ALLOWABLE} = \frac{M \times 10}{l^2} = \frac{32,400^{\text{IN-\#}}(10)}{(33)^2 \text{ IN}^2} = 298^{\#}/\text{INCH} = 3576^{\#}/\text{FT}$$

$$Q_{ALLOWABLE} = \frac{w(2)}{l} = \frac{3576^{\#}/\text{FT}\ (2)}{16.96'} = 422^{\#}/\text{FT}^2$$

422 PSF > 360 PSF > 265 PSF O.K.

ATTACHMENT 3

consists of a 53-page (Sheets B001 0-52) example of arching action analysis.

EES

CALCULATION COVER SHEET

Job No. 80014
File No. 12-F
Calc. Set No. 80-1-0
No. of Sheets 52

PROJECT MILLSTONE, UNIT # 1

CLIENT NORTHEAST UTILITIES SERVICE COMPANY

SUBJECT

GENERIC CALCULATIONS OF MASONRY BLOCKOUT ANALYSIS FOR NRC IE BULLENTIN 80-11

STATEMENT OF PROBLEM

TO DEVELOP AN ACCEPTACE CRITERIA FOR USE OF "ARCHING ACTION" TO EVALUATE OR MODIFY EXISTING 6" solid BLOCKOUTS AT THE MILLSTONE NUCLEAR POWER STATION, UNIT #1.

SOURCES OF DATA

① EES DESIGN CRITERIA (DC-1) REV. 1
② EES MILLSTONE UNIT #1 CALC. SET NO. MOCO SEET. 13

SOURCES OF FORMULAE & REFERENCES

SEE PAGE 6

REMARKS ① FOR LIMITATIONS OF USING GENERIC CALCULATION SEE PAGE 3
② FOR SUMMARY OF CALCULATIONS SEE PAGE 5, 53, 54.

ORIGINATORS	CHECKERS	DISTRIBUTION	REVISION NO. 0
C. T. LIN	B. MONAJARIA		SUPERSEDES CALCULATION SET NO. NA
11-1-80	11-5-80		APPROVED BY: [signature] DATE 12-16-80

EES

CALCULATION SHEET

PREPARED BY: C.T.LIN	DATE: 10-31-80
CHECKED BY: B.HONARARIAN	DATE: 11-5-80
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: MASONRY BLOCKOUT	JOB NO.: 80014 / FILE NO.: 12-F
SYSTEM: GENERIC CAL'S OF ARCHING ACTION	
ANALYSIS NO.: 1 / REV. NO.: 2	SHEET NO.: B001 - 1

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SECTION		PAGE
1.0	GENERAL	2
2.0	DEFINITIONS	2
3.0	LIMITATIONS OF USING GENERIC CAL'S	3
4.0	ASSUMPTIONS	4
5.0	SUMMARY	5
6.0	REFERENCES	6
7.0	NOTATION & FORMULA FOR ARCHING ACTION	7 ~ 9
8.0	ALLOWABLE LINE LOAD REQUIREMENT	10 ~ 29
9.0	DEFLECTION REQUIREMENT	30 ~ 41
10.0	FREQUENCY REQUIREMENT	42 ~ 49
11.0	MAX. GAP SIZES FOR CONVERTING GAP ARCHING TO RIGID ARCHING	50 ~ 52

EES

CALCULATION SHEET

PREPARED BY: C.T. LIN	DATE: 10-31-80
CHECKED BY: B. HONARIAN	DATE: 11-5-80
PROJECT: MILLSTONE #1, NUSCO	JOB NO.: 80014 / FILE NO.: 12-F
SUBJECT: BLOCKOUT	
SYSTEM: GENERIC CAL'S OF ARCHING ACTION	
ANALYSIS NO.: Rev. 1 / REV. NO.:	SHEET NO.: B001 - 2

REF.

MASONRY BLOCKOUT (UNREINFORCED)

GENERAL :

THESE GENERIC CALCULATIONS PROVIDE THE TECHNICAL BASIS AND ANALYSIS METHOD BASED ON ARCHING ACTION THEORY FOR THE RE-EVALUATION OF MASONRY BLOCKOUTS AND THE DESIGN OF MODIFICATIONS AT THE MILLSTONE NUCLEAR POWER STATION, UNIT 1 AS PER THE REQUIREMENTS OF NRC I&E BULLETIN 80-11

DEFINITIONS :

1) MASONRY BLOCKOUT — A WALL MADE OF MASONRY UNITS AND CONFINED (OR CONSTRAINED) BETWEEN RIGID SUPPORTS SUCH AS INSIDE THE REINFORCED CONCRETE WALL.

2) ARCHING ACTION THEORY — WHEN THE WALL IS BUTTED UP AGAINST SUPPORTS WHICH ARE ESSENTIALY RIGID, ADDITIONAL STRENGTH CAN BE DEVELOPED FOR WALL BY FORMING A THREE-HINGED ARCH. SUCH ARCH IS ABLE TO WITHSTAND MUCH HIGHER LATERAL LOADS THAN WOULD BE PREDICTED ON THE BASIS OF CONVENTIONAL BENDING ANALYSIS.

3) RIGID ARCHING WALL : NO GAP IN MORTAR JOINT BETWEEN WALL AND SUPPORTS (NO VISIBLE CRACK)

4) GAPPED ARCHING WALL : WITH GAP IN MORTAR JOINT BETWEEN WALL AND SUPPORTS (MAX. ALLOWABLE GAP = $\frac{1}{8}$" FOR MILLSTONE #1)

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT
SYSTEM	GENERIC CAL'S (ARCHING ACTION)
ANALYSIS NO.	REV. NO.

PREPARED BY	C.T.LIN
DATE	10-31-85
CHECKED BY	B. MONAJARIAN
DATE	11-5-85
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001 - 3

LIMITATIONS OF USING GENERIC CALCULATIONS.:

1) MASONRY UNIT OF BLOCKOUT TO BE ASTM C-145 SOLID CONC. BLOCK

2) TYPE S MORTAR TO BE USED

3) MAX. GAP SIZE SHALL BE LIMITED TO $\frac{1}{8}$" FOR GAPPED ARCHING WALL

4) NO MAJOR OPENINGS OR PENETRATIONS EXIST IN WALL TO INTERUPT THE LOAD PATH WHEN APPLYING ARCHING ACTION

5) ARCHING ACTION IS ONLY APPLICABLE TO THE VERTICAL STRIP OF WALL SPANNING BETWEEN TOP AND BOTT. EDGES OF EXISTING SUPPORTS..

6) EXTERNAL LOADS ON WALL IN THE LATERAL DIRECTION SHALL BE THE UNIFORM LOAD FROM PRESSURE, INERTIA FORCE OF SEISMIC, OR EOTH, ACROSS ENTIRE SPAN.
THE MAXIMUM TOTAL UNIFORM LOAD IS 0.72 PSI

7) GAPPED ARCHING WALL CAN BE CONVERTED TO RIGID ARCHING WALL BY PROVIDING LATERAL RESTRAINTS AT GAPPED END ONLY IF GAP SIZE IS LESS OR EQUAL TO $\frac{1}{8}$"

8) TO QUALIFY A BLOCKOUT PASSING GENERIC CALCULATIONS, MAXIMUM ALLOWABLE SPAN OF WALL SHALL SATISFY THE LIMITATIONS OF ALLOWABLE LINE LOAD, DEFLECTION AND FREQUENCY WITHIN RIGID RANGE

EES 08/80 - 101

REF

BLOCKOUT

ASSUMPTIONS:

① THE ENTIRE ANALYSIS OF THESE GENERIC CALCULATIONS IS BASED ON SINGLE WYTH OF 6" MASONRY WALL. IF A WALL HAVING MORE THAN ONE WYTH IN ITS THICKNESS, EACH WYTH SHALL BE ASSUMED TO BE MADE OF 6 x 8 x 16 MASONRY BLOCKS AND TO ACT INDEPENDENTLY FROM THE OTHER WYTHS WITHOUT CONSIDERING THE BOND BETWEEN THEM.
IT IS BELIEVED CONSERVATIVE IF ANY ONE OF SINGLE WYTH OF WALL PASS THE REQUIREMENTS OF ARCHING ACTION THE WHOLE WALL IS ALSO VERIFIED.

② MASONRY BLOCKOUTS ARE NOT PART OF THE STRUCTURAL LOAD RESISTING SYSTEM AND, THEREFORE, DO NOT CARRY SEISMIC SHEARS OR VERTICAL SEISMIC LOADS DUE TO BUILDING INERTIA FORCES.

DC-1, REV 1

③ THE ANALYSIS OF GENERIC CALCULATIONS IS BASED ON CRACKS OCCURING AT CENTER OF WALL (MIDSPAN) DURING ARCHING ACTION.

④ MINOR PENETRATIONS OR OPENINGS DUE TO SMALL PIPES OR CONDUITS DO NOT INTERRUPT THE LOAD PATH IN THE ANALYSIS WHEN APPLYING ARCHING ACTION

⑤ SEE PAGE 11 FOR ALLOWABLE LINE LOAD ASSUMPTIONS

⑥ SEE PAGE 31 FOR ALLOWABLE DEFLECTION ASSUMPTIONS

⑦ SEE PAGE 43 FOR NATURAL FREQUENCY ASSUMPTIONS

EES

CALCULATION SHEET

PREPARED BY	C.T. LIN	DATE	10-31-80
CHECKED BY	B. HONARVAR	DATE	11-5-80
PROJECT	MILLSTONE #1, NUSCO		
SUBJECT	BLOCKOUT	JOB NO.	80014
SYSTEM	GENERIL CAL'S OF ARCHING ACTION	FILE NO.	12-F
ANALYSIS NO.	REV. NO. 0	SHEET NO.	B001- 5

SUMMARY :

UNREIFORCED MASONRY BLOCKOUT (ASTM-C145)
VERTICAL DESIGN SPAN BASED ON ARCHING ACTION

	ARCHING ACTION TYPE	CASE	M' MAX. EARTHQUAKE	M DESIGN EARTHQUAKE	MAX. ALLOWABLE SPAN (FT) ALLOWABLE LINE LOAD LIMITATION	$\Delta = 0.15\,t$ DEFLECTION LIMITATION	RIGID RANGE FREQUENCY LIMITATION	GOVERNING VERTICAL DESIGN SPAN (FT)
Pg 15, 36, 46, 49	RIGID ARCHING	A'	✓		18.75	10.42	CHART 2	7.25
Pg 16, 36, 46, 49	(1/16 GAP)	A		✓	14.16	10.42	CHART 2	
Pg 17, 39, 47, 48	GAPPED ARCHING	B'	✓		13.5	7.25	CHART 1	SHORTER SPAN OF 7.25 FT OR FROM CHART 1
Pg 18, 39, 47, 48	($\frac{1}{16}$" GAP)	B		✓	10.33	7.25	CHART 1	
Pg 17, 41, 47, 48	GAPPED ARCHING	C'	✓		13.5	5.4	CHART 1	SHORTER SPAN OF 5.4 FT OR FROM CHART 1
Pg 18, 41, 47, 48	($\frac{1}{8}$" GAP)	C		✓	10.33	5.4	CHART 1	

ARCHING ACTION BASED ON :

① SINGLE WYTH OF 6x8x16 BLOCK WALL (ACTUAL WALL THK. $t = 5.625$")

② MAX. HORIZ. UNIFORM LOAD ON WALL, $p = 0.72$ PSI FOR M'
REF. 9 (p = 0.5 PSI PRESSURE LOAD + 0.22 PSI INERTIA LOAD BASED ON
DC-1, REV 1 HORIZ. g_H FACTOR = 0.452 @ EL. 54.5' OF TURBINE BLDG)

③ MAX. HORIZ. UNIFORM LOAD ON WALL, $p = 0.59$ PSI FOR M
REF 9 (p = 0.5 PSI PRESSURE LOAD + 0.09 PSI INERTIA LOAD BASED ON
DC-1, REV 1 HORIZ. g_H FACTOR = 0.186)

④ f'_m = 1080 PSI = ULTIMATE COMPRESSIVE STRENGTH OF MASONRY

⑤ CRACK WITHIN TWO SUPPORTS SHALL BE AT MIDSPAN OF WALL

⑥ ATTACHMENT WT. OF WALL NOT INCLUDED IN FINDING NATURAL FREQUENCY

EES
EARTHQUAKE ENGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT
	GENERIC CAL'S (ARCHING ACTION)

PREPARED BY	C.T. LIN
DATE	10-10-80
CHECKED BY	B. HONARVARIAN
DATE	10/23/80
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001 - 6

REFERENCES:

1. RESPONSE OF ARCHING WALLS AND DEBRIS FROM INTERIOR WALLS CAUSED BY BLAST LOADING BY URS RESEARCH COMPANY (FEBRUARY 1975, URS 7030-23)

2 SHOCK TUNNEL TESTS OF ARCHED WALL PANELS BY URS RESEARCH COMPANY (DECEMBER 1974 URS 7030-19)

3. BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-77)

4 EES DESIGN CRITERIA (DC-1) REV. 1

5. EES MILLSTONE UNIT #1, CAL'S SET NO MOCO SECT 13 BY C.T. LIN

6. REINFORCED MASONRY ENGINEERING HANDBOOK (2nd EDITION) BY J.E. AMRHEIN (M.I.A.)

8. CONCRETE MASONRY HANDBOOK FOR ARCHITECTS, ENGINEERS, BUILDERS BY FRANK. A. RANDALL JR & WILLIAM C. PANARESE (P.C.A)

9 NUSCO LETTER GCE-80-603 FROM John R. WHITFIELD TO J. G. DYCKMAN 10-14-80

10. FORMULAS FOR NATURAL FREQUENCY AND MODE SHAPE BY ROBERT D. BLEVINS, VAN NOSTRAND REINHOLD CO., 1979

REF

NOTATION FOR ARCHING ACTION

ℓ : DESIGN SPAN (INCH)

t : THICKNESS OF ONE WYTH WALL (IN)

W : WALL WT. + ATTACHMENT WT. (PSF)

g_H : HORIZONTAL ACC. FACTOR (MAX. EARTHQUAKE)

g_V : VERTICAL ACC. FACTOR (")

N : NUMBER OF WYTH TO BE CONSIDERED IN DESIGN FOR INERTIA FORCE

P_1 : PRESSURE FORCE ON WALL (PSI)

P_2 : INERTIA LOAD DUE TO HORIZONTAL SEISMIC $= \frac{W g_H N}{144}$ (PSI)

V : DEAD LOAD AND INERTIA LOAD DUE TO VERTICAL SEISMIC (PSI)

$= \frac{W}{144} \times (1 + g_V) \times N$

P : TOTAL UNIFORM LOADS (PSI) $= P_1 + P_2$

F_L : ALLOWABLE LIND LOAD AT ARCHING PT. ($^{LB}/_{IN}$)

R_A, R_B & R_C = RESULTANT REACTIONS @ SUPPORTS A, B, & C RESPECTIVELY ($^{\#}/_{IN}$)

S_A, S_B & S_C = HORIZ. REACTIONS @ SUPPORTS A, B, & C RESPECTIVELY ($^{\#}/_{IN}$)

H : VERTICAL REACTION ($^{\#}/_{IN}$)

δ_o : INITIAL GAP (IN)

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CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO

SUBJECT: BLOCKOUT (ARCHING ACTION)

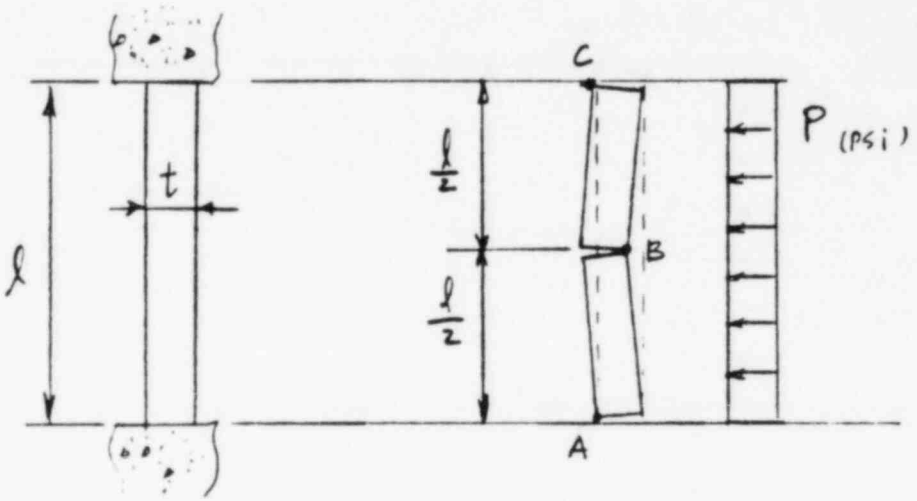
FORCES IN RIGID ARCHING

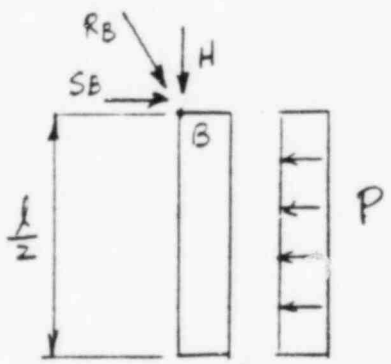
PREPARED BY	DATE
C.T. LIN	10-16-80
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B. MONAVARIAN	10/23/80
JOB NO. 80014	FILE NO. 12-F
SHEET NO. B001 - 8	

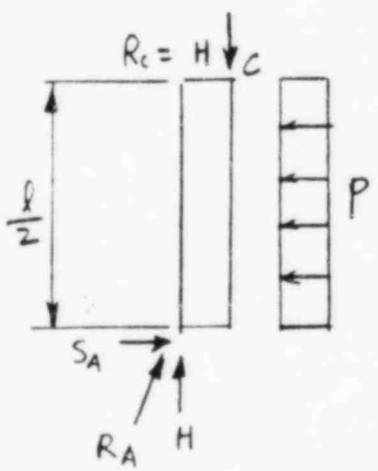
REF.

PAGE 2-6
REF. 1.

RIGID ARCHING SUBJECT TO UNIFORM LD P (NO GAP EXISTS)







$$H = \frac{P\ell^2}{8t}$$

$$S_B = \frac{P\ell}{2}$$

$$R_B = \frac{H}{\ell}\sqrt{\ell^2+(4t)^2} = \frac{P\ell}{8t}\sqrt{\ell^2+(4t)^2}$$

$$S_C = 0$$

$$R_C^{*} = H$$

$$S_A = \frac{P\ell}{2}$$

$$R_A^{*} = \frac{H}{\ell}\sqrt{\ell^2+(4t)^2} = \frac{P\ell}{8t}\sqrt{\ell^2+(4t)^2}$$

* NOTE: DL OF WALL NOT INCLUDED

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CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO

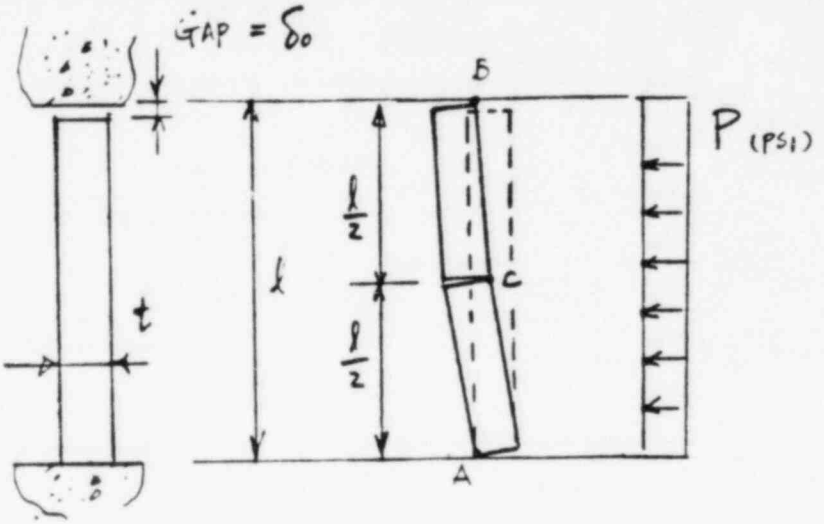
SUBJECT: BLOCKOUT - GENERIC CAL'S

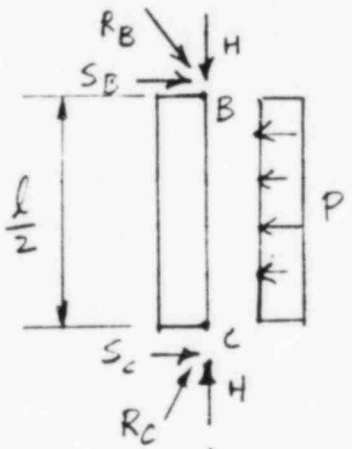
FORCES IN GAPPED ARCHING

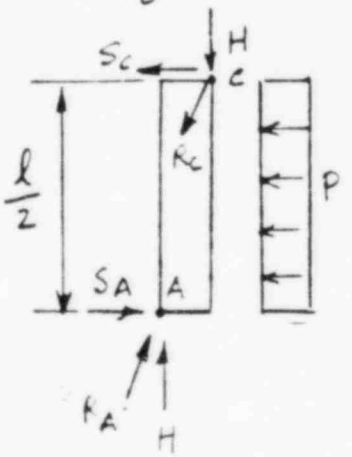
PREPARED BY	DATE
C.T. LIN	10-16-87
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B. HONARVAR	10/23/87
JOB NO. 80014	FILE NO. 12-F
SHEET NO. B001 - 9	

REF. 1
FINAL REPORT
RESPONSE OF
ARCHING WALLS
AND DEBRIS
FROM INTERIOR
WALLS CAUSED
BY BLAST LOADING
FROM U.R.S
RESEARCH
COMPANY
PAGE 2-6

GAPPED ARCHING SUBJECTED UNIFORM LD P







$$H = \frac{Pl^2}{4t}$$

$$S_B = \frac{Pl}{4}$$

$$R_B = \left(\frac{H}{l}\right)\sqrt{l^2+t^2} = \frac{Pl}{4t}\sqrt{l^2+t^2}$$

$$S_C = \frac{Pl}{4}$$

$$R_C^{*} = \frac{H}{l}\sqrt{l^2+t^2}$$

$$S_A = \frac{3Pl}{4}$$

$$R_A^{*} = \frac{H}{l}\sqrt{l^2+(3t)^2} = \frac{Pl}{4t}\sqrt{l^2+(3t)^2}$$

* NOTE: DEAD LOAD OF WALL NOT INCLUDED

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CALCULATION SHEET

PREPARED BY C. T. LIN			DATE 10-31-?
CHECKED BY B. MONAVARIAN			DATE 11-5-8?
PROJECT MILLSTONE #1, NUSCO	JOB NO. 80014	FILE NO. 12-F	
SUBJECT BLOCKOUT (ARCHING ACTION)			
SYSTEM ALLOWABLE LINE LOAD			
ANALYSIS NO. ____ REV. NO. ____	SHEET NO. B001 - 10		

PAGE

SECT 8.0 ALLOWABLE LINE LOAD REQUIREMENT 10~29

8.1 ASSUMPTIONS 11

8.2 CONCLUSION 12

8.3 ALLOWABLE LINE LOAD FOR SOLID CONC. MASONRY WALL 13~14

8.4 MAX. ALLOWABLE SPAN OF RIGID ARCHING

CASE A' - $F_\ell = 942^{\#/in}$ (MAX. EARTHQUAKE LOADING CONDITION) 15

CASE A. - $F_\ell = 471^{\#/in}$ (DESIGN EARTHQUAKE LOADING CONDITION) 16

8.5 MAX. ALLOWABLE SPAN OF GAPPED ARCHING

CASE B' - $F_\ell = 942^{\#/in}$ (MAX. EARTHQUAKE) 17

CASE B - $F_\ell = 471^{\#/in}$ (DESIGN EARTHQUAKE) 18

8.6 SUMMARY OF REACTIONS 19

REACTIONS OF DIFFERENT CASES 20~29

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CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	GENERIC CAL'S - BLOCKOUT
SYSTEM	ALLOWABLE LINE LOAD
ANALYSIS NO.	REV. NO.

PREPARED BY	C.T. LIN
DATE	10-31-83
CHECKED BY	B. MONAVARIAN
DATE	11-5-83
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001-11

REF.

ALLOWABLE LINE LOAD

ASSUMPTIONS:

① ALLOWABLE LINE LOAD F_L IS DERIVED FROM PAGE 13 WHICH IS BASED ON FORMULA FROM PAGE 2-16 OF REF 1.

Pg 8.9

RESULTANT REACTIONS (R_A, R_B & R_C) DURING ARCHING ACTION SHALL BE LESS THAN F_L

② SHEAR FORCE (S_A, S_B, & S_C) DUE TO ARCHING ACTION SHALL BE LESS THAN ALLOWABLE SHEAR (V_{mu} FROM DC-1, REV 1, WITH 1" CONTACT WIDTH.) OTHERWISE, HELP FROM FRICTION EFFECT BETWEEN MASONRY AND CONC. OR STEEL FRAME SHALL BE CONSIDERED IN ORDER TO DEVELOP A THREE-HINGED ARCH MECHANISM.

Pg 191 OF REF. 8

USE OF COEFF. OF FRICTION = 0.2 FOR MASONRY AND CONC OR STEEL FRAME IS CONSERVATIVE.

③ MAX. SPAN DUE TO ALLOWABLE LINE LOAD IS BASED ON UNIFORM LOAD ON WALL AND CRACK AT CENTER (MIDSPAN) OF WALL. FOR CRACKS NOT IN THE MIDSPAN OF WALL, ADDITIONAL CALCULATION SHALL BE MADE TO ADJUST THE INFLUENCE OF RADOM CRACK. OR WE CAN MULTIPLY THE LONGER SPAN OF CRACK SECTION BY TWO AND LIMIT THIS NEW SPAN TO BE LESS THAN THE ALLOWABLE SPAN BASED ON CRACK AT MIDSPAN. THIS METHOD IS CONSERVATIVE.

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CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT (ARCHING ACTION)
SYSTEM	ALLOWABLE LINE LD.
ANALYSIS NO.	REV. NO. 0

PREPARED BY	C.T. LIN
DATE	10-31-?
CHECKED BY	B. MONAJJAR, AN
DATE	11-5-8?
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001-12

REF

ALLOWABLE LINE LOAD

CONCLUSION: MAXIMUM SPAN BASED ON ALLOWABLE LINE LOAD FOR SINGLE WYTH OF 6×8×16 SOLID MASONRY BLOCKOUT WITH 0.72 PSI UNIFORM LOAD FROM PRESSURE & INERTIA LOAD ARE AS FOLLOWS

① RIGID ARCHING

Pg 15 — MAX. SPAN = 18.75 FT (MAX. EARTHQUAKE)

Pg 16 — MAX. SPAN = 14.16 FT (DESIGN EARTHQUAKE)

② GAPPED ARCHING

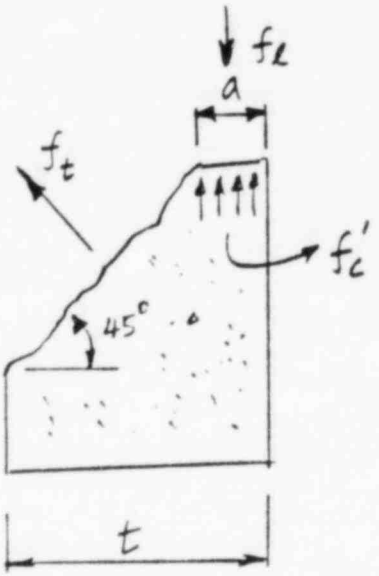
Pg 7 — MAX. SPAN = 13.5 FT (MAX. EARTHQUAKE)

Pg 18 — MAX. SPAN = 10.33 FT (DESIGN EARTHQUAKE)

EES	CALCULATION SHEET	PREPARED BY C.T. LIN	DATE 10-16-80
		CHECKED BY B. MONAVARIAN	DATE 10-23-80
	PROJECT MILLSTONE #1, NUSCO	JOB NO. 20014	FILE NO. 12-F
	SUBJECT BLOCK O.T (ARCHING ACTION)		
	SYSTEM ALLOWABLE LINE LD		
	ANALYSIS NO. B001 REV. NO. 0	SHEET NO. B001-13	

REF.

ALLOWABLE LINE LOAD FOR SOLID CONC. MASONRY WALL (ASTM-C145)



USE FORMULA FROM URS 7030-23 REPORT

$$f_l = [f_c' a + f_t (t - aK)\cos 45^\circ] K$$

(f_m' → f_c')

Pg 2-12, 2-13 OF REF. 1

ASSUME $a = 1"$ OF CRUSHING & SPALL

$K = \frac{b_s}{b} = 1$ FOR SOLID BLOCK WITH NO CORE WHERE

b_s = THE WIDTH OF SHEARED MATERIAL

b = THE WIDTH OF BLOCK

REF. 4

① TYPE S MORTAR STRENGTH = 1800 PSI

② FOR ASTM C-145 SOLID CONC BLOCK

$f_c' = 1800$ PSI

REF. 3

$f_m' = 1020$ PSI

TO BE CONSERVATIVE, USE

$f_c' = f_m' = 1080$ PSI

f_t = TENSILE RUPTURE STRENGTH

$= 7.5\sqrt{f_c'} = 7.5\sqrt{1080}$

$= 246$ PSI (ACI CODE 318-77)

t = THICKNESS OF 1 WYTH

FOR BLOCK 6 x 8 x 16 SIZE ($t = 5.625"$)

$$f_l = [1080 \times 1 + 246\,(5.625 - 1) \times 0.707] \times 1$$

$$= 1824 \text{ #/IN}$$

TAKE A SAFTY FACTOR OF 4 FOR M CASE (DESIGN EARTHQUAKE LOADS)

$$F_l = 1824/4 = 471 \text{ #/IN}$$

TAKE A SAFTY FACTOR OF 2 FOR M' CASE (MAX. EARTHQUAKE LOADS)

$$F_l = 1824/2 = 942 \text{ #/IN}$$

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOWOUT (ARCHING ACTION)
SYSTEM	ALLOWABLE LINE LD
ANALYSIS NO.	B001
REV. NO.	0

PREPARED BY	C. T. LIN
DATE	10-16-80
CHECKED BY	B MONAVARIAN
DATE	10/23/80
JOB NO.	80014
FILE NO.	12-T
SHEET NO.	B001-14

REF

∴ ALLOWABLE LINE LOAD FOR 6×8×16 CONC BLOCK (ASTM-C145)

F_ℓ = 471 #/IN FOR DESIGN EARTHQUAKE LOADS (M)

F_ℓ = 942 #/IN FOR MAX. EARTHQUAKE LOADS (M')

CALCULATION SHEET

EES

PREPARED BY: C. T. LIN	DATE: 10-16-2
CHECKED BY: B. HONARVARIAN	DATE: 11-5-9
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: BLOCKOUT (ARCHING ACTION)	
SYSTEM: ALLOWABLE LINE LD	JOB NO.: 80014 / FILE NO.: 12-F
ANALYSIS NO.: B001 / REV. NO.: 0	SHEET NO.: B001-15

REF Pg 14

FIND MAX. RIGID ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 942$ #/IN FOR 6×8×16 SOLID CONC. BLOCK WITH 0.72 PSI UNIFORM LOAD ON WALL

Pg 7&8

$$R_A = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} + \underbrace{V\ell}_{\text{DL \& VERT. SEISMIC OF WALL}} \quad \cdots \text{GOVERNS}$$

∴ V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT. (WALL WT.)

Pg 7 OF REF 5 & DC-2, REV.1

$$= W \times (1 + g_V) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557 \text{ PSI}$$

(g_V: VERT. g FACTOR; N: NO OF WYTH)

LET $R_A = F_\ell = 942$ #/IN, $P = 0.72$ PSI $= P_1 + P_2$

$t = 5.625"$ FOR 6×8×16 BLOCK, $P_1 = 0.5$ PSI FROM PRESSURE

Pg 5, 21

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC

$= \frac{70}{144} \times 0.452 \times 1 = 0.22$ PSI

$$942 = \frac{0.72 \times \ell}{8 \times 5.625}\sqrt{\ell^2 + (4 \times 5.625)^2} + 0.557 \times \ell \quad \cdots\cdots (1)$$

SOLVE ℓ BY TRY & ERROR METHOD

Try $\ell = 18.75' = 225"$, SUBSTITUTING INTO (1)

WE HAVE $942 \doteq 939$, O.K CLOSE ENOUGH USE $\ell = 225"$

∴ MAX. RIGID ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 942$ #/IN IS 18.75' FOR 6×8×16 SOLID CONC BLOCK WITH 0.72 PSI UNIFORM LD ON WALL

EES

CALCULATION SHEET

PREPARED BY	C.T.LIN	DATE	10-16-80
CHECKED BY	B.MONAURIAN	DATE	11-5-80
PROJECT	MILLSTONE #1, NUSCO	JOB NO.	20014
SUBJECT	BLOCKOUT (ARCHING ACTION)	FILE NO.	12-F
SYSTEM	ALLOWABLE LINE LD		
ANALYSIS NO.	B001 REV. NO. 0	SHEET NO.	B001-16

REF Pg 14

FIND MAX. RIGID ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 471$ #/IN FOR 6×8×16 SOLID CONC. BLOCK WITH 0.59 PSI UNIFORM LOAD ON WALL

Pg 7&8

$$R_A = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} + V\ell \quad \cdots \text{ GOVERNS}$$

∴ V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT. (WALL WT.)

Pg 7 OF DCF 5 & DC-1, REV 1

$$= W \times (1 + q_V) \times N = \frac{70}{144} \times (1 + 0.067) \times 1 = 0.519 \text{ PSI}$$

(q_V: VERT. Q FACTOR; N: NO. OF WYTH)

LGT. $R_A = F_\ell = 471$ #/IN, $P = 0.59$ PSI $= P_1 + P_2$

Pg 23

$t = 5.625$" FOR 6×8×16, $P_1 = 0.5$ PSI FROM PRESSURE

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC

Pg 5

$$= \frac{70}{144} \times 0.186 = 0.09 \text{ PSI}$$

$$471 = \frac{0.59 \times \ell}{8 \times 5.625}\sqrt{\ell^2 + (4 \times 5.625)^2} + 0.519 \times \ell \quad \cdots\cdots (1)$$

SOLVE ℓ BY TRY & ERROR METHOD

TRY $\ell = 14.16^{FT} = 170$ IN, SUBSTITUTING INTO (1)

WE HAVE $471 \doteq 470$, O.K CLOSE ENOUGH USE $\ell = 170$"

∴ MAX. RIGID ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 471$ #/IN IS 14.16^{FT} FOR 6×8×16 SOLID CONC BLOCK WITH 0.59 PSI UNIFORM LD ON WALL

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT (ARCHING ACTION)
SYSTEM	ALLOWABLE LINE LD
ANALYSIS NO.	B001 REV. NO. 0

PREPARED BY	C.T. LIN
DATE	10-16-80
CHECKED BY	B. MONAJARIAN
DATE	11-5-80
JOB NO.	P0014
FILE NO.	12-F
SHEET NO.	B001-17

REF

Pg 14 FIND MAX. GAPPED ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 942$ #/IN FOR 6×8×16 SOLID CONC BLOCK WITH 0.72 psi UNIFORM LOAD ON WALL

Pg 9

$$R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell \quad \cdots \text{ GOVERNS}$$

Pg 24 LET $R_A = F_\ell = 942$ #/IN, $t = 5.625$, $P = 0.72$ psi

$V = 0.557$ psi

$$\therefore 942 = \frac{0.72 \times \ell}{4 \times 5.625}\sqrt{\ell^2 + (3 \times 5.625)^2} + 0.557 \times \ell \quad \cdots (2)$$

SOLVE ℓ, BY TRIAL & ERROR METHOD

TRY $\ell = 13.5' = 162''$, SUBSTITUTING INTO (2)

WE HAVE $942 \doteq 934$. O.K CLOSE ENOUGH

USE $\ell = 162''$

$\therefore$ MAX. GAPPED ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 942$ #/IN IS 13.5' OR 162"

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CALCULATION SHEET

PREPARED BY	C. T. LIN
DATE	10-16-80
CHECKED BY	B. MONAVARIAN
DATE	11-5-80
PROJECT	MILLSTONE #1, NUSCC
JOB NO.	80014
FILE NO.	12-F
SUBJECT	BLOCKOUT (ARCHING ACTION)
SYSTEM	ALLOWABLE LINE LD
ANALYSIS NO.	B001
REV. NO.	0
SHEET NO.	B001-18

REF

FIND MAX. GAPPED ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 471$ #/IN FOR 6×8×16 SOLID CONC BLOCK WITH 0.59 psi UNIFORM LOAD ON WALL

pg 9,

$$R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell \quad \cdots \text{ GOVERNS}$$

LET $R_A = F_\ell = 471$ #/IN, $t = 5.625$, $P = 0.59$ psi

$V = 0.519$ psi (PAGE 26)

$$\therefore \quad 471 = \frac{0.59 \times \ell}{4 \times 5.625}\sqrt{\ell^2 + (3 \times 5.625)^2} + 0.519 \times \ell \quad \cdots (2)$$

SOLVE ℓ, BY TRIAL & ERROR METHOD

Try $\ell = 10.33^{FT} = 124^{IN}$, SUBSTITUTING INTO (2)

WE HAVE $471 \doteq 471.2$. O.K CLOSE ENOUGH

USE $\ell = 124$"

$\therefore$ MAX. GAPPED ARCHING SPAN BASED ON ALLOWABLE LINE LOAD $F_\ell = 471$ #/IN IS 10.33^{FT} OR 124^{IN}

CALCULATION SHEET

EES

PROJECT: MILLSTONE #1, NUSCO
SUBJECT: BLOCKOUT (ARCHING ACTION)
SYSTEM: REACTIONS
ANALYSIS NO.: B001 REV. NO.: 0

PREPARED BY: C.T. LIN — DATE: 10-31-80
CHECKED BY: B. HONARARIAN — DATE: 11-5-80
JOB NO.: 20014 — FILE NO.: 12.F
SHEET NO.: B001-19

REF

SUMMRY OF REACTIONS DUE TO ARCHING ACTION

* INCLUDES DL & VERTICAL SEISMIC OF WALL

REF	CASE	SPAN (FT)	HORIZ. UNIFORM LOAD (PSI)	F_L (#/IN)	R_A* (#/IN)	R_B (#/IN)	S_A (#/IN)	REACTION PASS ?	REMARK
Pg 20	A'+A (NO GAP)	7.25	0.72	942 471	174	125	31	YES	RIGID ARCHING GOVERNING CASE
Pg 21	A' (NO GAP)	18.75	0.72	942	939	814	81	YES	ALLOW. LINE LOAD (M')
Pg 22	A' (NO GAP)	10.42	0.72	942	324	254	45	YES	DEFLECTION $\Delta = 1.5t$
Pg 23	A (NO GAP)	14.16	0.59	471	470	382	50	YES	ALLOW. LINE LOAD (M)
Pg 24	B' ($\frac{1}{16}$" GAP)	13.5	0.72	942	934	240	87	YES	ALLOW. LINE LOAD (M')
Pg 25	B' ($\frac{1}{16}$" GAP)	7.25	0.72	942	295	243	47	YES	DEFLECTION $\Delta = 1.5t$
Pg 26	B ($\frac{1}{16}$" GAP)	10.33	0.59	471	471	404	55	YES	ALLOW LINE LOAD (M)
Pg 27	C' ($\frac{1}{8}$" GAP)	13.5	0.72	942	934	240	87	YES	ALLOW. LINE LOAD (M')
Pg 28	C ($\frac{1}{8}$" GAP)	10.33	0.59	471	471	404	55	YES	ALLOW. LINE LOAD (M)
Pg 29	D' (WITH GAP)	7.0	0.72	942	277	226	45	YES	

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCK OUT (ARCHING ACTION) REACTIONS

PREPARED BY C. T. LIN DATE 10-16-80

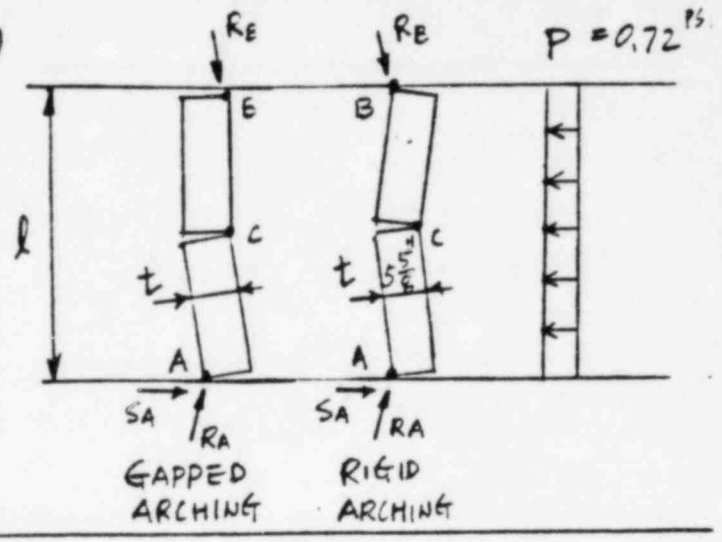
CHECKED BY B. HONARIAN DATE 11-5-80

JOB NO. 80014 FILE NO. 12-F

SHEET NO. B001-20

REF

CASE : A'+A (SPAN = 7.25', F_ℓ = 471 #/in OR F_ℓ = 942 #/in)



TYPE OF ARCHING USED :

☐ GAPPED ☑ RIGID

DESIGN SPAN ℓ = 7.25 FT = 87 IN

Pg 5

Pg 49

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

Pg 5

$$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{N: OF WYTH}} = 0.22 \text{ PSI}$$

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557 \text{ PSI}$$

Pg 8, 14

$$R_A = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} + V\frac{\ell}{2} = 173.7P + 0.557 \times 87 = 174 \text{ #/in} < F_\ell = 471 \text{ #} < F_\ell = 942 \text{ #/in} \quad \text{O.K}$$

$$R_B = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} = 173.7P = 125 \text{ #/in} < 471 \text{ #/in} < 942 \text{ #/in} \quad \text{O.K}$$

$$S_\ell = \frac{P\ell}{2} = 43.5P = 31 \text{ #/in} < 36 \text{ #/in} < 56 \text{ #/in} \quad \text{O.K}$$

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION)

REACTIONS

PREPARED BY	DATE
C. T. LIN	10-16-?
CHECKED BY	DATE
B. HONARVARIAN	10/23/?
JOB NO.	FILE NO.
8001.4	12-F

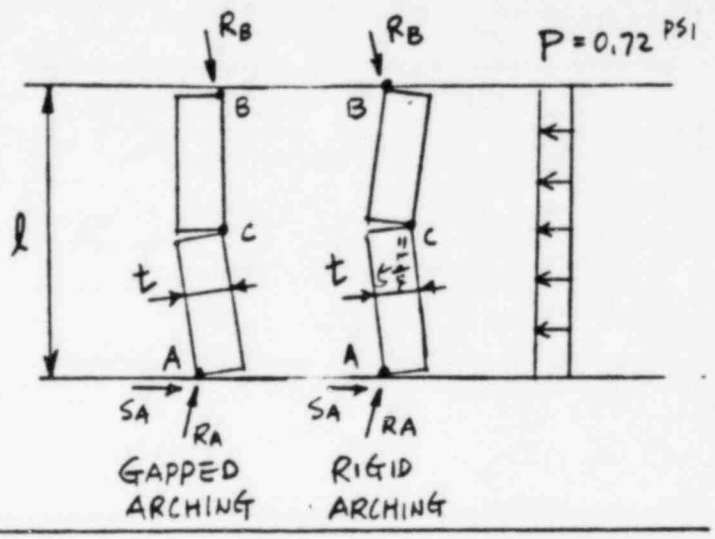
SHEET NO. B001-21

REF

CASE : A' (SPAN = 18.75', NO GAP, ALLOWABLE LINE LD F_L = 942 #/IN)

TYPE OF ARCHING USED :

☐ GAPPED ☒ RIGID



DESIGN SPAN l = 18.75 FT = 225 IN

Pg 15, 19

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.22 \text{ PSI}$$

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557 \text{ PSI}$$

Pg 2, 14

$$R_A = \frac{Pl}{8t}\sqrt{l^2 + (4t)^2} + Vl = 1131P + 0.557 \times 225 = 939 \text{ #/IN} \approx 942 \text{ #/IN}$$ O.K.

$$R_B = \frac{Pl}{8t}\sqrt{l^2 + (4t)^2} = 1131P = 814 \text{ #/IN} < 942 \text{ #/IN} = F_L$$ O.K.

ASSUME 1" BEARING WIDTH

$V_m = 1.7\sqrt{f'_m}$

Pg 11

$$S_A = \frac{Pl}{2} = 112.5P = 81 \text{ #/IN} > 56 \text{ #/IN}$$ N.G.

$< 0.2 \times 939 = 188$ #/IN O.K. IF FRICTION

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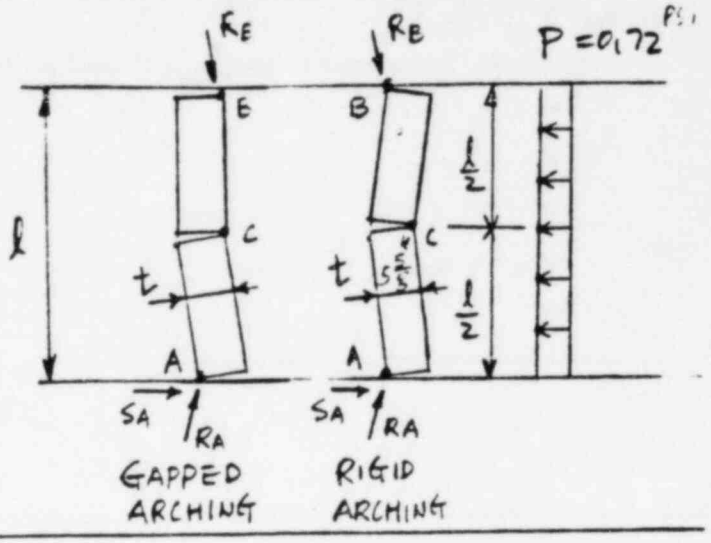
CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO
SUBJECT: BLOCKOUT (ARCHING ACTION) REACTIONS

PREPARED BY	DATE
C. T. LIN	10-16-85
CHECKED BY	DATE
B. MUNAVARAN	11-5-85
JOB NO.	FILE NO
80014	12-F
SHEET NO.	8001-22

REF

CASE : A' (SPAN = 10.42', NO GAP, HORIZ. DEFLECTION @ PT 'c' Δ = 1.5 t)



TYPE OF ARCHING USED :

☐ GAPPED ☑ RIGID

Pg 36, DESIGN SPAN ℓ = 10.42 FT = 125 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.50 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

Pg 5 $= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 22$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_V) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557$ PSI

Pg 8,14 $R_A = \frac{P\ell}{2t}\sqrt{\ell^2 + (4t)^2} + V\frac{\ell}{2} = 352.8P + 0.557 \times 125 = 324$ #/IN < F_E 342 #/IN OK

$R_B = \frac{P\ell}{2t}\sqrt{\ell^2 + (2t)^2} = 352.8P = 254$ #/IN < 342 #/IN OK

$S_A = \frac{P\ell}{2} = 62.5P = 45$ #/IN < 56 #/IN OK

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION) REACTIONS

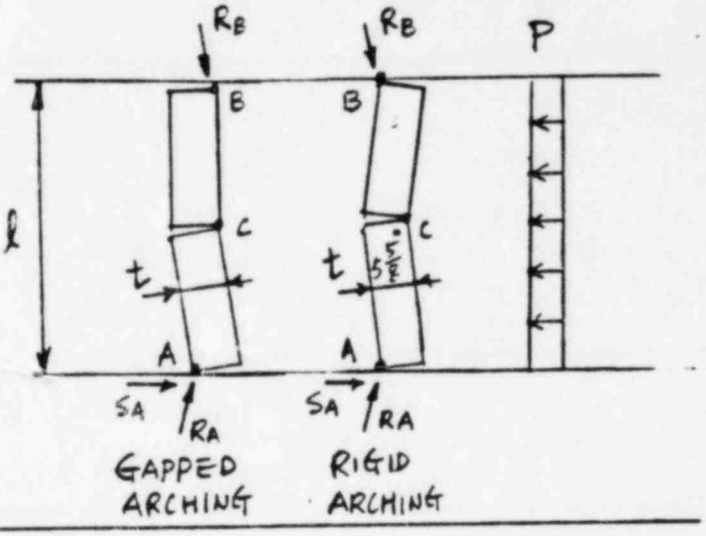
PREPARED BY	DATE
C. T. LIN	10-16-80
CHECKED BY	DATE
B. MONAHARAN	11-5-80
JOB NO.	FILE NO.
80014	12-F
SHEET NO.	B001-23

REF

CASE : A (SPAN = 14.16', NO GAP) F_t = 471 #/IN

TYPE OF ARCHING USED :

☐ GAPPED ☑ RIGID



Pg 16 DESIGN SPAN ℓ = 14.16 FT = 170 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

Pg 5 $= \frac{70}{144} \times \underbrace{0.186}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.09$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.09 = 0.59 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.067) \times 1 = 0.519$ PSI

Pg 8, 14 $R_A = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} + V\ell = 648\,P + 0.519 \times 170 = 470$ #/IN $< F_t = 471$ #/IN O.K

Pg 8 $R_B = \frac{P\ell}{8t}\sqrt{\ell^2 + (4t)^2} = 648\,P = 382$ #/IN $< F_t = 471$ #/IN

Pg 11 $S_A = \frac{P\ell}{2} = 85\,P = 50$ #/IN > 36 #/IN N.G ($1.1\sqrt{f'_m}$ WITH 1" BEARING ... TRY CONSIDERING FLEXURE EFFECT.

$< 0.2 \times 470 = 94$ #/IN O.K

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CALCULATION SHEET

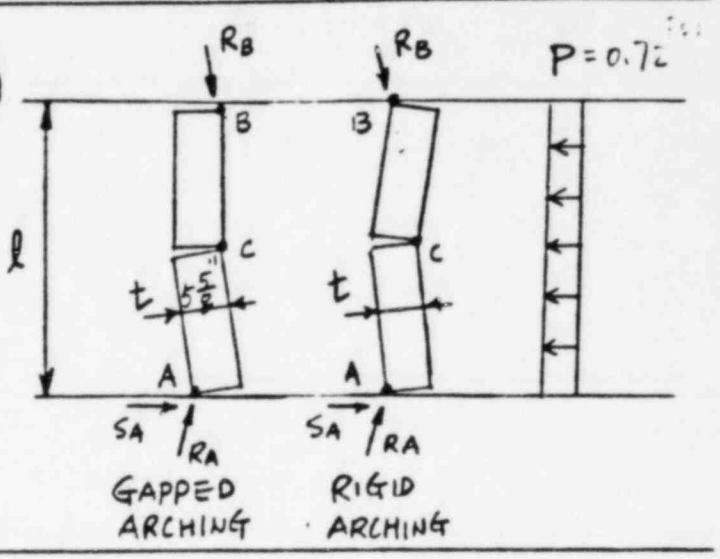
PROJECT	MILLSTONE #1, NUSCO	PREPARED BY: C. T. LIN	DATE: 10-16-87
SUBJECT	BLOCKOUT (ARCHING ACTION) REACTIONS	CHECKED BY: B. MONAVARIAN	DATE: 10/23/87
		JOB NO.: 80014	FILE NO.: 12-F
		SHEET NO.: B001-24	

REF

CASE : B'. (SPAN = 13.5', $\frac{1}{16}$" GAP, F_ℓ = 942 #/IN)

TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID



Pg 5, 17

DESIGN SPAN ℓ = 13.5 FT = 162 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.22 \text{ PSI}$$

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557 \text{ PSI}$$

Pg 9, 14

$$R_A = \frac{H}{\ell}\sqrt{\ell^2 + (3t)^2} + V\ell = 1173\,P + 0.557 \times 162 = 934 \doteq 942 \text{ #/IN} = F_\ell \text{ O.K}$$

$$R_B = \frac{H}{\ell}\sqrt{\ell^2 + t^2} = 1167\,P = 840 \text{ #/IN} < 942 \text{ #/IN} = F_\ell \text{ O.K}$$

Pg 11

$$S_A = \frac{3P\ell}{4} = 121.5\,P = 87 \text{ #/IN} > 56 \text{ #/IN} \quad \text{N.G}$$

$< 0.2 \times 934 = 188$ #/IN

(SINCE $R_A < F_\ell$, WE CAN TRY TO INCREASE THE ALLOWABLE DUE TO FRICTION EFFECT BETWEEN MASONRY AND CONCRETE)

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION) REACTIONS

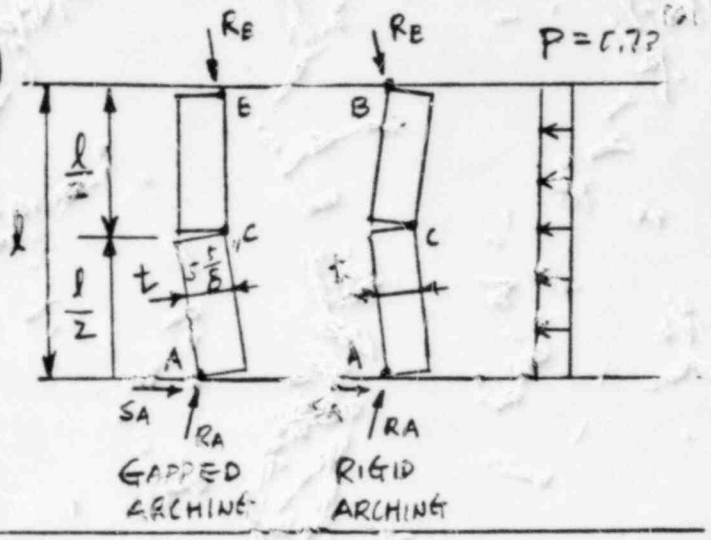
PREPARED BY	DATE
C. T. LIN	10-16-81
CHECKED BY	DATE
B. HONARVAR	11-5-81
JOB NO.	FILE NO.
80014	12.F
SHEET NO.	8001 - 25

REF

CASE : B' (SPAN = 7.25', 1" GAP, HORIZ. DEFLECTION @ PT 'C' Δ = 1.5 t)

TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID



Pg 5, 39 DESIGN SPAN ℓ = 7.25 FT = 87 IN

ACTURAL WALL THICKNESS = 5 5/8 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 22$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.46) \times 1 = 0.557$ PSI

Pg 9,14 $R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell = 343P + 0.557 \times 87 = 295$ #/IN < F_c = 9[illegible] #/IN C.K

Pg 9,14 $R_E = \frac{P\ell}{4t}\sqrt{\ell^2 + t^2} = 337P = 243$ #/IN < F_c = 942 #/IN C.K

Pg 11 $S_L = \frac{3P\ell}{4} = 65.3P = 47$ #/IN < 56 #/IN C.K

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION) REACTIONS

PREPARED BY	DATE
C. T. LIN	10-16-8
CHECKED BY	DATE
B. HOVANARIAN	11-5-8
JOB NO.	FILE NO.
80014	12-F
SHEET NO.	B001-26

REF

CASE : B (SPAN = 10.33', $\frac{1}{16}$" GAP, F_c = 471 #/IN)

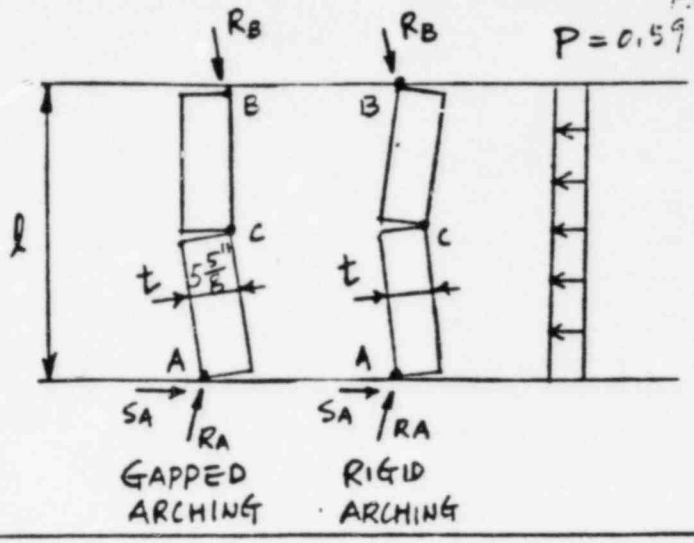
TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID

Pg 18 DESIGN SPAN ℓ = 10.33 FT = 124 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT



P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$= \frac{70}{144} \times \underbrace{0.186}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.09$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.09 = 0.59 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.067) \times 1 = 0.519$ PSI

Pg 9,14 $R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell = 689P + 0.519 \times 124 = 471.2 \simeq F_c = 471$ #/IN O.K.

Pg 9,14 $R_B = \frac{P\ell}{4t}\sqrt{\ell^2 + (t)^2} = 684P = 404$ #/IN $< F_c = 471$ #/IN O.K.

Pg 11 $S_A = \frac{3P\ell}{4} = 93P = 55$ #/IN > 36 #/IN N.G. $(1.1\sqrt{F'_m})$ TRY USING KEYWAY FOR HELP FROM FRICTION

$< 0.2 \times 471 = 92$ #/IN O.K.

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION) REACTIONS

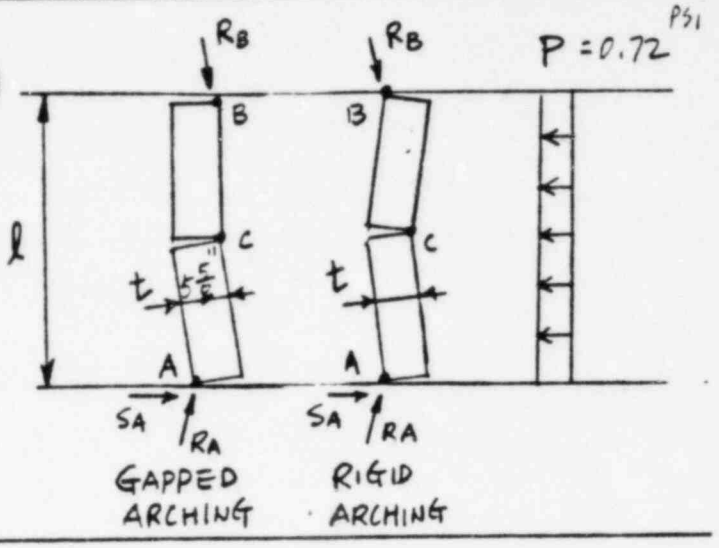
PREPARED BY	DATE
C. T. LIN	10-16-80
CHECKED BY	DATE
B. MONAJARIAN	10/23/80
JOB NO. 80014	FILE NO. 12-F
SHEET NO. B001-27	

REF

CASE : C'. (SPAN = 13.5', $\frac{1}{8}$" GAP, F_ℓ = 942 #/IN)

TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID



Pg 5, 17 DESIGN SPAN ℓ = 13.5 FT = 162 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.22$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557$ PSI

Pg 9, 14 $R_A = \frac{H}{\ell}\sqrt{\ell^2 + (3t)^2} + V\ell = 1173\,P + 0.557 \times 162 = 934 \doteq 942$ #/IN $= F_\ell$ O.K

$R_B = \frac{H}{\ell}\sqrt{\ell^2 + t^2} = 1167\,P = 840$ #/IN < 942 #/IN $= F_\ell$ O.K

Pg 11 $S_A = \frac{3P\ell}{4} = 121.5\,P = 87$ #/IN > 56 #/IN N.G

$< 0.2 \times 934 = 188$ #/IN O.K

(SINCE $R_A < F_\ell$, WE CAN TRY TO INCREASE THE ALLOWABLE DUE TO FRICTION EFFECT BETWEEN MASONRY AND CONCRETE)

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CALCULATION SHEET

PROJECT MILLSTONE #1, NUSCO

SUBJECT BLOCKOUT (ARCHING ACTION) REACTIONS

PREPARED BY C. T. LIN DATE 10-16-80

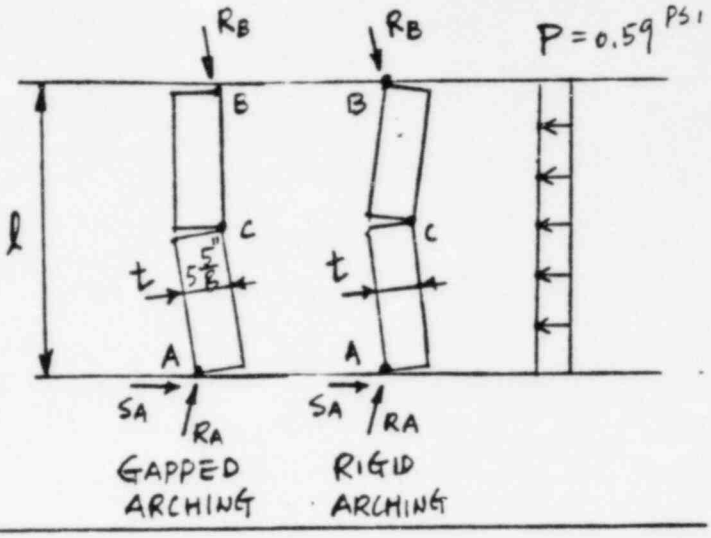
CHECKED BY B. MUNAWAR-IN DATE 11-5-80

JOB NO. 20014 FILE NO. 12-F

SHEET NO. B001-28

REF

CASE : C (SPAN = 10.33', $\frac{1}{8}$" GAP, F_e = 471 #/IN)



TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID

Pg 18 DESIGN SPAN ℓ = 10.33 FT = 124 IN

Pg 5 ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$$= \frac{70}{144} \times \underbrace{0.186}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.09 \text{ PSI}$$

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.09 = 0.59 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$$= W \times (1 + g_v) \times N = \frac{70}{144} \times (1 + 0.067) \times 1 = 0.519 \text{ PSI}$$

Pg 9, 14 $R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell = 689P + 0.519 \times 124 = 471 \approx F_e = 471$ #/IN

$R_B = \frac{P\ell}{4t}\sqrt{\ell^2 + (t)^2} = 684P = 404$ #/IN $< F_e = 471$ #/IN

DC1, Rev 2 & Pg 11 $S_A = \frac{3P\ell}{4} = 93P = 55$ #/IN > 36 #/IN N.G. $(1.1\sqrt{f'_c})$

$< 0.2 \times 471$

$= 92$ #/IN O.K.

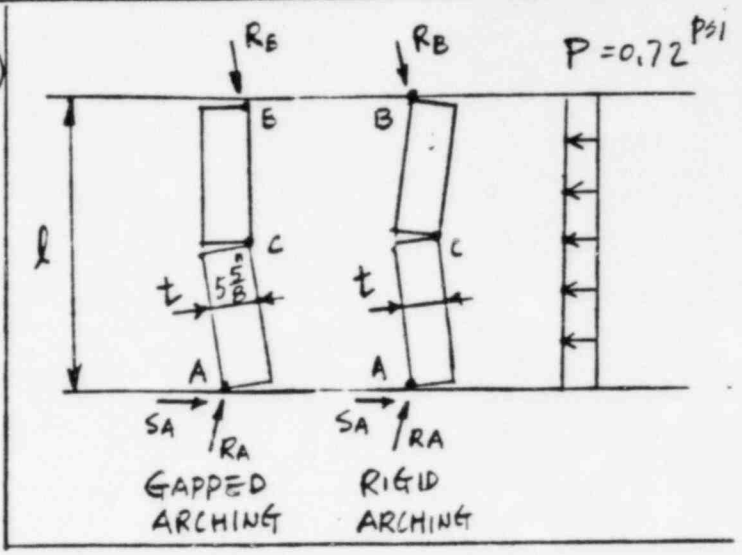
EARTHQUAKE ENGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO	PREPARED BY: C. T. LIN	DATE: 10-16-20
SUBJECT	BLOCKOUT (ARCHING ACTION) REACTIONS	CHECKED BY: B. HONARIAN	DATE: 11-5-20
		JOB NO.: 20014	FILE NO.: 12-F
		SHEET NO.: 2001-29	

REF

CASE : D' (SPAN = 7.0', WITH GAP F_L = 942 #/IN)



TYPE OF ARCHING USED :

☑ GAPPED ☐ RIGID

DESIGN SPAN ℓ = 7.0 FT = 84 IN

ACTURAL WALL THICKNESS = 5.625 IN

W = WALL WT + ATTACHMENT

P_1 = UNIFORM PRESSURE LOADS = 0.5 PSI

P_2 = INERTIA LOADS OF WALL DUE TO HORIZONTAL SEISMIC = $W g_H N$

$= \frac{70}{144} \times \underbrace{0.452}_{\text{HORIZONTAL } g \text{ FACTOR}} \times \underbrace{1}_{\text{NO OF WYTH}} = 0.22$ PSI

P = TOTAL UNIFORM LOADS = $P_1 + P_2$ = 0.5 + 0.22 = 0.72 PSI

V = INERTIA LOADS OF WALL DUE TO VERTICAL SEISMIC AND ITS DEAD WT.

$= W \times (1 + g_V) \times N = \frac{70}{144} \times (1 + 0.146) \times 1 = 0.557$ PSI

Pg 9, 14

$R_A = \frac{P\ell}{4t}\sqrt{\ell^2 + (3t)^2} + V\ell = 320P + 0.557 \times 84 = 277^{\#/IN} < F_L = 942^{\#/IN}$ O.K.

$R_B = \frac{P\ell}{4t}\sqrt{\ell^2 + (t)^2} = 314P = 226^{\#/IN} < F_L = 742^{\#/IN}$ O.K.

DC-1, REV 1

$S_A = \frac{3P\ell}{4} = 63P = 45^{\#/IN} < 56^{\#/IN}$ O.K.

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCK OUT (ARCHING ACTION)
SYSTEM	DEFLECTION
ANALYSIS NO.	B001
REV. NO.	0

PREPARED BY	C.T.LIN
DATE	10-21-80
CHECKED BY	B.HONARVARIAN
DATE	11-5-80
JOB NO.	20014
FILE NO.	12-F
SHEET NO.	B001-30

PAGE

SECT. 9.0 DEFLECTION REQUIREMENTS 30~41

9.1 ASSUMPTIONS 31

9.2 CONCLUSION 32~33

9.3 DEFLECTION DUE TO RIGID ARCHING 34

9.4 MAX. ALLOWABLE SPAN FOR RIGID ARCHING 35~36

9.5 DEFLECTION DUE TO GAPPED ARCHING 37

9.6 MAX. ALLOWABLE SPAN FOR $\frac{1}{16}$" GAPPED ARCHING 38~39

9.7 MAX. ALLOWABLE SPAN FOR $\frac{1}{8}$" GAPPED ARCHING 40~41

EES

CALCULATION SHEET

PREPARED BY: C.T. LIN	DATE: 10-21-80
CHECKED BY: B. MONAJARIAN	DATE: 11-5-80
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: BLOCKOUT (ARCHING ACTION)	JOB NO.: 80014 / FILE NO.: 12-F
SYSTEM: DEFLECTION	
ANALYSIS NO.: B001 REV. NO.: 0	SHEET NO.: B001-31

REF

DEFLECTION

ASSUMPTIONS :

① BLOCKOUT IS ASSUMED TO BE CONFINED BETWEEN TWO NON-YIELDING SUPPORTS AT TOP EDGE AND BASE DURING ARCHING ACTION (THAT IS BOUNDARY SUPPORT STIFFNESS AT TOP & BOTT. EDGES TO BE INFINITE)

② MAXIMUM ALLOWABLE HORIZONTAL DEFLECTION @ MIDSPAN OF WALL (CRACKED SECTION) SHALL BE LIMITED TO 0.15 THICKNESS OF WALL (ONE WYTH WIDTH)

③ DEFLECTION IS DERIVED BY ASSUMING THAT WALL CRACKS AT MIDSPAN DUE TO EXTERNAL UNIFORM LOAD AND THEN FORMS A THREE-HINGED ARCH IN THE VERTICAL DIRECTION

④ GEOMETRY RELATIONSHIP CAN BE OBTAINED FROM THE SHORTENING OF DIAGONAL (AS SHOWN ON PAGE 34, $\overline{AC} - \overline{AC'}$) THROUGH ULTIMATE STRAIN $\varepsilon = \frac{f_\ell}{E'}$.

WHERE f_ℓ = LINE LOAD AT SUPPORT OF THREE-HINGED ARCH

E' = MODULUS OF ELASTICITY IN LINE LOADING

$= 29300^{\#}$ $^{\text{lb-in/in}^2}$ FROM PAGE C-29 OF REF. 2

⑤ ASSUME THAT DEFLECTION DUE TO GAPPED ARCHING IS THE DEFLECTION DUE TO RIGID ARCHING PLUS THE DEFLECTION DUE TO ITS OWN RATATION @ BOTT. PIVOT PT 'A' BEFORE ARCHING ACTION OCCURING.

⑥ NEGLECT THE EFFECT OF ADDITIONAL DEFLECTION DUE TO RELATIVE FLOOR DISPLACEMENT (IN-PLANE & OUT OF PLANE)

EES	CALCULATION SHEET	PREPARED BY C. T. LIN	DATE 10-21-80
		CHECKED BY B MONAVARIAN	DATE 11-5-80
	PROJECT MILLSTONE #1, NUSCO	JOB NO. 80014	FILE NO. 12-F
	SUBJECT BLOCKOUT (ARCHING ACTION)		
	SYSTEM DEFLECTION		
	ANALYSIS NO. B001 REV. NO. 0	SHEET NO. B001-32	

REF.

DEFLECTION

CONCLUSION :

① RIGID ARCHING : DEFLECTION @ MIDSPAN OF BLOCKOUT CAN BE OBTAINED BY FORMULA FROM PAGE 34

$$\Delta = t - \sqrt{t^2 - m} \quad \cdots\cdots ①$$

THIS TERM CAN BE NEGLECTED

WHERE $m = S^2 \varepsilon (2 - \varepsilon)$, OR $m = 2\varepsilon S^2$

$$\varepsilon = \frac{fl}{E'}$$

$$S = \left[t^2 + \left(\frac{l}{2}\right)^2 \right]^{\frac{1}{2}}$$

② GAPPED ARCHING : DEFLECTION @ MIDSPAN OF BLOCKOUT CAN BE OBTAINED BY FORMULA FROM PAGE 37

$$\Delta = \frac{l \varepsilon_0}{2t} + t - \sqrt{t^2 - m}$$

THIS TERM CAN BE NEGLECTED

WHERE $m = S^2 \varepsilon (2 - \varepsilon)$ OR $m = 2\varepsilon S^2$

$$\varepsilon = \frac{fl}{E'}$$

$$S = \left[t^2 + \left(\frac{l}{2}\right)^2 \right]^{\frac{1}{2}}$$

Pg 36

③ RIGID ARCHING : 10.42 FT IS THE MAXIMUM ALLOWABLE SPAN FOR SOLID CONC. MASONRY BLOCKOUT BASED ON

1) $\Delta = 0.15\ t = 0.15 \times 5.625 = 0.84"$ @ MIDSPAN
2) $P = 0.72$ psi UNIFORM LOAD ON WALL
3) SINGLE WYTH OF 6 X 8 X 16 BLOCK UNIT

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCK OUT (ARCHING ACTION)
SYSTEM	DEFLECTION
ANALYSIS NO.	B001 REV. NO. 0

PREPARED BY	C.T. LIN
DATE	10-21-85
CHECKED BY	B. MOJAVARIAN
DATE	11-5-85
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001-33

DEFLECTION

CONCLUSION : (CONT')

④ GAPPED ARCHING :

Pg 39 WITH $\frac{1}{16}$" GAP , MAX. ALLOWABLE SPAN IS 7.25 FT

Pg 40 WITH $\frac{1}{8}$" GAP . MAX ALLOWABLE SPAN IS 5.42 FT

FOR SOLID CONC. MASONRY BLOCKOUT BASED ON

1) $\Delta = 0.15\ t = 0.15 \times 5.625 = 0.84$" @ MIDSPAN

2) P = 0.72 PSI UNIFORM LOAD ON WALL

3) SINGLE WYTH OF 6 x 8 x 16 BLOCK UNIT

CALCULATION SHEET

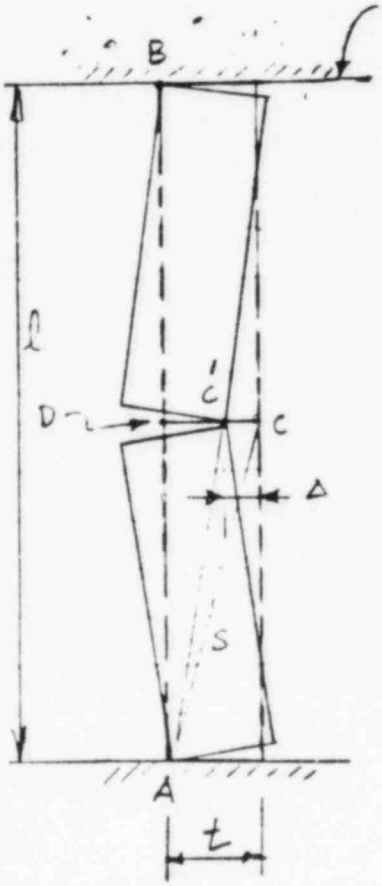
EES

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT (ARCHING ACTION)
SYSTEM	DEFLECTION
ANALYSIS NO.	B001
REV. NO.	0

PREPARED BY	C. T. LIN
DATE	10-21-80
CHECKED BY	B. HONARVARIAN
DATE	11-5-80
JOB NO.	20014
FILE NO.	12-F
SHEET NO.	B001-34

REF

DEFLECTION DUE TO RIGID ARCHING ACTION (NO GAP)



GEOMETRY RELATIONSHIP :

$$\Delta = \overline{CC'}$$

$$\overline{AC} = \left[t^2 + \left(\frac{l}{2}\right)^2\right]^{\frac{1}{2}} = S$$

$$\overline{AC'}^2 = \overline{DC'}^2 + \left(\frac{l}{2}\right)^2$$

$$\overline{AC'} = \left[(t-\Delta)^2 + \left(\frac{l}{2}\right)^2\right]^{\frac{1}{2}}$$

$$\varepsilon = \frac{\overline{AC} - \overline{AC'}}{\overline{AC}}, \qquad \varepsilon = \frac{f_l}{E'}$$

$$\varepsilon = \frac{S - \left[(t-\Delta)^2 + \left(\frac{l}{2}\right)^2\right]^{\frac{1}{2}}}{S}$$

$$\Delta^2 - 2t\Delta + S^2\left[1 - (1-\varepsilon)^2\right] = 0$$

$$\Delta^2 - 2t\Delta + S^2\varepsilon(2-\varepsilon) = 0$$

LET $S^2\varepsilon(2-\varepsilon) = m$

$$\therefore \Delta = \frac{2t \pm \sqrt{4t^2 - 4m}}{2}$$

$$= t \pm \sqrt{t^2 - m}$$

(DISCARD $t + \sqrt{t^2-m}$ SINCE IT IS LARGER THAN t)

$$\therefore \Delta = t - \sqrt{t^2 - m}$$

WHERE $m = S^2\varepsilon(2-\varepsilon)$ AND $S = \left[t^2 + \left(\frac{l}{2}\right)^2\right]^{\frac{1}{2}}$, $\varepsilon = \frac{f_l}{E'}$

NOTE: ① f_l = LINE LOADING (lb/in)

② ASSUME $E' = 29300^{(lb/in^2)}$ FOR MODULUS OF ELASTICITY IN LINE LOADING WHICH IS FROM PAGE C-29 OF REF 2

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO	PREPARED BY C.T. LIN	DATE 10-21-80
SUBJECT	BLOCKOUT (ARCHING ACTION)	CHECKED BY B. MORAVARIAN	DATE 11-5-80
SYSTEM	DEFLECTION	JOB NO. 80014	FILE NO. 12-F
ANALYSIS NO.	A.001 REV. NO. 0	SHEET NO.	B001-35

REF.

Pg 31 FIND ALLOWABLE SPAN FOR $\Delta = 0.15t$ OF 6×8×16 SOLID BLOCK WALL DURING RIGID ARCHING ACTION

Pg 34

$$\Delta = t - \sqrt{t^2 - m}$$

$$(t - \Delta)^2 = t^2 - m$$

$$(t - 0.15t)^2 = t^2 - m$$

$t = 5.625''$ $\therefore m = t^2 - (0.85t)^2 = 0.2775\,t^2 = 8.78$

Pg 5

$$\varepsilon = \frac{f_\ell}{E'} = \frac{R_A}{E'} = \frac{\left\{\frac{Pl}{8t}\sqrt{l^2 + (4l)^2} + Vl\right\}}{E'}$$

Pg 22

$V = 0.557$ psi — VERTICAL SEISMIC & DL PER UNIT AREA

$P = 0.72$ psi

$$\varepsilon = \frac{\frac{0.72 \times l}{8 \times 5.625}\sqrt{l^2 + (4 \times 5.625)^2} + 0.557 \times l}{29{,}000}$$

$$\varepsilon = \frac{l\left[0.016\sqrt{l^2 + 506.25} + 0.557\right]}{29{,}000}$$

SINCE $m = S^2\varepsilon(2 - \varepsilon) = 8.78$, $S^2 = \left(\frac{l}{2}\right)^2 + (t)^2$

$$\therefore \left[\left(\frac{l}{2}\right)^2 + (5.625)^2\right]\varepsilon(2 - \varepsilon) = 8.78$$

SOLVE l:

FIRST TRY $l = 100$, $\varepsilon = 0.000749$, $m = 3.79 < 8.78$

2nd TRY $l = 150$, $\varepsilon = 0.001577$ $m = 17.26 > 8.78$

3rd TRY $l = 120$, $\varepsilon = 0.001028$ $m = 7.46 < 8.78$

EES

CALCULATION SHEET

PREPARED BY C.T.LIN	DATE 10-21-80
CHECKED BY B.MONAHARIAN	DATE 11-5-80
PROJECT MILLSTONE #1, NUSCO	JOB NO. 80014 / FILE NO. 12-F
SUBJECT BLOCK OUT (ARCHING ACTION)	
SYSTEM DEFLECTION	
ANALYSIS NO. B001 REV. NO. 0	SHEET NO. B001-36

REF

4TH TRY $\ell = 124"$, $\varepsilon = 0.001024$ $M = 8.43 < 8.78$

5TH TRY $\ell = 126"$ $\varepsilon = 0.001112$ $M = 8.95 > 8.78$

FINAL TRY $\ell = 125"$ $\varepsilon = 0.001091$ $M = 8.68 < 8.78$

O.K.

USE 125"

∴ 10.42 FT IS THE MAXIMUM ALLOWABLE SPAN FOR 6X8X16 SOLID BLOCK WALL DURING RIGID ARCHING ACTION WITH HORIZONTAL DISPLACEMENT @ MIDSPAN $\Delta = 0.15\,t$ (THICKNESS OF WALL) AND 0.72 PSI UNIFORM LOAD ON THE WALL

EES

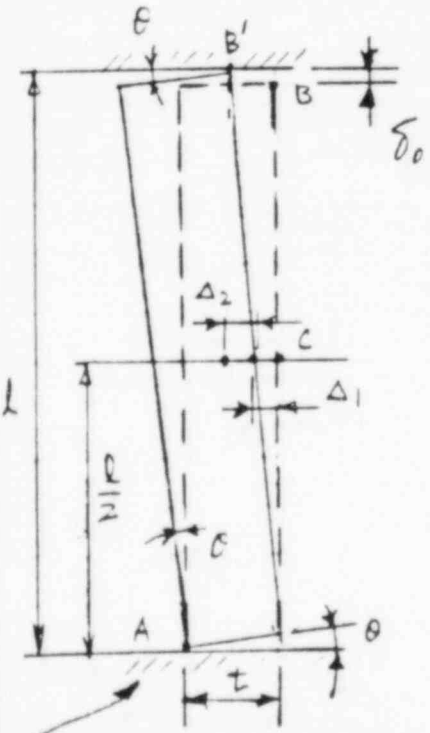
CALCULATION SHEET

PREPARED BY C. T. LIN	DATE 10-21-80
CHECKED BY B. HONARIAN	DATE 11-5-80
JOB NO. 80014	FILE NO. 12-F
SHEET NO. B001-37	

PROJECT MILLSTONE #1, NUSCO
SUBJECT BLOCKOUT (ARCHING ACTION)
SYSTEM DEFLECTION
ANALYSIS NO. B001 REV. NO. 0

DEFLECTION DUE TO GAPPED ACHING ACTION

ASSUME THE MAX. DEFLECTION OF GAPPED ARCHING @ PT 'C' IS EQUAL TO $\Delta_1 + \Delta_2$, WHERE Δ_2 IS THE HORIZ. DEFLECTION @ PT 'C' DUE TO THE SHORTENING OF DIAGONAL AFTER ARCHING ACTION TAKING PLACE AND Δ_1 IS THE HORIZONTAL DEFLECTION @ PT 'C' DUE TO THE GAP EXISTENCE WHICH CAUSES THE WHOLE WALL TO ROTATE AS A RIGID BODY MOVEMENT @ PT 'A' UNTIL PT 'E' (UPSTEAM PT @ TOP OF WALL) TOUCHES THE BOUNDARY OF UPPER SUPPORT.



δ_0 = INTIAL GAP, $\theta \approx \frac{\delta_0}{t}$ AND

$\theta \approx \frac{\Delta_1}{\frac{\ell}{2}}$, LET $\frac{\Delta_1}{\frac{\ell}{2}} = \frac{\delta_0}{t}$

$\therefore \Delta_1 = \frac{\ell \delta_0}{2t}$

Pg 34

LET $\Delta_2 = t - \sqrt{t^2 - m}$ (ASSUME SIMILAR RELATIONSHIP AS RIGID ARCHING)

SINCE $\Delta = \Delta_1 + \Delta_2$

$\therefore$ $\Delta = \frac{\ell \delta_0}{2t} + t - \sqrt{t^2 - m}$

WHERE $m = S^2 \varepsilon (2 - \varepsilon)$
AND $S = [t^2 + (\frac{\ell}{2})^2]^{\frac{1}{2}}$, $\varepsilon = \frac{t}{\ell'}$

EES

CALCULATION SHEET

PREPARED BY C.T.LIN			DATE 10-21-80
CHECKED BY B.MONAVARIAN			DATE 11-5-80
PROJECT MILLSTONE #1, NUSCO		JOB NO. 80014	FILE NO. 12-F
SUBJECT BLOCKOUT, (ARCHING ACTION)			
SYSTEM DEFLECTION			
ANALYSIS NO. B001	REV. NO. 0	SHEET NO. B001-38	

REF

AS PER PAUL BAUGMAN

FIND ALLOWABLE SPAN FOR $\Delta = 0.15t$ OF 6×8×16 BLOCK WALL DURING GAPPED ARCHING ACTION (INITIAL GAP = $\frac{1}{16}$ INCH & EXTERNAL UNIFORM LD = 0.72 PSI)

Pg 37

$$\Delta = \frac{l\,\delta_0}{2t} + t - \sqrt{t^2 - m}$$

$$t^2 - m = \left(\frac{l\,\delta_0}{2t} + t - \Delta\right)^2$$

$$m = t^2 - \left(\frac{l\,\delta_0}{2t} + t - 0.15t\right)^2$$

$$m = t^2 - \left(\frac{l\,\delta_0}{2t} + 0.85t\right)^2$$

LET $t = 5.625''$, $\delta_0 = \frac{1}{16}''$

$$\therefore m = (5.625)^2 - \left(\frac{l}{2\times 5.625\times 16} + 0.85\times 5.625\right)^2$$

$$\therefore m = 31.64 - \left(\frac{l}{180} + 4.78\right)^2 \quad \cdots\cdots \textcircled{1}$$

$$\varepsilon = \frac{f_l}{E'} = \frac{R_A}{E'} = \frac{\left[\frac{Pl}{4t}\sqrt{l^2 + (3t)^2} + Vl\right]}{293000}$$

Pg 5, Pg 25

$V = 0.557$ PSI (VERTICAL SEISMIC & DL PER UNIT AREA)

$P = 0.72$ PSI

$$\varepsilon = \frac{l\left[\frac{0.72}{4\times 5.625}\sqrt{l^2 + (3\times 5.625)^2} + 0.557\right]}{293000}$$

$$\varepsilon = l\left\{\frac{0.032\sqrt{l^2 + 284.7} + 0.557}{293000}\right] \quad \cdots\cdots \textcircled{2}$$

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT (ARCHING ACTION)
SYSTEM	DEFLECTION
ANALYSIS NO.	B001 REV. NO. 0

PREPARED BY	C. T. LIN
DATE	10-21-2
CHECKED BY	B. HONARIAN
DATE	11-5/9
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001-39

REF.

SINCE $m = S^2 \varepsilon (2-\varepsilon)$ WHERE $S = \left\{t^2 + \left(\frac{l}{2}\right)^2\right\}^{\frac{1}{2}}$.

$$m = \left[(5.625)^2 + \left(\frac{l}{2}\right)^2\right] \times \varepsilon (2-\varepsilon) \quad \cdots\cdots (3)$$

LET (1) = (3) AND SOLVE l BY TRIAL & ERROR METHOD

FIRST TRY $l = 100"$, $\varepsilon = 0.001297$

(1) = 3.17 < (3) = 6.57

2nd TRY $l = 80"$

$\varepsilon = 0.000824$

(1) = 4.345 > (3) = 2.226

3rd TRY $l = 90"$ $\varepsilon = 0.00107$

(1) = 3.76 < (3) = 4.4

4TH TRY $l = 86"$ $\varepsilon = 0.000986$

(1) = 3.99 > (3) = 3.71

5TH TRY $l = 88"$ $\varepsilon = 0.001028$

(1) = 3.87 < (3) = 4.045

FINAL TRY $l = 87"$ $\varepsilon = 0.001$

(1) = 3.937 > (3) = 3.84 O.K USE 87"

∴ ALLOWABLE SPAN FOR 6" THICK SOLID BLOCK WALL WITH $\frac{1}{16}$" GAP, 0.72 PSI UNIFORM LOAD, AND $\Delta = 0.05$" @ MIDSPAN IS 7.25 FT DURING GAPPED ARCHING ACTION.

EES

CALCULATION SHEET

PREPARED BY: C.T.LIN	DATE: 10-21-80
CHECKED BY: B.HONARIAN	DATE: 11-5-80
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: BLOCKOUT (ARCHING ACTION)	
SYSTEM: DEFLECTION	
JOB NO.: 80014	FILE NO.: 12-F
ANALYSIS NO.: B001 REV. NO.: 0	SHEET NO.: B001-40

REF

FIND ALLOWABLE SPAN FOR $\Delta = 0.15t$ (AS PER PAUL BAILEMAN) OF 6×8×16 BLOCK WALL DURING GAPPED ARCHING ACTION (INITIAL GAP = $\frac{1}{8}$ INCH & EXTERNAL UNIFORM LD = 0.72 PSI)

Pg 37

$$\Delta = \frac{l\,\varepsilon_c}{2t} + t - \sqrt{t^2 - m}$$

$$t^2 - m = \left(\frac{l\,\varepsilon_c}{2t} + t - \Delta\right)^2$$

$$m = t^2 - \left(\frac{l\,\varepsilon_c}{2t} + t - 0.15t\right)^2$$

$$m = t^2 - \left(\frac{l\,\varepsilon_c}{2t} + 0.85t\right)^2$$

LET $t = 5.625''$, $\varepsilon_c = \frac{1}{8}''$

$$m = (5.625)^2 - \left(\frac{l}{2\times 5.625\times 8} + 0.85\times 5.625\right)^2$$

$$\therefore m = 31.64 - \left(\frac{l}{90} + 4.78\right)^2 \quad \cdots\cdots \textcircled{1}$$

$$\varepsilon = \frac{f_l}{E'} = \frac{R_A}{E'} = \frac{\left[\frac{Pl}{4t}\sqrt{l^2 + (3t)^2} + Vl\right]}{29{,}000}$$

Pg 5.

$V = 0.557$ psi (VERTICAL SEISMIC & DL PER UNIT AREA)
$P = 0.72$ psi

$$\varepsilon = \frac{l\left[\frac{0.72}{6\times 5.625}\sqrt{l^2 + (3\times 5.625)^2} + 0.557\right]}{29{,}000}$$

$$\varepsilon = l\left\{\frac{0.032\sqrt{l^2 + 284.7} + 0.557}{29{,}000}\right] \quad \cdots\cdots \textcircled{2}$$

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCK OUT (ARCHING ACTION)
SYSTEM	DEFLECTION
ANALYSIS NO.	B001 REV. NO. 0

PREPARED BY	C. T. LIN
DATE	10-21-80
CHECKED BY	B. HONARIAN
DATE	11-5-80
JOB NO.	80014
FILE NO.	12-F
SHEET NO.	B001-41

REF.

SINCE $m = s^2 \varepsilon (2-\varepsilon)$ WHERE $s = \left[t^2 + \left(\frac{\ell}{2}\right)^2\right]^{\frac{1}{2}}$

$$m = \left[(5.625)^2 + \left(\frac{\ell}{2}\right)^2\right] \times \varepsilon (2-\varepsilon) \cdots\cdots (3)$$

LET (1) = (3) AND SOLVE ℓ BY TRY & ERROR METHOD

FIRST TRY $\ell = 100''$ $\varepsilon = 0.001297$

(1) = 3.065 < (3) = 6.57

2nd TRY $\ell = 65''$ $\varepsilon = 0.0006$

(1) = 1.36 $\simeq$ (3) = 1.3 O.K.

USE $\ell = 65''$
$= 5.4'$

∴ ALLOWABLE SPAN FOR 6" THICK SOLID BLOCK WALL WITH $\frac{1}{8}''$ GAP, 0.72 psi UNIFORM LOAD AND $\Delta = 0.15\,t$ @ MIDSPAN IS 5.4 FT DURING GAPPED ARCHING

EES

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKCUT (ARCHING ACTION)
SYSTEM	FREQUENCY
ANALYSIS NO.	B001 REV. NO. 0

PREPARED BY	C. T. LIN
DATE	10-31-80
CHECKED BY	B. MONAVARIAN
DATE	11-5-80
JOB NO.	20014
FILE NO.	12-F
SHEET NO.	B001-42

			PAGE
SECT	10.0	FREQUENCY REQUIRMENT	42 ~ 49
	10.1	ASSUMPTIONS	43
	10.2	CONCLUSION	44
	10.3	FORMULA FOR FINDING NATURAL FREQUENCY	45
	10.4	ALLOWABLE SPAN FOR RIGID ARCHING @ 20 HERTZ	46
	10.5	ALLOWABLE SPAN FOR GAPPED ARCHING @ 20 HERTZ	47
	10.6	CHART 1 (GAPPED ARCHING)	48
	10.7	CHART 2 (RIGID ARCHING)	49

EES

CALCULATION SHEET

PREPARED BY: C. T. LIN	DATE: 10-21-80
CHECKED BY: B. MONAVARIAN	DATE: 11/5/80
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: BLOCKOUT (ARCHING ACTION)	
SYSTEM: FREQUENCY	
JOB NO.: 80014	FILE NO.: 12-F
ANALYSIS NO.: B001 REV. NO.: 0	SHEET NO.: B001-43

FREQUENCY

ASSUMPTIONS:

① NATURAL FREQUENCY OF A SOLID CONC. MASONRY BLOCKOUT CAN BE ANALYZED AS A PLATE BY ASSUMING AN UNCRACKED SECTION OF WALL SPANNING VERTICALY BETWEEN TOP & BOTT. EDGES AND USING LINEAR, ELASTIC ANALYSIS BEFORCE ARCHING ACTION TAKING PLACE (I.E. CRACK STARTS AT MIDSPAN OF WALL)

② SUPPORT CONDITIONS FOR FINDING NATURAL FREQUENCY OF WALL BEFORE ARCHING ACTION ARE AS FOLLOWS

FOR RIGID ARCHING: (NO VISIBLE CRACK AT TOP EDGE)

a) TOP & BOTT EDGES TO BE ASSUMED AS HINGED SUPPORT

b) TWO VERTICAL EDGES TO BE ASSUMED AS FREE EDGES WITHOUT CONSIDERING WHETHER THEY ARE CRACKED OR NOT.

FOR GAPPED ARCHING: (VISIBLE CRACK AT TOP EDGE)

a) TOP & TWO VERTICAL SIDES TO BE ASSUMED FREE EDGES

b) BOTT. EDGE TO BE TREATED AS A HINGED SUPPORT

③ IN ORDER TO APPLY THE RIGID RANGE ACCELERATION VALUES IN EVALUATING SEISMIC LOADING FOR ARCHING ACTION, ALL BLOCKOUTS SHALL SATISFY THE NATURAL FREQUENCY OF 20 CPS OR GREATER. DC-1, REV-1

④ FACE SHELL AREAS SHALL BE USED IN COMPUTING THE SECTION PROPERTIES TO ACCOUNT FOR FULL MORTAR BEDDING JOINT NOT COMPLETELY DONE REF 5

EES

CALCULATION SHEET

PREPARED BY C.T.LIN	DATE 10-27-80
CHECKED BY B.HONARVARIAN	DATE 11-5-80
PROJECT MILLSTONE #1, NUSCO	
SUBJECT BLOCKOUT (ARCHING ACTION)	JOB NO. 2001.4 / FILE NO. 12-F
SYSTEM FREQUENCY	
ANALYSIS NO. 2001 REV. NO. 0	SHEET NO. 5001-44

⑤ NATURAL FREQUENCY OF BLOCKOUT IS CALCULATED BY THE ANALYSIS BASED ON SINGLE WYTH OF 6×8×16 SOLID MASONRY WALL. FOR A WALL CONSISTING OF MORE THAN ONE WYTH IN ITS THICKNESS SHALL BE ASSUMED THAT EACH WYTH IS ACTING INDEPENDENTLY FROM THE OTHERS. IT IS BELIEVED THAT SUCH ASSUMPTION IS CONSERVATIVE.

⑥ ATTACHMENT WEIGHT ON WALL ARE NOT INCLUDED FOR FINDING MAX. ALLOWABE SPAN DUE TO FREQUENCY REQUIREMENTS.

CONCLUSION: MAX. PLATE DIMENSIONS FOR A SOLID UNREINFORCED MASONRY BLOCKOUT TO SATISFY THE NATURAL FREQUENCY OF 20 CPS REQUIRMENTS ARE CONTAINED IN CHART 1, PAGE 48, FOR GAPPED ARCHING ACTION AND IN CHART 2, PAGE 49 FOR RIGID ARCHING ACTION.

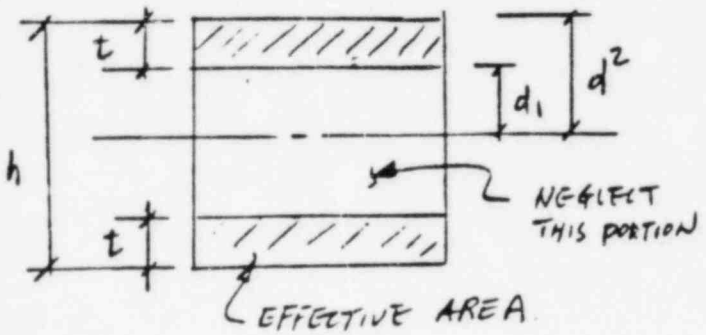
EES
EARTHQUAKE ENGINEERING SYSTEMS, INC.

CALCULATION SHEET

PROJECT	MILLSTONE #1, NUSCO
SUBJECT	BLOCKOUT (ARCHING ACTION)
	NATURAL FREQUENCY OF SOLID CONC. BLOCK

PREPARED BY	C.T. LIN	DATE 9/5/80
CHECKED BY	B. HOVANARIAN	DATE 11/5/80
JOB NO. 8001F	FILE NO. 12-F	
SHEET NO. E001-45		

REFERENCE : PAGE 237, 253 TO 261 OF FORMULAS FOR NATURAL FREQUENCY AND MODE SHAPE BY ROBERT D. BLEVINS.



CONC. BLOCK TREATED AS SANDWICH PLATES FOR FINDING $\frac{Eh^3}{12}$

$$\frac{Eh^3}{12} = \frac{2}{3}\sum_{K=0,1,} E_K (d_{K+1}^3 - d_K^3)$$

$$= \frac{2}{3}\left[\cancel{E_0}^{0}(d_1^3 - d_0^3) + E_1(d_2^3 - d_1^3)\right]$$

$$= \frac{2}{3} E_1 (d_2^3 - d_1^3) \quad \cdots \text{(ASSUME } E_0 = 0)$$

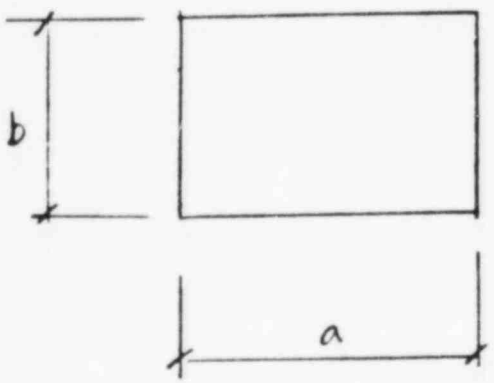
$E_1 = 1080000$ psi $= 1000\, f'_m$ ($f'_m = 1080$ psi)

USE 810,000 PSI AS PER PAUL BAUGMAN

γ = MASS PER UNIT AREA

CONSERVATIVE IF USING 0.2 INSTEAD OF 0.3

$\nu = 0.2$ (POISSON'S RATIO)



FIND NATURAL FREQUENCY FOR A PLATE :

$$f_{ij} = \frac{\lambda_{ij}^2}{2\pi a^2}\left[\frac{Eh^3}{12\gamma} \times \frac{1}{(1-\nu^2)}\right]^{\frac{1}{2}}$$

$$f_{ij} = \frac{\lambda_{ij}^2}{2\pi a^2}\left[\frac{2E_1(d_2^3 - d_1^3)}{3\gamma} \times \frac{1}{1-\nu^2}\right]^{\frac{1}{2}}, \quad (i=1, j=1)\ (E_1 = E)$$

$$f_n = \frac{\lambda^2}{2\pi a^2}\left[\frac{2E(d_2^3 - d_1^3)}{3\gamma(1-\nu^2)}\right]^{\frac{1}{2}}$$

LET $C_1 = \frac{2E}{3\gamma(1-\nu^2)}$, $C_2 = (d_2^3 - d_1^3)$, $f_n = \frac{\lambda^2}{a^2} \times \frac{[C_1 \times C_2]^{\frac{1}{2}}}{2\pi}$

LET $C_3 = \frac{[C_1 \times C_2]^{\frac{1}{2}}}{2\pi}$ $f_n = \frac{\lambda^2}{a^2} \times C_3$

CALCULATION SHEET

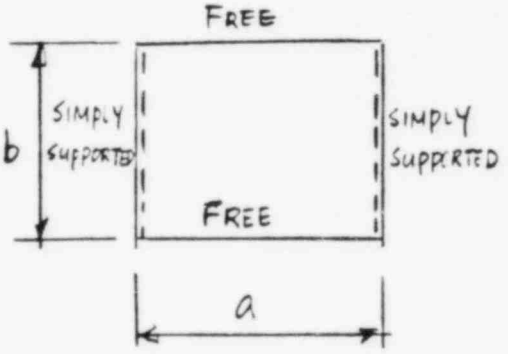
EES

PREPARED BY: C. T. LIN	DATE: 10-27-86
CHECKED BY: B. MONAVARIAN	DATE: 11-5/86
PROJECT: MILLSTONE #1, NUSCO	
SUBJECT: BLOCKOUT (ARCHING ACTION)	
SYSTEM: NATURAL FREQUENCY	
ANALYSIS NO.: B001 REV. NO.: 0	JOB NO.: 20014 FILE NO.: 12-F
	SHEET NO.: B001-46

REF

ALLOWABLE SPAN OF RIGID ARCHING WALL @ NATURAL FREQUENCY = 20 cps

Pg 45

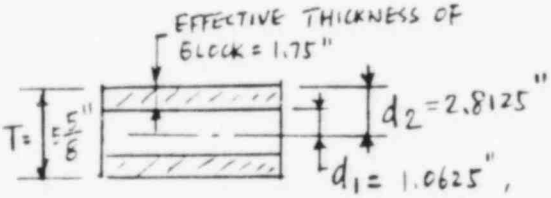


$$f_{n_1} = \frac{\lambda^2}{2\pi a^2}\left[\frac{2E(d_2^3 - d_1^3)}{3\gamma(1-\nu^2)}\right]^{\frac{1}{2}}$$

$$= \frac{\lambda^2}{a^2} \times \frac{[C_1 \times C_2]^{\frac{1}{2}}}{2\pi} = \frac{\lambda^2}{a^2} \times C_3$$

$$\therefore\ a = \lambda\left(\frac{C_3}{f_n}\right)^{\frac{1}{2}} \quad \text{.... MAX. ALLOWABLE SPAN}$$

$f_n \geq 20$ cps



LET $E = 810{,}000$ psi,

$\nu = 0.2$ (CONSERVATIVE IN STEAD OF USING 0.3)

λ^2 = SEE PAGE 254 OF REF 10

γ = MASS PER UNIT AREA = (WALL WT + ATTACHMENT WT) $\times \frac{1}{144 \times 386.1}$

Pg 49

NORMINAL BLOCK SIZE	RATIO $\frac{a}{b}$	λ^2	$d_2^3 - d_1^3 = C_2$	WALL WT. (PSF)	ATTACH. WT. (PSF)	γ	$\frac{2}{3}\frac{E}{\gamma(1-\nu^2)} = C_1$	$\frac{(C_1 \times C_2)^{\frac{1}{2}}}{2\pi} = C_3$	MAX. SPAN $a = \lambda\left(\frac{C_3}{20}\right)^{\frac{1}{2}}$	SPAN b
6×8×16	0.4	9.76	21.048	70	–	12.59×10^{-4}	4.468×10^{8}	$1{,}543 \times 10^{4}$	86.78" (7.23')	216.95" (12.08')
6×8×16	2/3	9.698	21.048	70	–	12.59×10^{-4}	4.468×10^{8}		86.51" (7.21')	129.76" (10.81')
6×8×16	1.0	9.631	21.048	70	–	12.59×10^{-4}	4.468×10^{8}		86.20" (7.18')	86.20" (7.18')
6×8×16	1.5	9.558	21.048	70	–	12.59×10^{-4}	4.468×10^{8}		85.87" (7.16')	57.24" (4.77')
6×8×16	2.5	9.484	21.048	70	–	12.59×10^{-4}	4.468×10^{8}		85.54" (7.13')	34.22" (2.85')

EES (1/86) - 101 PAD

CALCULATION SHEET

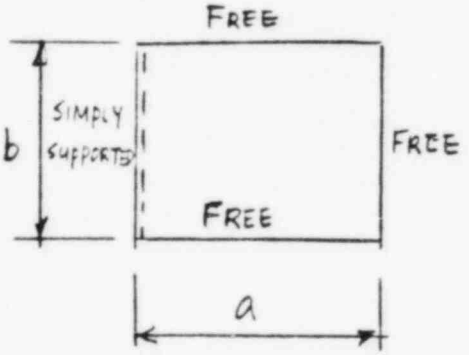
EES

PREPARED BY C. T. LIN		DATE 10-27-80	
CHECKED BY B. HONARARIAN		DATE 11/5/80	
PROJECT MILLSTONE #1, NUSCO		JOB NO. 80014	FILE NO. 12-F
SUBJECT BLOCKOUT (ARCHING ACTION)			
SYSTEM NATURAL FREQUENCY			
ANALYSIS NO. 0021	REV. NO. 0	SHEET NO. B001-47	

REF.

ALLOWABLE SPAN OF GAPPED ARCHING WALL @ NATURAL FREQUENCY = 20 cps

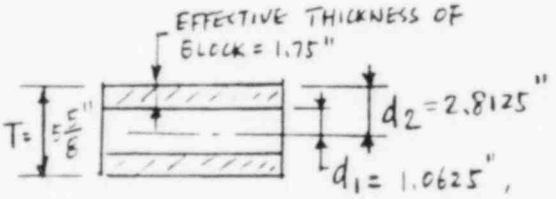
Pg 45



$$f_n = \frac{\lambda^2}{2\pi a^2}\left\{\frac{2E(d_2^3 - d_1^3)}{3\gamma(1-\nu^2)}\right\}^{\frac{1}{2}}$$

$$= \frac{\lambda^2}{a^2} \times \frac{[C_1 \times C_2]^{\frac{1}{2}}}{2\pi} = \frac{\lambda^2}{a^2} \times C_3$$

$$\therefore\ a = \lambda\left(\frac{C_3}{f_n}\right)^{\frac{1}{2}} \quad \text{..... MAX. ALLOWABLE SPAN}$$



LET E = 810,000 psi

ν = 0.2 (CONSERVATIVE IN STEAD OF USING 0.3)

λ^2 = SEE PAGE 254 OF REF

γ = MASS PER UNIT AREA

= (WALL WT + ATTACHMENT WT) $\times \frac{1}{144 \times 32.2}$

Pg 48

NORMINAL BLOCK SIZE	RATIO $\frac{a}{b}$	λ^2	$d_2^3 - d_1^3 = C_2$	WALL WT. (PSF)	ATTACH. WT. (PSF)	γ	$\frac{2}{3}\cdot\frac{E}{\gamma(1-\nu^2)} = C_1$	$\frac{(C_1 \times C_2)^{\frac{1}{2}}}{2\pi}$	MAX. SPAN $a = \lambda\left(\frac{C_3}{20}\right)^{\frac{1}{2}}$	SPAN b
6 x 8 x 16	0.4	2.692	21.042	70	-	12.59 x 10^{-4}	4.468 x 10^8	1.543 x 10^4	45.57" (3.79')	113.93" (9.49')
6 x 8 x 16	2/3	4.481	21.042	70	-	12.59 x 10^{-4}	4.468 x 10^8		58.79" (4.89')	88.19" (7.35')
6 x 8 x 16	1.0	6.642	21.042	70	-	12.59 x 10^{-4}	4.468 x 10^8		71.62" (5.97')	71.62" (5.97')
6 x 8 x 16	1.5	9.850	21.042	70	-	12.59 x 10^{-4}	4.468 x 10^8		87.17" (7.26')	58.11" (4.84')
6 x 8 x 16	2.5	14.94	21.042	70	-	12.59 x 10^{-4}	4.468 x 10^8	V	107.36" (8.94')	42.94" (3.58')

CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO
SUBJECT: BLOCKOUT, (ARCHING ACTION)
SYSTEM: FREQUENCY LIMITATION
ANALYSIS NO.: E001 REV. NO.: 0

PREPARED BY: C.T. LIN — DATE: 10-27-80
CHECKED BY: B. HONAVARIAN — DATE: 11/19/80
JOB NO.: 80014 — FILE NO.: 12-E
SHEET NO.: E001-48

EES

PLOT OF $\frac{a}{b}$
FOR 20 HERTZ @
GAPPED ARCHING ACTION

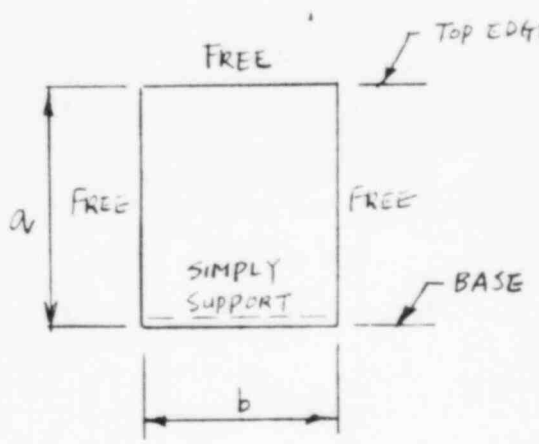


CHART 1

MAX. PLATE DIMENSIONS @ 20 HERTS FOR UNREINFORCED SOLID MASONRY BLOCKOUT WALL SUPPORTED ON THE BASE ONLY AND OTHER THREE EDGES FREE

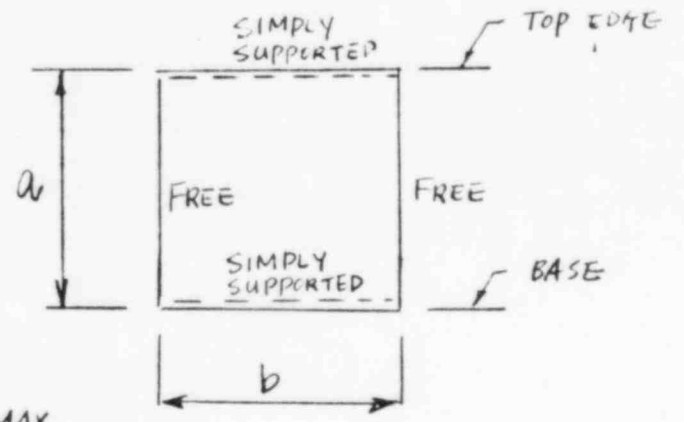
CALCULATION SHEET

PROJECT: MILLSTONE #1, NELSCO
SUBJECT: BLOCKOUT, (ARCHING ACTION)
SYSTEM: FREQUENCY LIMITATION
ANALYSIS NO.: 0001 REV. NO.: 0

PREPARED BY: C.T. LIN DATE: 10-27-80
CHECKED BY: B. MONAJARAN DATE: 11/5/80
JOB NO.: 80014 FILE NO.: 12-E
SHEET NO.: 8001-49

EES

PLOT OF $\frac{a}{b}$
FOR 20 HERTZ @
RIGID ARCHING ACTION



USE $a \approx 7.25'$ AS MAX. DESIGN SPAN

6" WALL (SINGLE WYTH $t = 5\frac{5}{8}$")

a (FT)

14.0'
12.0'
10.0'
8.0'
6.0'
4.0'
2.0'
0'

0' 2.0' 4.0' 6.0' 8.0' 10.0' 12.0' 14.0' 16.0'

b (FT)

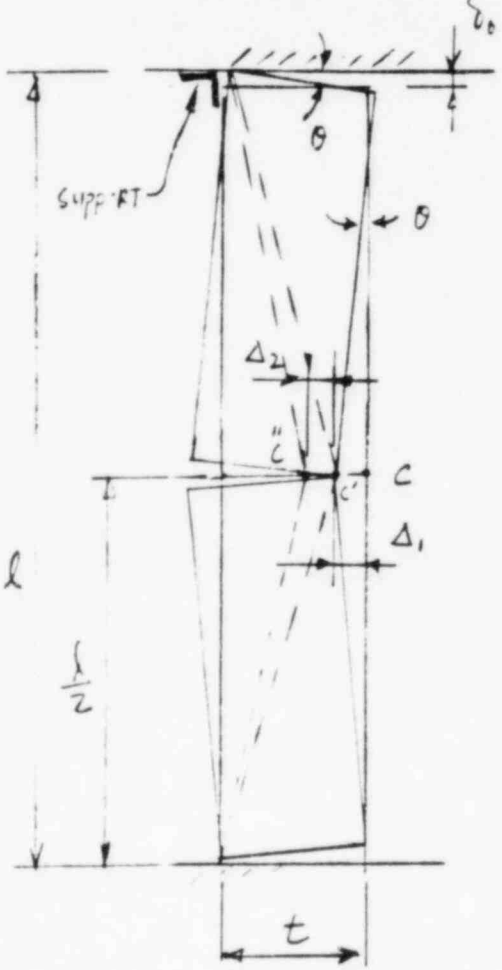
CHART 2

MAX. PLATE DIMENSIONS @ 20 HERTS FOR UNREINFORCED, SOLID MASONRY BLOCKOUT WALL SUPPORTED ON TOP EDGE AND THE BASE

EES	CALCULATION SHEET	PREPARED BY C. T. LIN	DATE 11-13-80
	PROJECT MILLSTONE #1, NUSCO	CHECKED BY [signature]	DATE 12-14-80
	SUBJECT —	JOB NO. 8 0014	FILE NO. 12-F
	SYSTEM —		
	ANALYSIS NO. B001 REV. NO. 2	SHEET NO. B001 - 50	

REF.

MAX. ALLOW. GAP SIZES FOR GAPPED ARCHING TO BE CONVERTED IN TO RIGID ARCHING BY PROVIDING SUPPORT AT TOP EDGE



$$\theta \doteq \frac{\Delta_1}{\frac{\ell}{2}} \qquad \theta \doteq \frac{\delta_0}{t}$$

LET $\dfrac{\Delta_1}{\frac{\ell}{2}} = \dfrac{\delta_0}{t}$

$$\Delta_1 = \frac{\ell \delta_0}{2t}$$ ... DEFLECTION DUE TO GAP EXISTENCE BEFORE ARCHING ACTION

$$\Delta_2 = t - \sqrt{t^2 - m}$$ ... DEFLECTION AFTER ARCHING ACTION

WHERE $m = 2\varepsilon s^2$ ..... (3)

$$s = \left[t^2 + \left(\frac{\ell}{2}\right)^2\right]^{\frac{1}{2}}, \quad \varepsilon = \frac{f_x}{E'}$$

$$\Delta = \Delta_1 + \Delta_2$$

$$\Delta = \frac{\ell \delta_0}{2t} + t - \sqrt{t^2 - m}$$

SET $\Delta = 0.15t$ FOR DEFLECTION LIMITATION

$$\therefore \delta_0 = \left(\sqrt{t^2 - m} - 0.85t\right) \times \frac{2t}{\ell} \quad \text{... (1)}$$

$$\varepsilon = \frac{f_\ell}{E'} = \frac{P_A}{E'} = \frac{P\ell}{Et}\sqrt{\ell^2 + (ut)^2} + v\ell \Big/ E' \quad \text{---- (2)}$$

LET $P = 0.72$ PSI, $t = 5.625''$, $E' = 29300000$ PSI

$v = 0.557$ PSI

EES

CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO		PREPARED BY: C. T. LIN	DATE: 11-13-80
SUBJECT: —		CHECKED BY: P. Guglielmino	DATE: 12-14-80
SYSTEM: —		JOB NO.: 80014	FILE NO.: 12-F
ANALYSIS NO.: B001	REV. NO.: 0	SHEET NO.: B001-51	

REF

$\ell = 72''$ $= 6'-0''$

$S^2 = (5.625^2 + 36^2) = 1327.6$

$$\varepsilon = \frac{0.72 \times 72}{8 \times 5.625}\sqrt{(72)^2 + (4 \times 5.625)^2} + 0.55) \times 72 \Big/ 293000$$

$= 0.000043$

$m = 2\varepsilon S^2 = 1.14$

$$\delta_0 = \left(\sqrt{(5.625)^2 - 1.14} - 0.25 \times 5.625\right) \times \frac{2 \times 5.625}{72}$$

$= 0.1158'' \doteq \frac{1}{8}''$

$\ell = 60''$ $= 5'-0''$

$S^2 = (5.625^2 + 30^2) = 931$

$$\varepsilon = \frac{0.72 \times 60}{8 \times 5.625}\sqrt{(60)^2 + (4 \times 5.625)^2} + 0.55 \times 60 \Big/ 293000 =$$

0.000324

$m = 2\varepsilon S^2 = 0.6$

$$\delta_0 = \left(\sqrt{(5.625)^2 - 0.6} - 0.25 \times 5.625\right) \times \frac{2 \times 5.625}{60}$$

$= 0.148'' \doteq \frac{1}{8}''$

$\ell = 84''$ $= 7'-0''$

$S^2 = (5.625^2 + 42^2) = 1776$

$$\varepsilon = \frac{0.72 \times 84}{8 \times 5.625}\sqrt{(84)^2 + (4 \times 5.625)^2} + 0.55 \times 84 \Big/ 293000$$

$= 0.000456$

$m = 2\varepsilon S^2 = 2$

$\delta_0 = \left(\sqrt{(5.625)^2 - 2}\right.$ $- 0.25 \times 5.625) \times \frac{2 \times 5.625}{72}$

$= 0.088 \doteq \frac{1}{16}''$

EES-NO.001 1st Rev.0

EES

CALCULATION SHEET

PROJECT: MILLSTONE #1, NUSCO		PREPARED BY: C. T. LIN	DATE: 11-13-90
SUBJECT: —		CHECKED BY: [signature]	DATE: 12-14-90
SYSTEM: —		JOB NO.: 20014	FILE NO.: 12-F
ANALYSIS NO.: B001	REV. NO.: 0	SHEET NO.: B001-52	

CONCLUSION: GAPPED ARCHING CAN BE CONVERTED INTO RIGID ARCHING IF SUPPORT IS PROVIDED AT TOP EDGE. BUT GAP SIZE SHALL NOT EXCEED $\frac{1}{16}$" FOR BLOCKOUT HEIGHT WITHIN 6'-0" AND 7'-0". AND SHALL NOT EXCEED $\frac{1}{8}$" FOR BLOCKOUT HEIGHT LESS THAN 6'-0"

ATTACHMENT 4

consists of a 13-page (Sheets D015 0-15 and D016 0-6) example of the analysis made to rectify wall deficiencies.

EES

CALCULATION COVER SHEET

Job. No. 80014
File No. 2F
Calc. Set No. D015
No. of Sheets 5

PROJECT MILLSTONE UNIT # 1

CLIENT NUSCO

SUBJECT

NRC IE BULLETIN 80-11 BLOCK OUT WALL RE-ANALYSIS

STATEMENT OF PROBLEM

DESIGN A MASONRY WALL (BLOCK OUT) BOUNDARY CONNECTION FOR SHEAR TRANSFER FROM A 3'-0" THICK BLOCK OUT WITH 5'-0" MAX SPAN TO AN EXISTING REINFORCED CONCRETE WALL HAVING A MAX HORIZONTAL ACCELERATION OF 0.452g and 0.5 PSI PRESSURE

SOURCES OF DATA

NES DESIGN CRITERIA (DC-1)(REV. 1)

SOURCES OF FORMULAE & REFERENCES

1. FORMULAS FOR STRESS AND STRAIN 4TH ED BY ROARK
2. AISC STEEL MANUAL 7TH ED.
3. Calc Set No H000 Section 9

REMARKS FOR SHEAR TRANSFER CONNECTION DETAILS SEE PG 5.
ALSO SEE PG 6. FOR USING ABOVE DETAIL FOR A 4'-6" THICK BLOCK OUT WITH 3'-3" MAX SPAN.

ORIGINATORS	CHECKERS	DISTRIBUTION	REVISION NO. 0
11-17-80	11-6-80		SUPERSEDES CALCULATION
			SET NO. NA
			APPROVED BY:
			DATE: 11-17-80

EES 08/80 - 100 Rev 0

CALCULATION SHEET

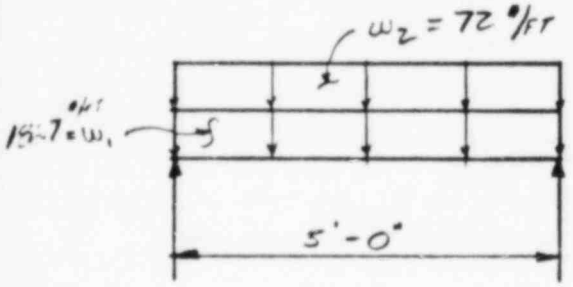
EES

PROJECT	MILLSTONE UNIT #1
SUBJECT	NRC - IE - BULLETIN 80-11
SYSTEM	SHEAR MOD FOR 3' THICK BLOCK WALLS
ANALYSIS NO.	REV. NO. 0

PREPARED BY	DIMITRIOS ANTYPAS — DATE 11/17/80
CHECKED BY	Vgn. Dsg. Rep. — DATE 11/18/80
JOB NO.	BCC14
FILE NO.	2F
SHEET NO.	D015 -1

GENERIC SHEAR MOD. FOR SEISMIC LOADS + 0.5 PSI PRESSURE. ON A Solid Masonry Wall - 5 ft max horiz SPAN

DESIGN CONDITIONS:



(Ref DC-1 Rev 1)

BLOCK OUT THICKNESS = 3'-0" SOLID BLOCK

" " WIDTH = 5'-0" MAX.

ASSUME 6-6" BLOCKS MAKE UP THE 3'-0" THK

ASSUME 5 PSF ATTACHMENT WT.

ASSUME MAX HORIZ. ACC = 0.452 g

DETERMINE MODIFICATION SUPPORT LOADS

TOTAL WALL WT + ATTACH. WT = 6 x 66 + 5 = 413 PSF

PRESSURE LOAD = $.5 \frac{\#}{IN^2} \times 144 \frac{IN^2}{FT^2}$ = 72 PSF

SEISMIC LOAD/FT = 0.452 x 413 = 186.7 #/FT

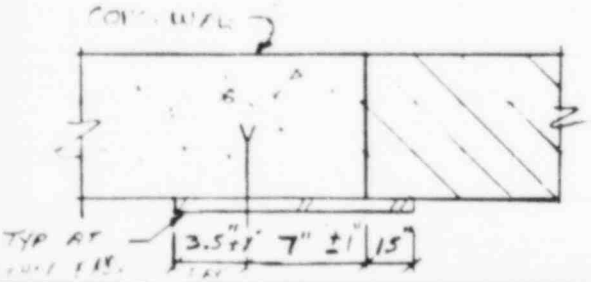
SEISMIC LOAD + PRESSURE: w = 186.7 + 72 = 258.7 #/FT

WALL REACTION/FT = $\frac{wl}{2} = \frac{258.7 \times 5}{2}$ = 646.75 #/FT

SAY WALL REACTION = 647 #/FT

DESIGN OF SHEAR MODIFICATION

TRY 5/8" x 12" x CONT R WITH 1" ϕ HILTI BOLTS @ 12" CTRS



CALCULATION SHEET

EES

PROJECT	MILLSTONE UNIT # 2
SUBJECT	NRC IE BULLETIN 80-11
SYSTEM	SHEAR MOD. FOR 3" THICK BULKHEADS
ANALYSIS NO.	REV. NO. 0

PREPARED BY	DIMITRIOS ANTONOPOULOS
DATE	11/17/82
CHECKED BY	[illegible]
DATE	[illegible]
JOB NO.	8C014
FILE NO.	2F
SHEET NO.	D015 -2

GENERIC SHEAR MOD. COND.

CHECK BEARING STRESS BETWEEN 5/8" ℄ & SOLID BULK OUT.

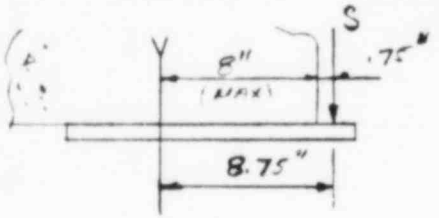
MAX REACTION = 647 #/FT = 53.92 #/IN (FOR DC-1 REV 1 CLASS PLATE)

$f_p = 53.92 \frac{\#}{IN} \times \frac{1}{1.5\,IN} = 36.0\ PSI < 540\ PSI$ ∴ O.K.

CONCLUSION: BEARING STRESS BETWEEN ℄ & BULK IS ADEQUATE.

CHECK 5/8" PLATE THICKNESS

ASSUME 1" TOLERANCE FOR BOLT SPACING.



MAX MOM = 647 #/FT × 8.75" = 5661.25 IN-#/FT

ASSUME 10" ℄ WIDTH TO BE EFFECTIVE IN BEARING.

$S = \frac{bd^2}{6} = \frac{10 \times .625^2}{6} = 0.651\ IN^3$

BENDING STRESS $f_b = 5661.25\ IN\text{-}\# / .651\ IN^3 = 8696\ PSI < 27000\ PSI$ ∴ O.K.

CONCLUSION: 5/8" ℄ THICKNESS IS ADEQUATE

CHECK 1" ϕ HILTI BOLTS @ 12" O.C SPACING

NOTE: IN ORDER TO ACCOUNT FOR PRYING ACTION DUE TO THE PLATE BEING FLEXIBLE ASSUME THE RESISTING MOMENT ARM WILL BE 2t (2 × PLATE THICKNESS).

∴ BOLT TENSION = $S + \frac{S \cdot l}{2t} = 647 + \frac{647 \times 8.75}{2 \times .625} = 5176\ \#/FT$ (FROM DC-1 REV. 1)

ALLOWABLE TENSION FOR 1" ϕ HILTI WITH 6" EMBED. = 5281# > 5176# ∴ O.K.

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT #1
SUBJECT	USI IE BULLETIN 80-11
SYSTEM	SHEAR AUX BLDG FOR 3" THKL CONC SLAB
ANALYSIS NO.	REV. NO. 0

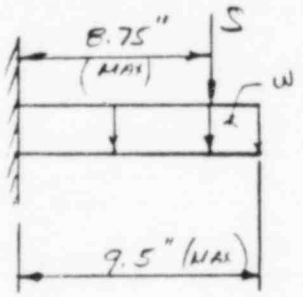
PREPARED BY	DIMITRIOS ANTHOPOULOS
DATE	11/17/80
CHECKED BY	[illegible]
DATE	[illegible]
JOB NO.	80014
FILE NO.	27
SHEET NO.	0015 -3

GENERIC SHEAR AUX. COND.

CHECK OF 1"ϕ HILTI BOLTS COND.

CONCLUSION: 1"ϕ HILTI BOLTS @ 12" O.C SPACING WITH 6" MIN. EMBED. IS ADEQUATE

CHECK SHEAR AUX. FREQUENCY



$S = 413 \times \frac{5}{2} = 10325^{\#}/FT$

$w = 490^{\#}/FT^3 \times 1\,FT \times 9.5\,IN \times .625\,IN \times \frac{1}{144\,IN^2}\,FT^2 = 20.2^{\#}/FT$

$f_n = \frac{3.13}{\sqrt{\frac{(W + 1.236\,w\,l)\,l^3}{3EI}}}$ (REF. 1.0)

WHERE: $W = 10325^{\#}/FT$

$l = 8.75\,IN$

$w = \frac{9.5}{8.75} \times 20.2 = 21.93^{\#}/FT$

$I = \frac{bd^3}{12} = \frac{12 \times .625^3}{12} = 0.244\ IN^3$

$\therefore f_n = \frac{3.13}{\sqrt{\frac{(10325 + 1.236 \times 21.93 \times 8.75)\,8.75^3}{3 \times 29000000 \times .244}}} = 16.97\ CPS < 20\ CPS$ N.G.

TRY 3/4" THKL SLAB

$w = 21.93 \times \frac{.75}{.625} = 26.32^{\#}/FT$

$I = \frac{12 \times .75^3}{12} = 0.422\ IN^4$

$\therefore f_n = \frac{3.13}{\sqrt{\frac{(10325 + 1.236 \times 26.32 \times 8.75)\,8.75^3}{3 \times 29000000 \times .422}}} = 22.23\ CPS > 20\ CPS$ ∴ O.K.

EES 08/80 - 10/1

EES

CALCULATION SHEET

PROJECT	MILLSTONE UNIT # 2
SUBJECT	NRC IE BULLETIN 80-11
SYSTEM	SHEAR MOD. FOR 3' THICK BUCK OUT
ANALYSIS NO.	REV. NO. 0

PREPARED BY	DATE
DWITTERS ASSOCIATES	11/17/80
CHECKED BY	DATE
JOB NO.	FILE NO.
80014	2F
SHEET NO.	D015 -4

GENTIL SHEAR MOD. COND.

SHEAR MOD FREQUENCY COND.

CONCLUSION IN ORDER TO SATISFY FREQUENCY REQUIREMENTS

3/8" X 12" X CONT. PL IS NEEDED.

NEW BOLT TENSION CALCULATION

$$T = 647 + \frac{647 \times 8.75}{2 \times .75} = 4422^{\#}/\text{BOLT}$$

ACTUAL MIN. BOLT SPACING = 11"

∴ BOLT ALL TENSION = $\frac{11}{12} \times 5281 = 4841.0^{\#} > 4422^{\#}$ O.K.

CONCLUSION: USE 3/4" X 12" X CONT. PL WITH

1" ϕ HILTI BOLTS MIN. EMBED = 6 IN @ 12" O.C.

NOTE STRESSES ON 3/4" PL WILL BE LOWER THAN THE VALUES CALCULATED FOR 5/8" THICK PLATE.

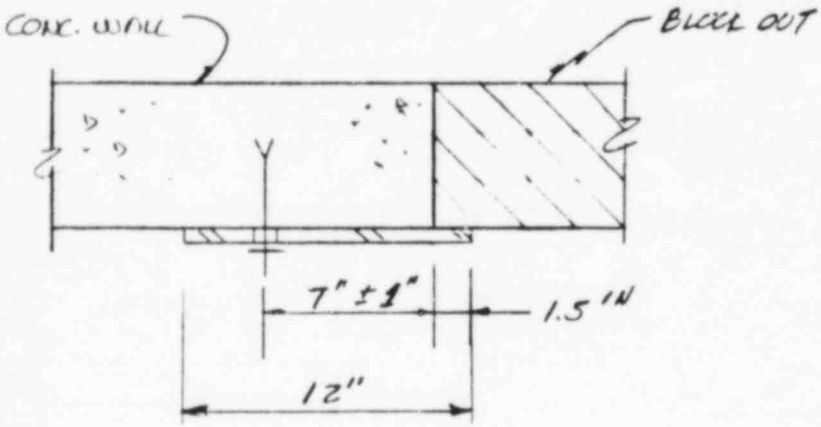
EES

CALCULATION SHEET

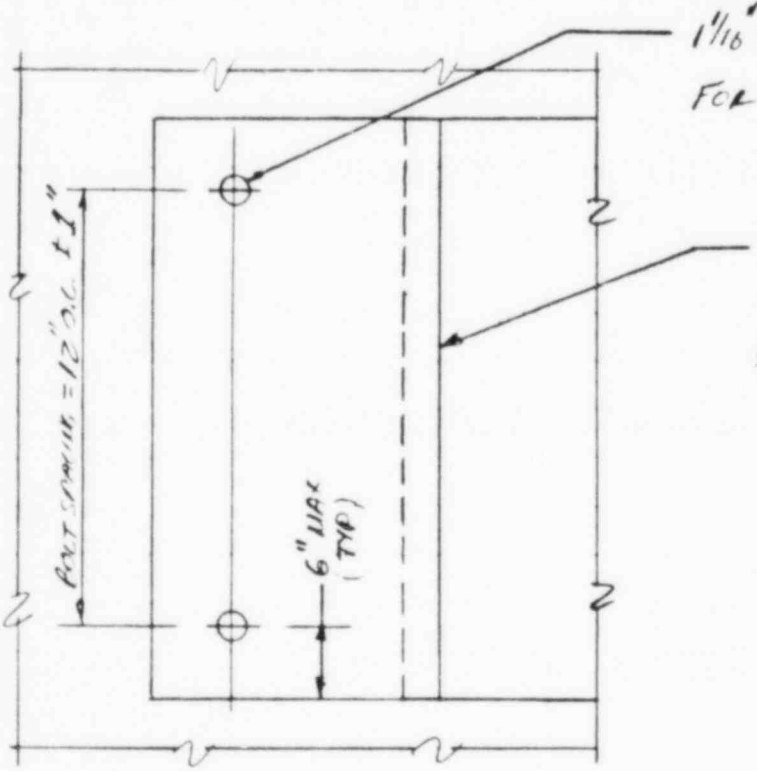
PROJECT	MILLSTONE UNIT #2
SUBJECT	NRC IE BULLETIN 80-11
SYSTEM	SHEAR MOD FOR 3' THRU BUCKOUTS
ANALYSIS NO.	REV. NO. 0

PREPARED BY	DATE 11/17/80
CHECKED BY	DATE 11/18/80
JOB NO. 80014	FILE NO. 2F
SHEET NO.	D015-5

GENERIC SHEAR MOD. DETAILS



SHEAR MOD. PLAN (TYP ON OTHER SIDE)



SHEAR MOD ELEV.

EES

CALCULATION COVER SHEET

Job. No. 80014
File No. 2-F
Calc. Set No. 0016
No. of Sheets 6

PROJECT MILLSTONE UNIT #1

CLIENT NUSCO

SUBJECT

N.R.C IE BULLETIN (80-11) BLOCK OUT WALL RE-ANALYSIS

STATEMENT OF PROBLEM

DESIGN A MASONRY WALL (BLOCK OUT) BOUNDARY CONNECTION FOR SHEAR TRANSFER FROM: 1.0) A 3' THK. BLOCKOUT WITH 4'-6" MAX SPAN TO AN EXISTING REINF. CONC. WALL HAVING A MAX HORIZ ACC. OF 0.346g AND 0.5 PSI PRESSURE. 2.0) A 4'-8" THK BLOCK OUT WITH 3'-6" MAX SPAN TO AN EXISTING REINF. CONC. WALL HAVING A MAX HORIZ ACC. OF 0.346g AND 0.5 PSI PRESSURE.

SOURCES OF DATA

EES DESIGN CRITERIA (DC-1) (Rev.1)

SOURCES OF FORMULAE & REFERENCES

1. FORMULAS FOR STRESS AND STRAIN 4th Ed BY ROARK
2. AISC STEEL MANUAL 7th Ed.
3. CALC SET NO MOOO SECTION 9

REMARKS

ORIGINATORS	CHECKERS	DISTRIBUTION	REVISION NO. 0
11/20/80	11-21-80		SUPERSEDES CALCULATION SET NO. N.A
			APPROVED BY:
			DATE: 11-26-80

EES 08/80 - 100 Rev 0

CALCULATION SHEET

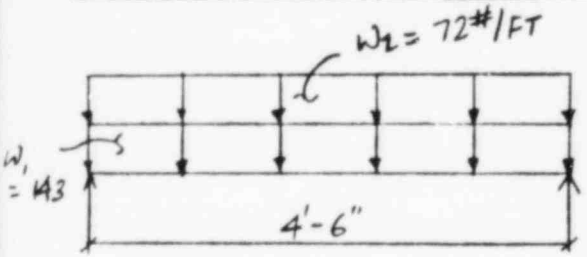
EES

PROJECT	MILLSTONE UNIT #1
SUBJECT	N.R.C IEBULLETIN (80-11)
SYSTEM	SHEAR MOD FOR 3" & 4'-8" THK BLOCK OUTS
ANALYSIS NO.	N/A
REV. NO.	0

PREPARED BY	Ugender Reddy	DATE 11/19/80
CHECKED BY	R Rickard	DATE 11/21/80
JOB NO.	80014	FILE NO. 2-F
SHEET NO.	D016 -1	

DESIGN OF SHEAR M.O. FOR 3'-0" THK BLOCK OUT

GENERIC SHEAR MOD. FOR SEISMIC LOADS + 0.5 PSI PRESSURE.



ON A SOLID MASONRY WALL 4-6" HORIZ SPAN

DESIGN CONDITIONS.

BLOCK OUT THICKNESS = 3'-0"
HORIZONTAL SPAN = 4'-6"

ASSUME 6 - 6" BLOCKS = 3'-0" THK.

MAX HORIZ. ACC. = .346 g

(REF. DC-1 Rev.1)

DETERMINE MODIFICATION SUPPORT LOADS

TOTAL WALL WT + ATTACH. WT = 6 × 68 + 5 = 413 PSF

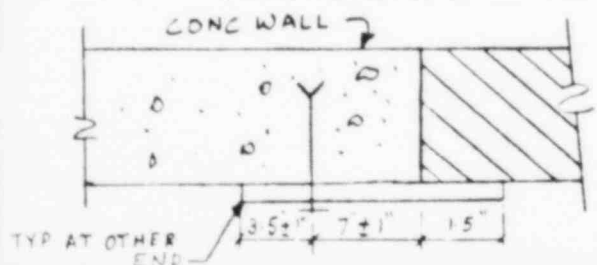
PRESSURE LOAD = $.5 \frac{\#}{m^2} \times 144 \frac{m^2}{FT^2}$ = 72 PSF.

SEISMIC LOAD/FT = .346 × 413 = 143 #/FT.

SEISMIC LOAD + PRESSURE = 143 + 72 = 215 #/FT.

WALL REACTION/FT = $\frac{wl}{2} = 215 \times \frac{4.5}{2}$ = 484 #/FT.

DESIGN OF SHEAR MODIFICATION



TRY 5/8" × 12" × CONT PL WITH 3/4" Ø HILTI BOLTS @ 12" SPACING O.C

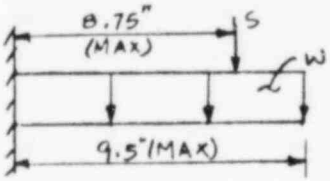
EES

CALCULATION SHEET

PREPARED BY: ugender Reddy	DATE: 11/19/80
CHECKED BY: RJ Richard	DATE: 11/21/80
PROJECT: MILLSTONE UNIT #1	JOB NO.: 80014
SUBJECT: N.R.C IE BULLETIN (80-11)	FILE NO.: 2-F
SYSTEM: SHEAR MOD FOR 3" & 4'-8" THK BLOCK OUTS.	
ANALYSIS NO.: N/A REV. NO.: 0	SHEET NO.: D016-2

CHECK SHEAR AND FREQUENCY FOR 3'-0" THK BLOCK OUT MAX SPAN = 4'-6"

TRY 5/8" X 12" PL



$$S = 413 \times \frac{4.5}{2} = 929\ \#/FT.$$

$$W = 490\ \#/FT^3 \times 1FT \times 9.5IN \times .625IN \times \frac{1}{144}\ FT^2/in^2$$

$$= 20.2\ \#/FT.$$

$$f_n = \frac{3.13}{\sqrt{\frac{(W + .236WL)L^3}{3EI}}}$$

$W = 929\ \#/FT.$

$L = 8.75\ IN$

$W = \frac{9.5}{8.75} \times 20.2 = 21.93\ \#/FT$ $\quad I = \frac{bd^3}{12} = \frac{12 \times .625^3}{12} = 0.244\ in^4$

$$f_n = \frac{3.13}{\sqrt{\frac{(929 + .236 \times 21.93 \times 8.75)\,8.75^3}{3 \times 29000000 \times .244}}} = 17.85\ CPS < 20\ CPS$$

(N.G.)

TRY 3/4" THICK PL

$W = 21.93 \times \frac{.75}{.625} = 26.32\ \#/FT$ $\quad I = \frac{12 \times .75^3}{12} = 0.422\ in^4$

$$\therefore f_n = \frac{3.13}{\sqrt{\frac{(929 + .236 \times 26.32 \times 8.75)\,8.75^3}{3 \times 29000000 \times .422}}} = 23.36\ CPS > 20\ CPS$$

(O.K.)

CHECK SHEAR AND FREQUENCY FOR 4'-8" THK BLOCK OUT MAX SPAN = 3'-6"

ASSUMING 7 Wythes of 8" BLOCK

TOTAL WALL WT + ATTACH WT = $7 \times 92 + 7 = 651\ PSF$

CALCULATION SHEET

EES

PREPARED BY: Ugender Reddy	DATE: 11/20/80
CHECKED BY: R.K. [illegible]	DATE: [illegible]
PROJECT: MILLSTONE UNIT #1	
SUBJECT: N.R.C I E BULLETIN (80-11)	
SYSTEM: SHEAR MOD FOR 3'&4'-8" THK BLOCK OUTS	JOB NO. 80014 / FILE NO. 2-F
ANALYSIS NO. 1/A REV. NO. 0	SHEET NO. D016-3

$$S = 651 \times \frac{3.5}{2} = 1139\ \#/FT$$

$W = 1139\ \#/FT.$
$\ell = 8.75\ IN$
$w = \frac{9.5}{8.75} \times 20.2 = 21.93\ \#/FT$

TRY 5/8" X 12" PL

$$I = \frac{bd^3}{12} = \frac{12 \times .625^3}{12} = 0.244$$

$$f_n = \frac{3.13}{\sqrt{\frac{(1139 + .236 \times 21.93 \times 8.75)\,8.75^3}{3 \times 29000000 \times 0.244}}} = 16.19\ CPS < 20 CPS$$

(N.G.)

TRY 3/4" PL

$$W = 21.93 \times \frac{.75}{.625} = 26.32\ \#/FT \qquad I = \frac{12 \times .75^3}{12} = 0.422\ in^4$$

$$\therefore f_n = \frac{3.13}{\sqrt{\frac{(1139 + .236 \times 26.32 \times 8.75)\,8.75^3}{3 \times 29000000 \times .422}}} = 21.21\ CPS > 20$$

(O.K.)

SHEAR MOD FREQ COND.

CONCLUSION In order to Satisfy Frequency Requirements

3/4" X 12" PL IS NEEDED.

CHECK BEARING STRESS BETWEEN 3/4" PL & SOLID BLOCK OUT.

MAX REACTION = 484 FOR (3' THK 4'-6" SPAN)

FOR (4'-6" THK 3'-6" SPAN) @ $\frac{34.5'}{\text{arc}}$ factor = .346

CALCULATION SHEET

EES

PREPARED BY: Ugender Reddy	DATE: 11/20/80
CHECKED BY: R.J. Richard	DATE: 11/21/80
PROJECT: MILLSTONE UNIT #1	JOB NO.: 80014 / FILE NO.: 2-F
SUBJECT: N.R.C IE BULLETIN (80-11)	
SYSTEM: SHEAR MOD FOR 3'4 4'-8" THK BLOCK OUTS	
ANALYSIS NO.: N/A / REV. NO.: 0	SHEET NO.: D016 - 4

TOTAL WALL WT + ATTACH WT $= 7 \times 92 + 7 = 651$ PSF

PRESSURE LOAD $= .5\ \#/in^2 \times 144\ \frac{in^2}{FT^2} = 72$ PSF

SEISMIC LOAD/FT $= 0.346 \times 651 = 225\ \#/FT.$

SEISMIC LOAD + PRESSURE $= 225 + 72 = 297\ \#/FT.$

WALL REACTION /FT $= \frac{wl}{2} = 297 \times \frac{3.5}{2} = 520\ \#/FT.$

SO CHECK BEARING FOR 520 #/FT (WHICH is 7480)

Max REACTION $= \frac{520}{12} = 43.33\ \#/in$

$f_p = 43.33 \frac{\#}{in} \times \frac{1}{1.5\ in} = 28.8$ PSI $<<$ 540 PSI $\therefore$ O.K.

CONCLUSION BEARING STRESS Between PL & BLOCK IS ADEQUATE.

CHECK 3/4" ϕ HILTI BOLTS @ 12" O.C SPACING.

NOTE In order to ACCOUNT FOR Prying Action Due to the plate BEING FLEXIBLE ASSUME THE RESISTING MOMENT ARM WILL BE 2t. (2 x plate thickness)

$\therefore$ BOLT TENSION $= S + \frac{S.l}{2t} = 520 + \frac{520 \times 6.75}{2 \times .75} = 3553\ \#$ /Bolt

ALLOWABLE TENSION FOR 3/4" ϕ HILTI WITH 6" EMBD = 3975# > 3553# FROM DC-1 Rev. 1 $\therefore$ OK.

CALCULATION SHEET

EES

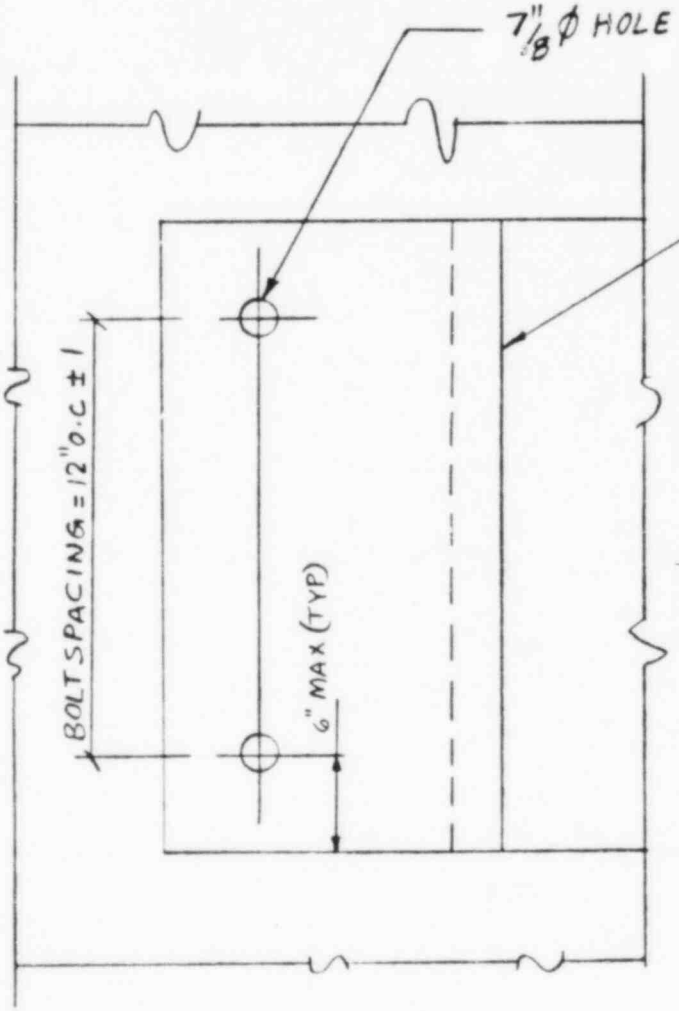
PROJECT	MILLSTONE UNIT #1
SUBJECT	N.R.C IE BULLETIN (80-11)
SYSTEM	SHEAR MOD FOR 3'-0" & 4'-8" THK BLOCK OUTS
ANALYSIS NO.	N/A
REV. NO.	0

PREPARED BY	Nginder Reddy
DATE	11/20/80
CHECKED BY	RJ Rudani
DATE	11/21/80
JOB NO.	80014
FILE NO.	2-F
SHEET NO.	D016-5

ALLOWABLE 3/4" ϕ BOLT TENSION DUE TO ± 1 TOLERANCE

$= \frac{11}{12} \times 3975 = 3644^{\#} > 3533^{\#}$:. (O.K.)

CONCLUSION : 3/4" ϕ BOLTS @ 12" O.C (± 1) SPACING WITH 6" EMBD IS ADEQUATE.



NOTE : PL MAY BE INTERUPTED TO AVOID INTERFERENCE.

SHEAR MOD ELEV.

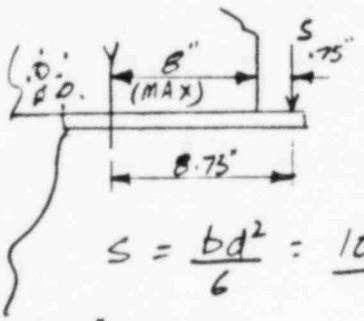
EES

CALCULATION SHEET

PREPARED BY: Ugender Reddy	DATE: 11/20/80
CHECKED BY:	DATE: 11/21/80
PROJECT: MILLSTONE UNIT #1	
SUBJECT: NRC IE BULLETIN (80-11)	JOB NO.: 80014 / FILE NO.: 2-F
SYSTEM: SHEAR MOD FOR 3'6 4'-8" THK BLOCKOUTS	
ANALYSIS NO.: N/A / REV. NO.: 0	SHEET NO.: D016-6

CHECK 3/4" PL THICKNESS

ASSUME 1" TOLERANCE FOR BOLT SPACING.



MAX MOM = $484^{\#}/_{FT} \times 8.75" = 4235\ ^{\#}/FT$

ASSUME 10" PL WIDTH TO BE EFFECTIVE IN BENDING.

$S = \frac{bd^2}{6} = \frac{10 \times .75^2}{6} = 0.9375\ in^3$

Bending stress $= \frac{4235}{0.9375} = 4517$ PSI < 27000 PSI (O.K.)

CONCLUSION 3/4" PL IS ADEQUATE.

ATTACHMENT 5

W. G. Counsil letter to B. B. Grier, NRC. Subject: Millstone Power Station, Unit 1-1E, Bulletin 80-11, Masonry Wall Design, Northeast Utilities, 04-Nov-80, A01021. Section 3, Page 2.

2.6 Final Safety Analysis Report, Millstone Nuclear Power Station, Unit 1, Section XII (Project Document No. M1-80014-DI-01)

2.7 Letter from D. Robinson of Northeast Utilities to C. T. Pitts of Earthquake Engineering Systems, Inc., dated September 11, 1980. (Project Document No. M1-80014-DI-07) | 1

2.8 Letter from T. L. Gould of EBASCO Services Incoror-ated to L. A. Chatfield of Northeast Utilities dated October 30, 1979.
(Project Document No. M1-80014-DI-03)

2.9 Reinforced Masonry Design, Robert R. Schneider and Walter L. Dickey, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1980. | 1

3.0 ASSUMPTIONS

3.1 The cutoff frequency for rigid range seismic acceleration values for buildings other than the reactor containment is 20 CPS.

3.2 Seismic displacement profiles and differential floor displacement values (story displacements) for the reactor building envelope those of other buildings.

3.3 There are no pipe whip, jet impingement, missile, flood, or pressurization loads acting on the masonry walls covered by this design criteria unless specifically mentioned herein. | 1

3.4 All components supported on or near masonry walls are rigid for the purposes of this evaluation, and therefore do not impose amplified loads or impact loads on the wall due to seismic displacements.

3.5 Unreinforced masonry walls do not carry building structure dead loads.

3.6 Masonry walls are not part of the structural load resisting system and, therefore, do not carry seismic shears or vertical seismic loads due to building inertia forces.

3.7 Masonry walls were constructed in accordance with masonry specifications and original drawings.

3.8 Surface mounted attachments which project no further from the wall surface than the wall thickness contribute only in plane loads to the wall.

Attachment 6 Final Report on Safety Related Masonry Walls at the Millstone Nuclear Power Station Unit 1.

This report summarizes the results of the reevaluation of safety-related masonry walls for the Millstone Nuclear Power Station Unit 1 and serves as a final repsonse to the requirements of the NRC I&E Bulletin 80-11.

The bulletin required that all licensees for nuclear power plants evaluate the design adequacy of all masonry walls which are in proximity to or have attachments from safety-related piping or equipment such that the wall failure could affect a safety-related system.

A walkdown of all masonry walls at Unit 1 was performed by CYGNA in accordance with the procedures established in the sixty day report submitted to the NRC by Northeast Utilities (letter from W. G. Counsil dated July 7, 1980).

The walls identified as safety related were analyzed and compared to the acceptance criteria as established in the 180 day report. All of the required design modifications have been issued to NUSCO.

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9.0	Exhibits:	10

A Rev. 3
B Rev. 3
C Rev. 3
D Rev. 3
E Rev. 3
F Rev. 3
G Rev. 3
H Rev. 3
I Rev. 3

Note: In August 1981, EES changed its name to Cygna Energy Services.

1.0 GENERAL

This Design Criteria provides the technical basis for the re-evaluation of masonry walls and the design of modifications at the Millstone Nuclear Power Station, Unit 1, required to satisfy the requirements of NRC I&E Bulletin 80-11 (Ref. 2.1).

2.0 REFERENCES

2.1 U.S. Nuclear Regulatory Commission, Office of Inspection and Enforcement, I&E Bulletin No. 80-11, dated May 8, 1980.

2.2 Report on the Earthquake Analysis of the Reactor Building for the Millstone Point Nuclear Power Station, prepared by John A. Blume & Associates, Engineers (Project Document No. M1-80014-DI-02)

2.3 Specification for the Design, Fabrication & Erection of Structural Steel for Buildings, American Institute of Steel Construction, New York, New York, dated February 12, 1969. (Including supplements 1, 2, & 3)

2.4 Masonry, EBASCO Services Incorporated Specification for Millstone Nuclear Power Station, issued Sept. 21, 1967, revised October 5, 1967. (Project Document No. M1-80014-DI-06)

2.5 American Society for Testing and Materials, Philadelphia, PA. Specifications:

C90-66T	Hollow Load-Bearing Concrete Masonry Units
C129-64T	Hollow Non-Load Bearing Concrete Masonry Units
C145-66T	Solid Load-Bearing Concrete Masonry Units
C270-64T	Mortar for Unit Masonry
A305-65	Minimum Requirements for the Deformations of Deformed Steel Bars
A15-66	Billet Steel Bars for Concrete Reinforcement
A36-67 -77A	Structural Steel

2.6 Final Safety Analysis Report, Millstone Nuclear Power Station, Unit 1, Section XII (Project Document No. M1-80014-DI-01)

2.7 Letter from D. Robinson of Northeast Utilities to C. T. Pitts of Earthquake Engineering Systems, Inc., dated September 11, 1980. (Project Document No. M1-80014-DI-07)

2.8 Letter from T. L. Gould of EBASCO Services incorporated to L. A. Chatfield of Northeast Utilities dated October 30, 1979.
(Project Document No. M1-80014-DI-03)

2.9 Reinforced Masonry Design, Robert R. Schneider and Walter L. Dickey, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1980.

2.10 Letter from J. R. Whitfield of NUSCO to J. G. Dyckman of EES dated Oct. 14, 1980. (Project Document No. M1-80014-DI-08)

2.11 "Response of Arching Walls and Debris from Interior Walls caused by Blast Loading", B. Gabrielsen, C. Wilton, and K. Kaplan, URS Report 7030-23, February 1975.

2.12 "Shock Tunnel Tests of Arched Wall Panels", B. Gabrielsen and C. Wilton, URS Report 7030-19, December 1974.

2.13 "Generic Criteria For Concrete Masonry Wall Evaluation," by EES dated Oct. 24, 1980.

2.14 Final Safety Analysis Report, Millstone Nuclear Power Station, Unit 1,Amendment 15, Section XII (Project Document No. 82028 DI-4) | 4

3.0 ASSUMPTIONS

3.1 The cutoff frequency for rigid range seismic acceleration values for buildings other than the reactor containment is 20 CPS. The Reactor Building cutoff frequency is 20 CPS per Ref. 2.2.

3.2 Seismic displacement profiles and differential floor displacement values (story displacements) for the reactor building envelope those of other buildings.

3.3 All components supported on or near masonry walls are rigid for the purpose of this evaluation, and therefore, do not impose amplified loads or impact loads on the wall due to seismic displacements.

3.4 Unreinforced masonry walls do not carry building structure dead loads.

3.5 Masonry walls are not part of the structural load resisting system and, therefore, do not carry significant seismic shears or vertical seismic loads due to building inertia forces. However, the effect of imposed displacements due to story drift will be evaluated.

3.6 Masonry walls were generally constructed in accordance with masonry specifications and original drawings.

3.7 Surface mounted attachments which project no further from the wall surface than the wall thickness contribute only in plane loads to the wall.

3.8 The compressive strength of concrete in existing structures is at least 3,000 PSI.

3.9 Out of plane story displacements on unreinforced masonry walls will not affect the flexural strength of the wall.

4.0 NATURAL FREQUENCY REQUIREMENT

Seismic analysis of safety related buildings at the time the station was designed did not include the development of amplified response spectra (ARS), Building floor accelerations were generated, however, (see Exhibit F) and these may be used to compute the seismic inertia forces for rigid systems supported by those floors.

All masonry walls covered by this design criteria shall be analyzed for natural frequency. Results which are greater than 20 CPS satisfy Assumption No. 1 and the appropriate building floor acceleration values shall be used to calculate seismic loadings. Results which are less than or equal to 20 CPS shall be reviewed by the Project Engineer on a case by case basis. This review shall determine whether or not a "stress amplification" approach may be appropriate. For example, if actual stresses computed with building acceleration values were smaller than the allowable stresses by a significant factor, the judgment might be made that the peak response of a flexible system could not exceed the floor input by that factor. In such cases, the masonry wall could be qualified for seismic loading independent from its natural frequency.

The stress amplification approach shall not be considered for unreinforced walls.

5.0 ANALYSIS AND DESIGN

5.1 Unreinforced Walls

5.1.1 Natural frequencies of, and stresses in, unreinforced walls shall be calculated using linear, elastic analysis assuming an uncracked section. Finite element analysis may be employed in special cases. Face shell areas shall be used in computing the section properties.

5.1.2 Support conditions for unreinforced walls shall be considered as hinged in the analysis. Necessary modifications shall be made to the supports to ensure that adequate shear transfer mechanisms exist. All unreinforced walls shall be considered free across the top edge unless otherwise indicated on the original design drawings. Control joints shall be considered as free edges. Further guidance on support condition classification is included in Exhibit E.

5.1.3 Unreinforced walls shall be analyzed as beams spanning horizontally between existing supports wherever existing support conditions permit, or between new intermediate supports where required. Maximum span lengths satisfying both a natural frequency of 20 CPS and stress requirements for various hollow and solid block thicknesses are tabulated in Exhibit A.

5.1.4 Unreinforced walls having only one vertical edge supported shall be stress analyzed as a plate on a case by case basis provided the natural frequency requirement is met. Maximum plate dimensions satisfying a natural frequency of 20 CPS is contained in Exhibit B.

5.1.5 The effect of in-plane loading on unreinforced walls shall be evaluated in accordance with the allowable displacements specified in Note (7) of Exhibit C-1. Imposed displacements due to story drift shall be evaluated using the displacement profile in Exhibit G. The effect of inertial reactions from adjoining walls shall be considered in the evaluation.

5.1.6 The effects of boundary structure flexibility, wall group interaction, and wall openings shall be evaluated on a case by case basis.

5.1.7 Masonry block pullout due to concentrated loadings imposed by attached components shall be evaluated using the allowable loads tabulated in Exhibit I.

5.1.8 Arching action may be used as the basis for qualification of blockouts in accordance with the guidelines given in Refs. 2.11 & 2.12. Blockouts are masonry walls which are constructed within, and completely fill a formed opening in a reinforced concrete wall. If arching action is used, the walls must be bounded top and bottom by rigid supports. Existing gaps at the upper boundary shall be taken into account in the analysis. Gaps must be no larger than 1/8". The maximum horizontal deflection due to arching action shall not exceed 15% of the wall thickness.

5.2 Reinforced Walls

5.2.1 Stresses in reinforced walls shall be calculated using the working stress method of analysis as described in Chapter 6 of Ref. 2.9.

5.2.2 Support conditions for reinforced walls shall be as shown in Exhibit E unless reinforcing bars pass through the support interface and are anchored in the adjacent structure. In this case the support condition shall be clamped for doubly reinforced sections and hinged for singly reinforced sections.

5.2.3 Reinforced walls shall be analyzed considering one-way or two-way behavior, whichever is appropriate for the boundary conditions, wall dimensions, and reinforcement configuration. Finite element methods may be employed in the analysis.

5.2.4 Reinforced walls shall first be analyzed using an uncracked, transformed moment of inertia (I_t). If the applied moment (M_a) resulting from this analysis exceeds the uncracked moment capacity (M_{cr}) an effective moment of inertia (I_e) shall be calculated as shown below.

$$I_e = \left[\frac{M_{cr}}{M_a}\right]^3 I_t + \left\{1 - \left[\frac{M_{cr}}{M_a}\right]^3\right\} I_{cr}$$

$$M_{cr} = F_t \left[\frac{I_t}{y}\right]$$

M_{cr} = Uncracked moment capacity

M_a = Applied maximum moment on the wall

I_t = Moment of inertia of uncracked, transformed section

I_{cr} = Moment of inertia of the cracked section

F_t = Allowable masonry tensile stress (M') for extreme environmental loadings.

y = Distance of neutral plane from tension face

The wall shall then be re-analyzed using I_e. If the use of I_e results in an applied moment M_a that differs from the previously calculatd M_a by more than 10%, a new I_e shall be calculated and the analysis repeated.

In lieu of the above procedure, the wall may be analyzed using the cracked transformed moment of inertia (I_{cr}). Since this procedure results in a lower bound in frequency, it is conservative with respect to the iteration procedure using I_e.

5.2.5 The effect of in-plane loading on reinforced walls shall be evaluated in the same manner as unreinforced walls per Section 5.1.5

5.2.6 The effects of boundary structure flexibility, wall group interaction, and wall openings shall be evaluated on a case by case basis.

5.2.7 Masonry block pullout due to concentrated inertial loadings imposed by attached components shall be evaluated using the allowable loads tabulated in Exhibit I.

5.3 Modifications

All modifications to masonry walls shall be designed in accordance with Ref. 2.3 subject to the limitations stated in Section 8 of this criteria. Selection of concrete anchors shall be in accordance with Exhibit D. Designs shall be based on conventional methods of structural analysis of linear elastic materials.

6.0 MATERIAL SPECIFICATIONS & PROPERTIES

6.1 Existing Construction

6.1.1 Concrete Block (Ref. 2.4)

Exterior Walls Battery Room Emergency Diesel Generator Room Boiler Room	ASTM C-90 (Ref. 2.5) Grade U-1 Lightweight, Hollow Load Bearing
Interior Partitions	ASTM C-129 (Ref. 2.5) Lightweight, Hollow Non-load Bearing
Shielding Block	ASTM C-145 (Ref. 2.5) Solid, Load-Bearing

6.1.2 Masonry Reinforcement (Ref. 2.4)

Horizontal	Dur-O-Wall Extra Heavy Weight
Vertical	ASTM A-15, Grade 40 ASTM A-305 (Ref. 2.5)

6.1.3 Mortar (Ref. 2.4)	ASTM C-270 (Ref. 2.5) Type 'S'
6.1.4 Concrete	f'c = 3000 PSI (Refer to assumption 3.8)
6.1.5 Structural Steel	ASTM A-36 (Ref. 2.5)

6.2 Modifications

Structural Steel, Threaded Rods	ASTM A-36 (Ref. 2.5)
Welding Electrode	E70-xx
Drilled-in Concrete Expansion Anchors	Hilti Kwik-Bolts (Refer to Exhibit D)

7.0 LOADS AND LOAD COMBINATIONS

The loads and load combinations in this section are based on the requirements for Class 1 structures of Paragraph 1.1.2 of Section XII of the Millstone Unit 1 FSAR (Ref. 2.6).

7.1 Loads to be considered in evaluating the masonry walls are described below.

D(1)	Dead load of wall and attached equipment, plus any other sustained loads.
E (E_v, E_h)(2)	Design earthquake loads. (E_v and E_h are vertical and horizontal components of the design earthquake loads, respectively.)
E' (E'_v, E'_h)(2)	Maximum earthquake loads. (E'_v and E'_h are vertical and horizontal components of the maximum earthquake loads, respectively.)
W'(3)	Tornado load.
R (8)	Pressure on structure due to a pipe rupture.

7.2 Masonry walls shall be evaluated for the following load combinations.

EQ 1 (9)	$D + R \pm E_v \pm E_h$ (4)	$\leq S$(5), B(6), M(7)
EQ 2	$D + R \pm E_v' \pm E_h'$ (4)	$\leq 1.33S$, B, M'(7)
EQ 3	$D + W'$	$\leq 1.33S$, B, M'

Notes:

(1) Dead load on each wall due to attachments shall be calculated using the data contained in Exhibit H.

(2) Seismic acceleration values and displacement profiles are given in Exhibits F and G, respectively. The accelerations have been increased by 33% to provide an overdesign margin.

(3) Tornado loads (W') shall consider Tornado wind load of 360 miles per hour (300 mph rotational and 60 mph translational) and Tornado depressurization load of 2.5 psi internal pressure. (Ref. 2.14, Section 12.1.2.9).

(4) The effects of one horizontal component and the vertical component of earthquake loading shall be combined in all loading combinations which include earthquake loads.

(5) S is the normal allowable stress in structural steel sections, bolts, and welds permitted by Ref. 2.3 (excluding the provisions of Section 1.5.6).

(6) B is the allowable concrete anchor bolt shear or tension load as given in Exhibit D.

(7) M and M' are the allowable stresses for evaluating existing masonry walls (including reinforcing steel) as given in Exhibit C.

(8) A differential pressure of 0.5 PSI shall be applied to masonry walls subject to pressure loading due to a postulated main steam line rupture in the condenser bay or a feedwater line rupture in the turbine building. (Ref. 2.10).

(9) The allowable stresses for EQ 1 apply to Class I structures. FSAR Section 1.1.3 states that for Class II structures, a one-third increase in allowable stress is permitted for load combinations which include seismic.

8.0 ACCEPTANCE CRITERIA

8.1 Unreinforced Walls

Allowable stresses for hollow and solid unit masonry are tabulated in Exhibit C.

Allowable block pullout loads are tabulated in Exhibit I.

8.2 Reinforced Walls

Allowable stresses for reinforced masonry are tabulated in Exhibit C.

Allowable block pullout loads are tabulated in Exhibit I.

8.3 Modifications

Stresses in structural steel sections, bolts, and welds shall conform to the requirements of Part 1 of the AISC Code (Ref. 2.3).

Allowable loads for Hilti Kwik-Bolt Concrete anchors are tabulated in Exhibit D.

9.0 EXHIBITS

A. Maximum Horizontal Span Lengths for Unreinforced, Masonry Walls Supported on Both Vertical Edges:

A-1 Hollow Masonry
A-2 Solid Masonry

B. Maximum Plate Dimensions for Unreinforced, Hollow Masonry Walls Supported on One Vertical Edge and the Base

C. Allowable Stresses in Concrete Masonry:

C-1 Unreinforced Walls
C-2 Reinforced Walls

D. Allowable Loads on Hilti-Kwik Concrete Anchors

E. Support Conditions for Unreinforced Walls

F. Rigid Range Seismic Acceleration Values

G. Reactor Building Seismic Displacement Profile

H. Attached Component/Equipment Weights for Dead Load Calculations

I. Allowable Block Pullout Loads

Building	Elevation (ft.)	Wall Thickness (in.)	Wall Weight (psf)	Attachment Weight (psf)	Maximum Span (ft.)
Turbine Building	14.5	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	34.5	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	54.5	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	104.75	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
Reactor Building	14.5 & 42.5	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	65.75 & 82.75	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	108.5 & 129.0	4	17	4	6.7
		6	26	5	8.3
		8	34	7	9.6
		12	48	10	11.6
	147.0	4	17	4	6.4
		6	26	5	7.8
		8	34	7	9.0
		12	48	10	10.7

Maximum Horizontal Span Lengths for
Unreinforced, Hollow Masonry Walls
Supported on Both Vertical Edges

(Natural Frequency $\geq$ 20 CPS and flexural tension stress $< F_t$)

Exhibit A-1

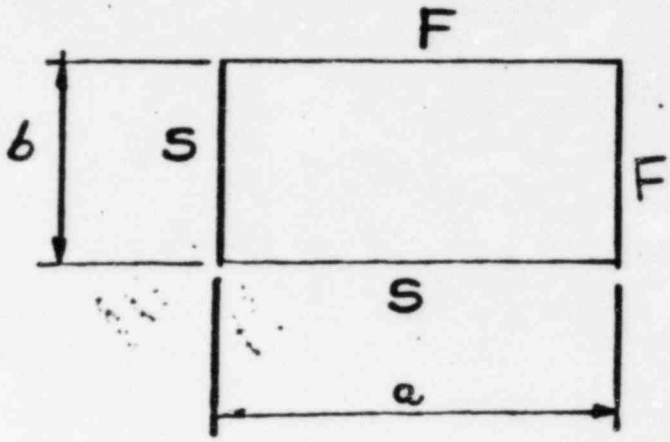
Building	Elevation (ft.)	Wall Thickness (in.)	Wall Weight (psf)	Attachment Weight (psf)	Maximum Span (ft.)
Turbine Building	14.5	4	44	4	5.7
		6	68	5	7.1
		8	92	7	8.2
		12	141	10	10.0
	34.5	4	44	4	5.7
		6	68	5	7.1
		8	92	7	8.2
		12	141	10	10.0
	54.5	4	44	4	5.7
		6	68	5	7.1
		8	92	7	8.2
		12	141	10	10.0
	104.75	4	44	4	5.3
		6	68	5	6.7
		8	92	7	7.7
		12	141	10	9.1
Reactor Building	14.5 & 42.5	4	44	4	5.7
		6	68	5	7.1
		8	92	7	8.2
		12	141	10	10.0
	65.75 & 82.75	4	44	4	5.7
		6	68	5	7.1
		8	92	7	8.2
		12	141	10	10.0
	108.5 & 129.0	4	44	4	5.1
		6	68	5	6.5
		8	92	7	7.4
		12	141	10	8.8
	147.0	4	44	4	4.6
		6	68	5	5.8
		8	92	7	6.6
		12	141	10	7.9

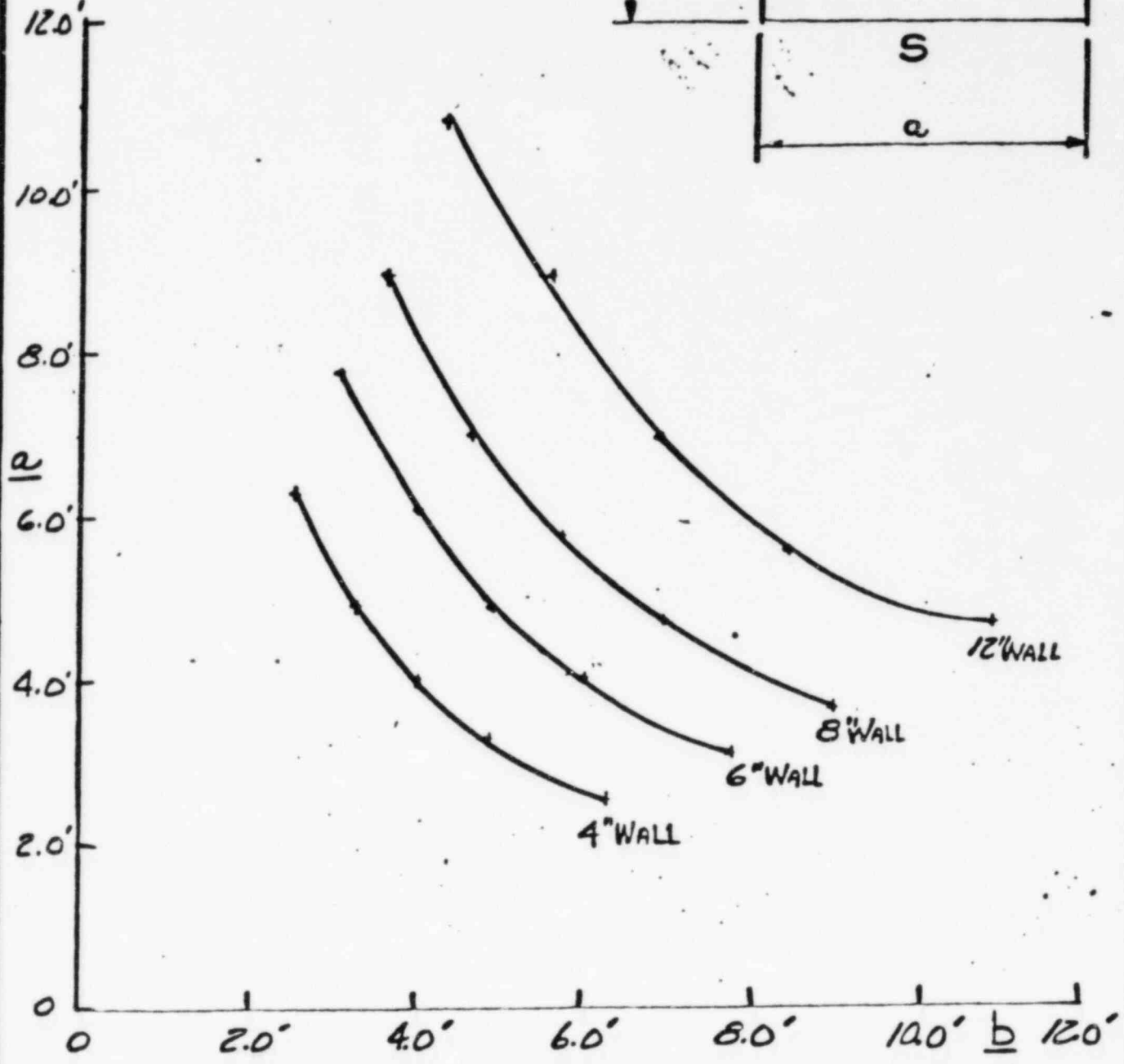
Maximum Horizontal Span Lengths for
Unreinforced, Solid Masonry Walls
Supported on Both Vertical Edges
(Natural Frequency $\geq$ 20 CPS and flexural tension stress $< F_t$)

Exhibit A-2

Exhibit B

PLOT OF a/b
FOR 20 Hz
WEIGHT OF ATTACHMENTS
INCLUDED





Maximum Plate Dimensions for Unreinforced, Hollow Masonry Walls Supported on One Edge and the Base

EES	DESIGN CRITERIA 80014 & 82028 MILLSTONE POINT UNIT 1 NORTHEAST UTILITIES SERVICE COMPANY	DC-1 Rev 3 Exhibit B Page 1 of 1

3

Exhibit C-1 Allowable Stresses for Unreinforced Masonry

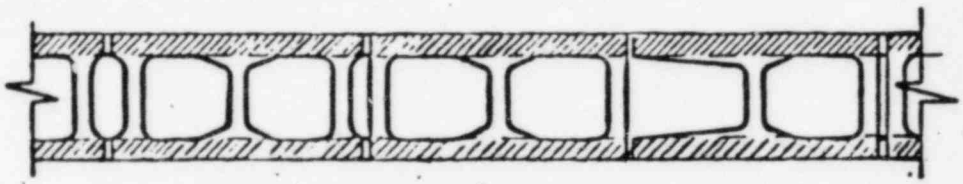
Description		Allowable Stresses (psi) (6)							
		M (1)				M' (1)			
			ASTM Block Type				ASTM Block Type		
			C90	C129	C145		C90	C129	C145
		Related to f'm & m_o	f'm=1350 psi m_o =1800 psi	f'm=630 psi m_o =1800 psi	f'm=1080 psi m_o =1800 psi	Related to f'm & m_o	f'm=1350 psi m_o =1800 psi	f'm=630 psi m_o =1800 psi	f'm=1080 psi m_o =1800 psi
COMPRESSION (3)									
Axial (4)(2)	F_a	0.22 f'm	297	139	238	0.44 f'm	594	277	475
Flexural	F_m	0.33 f'm	446	208	356	0.66 f'm	891	416	713
BEARING (3)(5)	F_a	0.25 f'm	338	158	270	0.50 f'm	675	315	540
SHEAR (3)									
Out of plane	v_m	$1.5\sqrt{f'm}$	55	38	49	$2.25\sqrt{f'm}$	83	56	74
In plane (7) (Shear Walls)	v_m	$0.9\sqrt{f'm}$	33	23	30	$1.35\sqrt{f'm}$	50	34	44
TENSION (3)									
Normal to bed joints (8)	F_t	$0.5\sqrt{m_o}$	21	21	21	$0.75\sqrt{m_o}$	32	32	32
Parallel to bed joints in running bond	F_t	$1.0\sqrt{m_o}$	42	42	42	$1.50\sqrt{m_o}$	64	64	64

Notes:

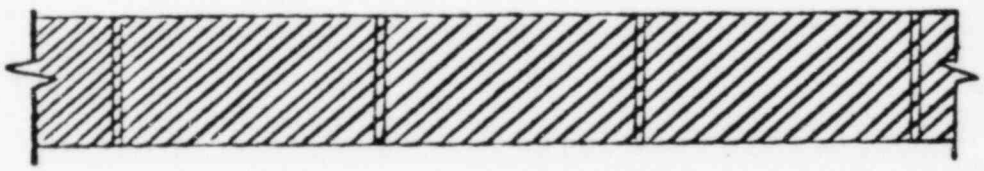
(1) M and M' shall be used for evaluating stress in accordance with Section 7.2.

(2) Multiply these values by $(1-(h/40t)^3)$ if the wall has significant vertical load at the top edge.

(3) The net area to be used for evaluating compression, bearing, shear and tension stresses is shown below. This area applies to wall sections on vertical planes as well as sections (shown) on horizontal planes.



HOLLOW MASONRY



SOLID MASONRY

(4) The effective length to be used to evaluate axial compressive stresses under concentrated loads is as given in Note (2) of Exhibit C-2.

(5) Allowable bearing stress may be increased to 0.375 f'm for M and 0.75 f'm for M' if load is applied on one-third of the compression area or less.

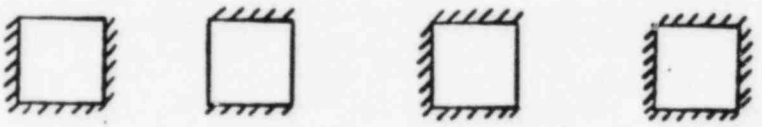
(6) Values for the moduli of elasticity and rigidity for all masonry strengths f'm are as follows:

Modulus of Elasticity E_s = 810,000 psi
Modulus of Rigidity E_v = 324,000 psi

(7) For non-shear walls which are confined in the structure and subjected to shear distortion due to relative floor displacements, the allowable relative displacement (Δ) is 0.1% of the height of the wall (h).

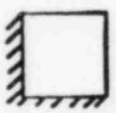
For non-shear walls which are subjected to shear distortions due to relative floor displacement but cannot be classified as confined walls, the allowable relative displacement is 0.01% of the height of the wall.

Confined walls are bounded by adjacent steel or concrete primary structures. As a minimum, confined walls are bounded top and bottom or bounded on three sides. Examples of confined walls are shown schematically below.



Confined walls: $e_v = \Delta/h \leqslant 0.001$

Unconfined walls are not bounded by adjacent steel or concrete primary structures sufficiently to create a confining effect. An example of an unconfined wall is shown schematically below.



Unconfined wall: $e_v = \Delta/h \leqslant 0.0001$

(8) Flexural tension normal to the bed joints may be used only where failure of the bed joint will not cause failure of the wall; i.e., there must be an alternate load path available.

Exhibit C-2 Allowable Stresses for Reinforced Masonry Walls

Description		Allowable Stresses (Masonry) (psi) (11)			
		M (1)		M' (1)	
		Related to f'm	ASTM C90 f'm=1350psi	Related to f'm	ASTM C90 f'm=1350psi
COMPRESSION					
Axial (2)(3)	F_a	0.22 f'm	297	0.44 f'm	594
Flexural (4)	F_m	0.33 f'm	446	0.66 f'm	891
BEARING (5)	F_a	0.25 f'm	338	0.50 f'm	675
SHEAR					
No special shear reinforcement					
Out of plane (6)	v_m	$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
In plane (12) (7) (Shear walls)	v_m				
$M/Vd_v \geq 1$ (8)		$0.9\sqrt{f'm}$	33	$1.35\sqrt{f'm}$	50
$M/Vd_v = 0$ (8)		$2.0\sqrt{f'm}$	73	$3.0\sqrt{f'm}$	110
Reinforcement taking shear					
Out of plane (6)	v_m	$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
In plane (12) (7) (Shear walls)	v_m				
$M/Vd_v \geq 1$ (8)		$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
$M/Vd_v = 0$ (8)		$2.0\sqrt{f'm}$	73	$3.0\sqrt{f'm}$	110

Description		Allowable Stresses (Reinforcement) (psi)	
		M (1)	M' (1)
BOND			
Deformed bars	u	140	140

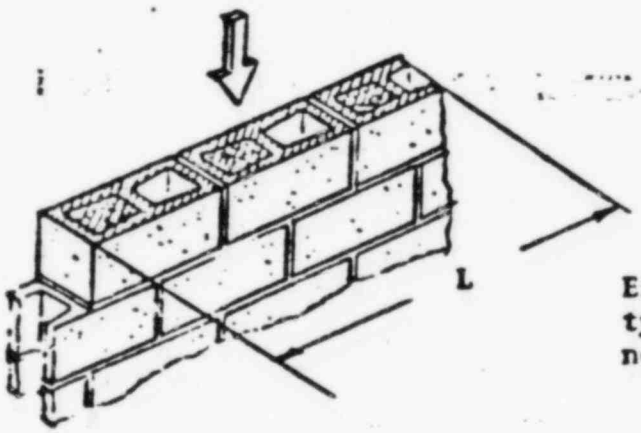
Exhibit C-2 (cont'd)

Description		Allowable Stresses (Reinforcement) (psi)	
		M (1)	M' (1)
TENSION & COMPRESSION			
Reinforcing steel	F_s		
Grade 40 bars		20,000	0.9 f_y (9)
Joint wire reinforcement		0.5 f_y (10)	0.9 f_y

Notes:

(1) M and M' shall be used for evaluating stress in accordance with Section 7.2.

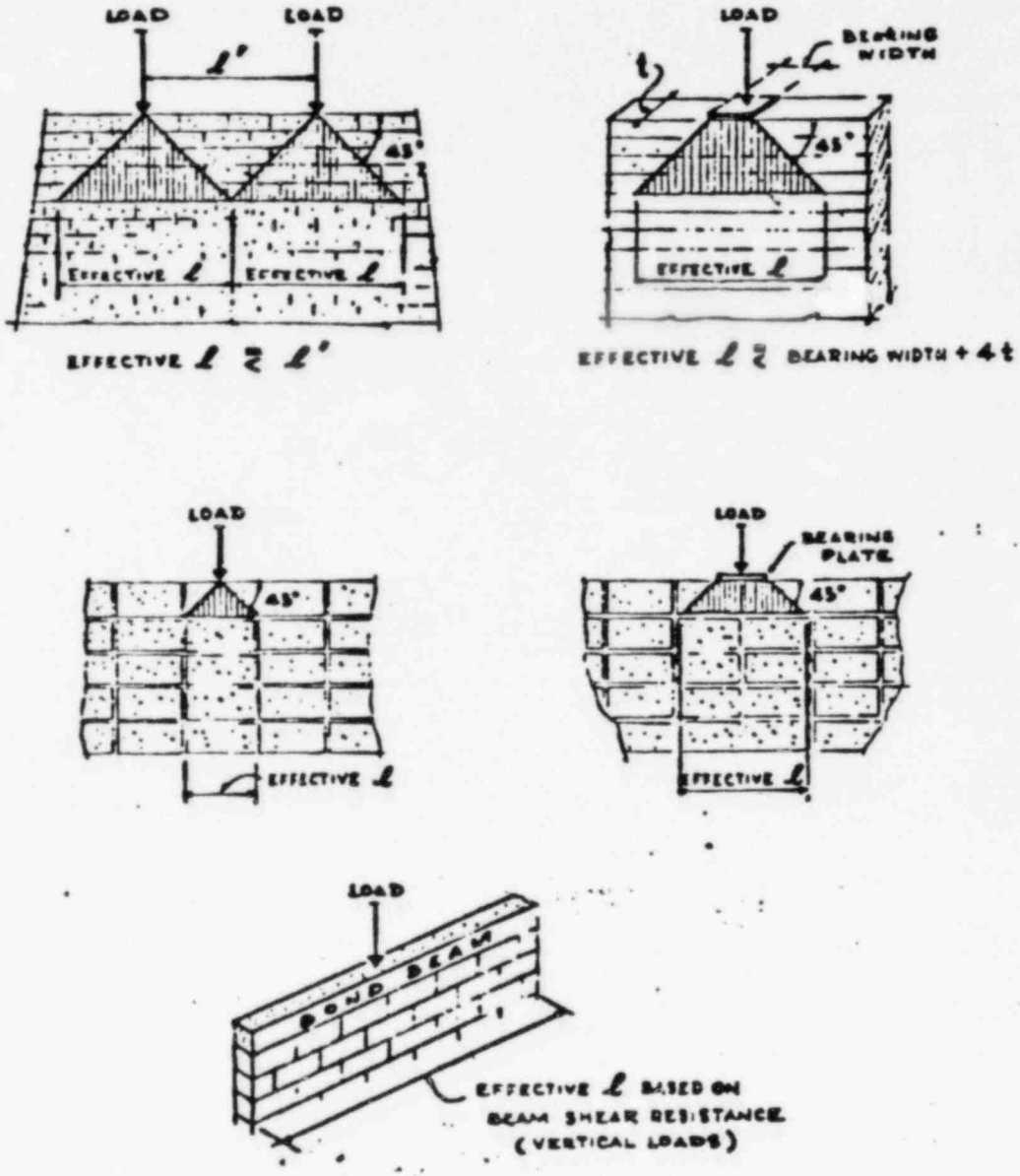
(2) The effective area to be used for evaluating axial compressive stress is shown below:



Effective length L depends on type of bond and loading (see next page).

▨ Effective area for axial compressive stress calculations on net section of masonry units plus grouted cores.

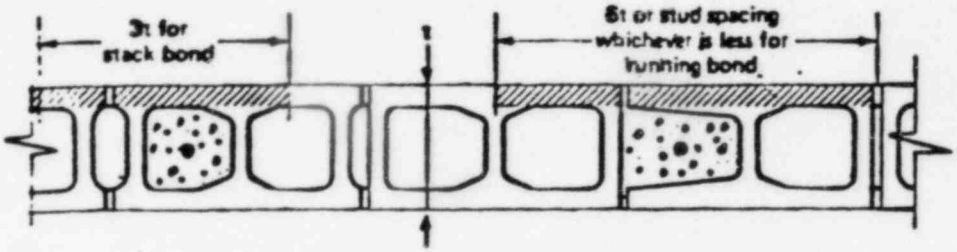
(2) (cont'd)



Effective lengths for axial compressive stress evaluation.

(3) Multiply these values by $(1-(h/40t)^3)$ if the wall has significant vertical load at the top edge.

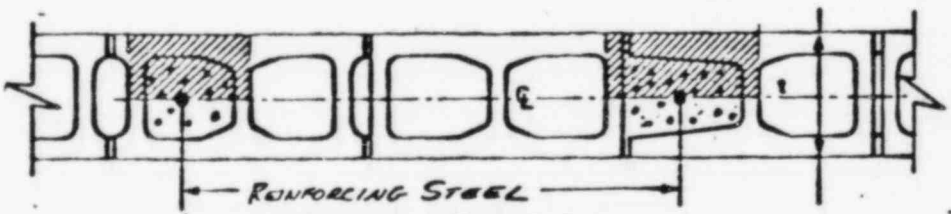
(4) The effective area to be used for evaluating flexural compressive stress is shown below.



Area assumed effective in flexural compression, force normal to face

(5) Allowable bearing stress may be increased to 0.375 f'm for M and 0.75 f'm for M' if load is applied on one-third of the compression area or less.

(6) The effective area for evaluating shear stress for walls in flexure is shown below.



Area assumed effective in shear, force normal to face

(7) The effective area for evaluating shear stress for shear walls is shown below.



Area assumed effective in shear, force parallel to face

(8) M is the maximum bending moment ocurring simultaneously with the shear load V at the section under consideration. d_v is the length of the wall in the direction of shear. Interpolate by straight line for M/Vd_v values between 0 and 1.

(9) f_y is the specified yield strength of the reinforcement. (f_y = 40 KSI for grade 40 reinforcement per Ref. 2.5)

(10) 0.5 f_y not to exceed 30,000 psi.

(11) Values for the moduli of elasticity and rigidity for all masonry strengths f'm are as follows:

Modulus of Elasticity E_s = 810,000 psi
Modulus of Rigidity E_v = 324,000 psi

(12) The in-plane distortion considerations discussed in note (7) of Exhibit C-1 also apply to reinforced walls.

Exhibit D Allowable Loads on Hilti Kwik Bolts

Bolt dia. (inches)	Embedment (inches)	Allowable Load	
		Tension (lb.) (Ta)	Shear (lb.) (Va)
1/2	2 1/4	951	2018
	2 3/4	1218	2018
	3 1/2	1541	2018
	4 1/2	1733	2391
	5 1/2	1902	2391
	6	1996	2391
5/8	2 3/4	1750	3400
	3 1/2	1918	3400
	4 1/2	2375	3400
	5 1/2	2731	3600
	6 1/2	3002	3600
	7 1/2	3250	3600
3/4	3 1/4	2359	4968
	4	2887	4968
	5	3525	4968
	6	3975	4968
	7	4600	4968
	8	4875	4968
	9	4937	4968

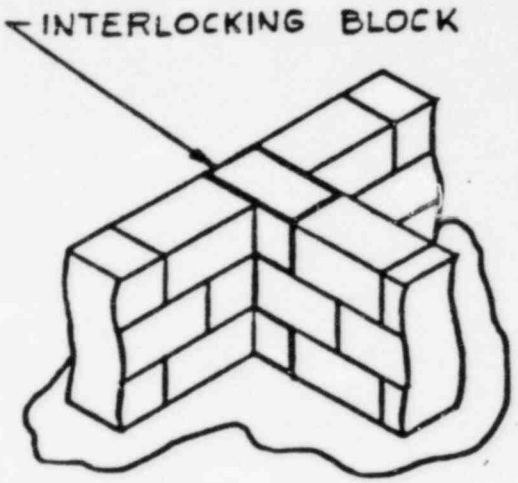
Exhibit D Allowable Loads on Hilti Kwik Bolts (cont.)

Bolt Dia. (inches)	Embedment (inches)	Allowable Load	
		Tension (lb.)	Shear (lb.)
1	4 1/2	3750	6777
	5	4300	6777
	6	5281	6875
	7	5712	6875
	8	5712	7730
	9	5712	7730
	10	5712	7730
1 1/4	5 1/2	5312	8150
	6 1/2	6087	8150
	7 1/2	6837	8150
	8 1/2	7462	8496
	9 1/2	8000	8496
	10 1/2	8462	8496

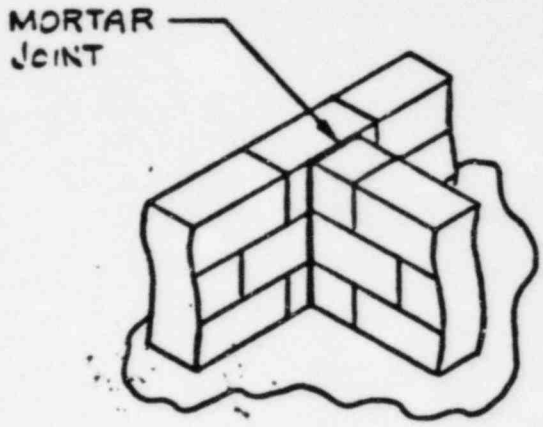
- Allowable Loads are based on a Factor of Safety of 4.0 and concrete strength of 3000 psi.
- Combined Tension and Shear shall be evaluated as follows:

$$\frac{T\ (actual)}{Ta} + \frac{V\ (actual)}{Va} \leqslant 1$$

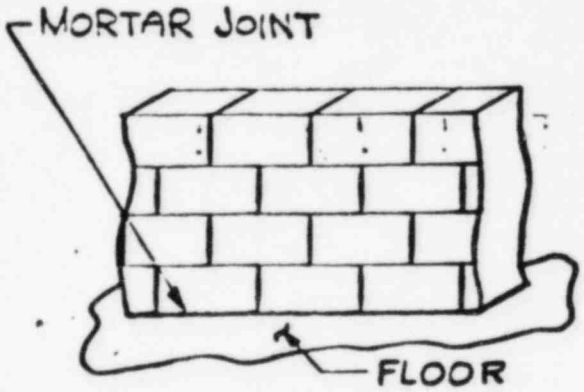
- A minimum edge distance of 6 inches shall be used unless otherwise justified by calculation.
- A minimum center to center bolt spacing of 10 bolt diameters or 2 times the embedment length, whichever is largest, shall be used unless otherwise justified by calculation.



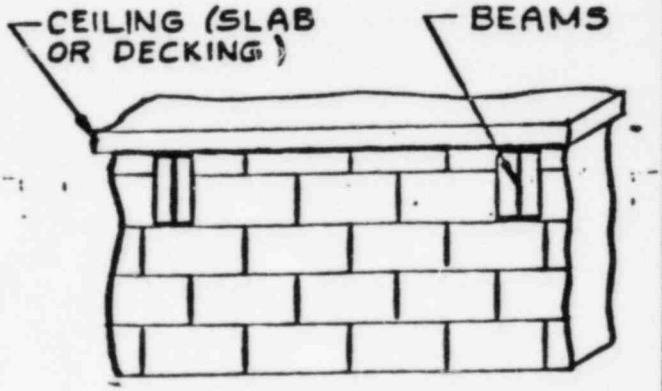
INTERLOCKING JOINT
(HINGED)



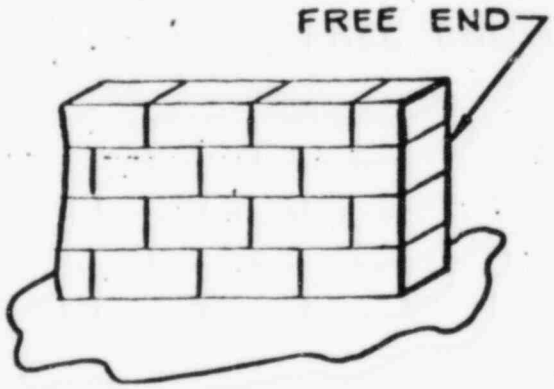
BOND JOINT
(FREE)



FLOOR JOINT
(HINGED)



CEILING JOINT
(FREE)



FREE JOINT
(FREE)

Support Conditions

for

Unreinforced Walls

Exhibit E

EES	DESIGN CRITERIA 80014 & 82028 MILLSTONE POINT, UNIT 1 NORTHEAST UTILITIES SERVICE COMPANY	DC-1 Rev. 3 Exhibit E Page 1 of 1

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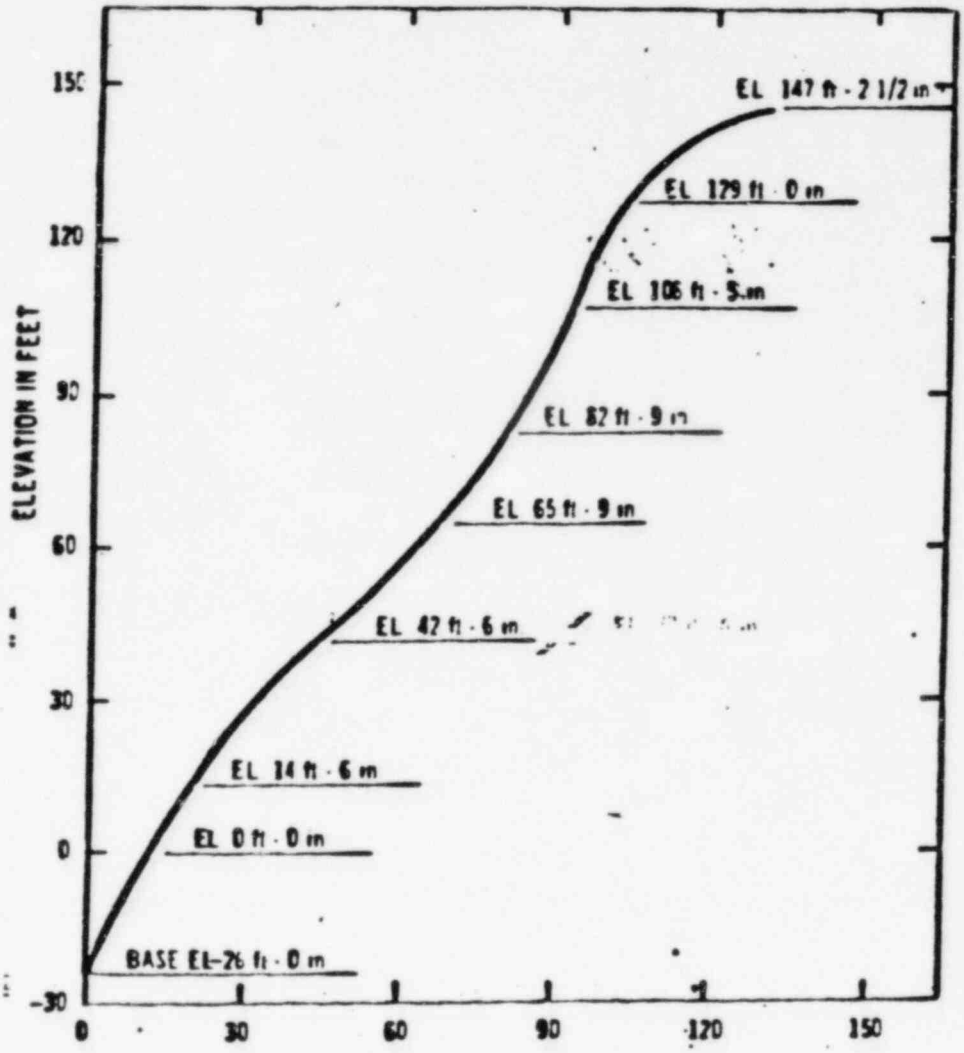
Exhibit F

Rigid Range Seismic Accelerations for Re-evaluation of Masonry Walls*

Building	El.	Design Earthquake E (acc. in 'g's)	Maximum Earthquake E' (acc. in 'g's)
Turbine Bldg. (Ref. 2.8)*	14.5'	.093 Horiz .067 Vert	.226 Horiz .146 Vert
	34.5'	.146 Horiz .067 Vert	.346 Horiz .146 Vert
	54.5'	.186 Horiz .067 Vert	.452 Horiz .146 Vert
	104.75'	.333 Horiz .067 Vert	.798 Horiz .146 Vert
Reactor Bldg. (Ref. 2.6)*	14.5'	.093 Horiz .067 Vert	.226 Horiz .146 Vert
	42.5'	.106 Horiz .067 Vert	.253 Horiz .146 Vert
	65.75'	.160 Horiz .067 Vert	.386 Horiz .146 Vert
	82.75'	.200 Horiz .067 Vert	.479 Horiz .146 Vert
	108.5'	.279 Horiz .067 Vert	.678 Horiz .146 Vert
	129.0'	.346 Horiz .067 Vert	.838 Horiz .146 Vert
	147.0'	.426 Horiz .067 Vert	1.04 Horiz .146 Vert

*Tabulated Accelerations are 1.33 times those in Refs. 2.6 and 2.8.

Exhibit G Reactor Building Seismic Displacement Profile (Ref. 2.6)



Note: Profile shown is for the Design Earthquake (E). For Maximum Earthquake (E') multiply displacements by 2.4

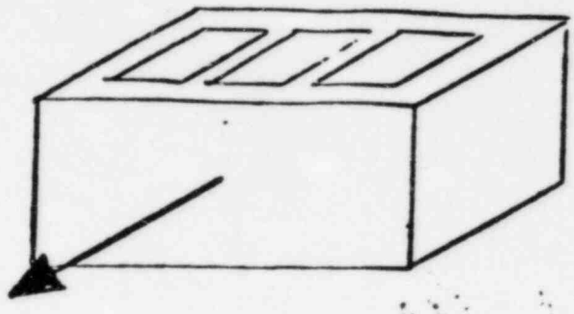
Exhibit H

Attached Component/Equipment

Weights for Dead Load Calculations

Seismic Conduit Support Standards		
Conduit Nom. Size (in.)	Conduit Plus Conductor Wt.	
	Steel (Lbs./Ft.)	Aluminum (Lbs./Ft.)
1	2.0	1.0
2	5.0	2.5
3	11.0	6.0
4	16.5	9.5
5	24.0	14.0
6	32.5	19.5
Electrical Panels	Surface Area of Panel x 3.50 Lb/Ft^2	
Piping	Flow of Fluids Through Valves, Fittings, and Pipes, Crane Co. Technical Paper No. 410, copyright 1979--Crane Co., New York, N. Y. (Appendix B)	

Exhibit I Allowable Block Pullout Loads



WALL THICKNESS (IN.)	ASTM BLOCK TYPE	ALLOWABLE BLOCK PULLOUT LOAD (LB)	
		M	M'
4	C-90	2302	3488
	C-129	1604	2372
	C-145	2093	3069
6	C-90	3070	4652
	C-129	2140	3163
	C-145	2791	4094
8	C-90	3836	5812
	C-129	2674	3952
	C-145	3487	5115
12	C-90	4604	6976
	C-129	3209	4744
	C-145	4186	6139

Refer to Section 7.2 for description of M and M'

Docket No. 50-336

ENCLOSURE III

MILLSTONE NUCLEAR POWER STATION, UNIT NO. 2

REQUEST FOR ADDITIONAL INFORMATION ON
IE BULLETIN 80-11, MASONRY WALL DESIGN

NOVEMBER, 1982

RESPONSE TO NRC QUESTIONS
MASONRY WALL RE-EVALUATION
(NRC IE BULLETIN 80-11)
MILLSTONE PLANT UNIT 2

1. QUESTION:

Indicate whether the walls are stack or running bond. If any stack bond walls exist, provide sample calculations used to obtain stresses of a typical wall.

RESPONSE:

All masonry walls at Millstone Unit #2 are running bond.

2. QUESTION:

a. Section 3 of the criteria (3) specifies ACI 531-79 (5) as the governing code for allowable stresses in masonry walls. In Section 5.1.1.3 of Reference 2, the Licensee has listed allowable shear stresses which exceed some allowables listed in ACI 531-79. Justify the following allowable shear stresses (the allowable stresses from the ACI 531-79 - "Special Inspection" category are shown in parentheses and based on an f'_m of 1350 psi).

No Shear Reinforcement

Flexural members
50 psi (33 psi = 0.9 $(f'_m)^{1/2}$)

Shear walls
50 psi (33 psi = 0.9 f'_m M/Vd $\geq$1)

Reinforcing Taking Entire Shear

Flexural members
120 psi (110 psi = 3.0 $(f'_m)^{1/2}$)

Shear walls
75 psi (55 psi = 1.5 f'_m M/Vd $\geq$1)

The Licensee should indicate whether construction practices were in conformance with the provisions specified in ACI 531-79.

b. Provide allowable stresses for unreinforced masonry. Justify these values if they exceed those given in ACI 531-79 or UBC-79.

RESPONSE:

a. The Millstone Unit #2 FSAR, which was used as the basis for our licensing document, specifies the UBC-67 to be used as the design code. Allowable shear stresses in masonry walls were determined in conformance with its provisions.

In general, shear did not govern in the design of the masonry walls. An investigation comparing the actual shear values with the ACI 531-79 failure criteria has not been made.

In conformance with the Millstone Unit #2 FSAR the specified construction practices were in conformance with the provisions specified in UBC-67. The actual construction practices did meet the intent of the ACI 531-79 although added allowable stress factors were not included for this reason.

b. Allowable stresses for unreinforced masonry were determined in conformance with the provisions of UBC-67, which are:

		OBE & 115 mph Wind Load Case	SSE & Other Accident Load Case
Axial Compression	= 175 psi	232.5 psi	350 psi
Flexural Compression	= 175 psi	232.5 psi	437.5 psi
Bearing	= 263 psi	349 psi	658 psi
Flexural Tension (Spanning horiz.-running bond)	= 24 psi	32 psi	40 psi
Flexural Tension (Spanning veritcally-running bond) (Tension = 0 across head jt. in stacked bond)	= 12 psi	16 psi	20 psi
Shear Stress	= 12 psi	16 psi	20 psi

These values are based upon using Type S mortar and having inspection provided.

3. QUESTION:

With respect to multiple wythes, clarify whether collar joint strength was used in the analysis. If so, justify by any existing test data the values used for allowable shear and tension of collar joints. Provide sample calculations illustrating the analysis of multiwythe wall.

RESPONSE:

All composite walls were assumed to have zero collar joint strength and, therefore, walls which satisfied the elastic criteria compositely were through bolted with prestressed bolts to assure composite action.

Sample calculations illustrating the analysis of a multiwythe wall are included in attachment 2.

4. QUESTION:

The Licensee uses an increase factor of 1.67 (Reference 2, Section 5.2.1) for allowable masonry stresses in tension and shear whereas the SEB criteria (3) allow only 1.3 for shear and tension normal to the bed joint and 1.5 for tension parallel to the bed joint. The Licensee justifies the factor of 1.67 by claiming it provides a factor of safety of 1.8 (3 / 1.67). Substantiate this assertion by any existing test data. The licensee is also requested to indicate the number of walls which would not be qualified if the SEB criteria were to be used.

RESPONSE:

The factor of 1.67 for shear bond stresses was included in the criteria for the re-evaluation of concrete masonry walls (Reference 1) submitted November 4, 1980, prior to the publishing of the Standard Review Plan (Reference 2) being published in July, 1981. Code allowable stresses for masonry shear, bond, and tension normal or parallel to the bed joint were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since ACI-531 Code Allowable Stresses (Reference 3, Chapter 10.1 of Commentary) are generally associated with a factor of safety of 3, the 1.67 increase still provides a factor of safety against failure of 1.8 (3 / 1.67). It is our engineering judgment that a factor of safety of 1.8 is conservative and allows sufficient margin for abnormal and/or extreme conditions. An analysis to determine the number of walls which would not be qualified if the SEB criteria were to be used has not been done at this time.

5. QUESTION:

In Reference 2, the Licensee indicated that the energy balance and arching action techniques have been used to qualify some of the masonry walls. The NRC does not accept the application of those methods to masonry walls in nuclear power plants without conclusive evidence to justify this application. The Licensee is requested to indicate the number of walls analyzed by each of these techniques and to provide sample calculations to illustrate the analysis by each technique. The Licensee is also requested to identify the types of structures (grouted, ungrouted, distribution of reinforcement) involved in these analysis. In addition, the following areas need technical verification before any conclusions can be made:

a. Energy Balance Technique

- Provide a technical basis to ensure that the ductile mode of failure will occur (if the wall fails).

- Provide justification for and test data (if available) to validate the applicability of the energy balance technique to the masonry structures at Millstone Unit 2, with particular emphasis on the following:

(1) Nature of the load
(2) Boundary conditions
(3) Material strengths
(4) Size of test walls

b. Arching Action

- Explain how the arching action theory handles cyclic loading, espcially when the load is reversed.

- Provide justification for and test data (if available) to validate the applicability of the arching action theory to the masonry structures at Millstone Unit 2, with particular emphasis on the following areas:

  (1) Nature of the load
  (2) Boundary conditions
  (3) Material strengths
  (4) Size of test walls

- If hinges are forced in the walls, the capability of the structures to resist in-plane shear force would be diminished, and shear failure might occur. This in-plane shear force would also reduce the out-of-plane stiffness. Explain how the effect of this phenomenon can be accurately determined.

RESPONSE:

A total of 57 masonry walls did not meet the acceptance criteria of the 180-day report and were modified. If feasible, walls were modified to meet elastic criteria. Of the 57 modifications, 43 were repaired to the elastic criteria and the remaining repaired to the operability criteria. The operability criteria included use of the energy balance technique (in 3 cases) and the arching action technique (in 18 cases). All three walls analyzed using the energy balance technique were grouted with #5 bars at 24 inches. All walls analyzed using the arching theory were grouted, with varying and without any reinforcement.

Sample calculations illustrating the energy balance and arching action techniques are included in attachments 1 and 2 respectively.

a. Energy Balance Technique

- The energy balance technique is used to analyze masonry walls subjected to out-of-plane loads. For this particular loading condition, a reinforced concrete block wall is a ductile structure, provided a flexural type of failure occurs with tensile yielding of the steel (Reference 4). Flexural failure by yielding of steel is ensured by checking the compressive stress in the block to determine that its crushing strength is not exceeded. Experimental work performed by Sheppard (Reference 5) indicates "considerable ductility" in concrete block walls having

reinforcement ratios of 0.15% or greater. All walls analyzed by energy balance at Millstone Unit #2 meet this criteria.

(1) The energy balance technique was developed to analyze ductile structures subjected to seismic loads (Reference 6). At Millstone Unit #2, energy balance is used primarily to determine the ability of reinforced masonry walls to resist earthquake loadings.

(2) (3) (4) Boundary conditions, material strengths and wall size are all conditions that quantitatively affect the response of a particular wall. In order to apply the energy balance technique, the wall need only be shown to qualitatively respond in a ductile manner. It is the inelastic energy absorption associated with a ductile response which is the foundation of this method of analysis.

Actual physical evidence in support of the energy balance technique is cited in Reference 7. This reference investigates the ductile failure of catalytic cracking towers affected by the 1952 Arvin-Tehachapi earthquake. The study shows "general agreement between observed and computed values" of displacement.

b. Arching Theory

o The arching theory of confined masonry walls is based on an extensive full-scale test program sponsored by the Defense Civil Preparedness Agency and the Veterans Administration (Reference 8). The tests were performed in a "shock tunnel" in which a wall was "struck by a blast wave, the wave reflected from the wall returned to its source area, then re-reflected to strike the wall again, about 0.3 seconds after the first loading. Thus, these tests provided information on the ability of (arching) walls to withstand a number of reversing loading cycles."

(1) At Millstone Unit #2, arching theory was used primarily to determine the ability of masonry walls to resist uniform pressure loadings and earthquake loadings. The test data of Reference 8 provides justification for applying arching theory to masonry walls subjected to pressure loads. Reference 8 also cites field evidence which confirms the increased resistance to out-of-plane earthquake loads of confined (arching) masonry walls over nonarching walls.

(2) Boundary conditions required for the application of arching theory were validated by walkdown inspection of the masonry walls to be analyzed.

(3) Experiments on arching have been performed on walls that range in crushing strength from 1000 psi to 3000 psi (Reference 9). The masonry walls at Millstone Unit #2 fall within this range.

(4) The tests of Reference 9 were performed on walls arching over an 8'-0" span while those of Reference 8 spanned 8'-6". We consider that application of arching theory to the range of wall sizes encountered at Millstone Unit #2 is appropriate.

- The walls at Millstone Unit #2 analyzed by the arching method are not designed to act as shear walls or as bearing walls and, therefore, are not subjected to significant in-plane load. This eliminates the need to determine how in-plane and out-of-plane loads interact.

6. QUESTION:

Section 3.7.2 of the Standard Review Plan requires that unless a dynamic analysis is performed, the effect of higher modes of vibration shall be accounted for by multiplying the peak acceleration of the floor response spectrum by a factor of 1.5. The Licensee is requested to explain how higher modes of vibration were accounted for in the analysis.

RESPONSE:

The "Blockwall" program was mainly used for the analysis of reinforced masonry for external and/or seismic loads. This program utilizes a response spectrum analysis to determine the seismic inertia loading including the effects of higher modes. When hand calculations were used the peak floor response acceleration factor was multiplied by 1.5.

7. QUESTION:

Indicate how earthquake forces in three directions were considered in the analysis.

RESPONSE:

The stresses were developed based upon the out-of-plane loads. The effects of vertical and in-plane loading were assumed to be negligible and thus not considered.

8. QUESTION:

Indicate how out-of-plane drift effects are considered in the analysis.

RESPONSE:

In the evaluation of Millstone Unit #2, all masonry walls were analyzed as having pinned boundaries. For this reason, drift was not a principle consideration in the re-evaluation of the masonry walls.

9. QUESTION:

Indicate if block pullout was considered in the evaluation. If yes, provide sample calculations of block pullout analysis.

RESPONSE:

The block pullout was considered in the re-evaluation of masonry walls at Millstone Unit #2. Sample calculations of the block pullout analysis are provided in Attachment 2.

10. QUESTION:

Specify the number of masonry walls analyzed for impact and suddenly applied loads. Provide the results (stresses, displacements) of these analyses. In addition, provide a sample calculation illustrating the analysis for impact and suddenly applied loads.

RESPONSE:

All masonry walls that were subject to possible impact and suddenly applied loads were reviewed. When the review determined these loads to be appreciable relative to other loading conditions an analysis was made to determine their impact.

Sample calculations illustrating the analysis for impact and suddenly applied loads (jet impingement) are included in Attachments 1 and 2.

11. QUESTION:

Provide the final report on the re-evaluation and modifications to masonry walls which was scheduled to be submitted by May 1, 1981.

RESPONSE:

The final report on the re-evaluation and modifications to masonry walls has been included in Attachment 3.

REFERENCES

1. Counsil, W. G. letter with attachments to B. B. Grier, NRC, Subject: Millstone Nuclear Power Station, Unit No. 2; I&E Bulletin No. 80-11; Masonry Wall Design, Northeast Utilities, 04-Nov-80, A01021.

2. Interim Criteria for Safety-Related Masonry Wall Evaluation NRC, July, 1981, SRP 3.8.4, Appendix A.

3. Building Code Requirements for Concrete Masonry Structures, Detroit: American Concrete Institute, 1979, ACI 531-79 and ACI 531-R-79.

4. Scrivener, J. C., "Reinforced Masonry - Seismic Behavior and Design," Bulletin of New Zealand Society for Earthquake Engineering, Vol. 5, No. 4, December, 1972.

5. Sheppard, P., et. al. "The Influence of Horizontally-Placed Reinforcement on the Shear Strength and Ductility of Masonry Walls," 6th World Conference on Earthquake Engineering, New Delhi, India, 1977.

6. Blume, J. A., Newmark, N. M., Corning, L. H., "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Illinois, 1961.

7. Housner, G. W., "Limit Design of Structures to Resist Earthquakes," World Conference on Earthquake Engineering, Berkeley, California, 1956.

8. Gabrielsen, B. L., Kaplan, K., "Arching in Masonry Walls Subjected to Out-of-Plane Forces," Earthquake Resistance of Masonry Construction, National Workshop, NBS 106, 1976.

9. McKee, K. E., Sevin, E., "Design of Masonry Walls for Blast Loading," ASCE Structural Journal, January, 1958.

Attachment 1 consists of an 18 page (sheets 0-17) example of energy balance and jet impingement analysis.

CALCULATION COVER SHEET

PROJECT Millstone Unit 2 JOB NO 11867-020 CALC. NO. 80-11-1.9 & 1.10

SUBJECT USNRC I.E. Bulletin 80-11 Design Verification of Block Wall 1.9 & 1.10

ORIGINATOR Jeffrey Castellano /REB DATE 5-28-81

CHECKER Thomas A Hogg / J. Mill DATE 5-28-81 NO. OF SHEETS 18

0

RECORD OF ISSUES

NO.	DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE	ACCEPT-ANCE	DATE
0	INITIAL CALC PG 1 T 17	JC	5-28-81	TAH	5-28-81	RWH	6/6/81		

☐ PRELIMINARY CALC. ☐ COMMITTED PRELIMINARY CALC.

☐ SUPERSEDED CALC. ☒ FINAL CALC.

GPD-2704 R[illegible]/78 (ED-1)

CALCULATION SHEET

CALC. NO. 8011-1.9&1.10 REV. NO. 0

ORIGINATOR Jeffrey Castelaz DATE 4.20.81 CHECKED JL DATE 4/27/81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 1

DESIGN CALCULATION CHECKLIST

Load Description WALL 1.9 / WALL 1.10

	Load Sheet Date	Description
1. Plant Design	9.8.80 / 12.16.80	N/A / PIPES
2. Electrical:	10.9.80 / 12.3.80	N/A / CONDUIT AND BOXES.
3. Mechanical:	4.6.81 / 4.6.81	JET IMPINGEMENT
4. Control Systems:	8.25.80 / 8.27.80	N/A / RECEIVER

Wall Description

1. Drawing Details: 25203 - CS-1, DET 12

2. Masonry Type: HOLLOW BLOCK 2 CELLS/BLOCK GROUTED W/REBAR @ 24"

3. Wall Thickness: 12" NOMINAL

4. Number of Wythes: 1

5. Rebar Size and Spacing: #5 @ 24"

6. Latest Page Revision of Survey Data and Date: △1 10.29.81

7. Boundary Connection Detail: DET 12, CS-1 BOTTOM 3/4" WELD @ 24" TOP DETAIL UNKNOWN, TO BEAM & DECKING



8. Safety-Related Attachments or II/I: II/I

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Cortelucci DATE 4.20.81 CHECKED FK DATE 4/27/81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 2

Summary of Results

1. Governing Load Combination: $D+L+E'+F_r$

2. Boundary Condition Assumed: SIMPLE - SIMPLE.

3. Horizontal or Vertical Span Checked: VERTICAL

4. In-Plane Shear Strain vs. Allowable: $\gamma = \frac{\Delta}{H} = \frac{.005 - .004}{545 - 385} = .00006 < 1.67(.0008)$.

5. Collar Joint Shear Stress vs. Allowable: N/A

6. Tensile Steel Stress or Masonry Tensile Stress vs. Allowable: $7.96^{ksi} > 54^{ksi}$ NG

7. Masonry Compressive Stress or Interaction Value vs. Allowable: 1.1 > 1.0

8. Masonry Shear Stress vs. Allowable: $.018 < .084^{ksi}$ O.K.

9. Cracked or Uncracked Section: CRACKED

10. Boundary Connections, Pass/Fail: BOTTOM O.K.
TOP TO DECKING, INADEQUATE

11. Block Pullout, Shear Stress vs. Allowable:
$.010^{ksi} < .020^{ksi}$ O.K.

12. Maximum Displacement Elastic Analysis: 1.06"

13. Repair, Required/Not Required: REQUIRED @ TOP BOUNDARY

14. Repair Type: PLACE ANGLES @ TOP

~~15. Arching, H_a vs. H_i~~ OPERABILITY: ENERGY BALANCE
μ vs. μ ALLOW

~~16. Maximum Displacement Arching Action:~~
1.22 < 5 OK

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Confalone DATE 4.20.81 CHECKED TE DATE 4/27/81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 3

REFERENCES:

- [x] 1) Specification for Re-evaluation of Concrete Masonry Walls - 11867-020-C003, Rev. 1.
- [x] 2) Survey Procedure No. 11867-020-C002, Rev. 5, (Including Survey Data).
- [x] 3) Loading Evaluation Procedure No. 11867-020-C004, Rev. 0, (Including Load Sheets).
- [x] 4) Floor Response Spectra-~~Auxiliary~~ WAREHOUSE Building Calculation No. 8011-00~~1~~6, Rev. 0.
- [ ] 5) Floor Response Spectra-Turbine Building Calculation No. 8011-002, Rev. 0.
- [x] 6) Masonry Block Section Properties Calculation No. 8011-003, Rev. 0.
- [ ] 7) Blockwall Pipe Hanger Load Summary Calculation No. 8011-004, Rev. 0.
- [ ] 8) Average Floor Response Spectra for Successive Floors In Auxiliary Building - Calculation No. 8011-005, Rev. 0.
- [x] 9) Millstone Unit 2, Seismic Report.
- [ ] 10) Civil Design Aid No. 8011-01-GPD, Rev. 0.
- [x] 11) Bechtel 'Blockwall' Program CE-020 (Version B).
- [ ] 12) Design Response Spectra, Pgs. E1 & E2 of FSAR for Millstone Unit 2.
- [x] 13) Damping Values for Wall Attachments, Pg. E3 of FSAR for Millstone Unit 2.
- [x] 14) IE BULLETIN 79-02, DESIGN GUIDELINES 11867-011-C-001 Rev. 1

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Contelouse DATE 4.20.81 CHECKED ZY DATE 4/27/81

PROJECT MILLSTONE UNIT No 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION of BLOCKWALLS SHEET NO. 4

WALL 1.9 & 1.10 WAREHOUSE @ ELEV 38'-6"

WALL DIMENSIONS;

LENGTH 19'-1"

HEIGHT: 13'-1"

THKNESS: 12" NOMINAL

EXTERNAL LOADS & ATTACHMENTS:

CONTROL SYSTEMS: RECORDER 60 LB

PLANT DESIGN: 3" ϕ PIPE 97 LB DEAD WT.

4" ϕ PIPE 107 LB DEAD WT.

MECHANICAL: JET IMPINGEMENT P = 0.63 K

(DLF = 2.0)

ELECTRICAL: TRANSFORMER 8 LB

JCT BOX 8 LB (T451-2A)

CONT BOX 92 LB (TE 245)

CONDUIT 3/4" ϕ, 1 LB/FT.

1" ϕ, 2 LB/FT.

2" ϕ, 7 LB/FT.

CALCULATION SHEET

CALC. NO. 8011 - 1.9 & 1.10 REV. NO. 0

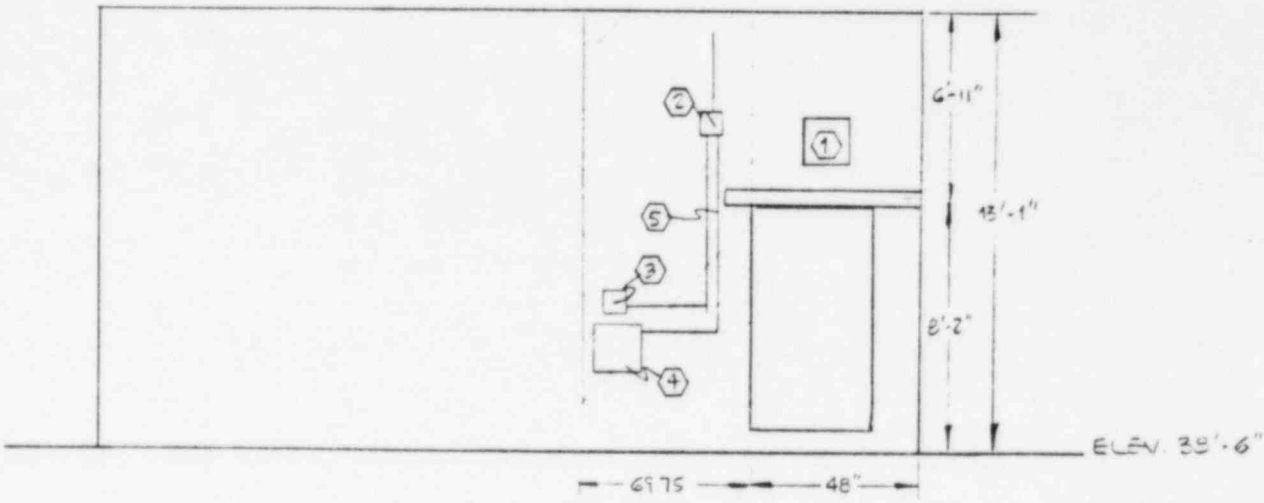
ORIGINATOR Tiffany Contelmo DATE 4.20.81 CHECKED TK DATE 4/27/81

PROJECT MILLSTONE UNIT Nº 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 5

DETERMINE WORST 'Gt' SECTION

Gt = G(11.625) = 69.75"



ELEVATION LOOKING WEST

THIS 'Gt' SECTION WAS CHOSEN AS THE CRITICAL SECTION BECAUSE THE DOOR LINTEL WILL TRANFER A LARGE MASS TO MASS PT 2 ON THE SEISMIC BEAM MODEL

(1)	TB 245	92 LB	RIGID ATTACHMENT - MASS PT. 2
(2)	T481-2A	8 LB	NEGLIGIBLE
(3)	INSTRUMENT	60 LB	RIGID ATTACHMENT - MASS PT 2
(4)	INSTRUMENT	60 LB	RIGID ATTACHMENT - MASS PT 1
(5)	3/4" Ø CONDUIT	NEGLIGIBLE	

WALL WT. ON LINTEL: $\frac{48"}{16"}$ = 3 BLOCKS ASSUME: 2 BLOCKS, 1 CELL GROUTED
1 BLOCK, NO CELLS GROUTED

$83" \left[2(.011) + (.0064) \right] = 2.37^K$

ASSUME HALF GOES TO EACH SIDE OF DOOR

WT: $\frac{2.37^K}{2} = 1.19^K$

CALCULATION SHEET

CALC. NO. 8011-1.9(1)0 REV. NO. 0

ORIGINATOR Jeffrey Confalone DATE 4.20.81 CHECKED JK DATE 4/27/81

PROJECT MILLSTONE UNIT Nº 2. JOB NO. 11867-020

SUBJECT 8311, DESIGN VERIFICATION OF BLOCKWALLS. SHEET NO. 6

EXTERNAL LOADS

DUE TO VERTICAL EARTHQUAKE (ASSUME WALL IS AXIALLY RIGID)

$\frac{69.75"}{16} = 4.36$ BLOCKS ASSUME { 3 1 CELL GROUTED; 1.36 NO CELLS GROUTED }

$w = 0.042^{k/in}$

$WT = 157" \left[3(.0111) + 1.36(.0064) \right] = 6.59^k$

ZPA ACCEL. = 0.47g (SSE WAREHOUSE $\beta = 2\%$ AVG ELEV 35'-6" & 54'-6")

AXIAL LOAD $= \left[1 \cdot \frac{2}{3}(.47) \right] (6.59^k + 1.19^k) = 10.22^k$

DUE TO JET IMPINGEMENT (ASSUME HITS @ ℄)

$P = 2(.63) = 1.26^k$

$MOMENT = \frac{1.26(157)}{4} = 49.5^{in\cdot k}$

$SHEAR = \frac{1.26}{2} = 0.63^k$

TOTAL WTS @ MASS PTS

MASS PT	1	0.06^k
MASS PT	2	$.092 + .060 + 1.19 = 1.34^k$
MASS PT	3	0.0^k

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Vffiley Cortelino DATE 4-20-81 CHECKED [illegible] DATE 4/27/81

PROJECT MILLSTONE UNIT No 2. JOB NO. 11867-020

SUBJECT 8211-DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 7

SECTION PROPERTIES

12" BLOCK GROUT W/ REBAR, #5 @ 24"

AS = 0.93 IN²
ASP = 0.0 IN²
DS = 4.31 IN
DP = ———
H = 11.625 IN
B = 69.75 IN
IUCR = 3872.8 IN⁴
ICR = 454.5 IN⁴

YCU = 4.38 IN
YTU = 5.74 IN
YCCR = 1.59 IN
YTCR = 8.54 IN
AAXIAL = 643.5 IN²
ASHEAR = 312.3 IN²
AC = 106.2 IN²

ALLOWABLE AXIAL COMPRESSIVE STRESS

$$AFMA = 540 - 0.02\left(\frac{157}{11.625}\right)^3 = 471 \text{ psi} = .491 \text{ ksi}$$

RESULTS FROM BLOCKWALL PROGRAM: (SNUM: T11151)

```
**SEISMIC MOMENT=     303.5KIPS.IN
** COMP. BENDING MASONRY STRESS GREATER THAN ALLOWABLE**
**TENSION STEEL OVERSTRESSED**
```

WALLS 1.09 & 1.10 FAIL ELASTIC CRITERIA

```
                    *****RESULTS OF ANALYSIS*****
MASONRY COMPRESSIVE BENDING STRESS=    1.2351KSI        ALLOWABLE =    1.114KSI
MASONRY AXIAL COMPRESSIVE STRESS=      .0159KSI         ALLOWABLE =     .491KSI
TENSILE STEEL STRESS=   71.9581KSI                     ALLOWABLE =  54.000KSI
COMPRESSIVE STEEL STRESS=     .0000 KSI                ALLOWABLE =  54.000KSI
MASONRY SHEAR STRESS=      .0180KSI                    ALLOWABLE =     .084KSI
MAXIMUM DEFLECTION =  1.057186 IN.
INTERACTION VALUE=1.1
```

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Kelly Cortelyou DATE 4.21.81 CHECKED ZK DATE 4/27/81

PROJECT MILLSTONE UNIT Nº 2. JOB NO. 11867-020

SUBJECT 011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 8

SINCE WALLS 1.9 & 1.10 FAIL ELASTIC CRITERIA, OPERABILITY ANALYSIS WILL BE USED. DUE TO THE FACT THAT THE WALL HAS DECKING AT ITS TOP BOUNDARY ARCHING ANALYSIS IS NOT APPLICABLE AND ENERGY BALANCE TECHNIQUE WILL BE USED.

ENERGY BALANCE:

ASSURE THAT THE SECTION IS UNDERREINFORCED
(I.E. YEILD FAILURE TAKES PLACE)

CHECK STRESSES @ FAILURE

MASONRY STRESS O.K. $< 1.114^{ksi}$

ULT MOMENT $\frac{Mc}{I} = 1.114$ $M_{ULT} = \frac{1.114\, I}{c}$

CRACKED SECTION $M_{ULT} = \frac{1.114\,(454.5)}{1.59} = 318.4^{in\text{-}k}$

STEEL STRESS

$$f_{st} = \frac{Mc(n)}{I} = \frac{318.4\,(8.54 - 4.31)\,21.9}{454.5} = 64.9^{ksi} > 60^{ksi}$$

STEEL IS OVER STRESSED, YIELD FAILURE OCCURS ENERGY BALANCE IS APPLICABLE.

CALCULATION SHEET

CALC. NO. 8011 - 1.9 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Cortelano DATE 4.21.81 CHECKED JR DATE 4/27/81

PROJECT MILLSTONE UNIT Nº 2. JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 9

FIND DUCTILITY RATIO μ

$$\mu = \frac{1}{2}\left[\left(\frac{\sigma_c}{\sigma_y}\right)^2 + 1\right] \quad < 5$$

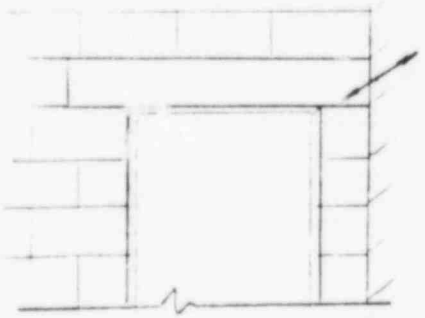
σ_c = 71.96 ksi (REF COMPUTER RUN T11151)

σ_y = 60 ksi

$$\mu = \frac{1}{2}\left[\left(\frac{71.96}{60}\right)^2 + 1\right] = 1.22 < 5 \quad \underline{OK.}$$

WALL 1.09 & 1.10 ARE O.K FOR OPERABILITY ENERGY BALANCE

CHECK FOR LOCAL FAILURE AROUND LINTEL



ASSUME EACH END OF LINTEL TAKE ½ THE LOAD

P = 1.19^k (fig. 5)

accel = 1.61 g (program T11151)

F = 1.19(1.61) = 1.92^k

A = (8×12)2 (BOTTOM & SIDE AREA)

= 192 in^2

$$\sigma = \frac{F}{A} = \frac{1.92^k}{192\ in^2} = 0.01\ ksi$$

< 0.02 ksi OK

BOTTOM CONNECTION:

V = (.010 ksi)(312.3 in^2) = 5.62^k

$\frac{69.75}{24}$ = 2.9 BOLTS

SAY 2 BOLTS $\frac{5.62}{2} < 3.96^k$ (SHEAR ALLOWABLE F.S. = 4 ¾" ⌀ WEJIT @ MIN EMB)

O.K

CALCULATION SHEET

CALC. NO. 8011-1.9 & 1.10 REV. NO. 0

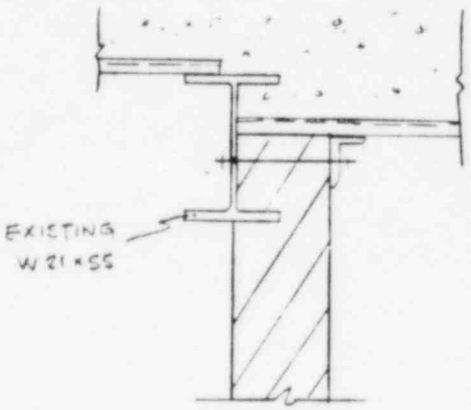
ORIGINATOR [illegible] DATE 4-22-81 CHECKED J. Wilks DATE 5-5-81

PROJECT MILLSTONE UNIT Nº 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 10

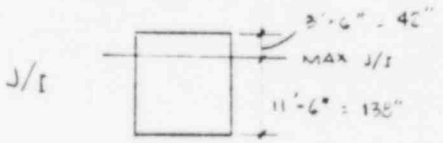
ENERGY BALANCE REQUIRES THE WALL TO BE ADEQUATELY ANCHORED TO THE SUPPORT

TOP BOUNDARY IS INADEQUATE



PROPOSED REPAIR OF TOP BOUNDARY

THROUGH BOLT WF WEB TO ANGLE



J/I $\frac{128}{180}$ (69.21) = .97^K

REACTION @ TOP

$$V = \frac{.042^K/_{IN}(157")(1.61_g)}{2} + \frac{134"(1.61_g)}{2} + \frac{1(0.060)1.61}{4} + .97 = 738^K$$

$$v = \frac{.738^K}{69.75'} = .106\ ^K/_{IN}$$ UNIFORM SHEAR @ TOP

CHECK 6 x 4 x ½" ANGLE W/ MAX SPAN OF 48"

$$M_{MAX} = \frac{106(48)^2}{8} = 305^{IN\text{-}K}$$

CONSERVATIVELY ASSUMING SIMPLE SPAN



$S_{YY} = 2.08\ in^3$

$b_f = 6"$ $t_f = 0.5"$

CHECK FOR ALLOWABLE STRESS OF $.6F_y$

MAX UNBRACED LENGTH : $\frac{76\,b_f}{\sqrt{F_y}} = \frac{76(6)}{\sqrt{36}} = 76" > 48"$ ok

$$\frac{b_f}{t_f} = \frac{6}{0.5} = 12 < \frac{76}{\sqrt{F_y}} = 12.7$$ ok

CALCULATION SHEET

CALC. NO. 8011-19 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Contolmo DATE 4.22.81 CHECKED F. Wilk DATE 5-5-81

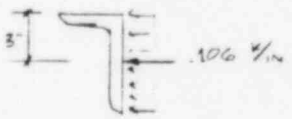
PROJECT MILLSTONE UNIT Nº 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 11

CHECK STRESS IN L

$$f_b = \frac{M}{S} = \frac{30.5}{2.08} = \underline{14.7}^{ksi} < 0.6F_y = 21.6^{ksi}$$

CHECK BENDING ON L LEG



PER INCH OF L

$M = .106(3) = 0.318^{in-k/in}$

$S = \frac{bt^2}{6} = \frac{1"(1.50)^2}{6} = .042^{in^3/in}$

$f_L = \frac{M}{S} = \frac{0.318}{0.042} = \underline{7.6}^{ksi}$ OK

CHECK 5/8" Φ THREADED ROD

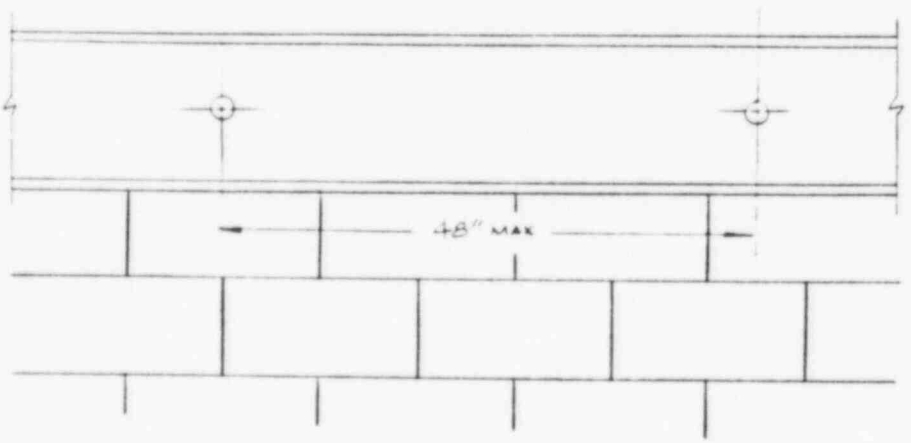
ASSUME EACH BOLT CARRIES THE LOAD FOR 24" ON EACH SIDE.

$F = .106(48") = \underline{5.09}^{k}$

NOMINAL AREA $= \frac{\pi(.625)^2}{4} = 0.307^{in^2}$

$f_T = \frac{5.09}{.307} = \underline{16.5}^{ksi} < 19.1^{ksi}$ $(.33F_u)$ (TABLE 2, APPENDIX A AISC 8TH ED.)

O.K.



ELEVATION (EASTSIDE)

CALCULATION SHEET

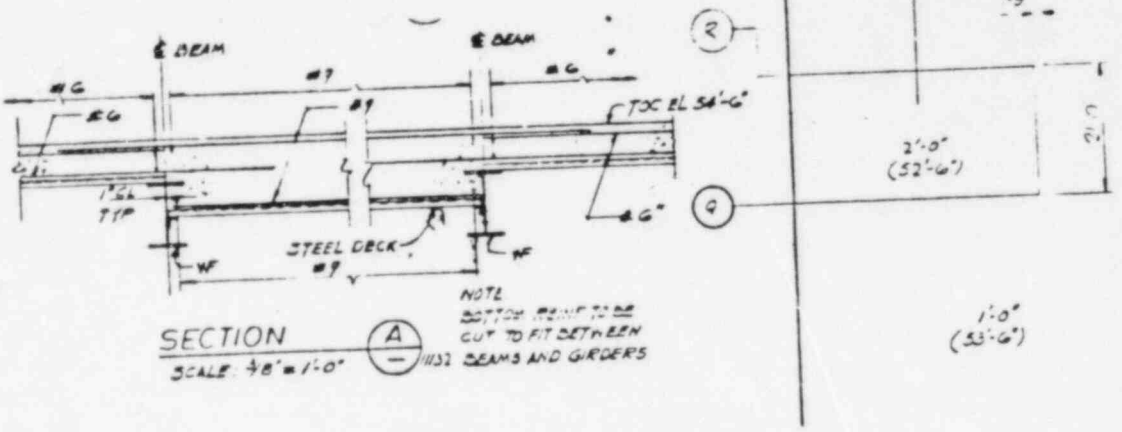
CALC. NO. 8011-1.9 ± 1.10 REV. NO. 0

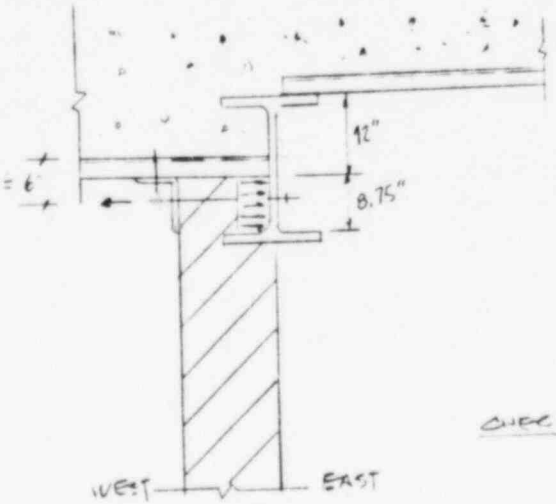
ORIGINATOR [illegible] DATE 4-22-81 CHECKED J. Mills DATE 5-5-81

PROJECT MILLSTONE UNIT NO 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALL SHEET NO. 12

CHECK STRESSES IN EXISTING WF





SECTION (LOOKING NORTH)

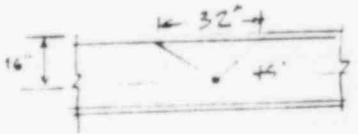
WF 21x55
$d = 20\ 3/4"$
$t_f = 1/2"$
$t_w = 3/8"$

$V = 5.09^{k}$

CHECK SHEAR (1" WIDTH) $\frac{5.09^{k}}{1 \times 3/8} = 13.6^{ksi} < 14.4^{ksi}$ $(.4F_y)$

WESTERLY LOAD (BENDING)

$M = 5.09(6") = 30.5^{in\cdot k}$



$S = \frac{32 \times (3/8)^2}{6} = 0.75\ in^3$

$f_b = \frac{M}{S} = \frac{30.5}{.75} = 40.7^{ksi} > 27^{ksi}$ N.G.

$> .9F_y = 32.4^{ksi}$

CALCULATION SHEET

CALC. NO. 8011-1.9 ₮1.10 REV. NO. 0

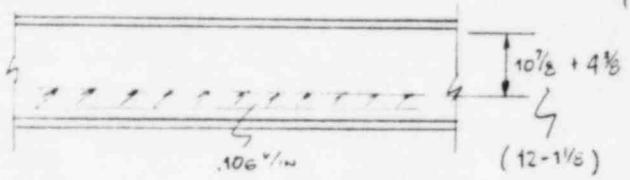
ORIGINATOR Jeffrey Castellow DATE 4.22.81 CHECKED J. Mich DATE 5-5-81

PROJECT MILLSTONE UNIT Nº 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 13

EASTERLY LOAD (BENDING)

(PER 1")



$M = .106(15.25^{\circ}) = 1.62^{IN\,K}$

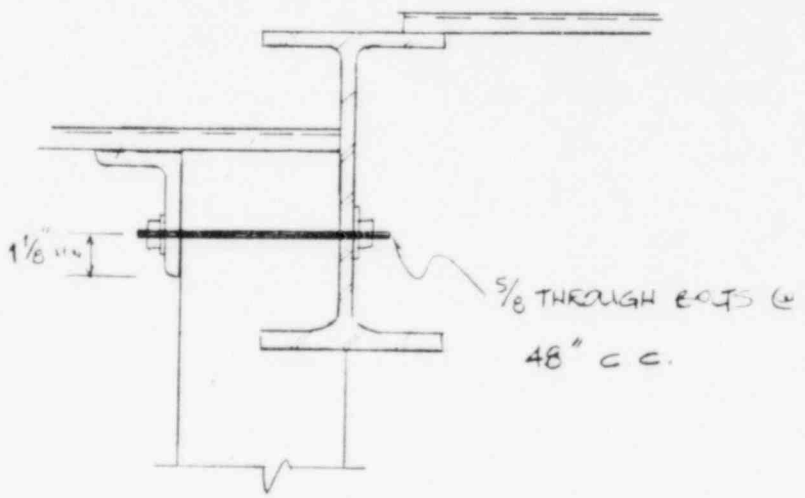
$S = \frac{1}{6}(1.375)^2 = 0.023\ IN^3$

$f_b = \frac{M}{S} = \frac{1.62\ IN^3}{.023\ IN^3} = 70.4\ KSI$

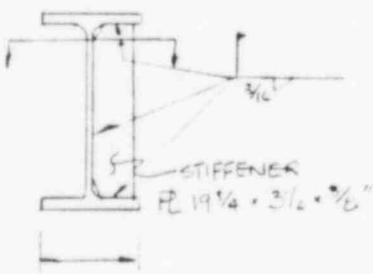
N.G.

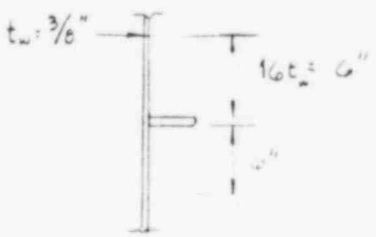
INCREASED ALLOWABLE FOR SEISMIC LOAD.

$1.5\ f_{allow} = 1.5(21.6) = 32.4\ KSI$



PROVIDE SUPPORT FOR WF WEB; ADD STIFFENERS





CALCULATION SHEET

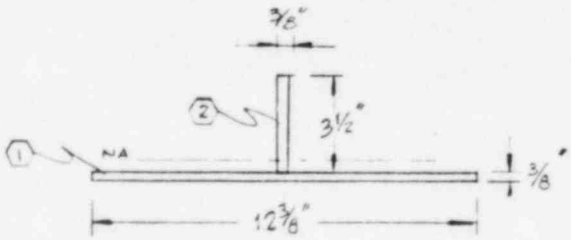
CALC. NO. B011-1.9&1.10 REV. NO. 0

ORIGINATOR [illegible] DATE 4-22-81 CHECKED J. Smith DATE 5-5-81

PROJECT MILLSTONE UNIT No. 2 JOB NO. 11867-020

SUBJECT B011 DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 14

FIND N.A. AND MOMENT OF INERTIA.



N.A. FROM BASE

	A	$\bar{y}$	$A\bar{y}$
①	.375(12.375) = 4.64	.1875	0.87
②	.375(3.5) = 1.313	2.125"	2.79
Σ	5.953 in²		3.66 in³

$\bar{Y} = \frac{3.66}{5.953} = 0.61"$

$C = 3.5 + .375 - .61 = 3.265"$

$$I = \frac{(.375)(3.5+.375-.61)^3}{3} + \frac{(.375)(0.61-.375)^3}{3} + \frac{(12.375)(.375)^3}{12} + (12.375 \times .375)\left(0.61 - \frac{.375}{2}\right)^2$$

$= 4.351 + .002 + .054 + .828 = 5.235 \text{ in}^4$

IRY SPACING STIFFENERS @ 24" C.C

WESTERLY LOAD (LOADS OUT)

$M = 5.09\,(6 - 1.125 + 2) = 35.0^{\text{in-k}}$

$f_b = \frac{Mc}{I} = \frac{35.0\,(3.265")}{5.235 \text{ in}^4} = 21.8^{\text{ksi}} < 27^{\text{ksi}}$ OK

EASTERLY LOAD (EFFECTIVE WIDTH ON STIFFENER = 24")

$M = 1.62 \frac{\text{in-k}}{\text{in}}\,(24") = 38.9^{\text{in-k}}$

↳ PG 13

$f_b = \frac{Mc}{I} = \frac{(38.9)(3.265)}{5.235} = 24.3^{\text{ksi}} < 27^{\text{ksi}}$ O.K.

CALCULATION SHEET

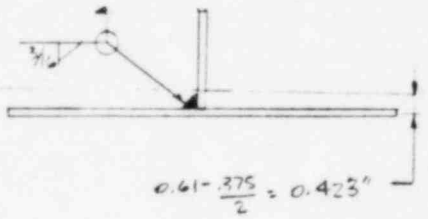
CALC. NO. 2011-1.9 & 1.10 REV. NO. 0

ORIGINATOR Jeffrey Castellano DATE 4-22-81 CHECKED F. Mich DATE 5-5-81

PROJECT MILLSTONE UNIT Nº 2 JOB NO. 11367-020

SUBJECT 2011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 15

CHECK 3/16" WELD ON ONE SIDE OF STIFFENER



V = 5.09

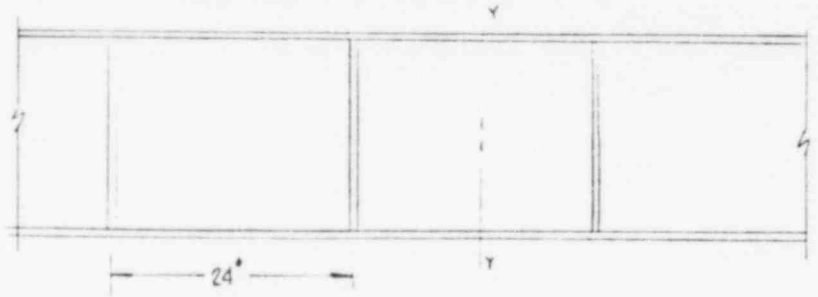
Q = (12.375 × .375) 0.423 = 1.96

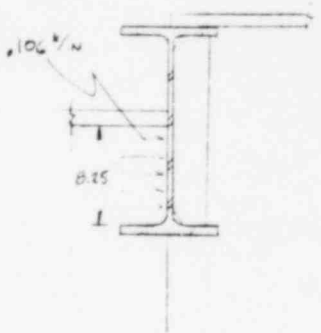
I = 5.245

t = 3/16

$$f = \frac{VQ}{It} = \frac{5.09(1.96)}{5.245(3/16)} = \underline{10.14\ ^{ksi}} < .3(70) = 21^{ksi} \quad O.K.$$

CHECK WEB BETWEEN STIFFENERS





BENDING ABOUT Y-Y

$$M = \frac{.106\ ^{k}/_{in}\ (24")^2}{12} = \underline{5.09\ ^{in\text{-}k}}$$

$$S = \frac{8.25(.375)^2}{6} = \underline{.193\ in^3}$$

$$f_b = \frac{M}{S} = \frac{5.09}{193} = \underline{26.4\ ^{ksi}} < 27^{ksi} \quad Ok.$$

CALCULATION SHEET

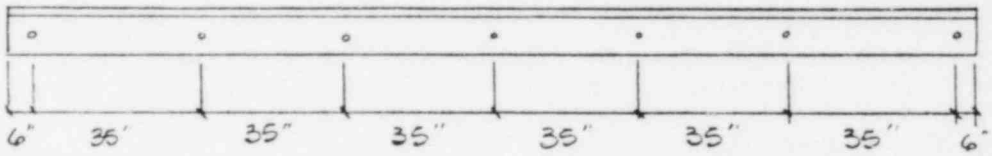
CALC. NO. 8011-J-1.9.110 REV. NO. 0

ORIGINATOR R E Rochlin Jr. DATE 5/28/81 CHECKED [illegible] DATE 5/28/81

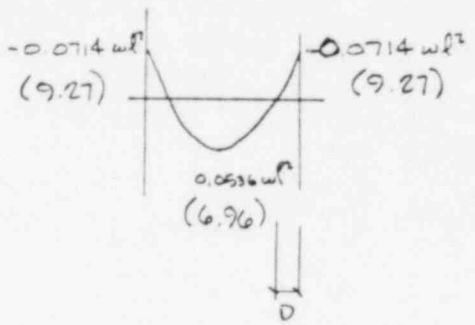
PROJECT MILLSTONE UNIT II JOB NO. 11867-020

SUBJECT WALL 1.9 & 1.10 SHEET NO. 16

WELD LOCATION



REF STEEL MANUAL PG 2-127 (39)



$w = 0.106$ k/in

$l = 35$

$$M = \frac{wlx}{2} - \frac{wx^2}{2}$$

$$-9.27 + \frac{(0.106)(35)(D)}{2} - \frac{(0.106)(D^2)}{2} = 0 \qquad 0.053D^2 - 1.855D + 9.27 = 0$$

$D = 6.04"$

ANGLE CUT @ $6" + 35" + 35" + (35" - 6.04") = 104.96"$

USE 8'-9"

3/8" PARTIAL PENETRATION WELD

CALCULATION SHEET

CALC. NO. 8011-J-1.9,1.10 REV. NO. 0

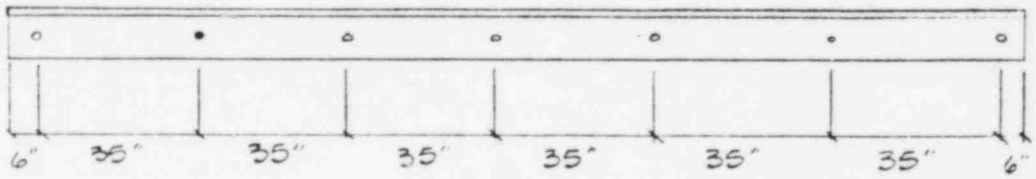
ORIGINATOR R. Goodhue Jr DATE 5/28/81 CHECKED TH DATE 5/28/81

PROJECT MILLSTONE II JOB NO. 11867-020

SUBJECT WALL 1.9 & 1.10 SHEET NO. 17

REF CALC. 8011-1.9 & 1.10

REPAIR CALLS FOR A 17'-6" ANGLE WITH 5 SPANS
CHANGE TO 6 SPANS AS FOLLOWS:



DROP ANGLE 2" BELOW TOP OF WALL ON WEST SIDE

SEE PAGE 14 OF REF CALC.

WESTERLY LOAD (BENDING)
MOMENT CHANGE FOR 2" DROP AND SHORTENED SPAN

$$M = (5.09)\left(\frac{35}{48}\right)(8 - 1.125 + 2) = 32.94\text{"K}$$ ← MULTIPY BY 1.1 FOR CONT. SPAN

$$= (32.94)(1.1) = 36.2\text{"K}$$

$$f_b = \frac{Mc}{I} = \frac{(36.2)(3.265)}{5.235} = 22.6 \text{ KSI VS. } 27 \text{ KSI} \quad \text{OK}$$

Attachment 2 consists of a 19 page (sheets 0-16) example of a multiwythe wall, arching action, block pullout, and jet impingement analysis.

CALCULATION COVER SHEET

PROJECT Millstone Unit 2 JOB NO 11867-020 CALC NO 8011 – 1.21

SUBJECT USNRC I.E. Bulletin 80-11 Design Verification of Block Wall 1.21

ORIGINATOR [signature] DATE 6 JULY 81

CHECKER Thomas Lavergne / [signature] DATE 7.7.81 NO. OF SHEETS 11

RECORD OF ISSUES

NO.	DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE	ACCEPTANCE	DATE
0	SHEETS 1 TO 12	TS	3.9.81	TL	3-9-81	[signature]	2/1/82		
↓	13 TO 16	R.M.	6 JULY 81	KLB	7781	↓			
1	ADDED PGS 12A & 12B.	RM	4/6/82	TL	4/7/82	[signature]	4-8-82		

☐ PRELIMINARY CALC. ☐ COMMITTED PRELIMINARY CALC.

☐ SUPERSEDED CALC. ☒ FINAL CALC.

SPD-2104 (9/76 IED-11)

BECHTEL

CALCULATION SHEET

CALC NO 8011-1.21 REV NO 0

ORIGINATOR Joe Anz Abu DATE 3/9/81 CHECKED TL DATE 3-9-81

PROJECT Millstone Unit 2 JOB NO 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO 1

DESIGN CALCULATION CHECKLIST

Load Description

	Load Sheet Date	Description
1. Plant Design	12/16/80	PIPE LOADS
2. Electrical:	12/3/80	CONDUITS TERMINAL BOX
3. Mechanical:	10/1/80	J/I IMPINGEMENT FORCE DIFFERENTIAL PRESSURE
4. Control Systems:	8/25/80	NONE

Wall Description

1. Drawing Details: 25203-•11094 (DETAIL 6)
2. Masonry Type: SOLID . 6 X 8 X 12
3. Wall Thickness: 24 IN.
4. Number of Wythes: 4 WYTHES
5. Rebar Size and Spacing: N/A
6. Latest Page Revision of Survey Data and Date: 10/29/80 REVISION 1
   MORTAR ; NON COMP.
7. Boundary Connection Detail: 3/4" Ø WET-ITS @ 18 IN AT SIDES ; COMP.
8. Safety-Related Attachments or II/I: SR AND SR II/I

* Note: The most recent mechanical load sheet noted 4/2/81 depicts the [illegible] load [illegible] the [illegible] load used in this calc.

CALCULATION SHEET

CALC NO. 8011-1.21 REV. NO. 1

ORIGINATOR Terry Shu DATE 3/7/81 CHECKED TL DATE 3-9-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 2

Summary of Results

1. Governing Load Combination: $D+L+E'+F_p$ (COMP)
   * NON - COMP. SECTION FAILS BY INSPECTION
2. Boundary Condition Assumed: SIMPLY SUPPORTED (COMP)
3. Horizontal or Vertical Span Checked: HORIZONTAL SPAN (COMP)
4. In-Plane Shear Strain vs. Allowable: 0.00007/0.00020 (S.S.E)
5. Collar Joint Shear Stress vs. Allowable: 0.009 KSI / 0.013 KSI (COMP.)
6. Tensile ~~Steel~~ MASONRY Stress vs. Allowable: 0.043 KSI / 0.040 KSI (COMP)
   * THE STRESS IS ACCEPTABLE. SEE NOTE ON PG 9
7. Masonry Compressive Stress or Interaction Value vs. Allowable: 0.043 KSI / 0.438 KSI (COMP)
8. Masonry Shear Stress vs. Allowable: 0.006 KSI / 0.020 KSI (COMP)
9. Cracked or Uncracked Section: UNCRACKED (COMP)
10. Boundary Connections, Pass/Fail: PASS (COMP)
11. Block Pullout, Shear Stress vs. Allowable: 0.022 KSI / 0.020 KSI
    * THE STRESS IS ACCEPTABLE. SEE NOTE ON PG 11
12. Maximum Displacement Elastic Analysis: IT IS ACCEPTABLE BY INSPECTION
13. Repair, Required/Not Required: N/A
14. Repair Type: N/A
15. Arching, H_a vs. H: 49.0 k < 50.28 k (NON - COMP VERTICAL SPAN) 1 4/9/81
16. Maximum Displacement Arching Action: 0.117 IN (NON - COMP. VERTICAL SPAN)

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tai Sung Soh DATE 3/9/81 CHECKED TL DATE 3-9-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 3

17. Rigid or gapped arching used: RIGID

18. Vertical or horizontal arching span used: VERTICAL

19. Assumed location of crack: CENTER

20. Any cracks along boundaries, yes/no: YES

21. Frequency of arched wall: USED PEAK ACC.

22. Is shear friction sufficient to transfer the shear from arching to the boundaries, yes/no: YES

23. Stiffness of boundary conditions: ALL FIXED

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Jas Sing Lohia DATE 11/15/81 CHECKED TL DATE 1-16-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 4

REFERENCES:

- [X] 1) Specification for Re-evaluation of Concrete Masonry Walls - 11867-020-C003, Rev. 1.
- [X] 2) Survey Procedure No. 11867-020-C002, Rev. 5, (Including Survey Data).
- [X] 3) Loading Evaluation Procedure No. 11867-020-C004, Rev. 0, (Including Load Sheets).
- [ ] 4) Floor Response Spectra-Auxiliary Building Calculation No. 8011-001, Rev. 0.
- [ ] 5) Floor Response Spectra-Turbine Building Calculation No. 8011-002, Rev. 0.
- [ ] 6) Masonry Block Section Properties Calculation No. 8011-003, Rev. 0.
- [ ] 7) Blockwall Pipe Hanger Load Summary Calculation No. 8011-004, Rev. 0.
- [X] 8) Average Floor Response Spectra for Successive Floors In Auxiliary Building - Calculation No. 8011-005, Rev. 0.
- [X] 9) Millstone Unit 2, Seismic Report.
- [X] 10) Civil Design Aid No. 8011-01-GPD, Rev. 0.
- [ ] 11) Bechtel 'Blockwall' Program CE-020 (Version D).
- [ ] 12) Design Response Spectra, Pgs. E1 & E2 of FSAR for Millstone Unit 2.
- [X] 13) Damping Values for Wall Attachments, Pg. E3 of FSAR for Millstone Unit 2.

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tai Sung Soh DATE 11/15/81 CHECKED TL DATE 1-16-81

PROJECT THE MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 5

° REFERENCES

CIVIL DRAWING ; 25203 – 27015
25203 – 11112
25203 – 11094 (SECT 6)

T.E.S TECHNICAL REPORT # TR-3501-2

▽* FOR VERTICAL ARCHING I'VE USED JET FORCE OF 0.75 K SINCE THIS IS REVISED VALUE

° EXTERNAL LOADS

CONTROL SYSTEM ; NONE

ELECTRICAL ; CONDUITS 2"Ø (7#/FT)
1"Ø (2#/FT)
TERMINAL BOXES 8"X6"X4" = 8#
1'-2"X1'-0"X6 5/8" = 26#

MECHANICAL LOAD ; J/I IMPINGEMENT FORCE – 15.0 K ▽*
D_{MINOR} = 9.11 in
DIFFRENTIAL PRESSURE – 1.4 PSI

PLANT DESIGN ; 4 3"X3"X 1/2" X 1'-6" SUPPORTING

PIPE LOADS APPLIED IN ANY DIRECTION
- 1" Ø C3 PIPE AND 8"X10"X 3/8" PL – 131 lb
- 4 1 1/2" X 1 1/2" X 3/16" X 1'-9" SUPPORTING 1 1/2" Ø COPPER AND 6"X8"X PL – 212 lb

1/2" Ø COPPER – 45 lb 3-3/4" Ø COPPER – 70 lb EACH

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Jac Jung Lo DATE 1/15/81 CHECKED TL DATE 1-16-81

PROJECT THE MILLSTONE UNIT JOB NO. 11867-020

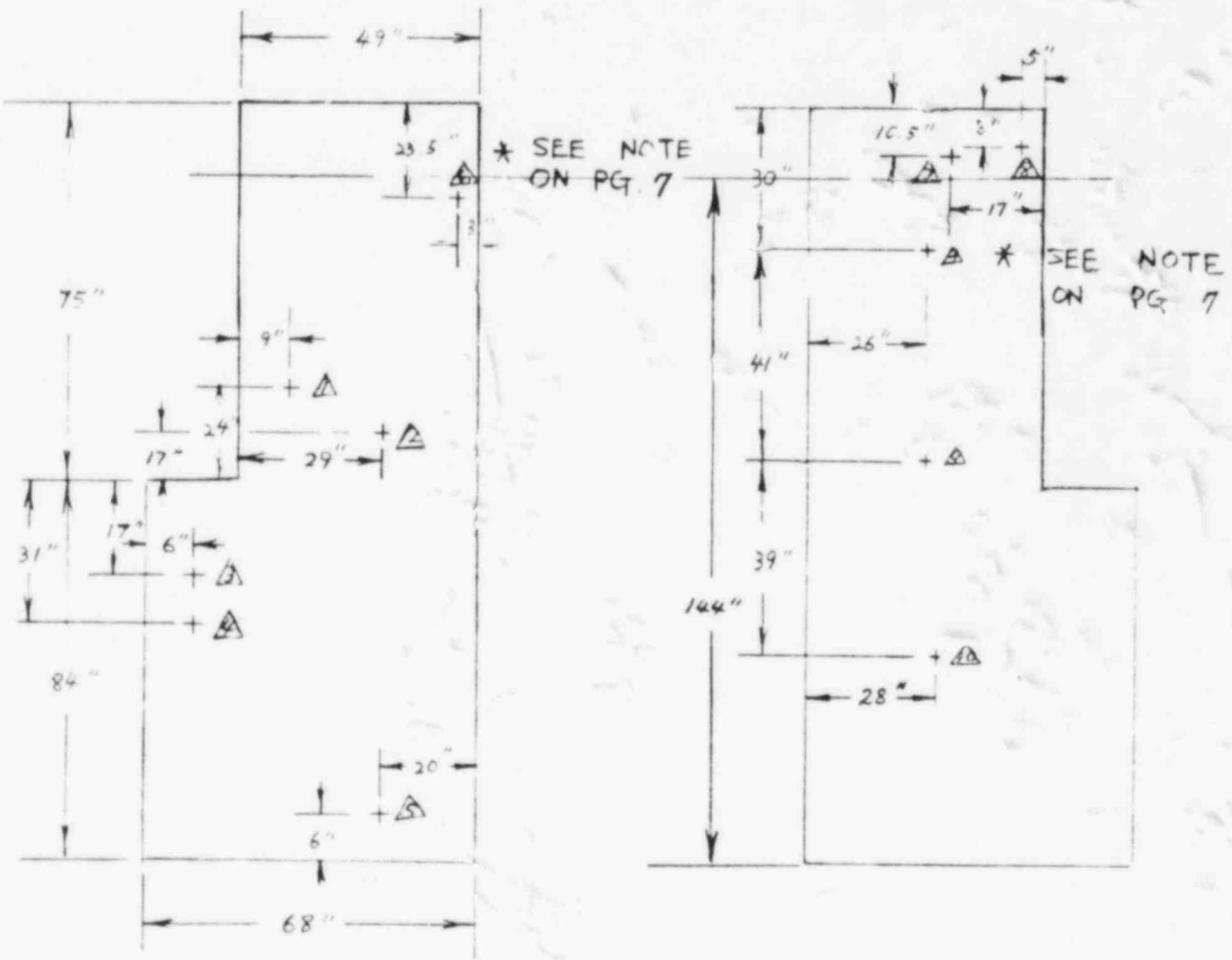
SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 6

- DIRECTION OF THE BLOCK WALL — EW (REF 9)

- DESCRIPTION

  THIS WALL IS LOCATED AT ELE 38 FT 6 IN. IN AUX. BUILDING. DIMENSIONS OF THE BLOCK WALL ARE 5 FT 8 IN. IN WIDTH, 7 FT AND 13 FT 3 IN. IN HEIGHT AND 24 IN. IN THICKNESS.
  TYPICAL BLOCK IS 6 X 8 X 12. SOLID.

- BLOCK WALL ELEVATION (REF 7)



CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tse Ang Shu DATE 11/15/81 CHECKED TL DATE 1-16-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 7

- BOUNDARY CONDITION

  ALL SIDES OF THE WALL ARE SUPPORTED BY Ls 4 x 3 x $\frac{1}{4}$ WITH $\frac{3}{4}$" Ø WEJ-ITS @ 18" (TYP).
  ASSUME THE WALL IS SIMPLY SUPPORTED AT ALL SIDES.

- MATERIAL PROPERTIES

  f_m' = 1500 PSI

  γ_m = 145 PCF

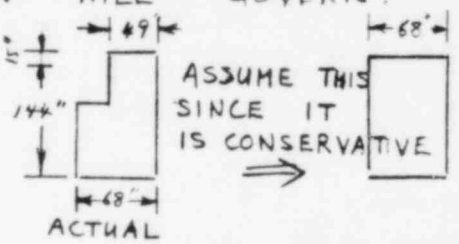
  E_m = 1500 KSI

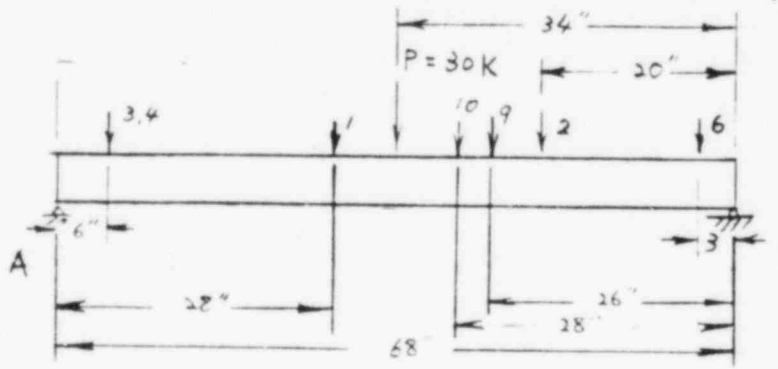
- LOAD COMBINATION

  SINCE THE WALL HAS A LARGE J/I IMPINGEMENT FORCE, D+L+F_p+E' COMBINATION WILL GOVERN.

- ANALYSIS OF THE BLOCKWALL.

  CHECK 144" WIDE STRIP SPANNING 68" (IT'S CONSERVATIVE)





* NOTE; THE LOADS 6 & 9 ARE LEFT OUT IN THE CALC. BUT THE CALC. IS O.K, SINCE THESE VALUES DO NOT CHANGE THE RESULT SIGNIFICANTLY

1. 2"Ø CONDUIT (7#/FT - 5 FT) = 35 lb
2. 1"Ø CONDUIT (2#/FT - 8FT) + 2"Ø CONDUIT (7#/FT - 8FT) = 72 lb
3, 4. 26 + 8 = 34 lb
6. 2 - 2"Ø CONDUIT (7#/FT - 5.5 FT) = 77 lb
7. 212 lb
8. 131 lb

} HANGER LOADS

9. (45 lb) + 3(70 lb) = 255 lb EACH 10, 45 lb

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tse Lung Shu DATE 1/15/81 CHECKED TL DATE 1-16-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 8

DUE TO THE INERTIA LOADING (COMPOSITE SECTION)
FIND THE FREQUENCY

$$f = \frac{\pi}{2(\ell)^2}\sqrt{\frac{EIg}{\gamma A}} = \frac{\pi}{2(68)^2}\sqrt{\frac{1.50\times10^6(165888)(386.4)\ 1728}{145\ (144)\ (24)}} = 195.6\ \text{CPS}$$

$$I = \frac{bh^3}{12} = \frac{144\ (24)^3}{12} = 165,888\ \text{IN}^4$$

FROM 2% DAMPING S S.E

$a = 0.51\ g$ (HORIZONTAL ACCE.)

UNIFORM LOAD $= 0.51\ (0.145)(144)(24)/1728 = 0.148\ \text{K/IN}$

MAX SEISMIC MOMENT $= \frac{W\ell^2}{8} = \frac{0.148(68)^2}{8} = 85.54\ \text{K-IN}$

MAX SHEAR $= \frac{W\ell}{2} = \frac{0.148(68)}{2} = 5.032\ \text{K}$

VERTICAL ACCE.

SINCE THE WALL IS RIGID

$A = 1 + \frac{2}{3}(0.51) = 1.34\ g$

TOTAL LOAD $= 1.34\ (0.145)[(84)(68) + (49)(75)](24)/1728 = 25.33\ \text{K}$

* NOTE: SINCE WE DON'T USE THE BLOCKWALL PROGRAM, WE MUST CONSIDER THESE AS EXTERNAL LOADS.

DUE TO THE EXTERNAL LOADS,

USE THE SAME HORI. ACCE. AS THE WALL FOR THE CONDUITS AND THE TERMINAL BOXES

	HORIZONTAL ACCE.	VERTICAL ACCE. (2/3 X PEAK ACCE.)
9	0.255 K	0.255 K
10	0.045 K	0.045 K
* 1	0.51 (0.035) = 0.018 K	$[1+\frac{2}{3}(4.28)](0.035) = 0.135$ K
2	0.51 (0.073) = 0.037 K	3.85 (0.073) = 0.277 K
3,4	0.51 (0.034) = 0.017 K	3.85 (0.034) = 0.131 K
6	0.51 (0.077) = 0.039 K	3.85 (0.077) = 0.296 K
7	0.212 K	0.212 K
8	0.131 K	0.131 K

SUM THE MOMENT ABOUT A

$$\circlearrowleft + \Sigma M_A = 0 = -0.017(6) - 0.018(28) - 30(34) - 0.045(40) - 0.255(42) - 0.037(48) + R_B(68)$$

$R_B = 15.219$ K $\quad R_A = 15.153$ K ↑

CALCULATION SHEET

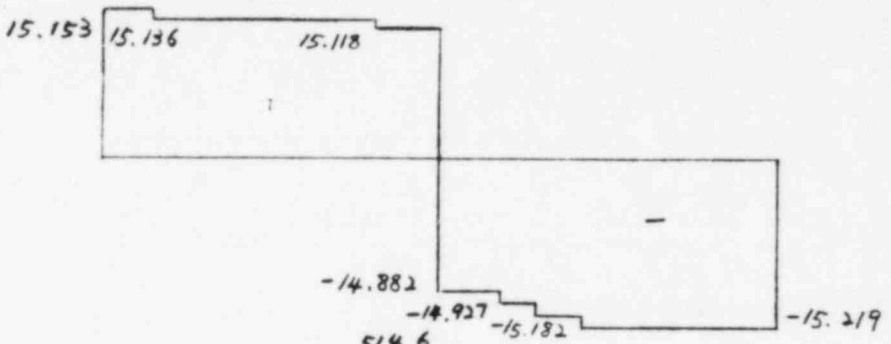
CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR [signature] DATE 1/15/81 CHECKED TL DATE 1-16-81

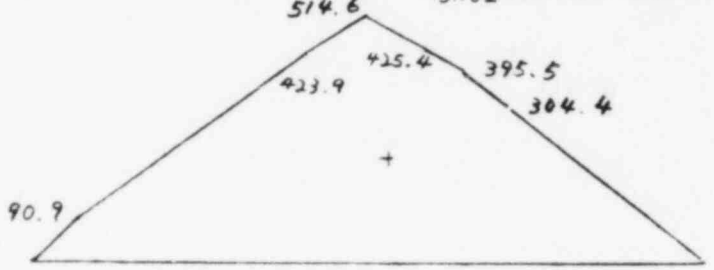
PROJECT MILLSTONE UNIT 2 JOB NO. 11267-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 9

SHEAR



MOMENT



MAX MOMENT = 85.54 + 514.6 = 600.1 K-IN

MAX SHEAR = 15.219 + 5.032 = 20.251 K

TOTAL AXIAL = 25.33 + 0.255 + 0.045 + 0.135 + 0.277 + 0.131 + 0.296 + 0.212 + 0.131 = 26.712 K

CALC. THE STRESSES

$f_{COM} = \frac{MC}{I} = \frac{600.1\,(12)}{165888} = 0.043$ KSI < 0.438 KSI O K

$f_{TEN} = 0.043$ KSI * $\simeq 0.040$ KSI

$f_{SHEAR} = \frac{V}{bd} = \frac{20.251}{144\,(24)} = 0.006$ KSI < 0.020 KSI O K

$f_{AXIAL} = \frac{26.712}{68\,(24)} = 0.016$ KSI < 0.350 KSI O K.

$f_{SHEAR\ IN\ JOINTS} = \frac{VQ}{Ib} = \frac{20.251\,(144)(12)(16)}{165888\,(144)}$

$= 0.009$ KSI < 0.013 KSI O K

* NOTE; THE TENSILE STRESS IS ACCEPTABLE SINCE WE USED TOO CONSERVATIVE D.L.F OF 2 FOR THE J/I IMPINGEMENT FORCE.

CALCULATION SHEET

CALC. NO. 8011-121 REV. NO. 0

ORIGINATOR Tae Jung Sohn DATE 1/15/81 CHECKED TL DATE 1-16-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL SHEET NO. 10

CHECK THE ANCHOR BOLT STRENGTH.

ALLOWABLE = 3.969 K

NUMBER OF BOLTS IN 6t

$= \frac{144 - 18}{18} = 7$

MAX SHEAR = 20.251 K (FOR 68" SPAN)

SHEAR ON A BOLT = $\frac{20.251}{7}$ = 2.89 K < 3.969k O.K

CHECK THE IN-PLANE STRAIN (S.S.E.)

$\gamma = \frac{\delta}{h} = \frac{0.0011}{16}$ 0.00007 (O.B.E) < 0.00012 < 0.00020 (S.S.E) O.K

δ = RELATIVE DISPLACEMENT BET ELE 38'-6" & 54'-6" = 0.0011 FT

h = DIFFERENCE OF TWO ELE 38'-6" & 54'-6"
= 16 FT.

SINCE THE STRAIN OF S.S.E < THE ALLOWABLE OF O.B.E. THE STRAIN OF O.B.E IS O.K TOO.

NON-COMPOSITE SECTION.

SINCE THE 68" COMPOSITE SECTION'S TENSILE STRESS IS VERY CLOSE TO THE ALLOWABLE, NON-COMPOSITE SECTION WILL FAIL AT LEAST IN TENSION.

THEREFORE WE MUST CHECK THE ARCHING ACTION FOR THE NON-COMPOSITE SECTION

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tue Aung Min DATE 1/15/81 CHECKED TL DATE 1-16-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL 1.21 SHEET NO. 11

- BLOCK PULL OUT

$D_{MINOR} = 9.11$ IN

ASSUME J/I FORCE AFFECT TWO BLOCKS.

$$\tau = \frac{P}{(16+12)(2)\,24} = \frac{30}{(28)(2)(24)} = 0.022 \text{ KSI} \simeq 0.020 \text{ KSI}$$

$P = 30$ K

* BUT, SINCE WE USED A CONSERVATIVE D.L.F. VALUE, THE PUNCHING SHEAR STRESS IS ACCEPTABLE.

- ARCHING ACTION

(36" STRIP SPANNED HORIZONTALLY)

ASSUME THE RIGID ARCHING (S.S.E.)

DETERMINE f_1

$$f_1 = K\left[f_c'(a) + f_t(t - Ka)\cos 45^\circ\right]$$

$$= (1)\left[1275(1) + 232(6-(1)(1))\,0.707\right] = 2.095 \text{ K/IN}$$

$f_c' = 0.85(1500) = 1275$ PSI

$f_t = 6\sqrt{1500} = 232$ PSI

$a = 1$ IN $\quad t = 6$ IN. $\quad K = 1$

DETERMINE ALLOWABLE AXIAL H

$H = f_1(36)/1.5 = 50.28$ K

DUE TO ITS OWN INERTIA TO BE ON THE CONSERVATIVE SIDE, USE PEAK ACCE USING 10% DAMPING. S S E AVE OF EL. 38'-6" & EL 54'-6" OF AUX BUILDING

$a = 1.66\,g$

$$P_a = a\,\gamma A/g = 1.66(0.145)(36)(6)/1728 = 0.030 \text{ K/IN}$$

$$H_a = \frac{P_a \ell^2}{8t} = \frac{0.03(68)^2}{8(6)} = 2.89 \text{ K}$$

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 0

ORIGINATOR Tse Ang Shu DATE 3/6/81 CHECKED TL DATE 3-9-81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL SHEET NO. 12

DUE TO THE EXTERNAL MOMENT WITHIN 36" STRIP, THE LOADS 3, 4, 10 & JET FORCE ARE ONLY FORCES EXISTING.

AND JET FORCE CAN BE REDUCED TO $^{30}/_{4}$ K, SINCE THERE ARE 4 COURSES AND EACH COURSE CAN TAKE $\frac{30}{4}$ K.

IF WE PLACE ALL THE LOADS AT THE CENTER OF THE SPAN, THEN THE MAX MOMENT WILL BE;

$$\text{MAX MOMENT} = \frac{Pl}{4} = \frac{(7.5 + 0.017 + 0.045)(68)}{4} = 128.55 \text{ K-IN}$$

DETERMINE P_a

$$P_a = \frac{8M}{l^2}$$

DETERMINE H_a

$$H_a = \frac{P_a l^2}{8t} = \frac{8M l^2}{l^2 8t} = \frac{M}{t} = \frac{128.55}{6} = 21.43 \text{ K}$$

TOTAL $H_a = 21.43 + 2.89 = 24.32 \text{ K} < H = 50.28 \text{ K}$ O.K

CONCLUSION

THEREFORE THE WALL PASSES COMPOSITELY AND NON - COMPOSITELY,

$$\Delta = \frac{U_a l}{2t} = \frac{H l^2}{2AE t} = \frac{50.28(68)^2}{2(36)(6)(1.5\times10^4)6} = 0.059 \text{ IN} < 0.3t = 1.8 \text{ IN}.$$

$$U_a = \frac{Hl}{AE}$$

O.K

FRICTION = 0.7 H_a = 0.7 (24.32) = 17.02K

SHEAR = $\frac{P_a l}{2} = \frac{(8.03 + 0.222)(68)}{2} = 8.57 \text{ K} <$ FRICTION = 17.02 K

THE SLIDING WOULD NOT OCCUR.

CALCULATION SHEET

CALC. NO. 8011-121 REV. NO. 1

ORIGINATOR Jeffrey Castillo DATE 4/6/82 CHECKED TL DATE 4/7/82

PROJECT MILLSTONE UNIT No 3 JOB NO. 11867-020

SUBJECT 80-11, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 12a

CHECK VERTICAL ARCHING (BOUNDARIES ∞ STIFF ~~Pgs~~ 13-16)

FROM PAGE 11

H_a ALLOWABLE = 50.28^k (for 36" STRIP)

SEISMIC LOAD

PEAK ACCELERATION, 10% DAMPING, SSE
AVERAGE EL 35'-6" & 54'-6"

$a = 1.66g$

$P_a = .030^{k/in}$ (see pg. 11)

$$H_a = \frac{P_a L^2}{8t} = \frac{.030(159)^2}{8(6)} = 15.8^k$$

EXTERNAL LOAD

CHOOSE WORSE PIPE BREAK LOAD

• JET IMPINGEMENT

$P = 2.0 \times .63 = 1.26^k$

$$M_{MAX} = \frac{PL}{4} = \frac{1.26(159)}{4} = 50.1^{in.k}$$

• DIFFERENTIAL PRESSURE

$q = 1.4^{psi}$

$w = 1.4^{psi}(36") = 50.4^{lb/in}$

$$M_{MAX} = \frac{wL^2}{8} = \frac{.0504(159)^2}{8} = 159.3^{in.k}, \text{ CONTROLS}$$

• ATTACHMENTS (see pg. 8)

$P = 1.66g(.035 + .072 + .034 + .077) + .212 + .131 + .255 + .045 = 1.00^k$

ASSUME TOTAL LOAD AT CENTER

$$M = \frac{PL}{4} = \frac{1.00(159)}{4} = 39.8^{in.k}$$

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 1

ORIGINATOR [illegible] DATE 4/6/82 CHECKED TL DATE 9/7/82

PROJECT MILLSTONE UNIT No. 2 JOB NO. 11867-020

SUBJECT 8011, DESIGN VERIFICATION OF BLOCKWALLS SHEET NO. 126

TOTAL MOMENT

EXTERNAL M_T = $^{\text{PIPE BREAK}}$ 159.3 + $^{\text{ATTACHMENTS}}$ 39.8 = 199.1 in-k

$P = \frac{8M}{l^2}$ $\quad H_a = \frac{PL^2}{8t} = \frac{M}{t}$

$H_a = \frac{199.1}{6} + 15.8^k = 49.0^k < 50.29^k$ O.K

∴ VERTICAL ARCHING IS O.K.

GPO 2705 212

BECHTEL

CALCULATION SHEET

CALC NO 8021-1.21 REV NO 0

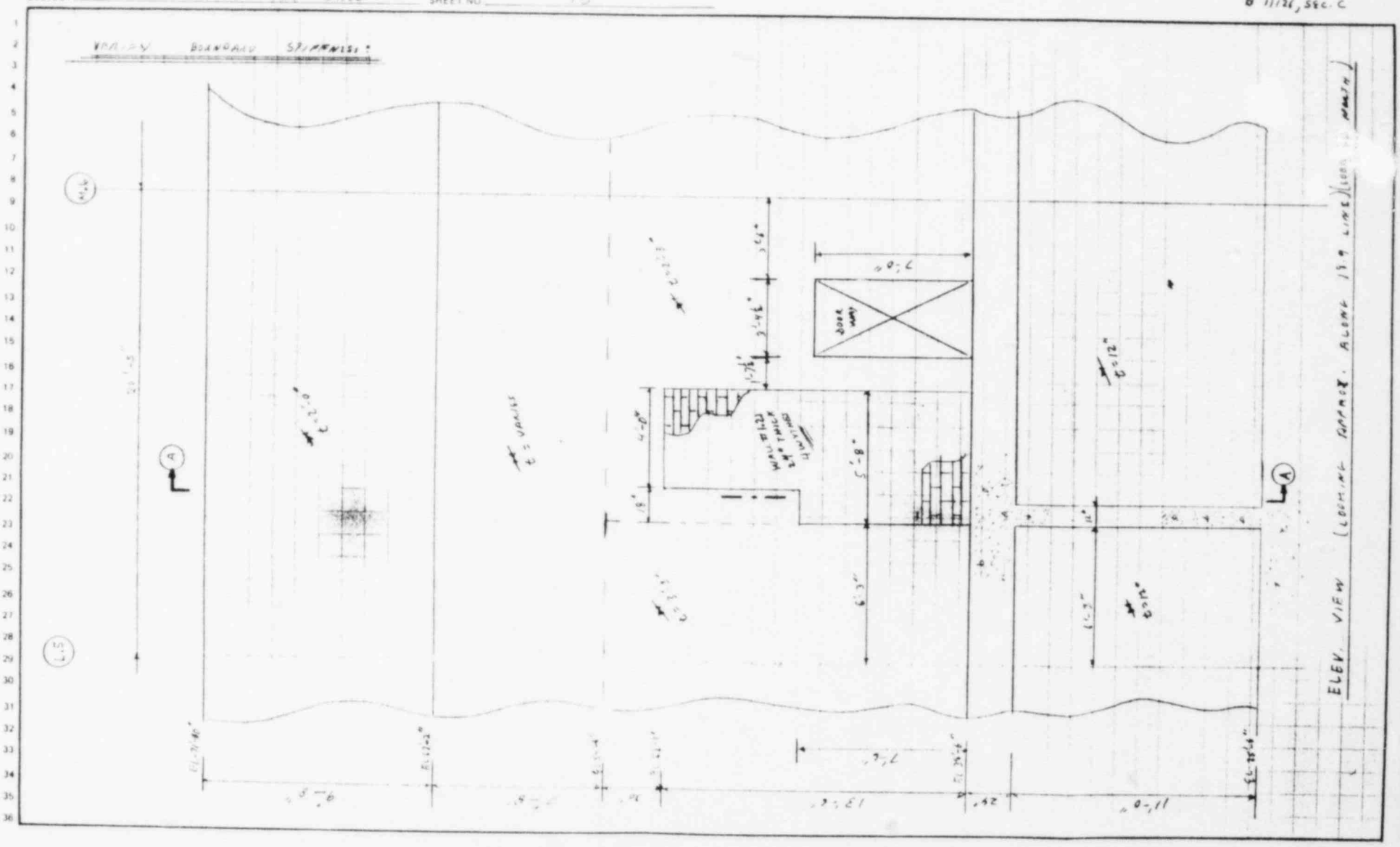
ORIGINATOR R Pham DATE 6 24 81 CHECKED KLB DATE 7.7.81

PROJECT MILLSTONE UNIT # 2 JOB NO 12827-020

SUBJECT ANALYSIS OF TAS WALL SHEET NO 13

NOTES: RSB. DWG. # 11111, SECT. D
1112
11115, SECT. F
11114
11130
11126, SEC. C

VARIOUS BOUNDARY SPRINGS:



BECHTEL

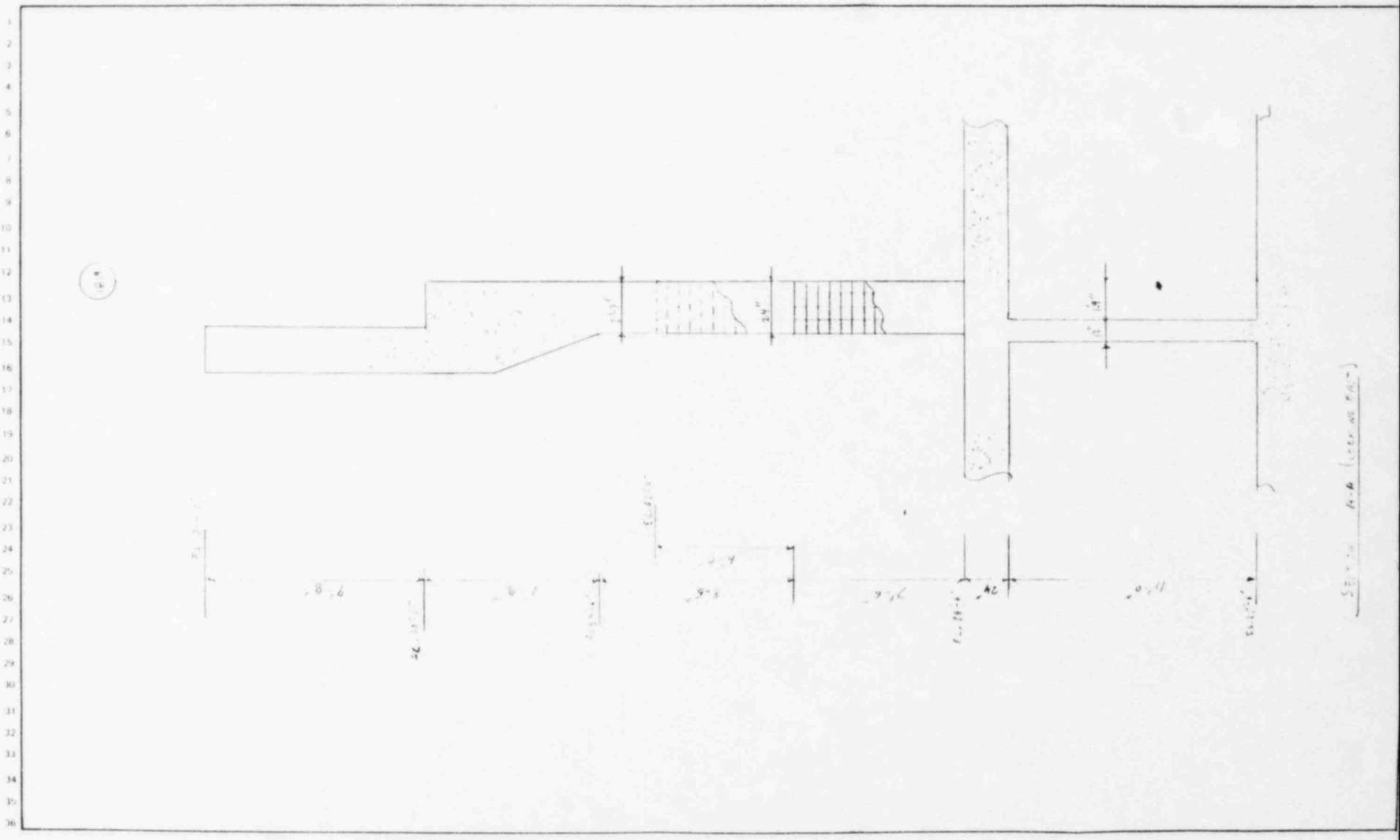
CALCULATION SHEET

CALC NO _____ REV NO _____

ORIGINATOR _____ DATE _____ CHECKED _____ DATE _____

PROJECT _____ JOB NO _____

SUBJECT _____ SHEET NO _____



CALCULATION SHEET

CALC. NO. 821-1.21 REV. NO. 1

ORIGINATOR R. Mann DATE 6 JULY 81 CHECKED KLB DATE 7.7.81

PROJECT MILLSTONE UNIT #2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL SHEET NO. 15

TOP BOUNDARY :

CONSIDER A CONCRETE BEAM ELEMENT FIXED AT BOTH ENDS.

$\ell = 5'-8'' = 68''$

DEPTH of BEAM = EL. 62'-2" TO EL 54'-6" (SEE SEC. A-A)

$= 7.666' = 92''$

WIDTH of BEAM = $2'-3'' \Rightarrow 27''$

$I = \frac{27(92)^3}{12} = 1752048.0 \text{ in}^4$

$E = 57000 (3000)^{\frac{1}{2}} \Rightarrow 3122.02 \text{ KSI}$

$K = \frac{W}{\Delta} = \frac{384 EI}{\ell^4} = \frac{384 (3122.02 \times 1752048.0)}{(68)^4}$

$= 99237.484 \text{ K/in}$

Σ FOR Δ

CONSIDERED STIFF ENOUGH FOR THE ASSUMPTION $\ell \to \infty$ IN PREVIOUS BUCKLING CALCS. (SEE SHT. 12a)

CALCULATION SHEET

CALC. NO. 8011-1.21 REV. NO. 1

ORIGINATOR K. Hasani DATE 6 JULY 81 CHECKED KLB DATE 7.7.81

PROJECT MILLSTONE UNIT #2 JOB NO. 11867-020

SUBJECT ANALYSIS OF THE WALL SHEET NO. 16

BOTTOM BOUNDARY:

CONSIDER A CONCRETE BEAM ELEMENT FIXED AT BOTH ENDS.

$\ell = 5'-8" = 68"$

DISCOUNT STIFFENING OF 12" THICK WALL ALMOST DIRECTLY UNDER BLOCKWALL.

$b = 24" + 24" + 24" = 72"$ (BASED ON 45° PUNCHING SHEAR PLANE PROJECTION THROUGH SLAB)

$d = 24"$

$I = \frac{72(24)^3}{12} = 82944.0 \text{ in}^4$

$E = 3122.02 \text{ ksi}$

$k = \frac{w}{\Delta} = \frac{384(3122.02)(82944.0)}{(68)^4} = 4652.677 \text{ k/in}$

Δ 4/6/82

CONSIDERED STIFF ENOUGH FOR THE ASSUMPTION $k \to \infty$ IN PREVIOUS ARCHING CALCS. (SEE SHT. 12a)

Attachment 3 consists of the Final Report on Concrete Masonry Wall Evaluation in response to NRC I.E. Bulletin 80-11 for Millstone Unit 2.

FINAL REPORT ON

CONCRETE MASONRY WALL EVALUATION

IN RESPONSE TO NRC I.E. BULLETIN 80-11

FOR THE

MILLSTONE NUCLEAR POWER STATION UNIT 2

BECHTEL POWER CORPORATION

GAITHERSBURG, MARYLAND

CONTENTS

		Page
I.	INTRODUCTION	1
II.	GENERAL DISCUSSION	1
III.	ANALYTICAL METHODS	2
IV.	MODIFICATION METHODS	3
V.	SUMMARY	5

ATTACHMENTS

A. Wall Location Plans

B. Safety Related Wall Summary

REFERENCES

1. Report on Concrete Masonry Wall Evaluation in Response to NRC I.E. Bulletin 80-11 (Bechtel Letter to NUSCo, BMC-163, dated October 29, 1980).

I. INTRODUCTION

This report contains the results of the reevaluation of concrete masonry walls for the Millstone Nuclear Power Station Unit 2, and serves as a final response to NRC I.E. Bulletin 80-11, dated May 8, 1980. The bulletin required all licensees for nuclear power plants to reevaluate the design adequacy of all masonry walls which are in proximity to or have attachments from safety-related piping or equipment such that the wall failure could affect a safety-related system.

II. GENERAL DISCUSSION

As a result of I.E. Bulletin 80-11, Northeast Utilities Service Company (NUSCo) retained Bechtel Power Corporation, Gaithersburg Division, to provide services including: walkdown, analysis, and modification if necessary of Millstone Unit 2 concrete masonry walls.

All walls which are in proximity to or have attachments from safety-related piping or equipment such that the wall failure could affect a safety-related system were identified and categorized in a systematic plant walkdown. The walkdown was performed in accordance with the procedures established in the 60-day report submitted to the NRC by Northeast Utilities (letter from W. G. Counsil, dated July 7, 1980). All Masonry walls included in the walkdown are shown on the wall location plans in Attachment A. A total of 312 masonry walls were included in the survey of which 155 were supporting or in the proximity of safety-related items, and the remaining 157 were determined to be non-safety-related.

The 155 walls identified in the walkdown as safety-related were analyzed and compared to the acceptance criteria as established in the 180-day report (reference 1). These walls were analyzed for the applicable loading conditions including earthquake, wind, tornado and pipe break effects. A summary of the analyses for the safety-related walls is tabulated in Attachment B, showing 98 meeting the established acceptance criteria and 57 requiring modification. All the required design modifications have been issued to NUSCo, and the construction work was completed during the last plant outage (December 1981).

III. ANALYTICAL METHODS

The reevaluation of the design adequacy of safety-related masonry walls was conducted in accordance with the acceptance criteria set forth in the 180-day report. The analyses included both elastic and operability criteria as summarized below.

The elastic criteria utilized two basic methods of analysis, the Bechtel computer program 'Blockwall' and hand calculations both of which are based upon working stress theory.

a. The 'Blockwall' program was mainly used for the analysis of reinforced concrete masonry walls for external and/or seismic loads. Based on modal analysis, a response spectrum analysis is performed to determine the seismic inertia loading. The reevaluation criteria and 'Blockwall' program descriptions were given in the 180-day report.

b. Hand calculations were performed for solid masonry walls using the same criteria and determination of loads as in the 'Blockwall' program.

The operability criteria also utilized two basic methods of analysis; energy balance technique and arching action.

a. The energy balance technique recognizes the energy absorbing capability of the wall beyond its elastic range. This analysis was used only on single panel, reinforced masonry walls on which flexural yielding of the reinforcement was the governing failure mode.

b. Arching action was primarily used on un-reinforced walls. This technique relies upon the ability of the wall after flexural cracking occurs to form a three-hinged arch between rigid boundaries. The analysis was only applied to walls which were capable of transfering the large vertical in plane forces developed to a relatively stiff structural boundary.

IV. MODIFICATION METHODS

A total of 57 masonry walls did not meet the acceptance criteria of the 180 day report and were modified. If feasible, walls were modified to meet elastic criteria. Of the 57 modifications 43 were repaired to elastic criteria, the remaining repaired to the operability criteria. A short description of each repair type is listed below.

a. Reduce Span - This repair consists of addition of structural steel, meeting certain stiffness requirements, to provide enough support to reduce the wall's span to satisfy the elastic criteria.

b. Provide Boundary Connection - Structural steel was provided at the top of the wall to assure a sound boundary connection. This was used with both elastic and energy balance analyses.

c. Provide Support at Free Edge - Walls with one or more free edges were modified to satisfy the elastic criteria through the addition of structural steel at their free edge.

d. Grout Decking Above Wall - In order to apply arching action to certain walls it was required to grout the decking above the wall to assure proper transfer of the reaction to the structural boundaries.

e. Through Bolt Wall - All composite walls were assumed to have zero collar joint strength and, therefore, walls which satisfied the elastic criteria compositely were through bolted with prestressed bolts to assure composite action.

f. Remove or Through Bolt Pipe Support - Removal of pipe supports from the wall reduced the external load so it would satisfy the elastic acceptance criteria. Through bolting of pipe support base plates redistributed the pipe support load to all the wythes so the wall would satisfy the elastic acceptance criteria.

g. Provide Structural Barrier - Structural barriers were provided to protect safety-related items attached to or in proximity of the walls.

V. SUMMARY

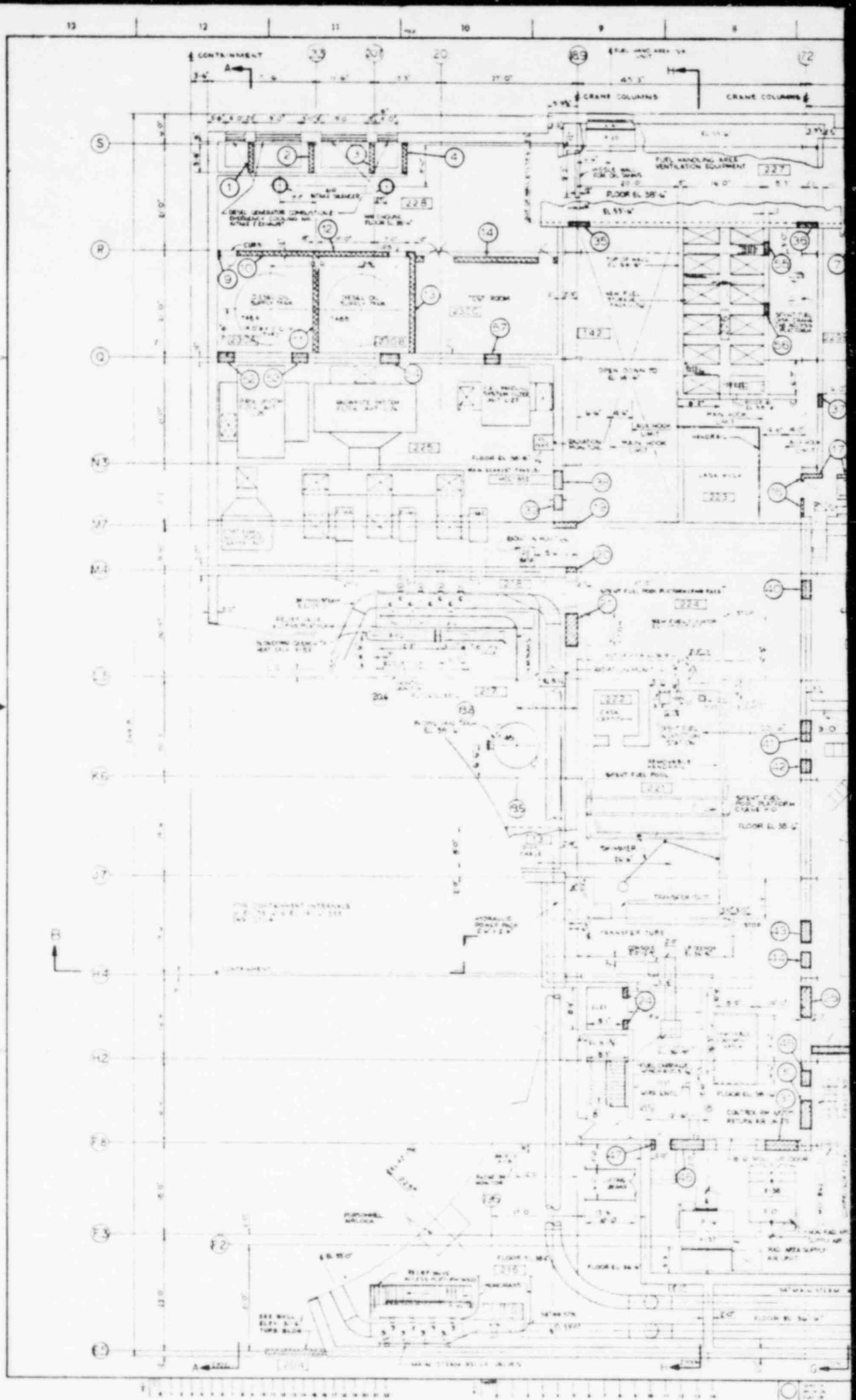
The I.E. Bulletin 80-11 program included 312 concrete masonry walls of which 155 were supporting or in the proximity of safety-related items, and the remaining 157 were determined to be non-safety-related.

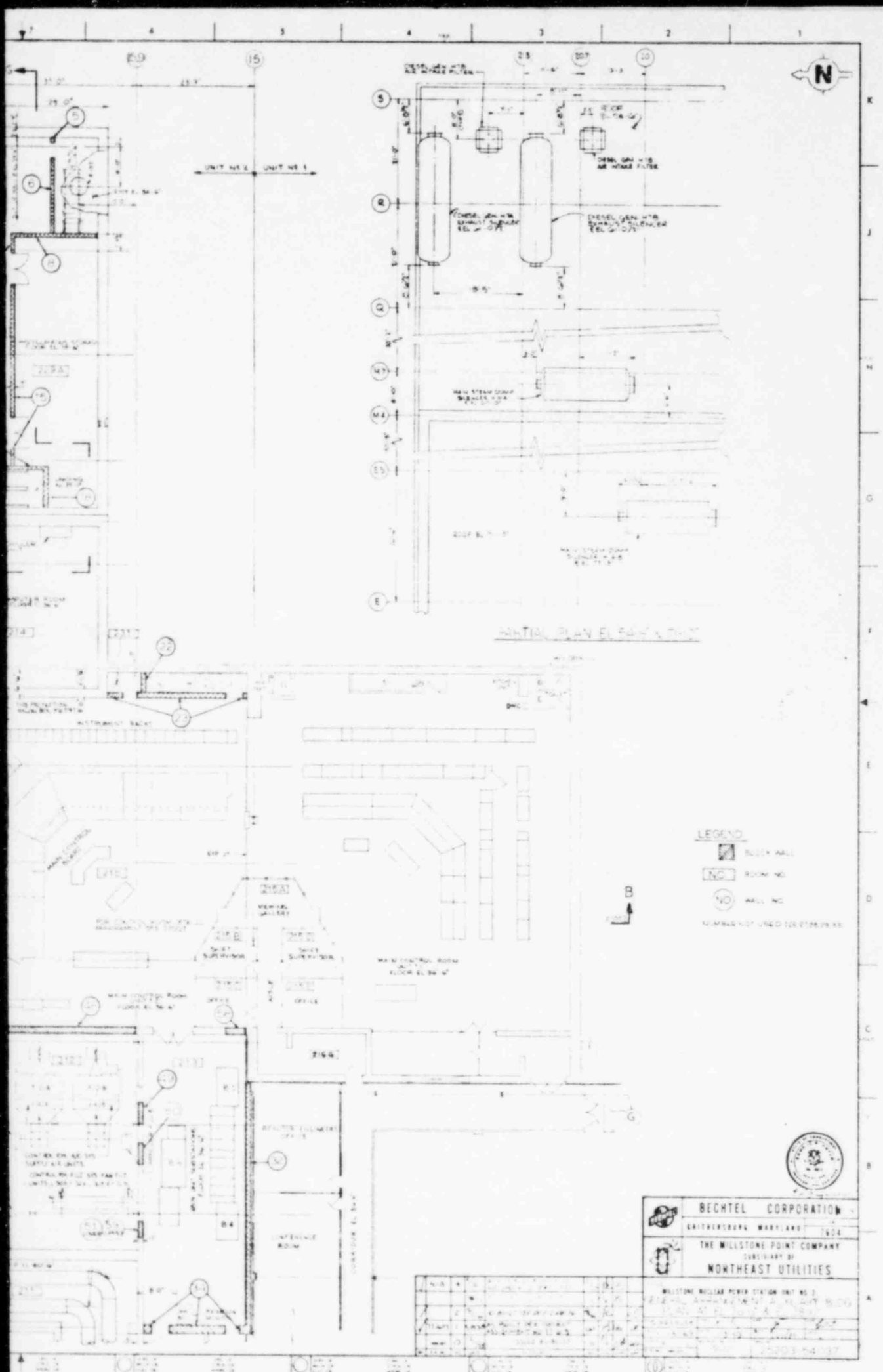
Non-Safety-Related Walls	157
Safety-Related Walls	155
98 Satisfied acceptance criteris	
57 Required modification	
TOTAL	312

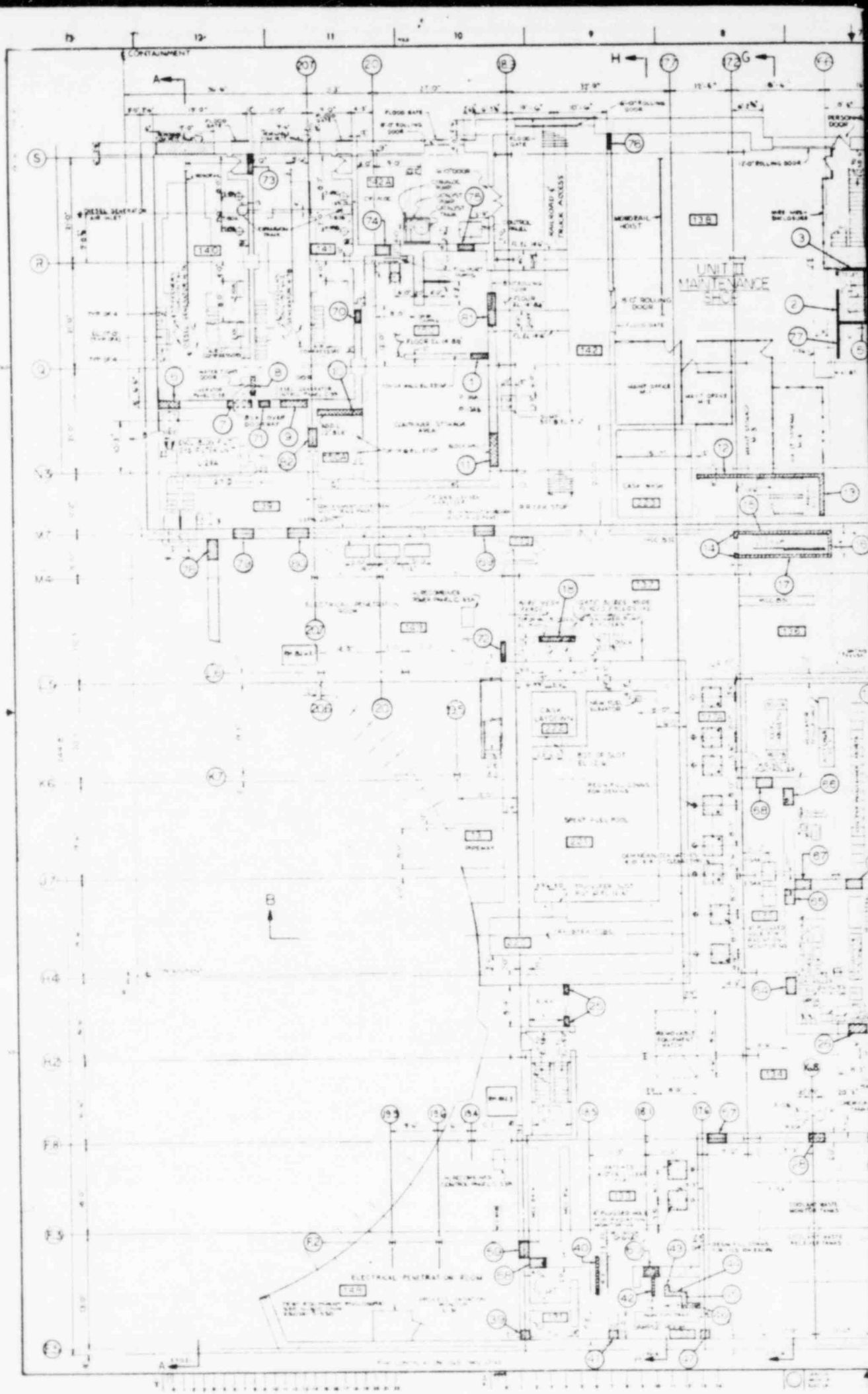
All modifications have been completed, and all safety-related walls meet the acceptance criteria.

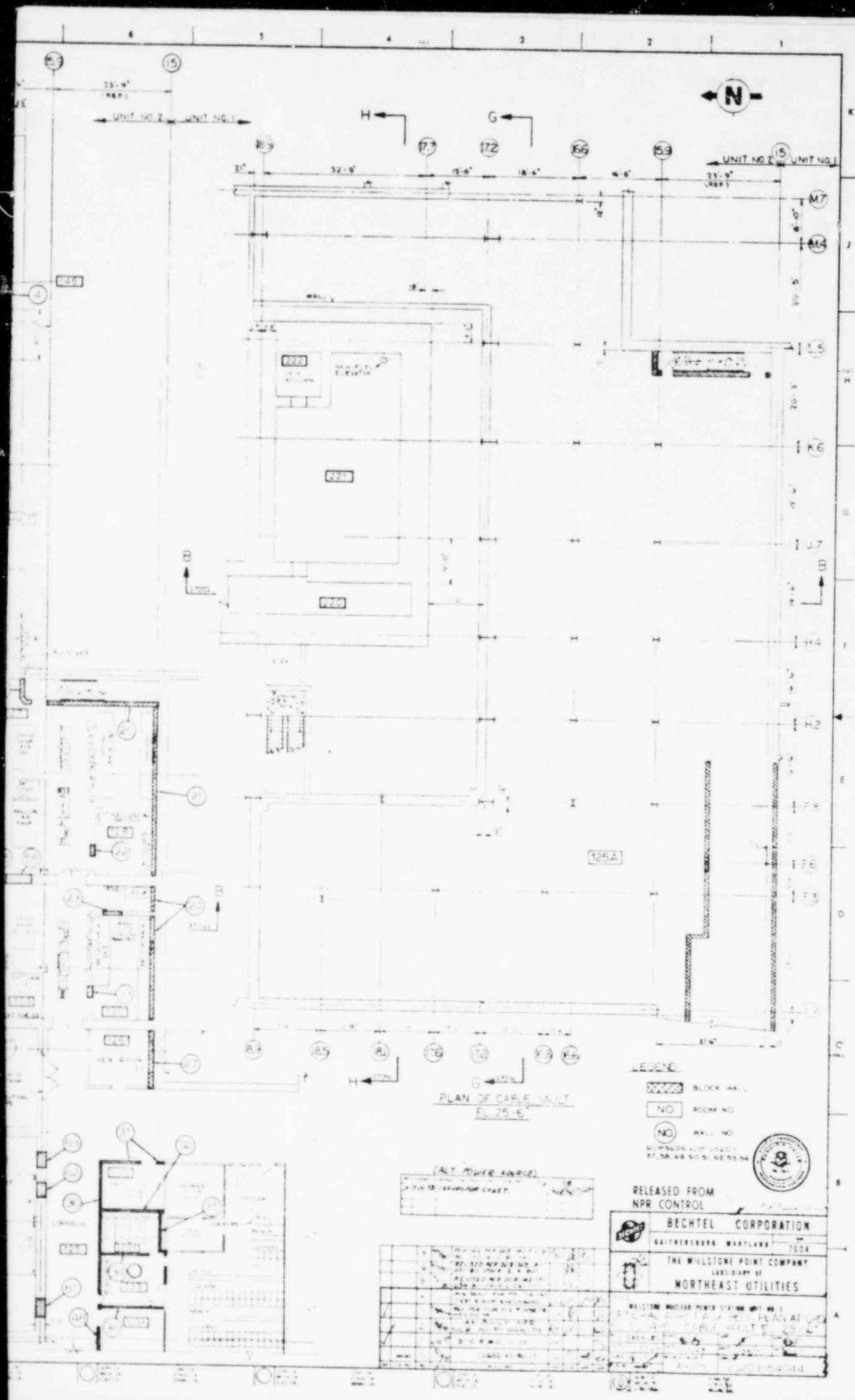
ATTACHMENT A

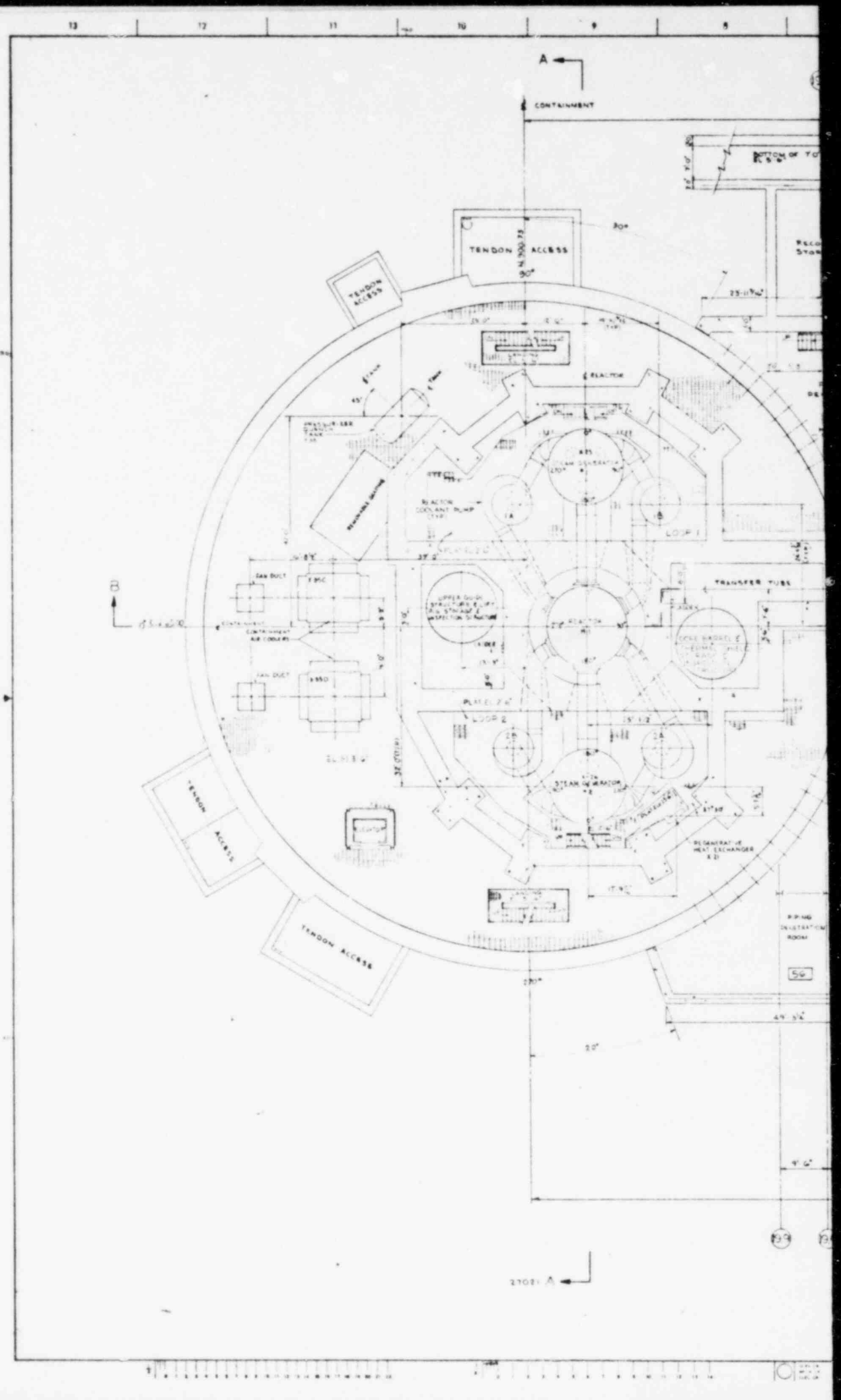
'WALL LOCATION PLANS'

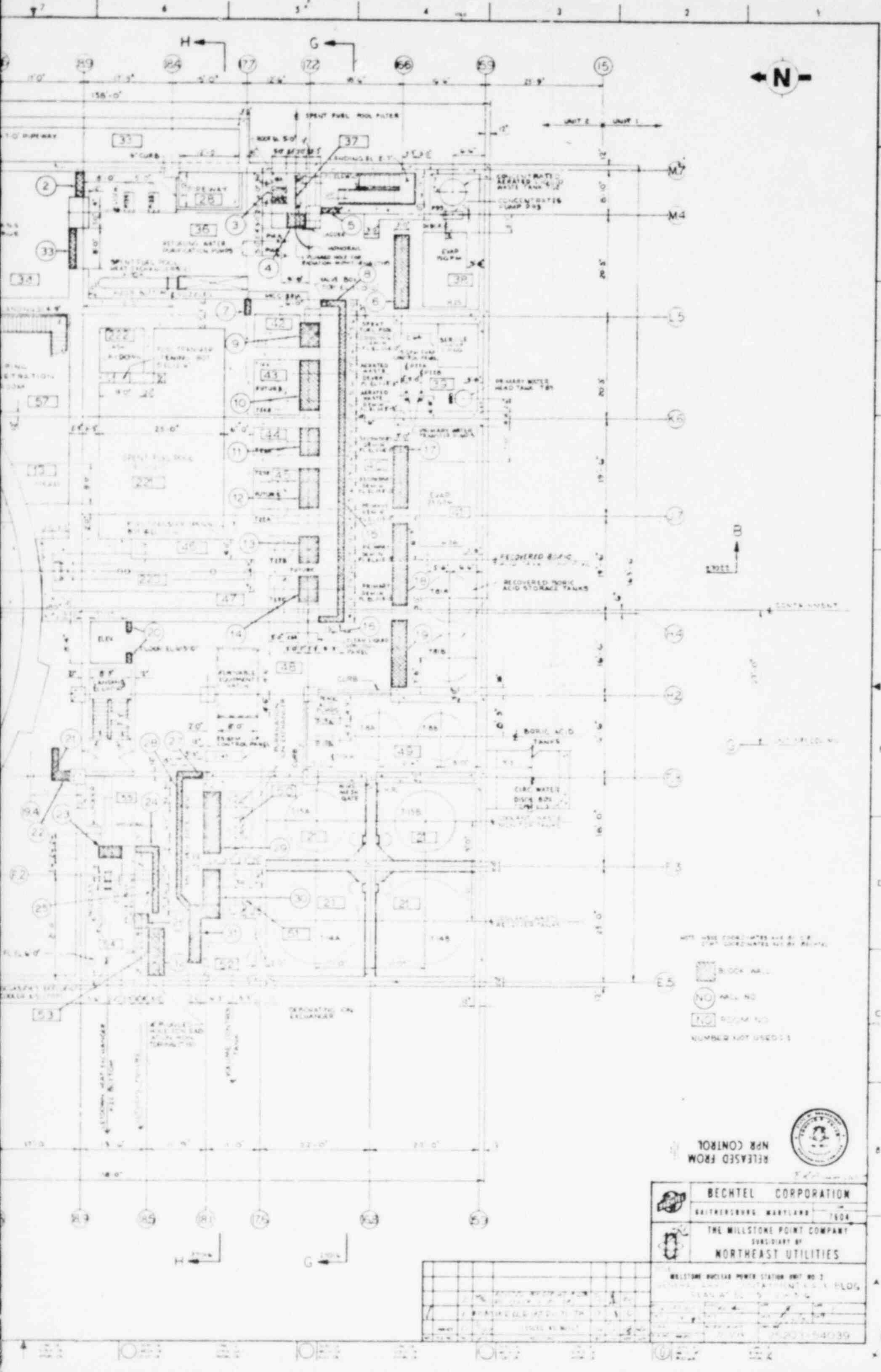

CONTAINMENT
CRANE COLUMNS
FUEL HANDLING AREA VENTILATION EQUIPMENT
FLOOR EL 38'-6"
EL 55'-6"
TEST ROOM
SPENT FUEL POOL
TRANSFER TUBE
CONTROL ROOM AIR RETURN AIR UNIT
RAD AREA SUPPLY AIR UNIT
PERSONNEL AIRLOCK
EQUIPMENT AIRLOCK
MAIN STEAM RELIEF VALVES

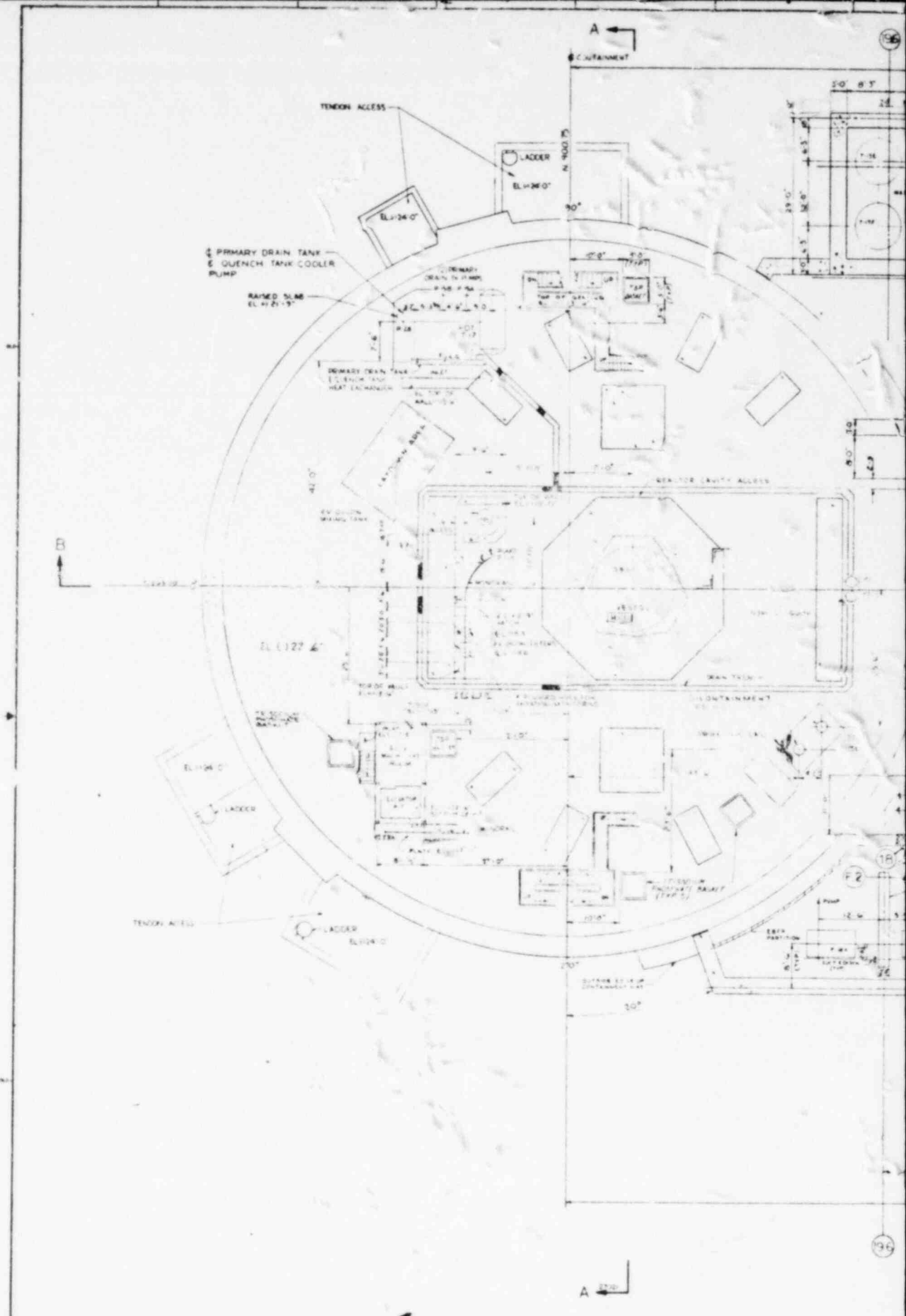

N
UNIT NO. 2
UNIT NO. 1
PARTIAL PLAN EL 54'-6"
LEGEND
BLOCK WALL
ROOM NO.
WALL NO.
MAIN CONTROL ROOM
FLOOR EL 54'-6"
INSTRUMENT RACKS
TURBINE ROOM
BECHTEL CORPORATION
GAITHERSBURG, MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

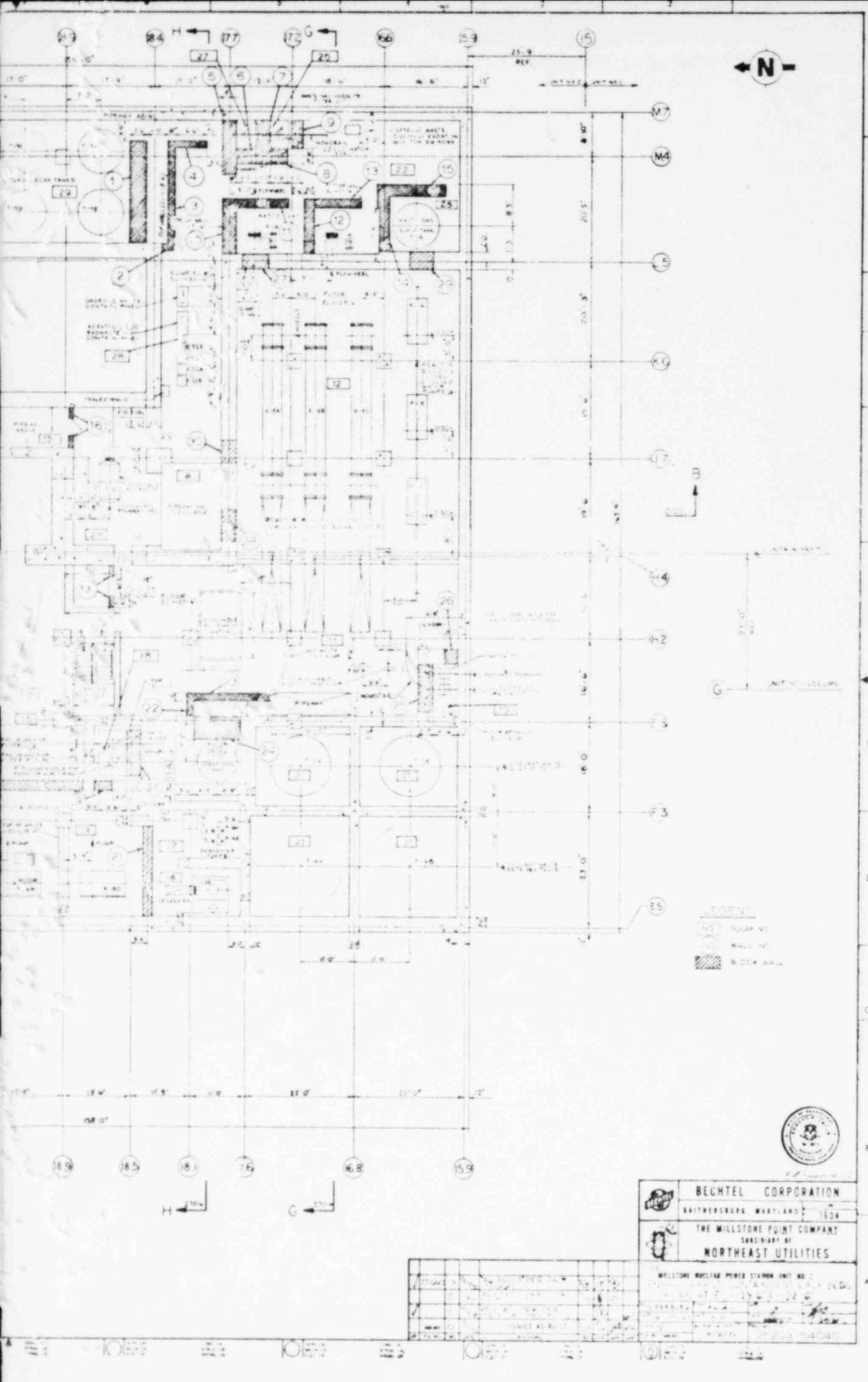

CONTAINMENT
UNIT II MAINTENANCE SHOP
ELECTRICAL PENETRATION ROOM

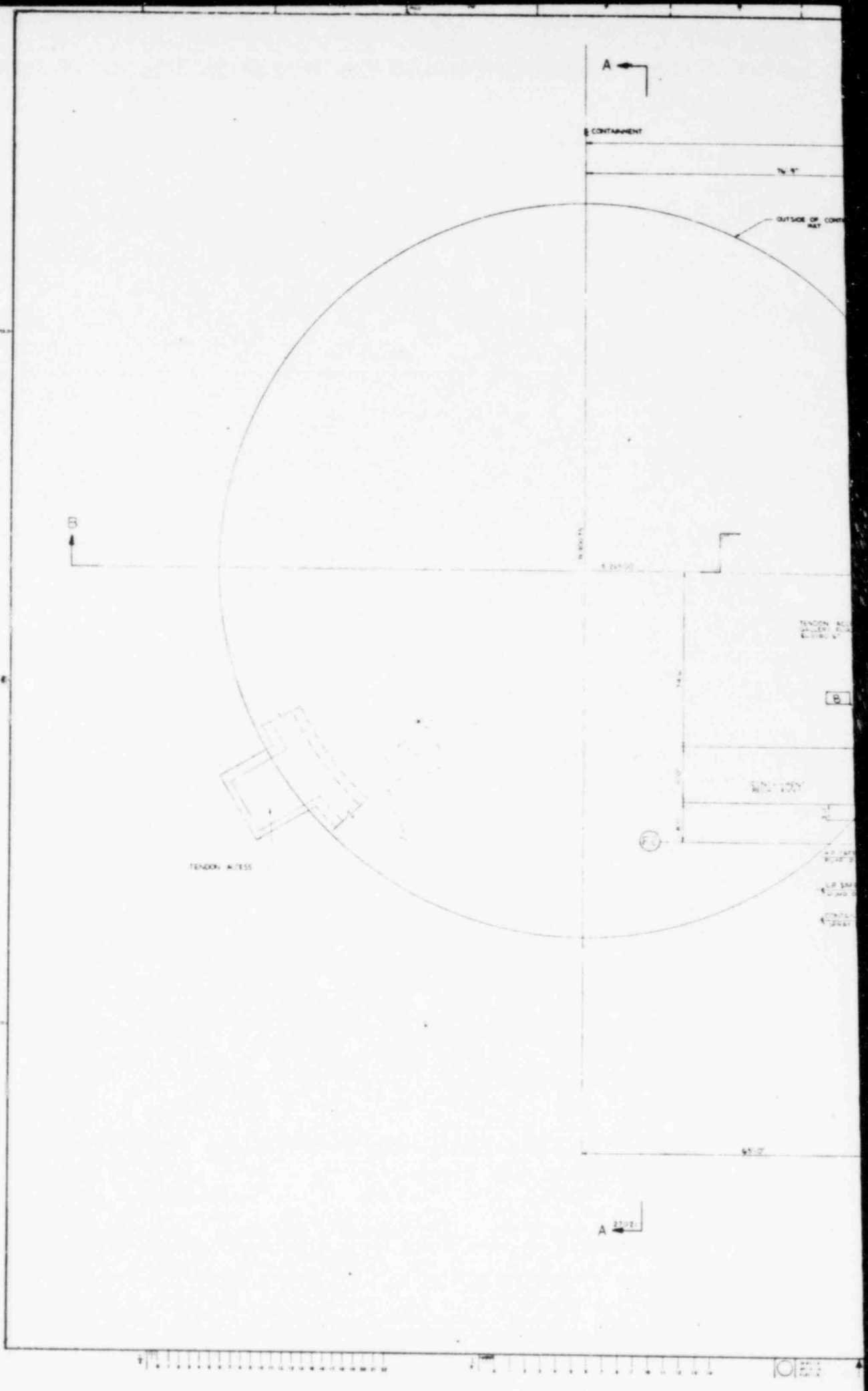

N
UNIT NO 2
UNIT NO 1
PLAN OF CABLE VAULT
EL 25'-6"
LEGEND
BLOCK WALL
ROOM NO
WALL NO
(ALT POWER SOURCE)
RELEASED FROM
NPR CONTROL
BECHTEL CORPORATION
GAITHERSBURG MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

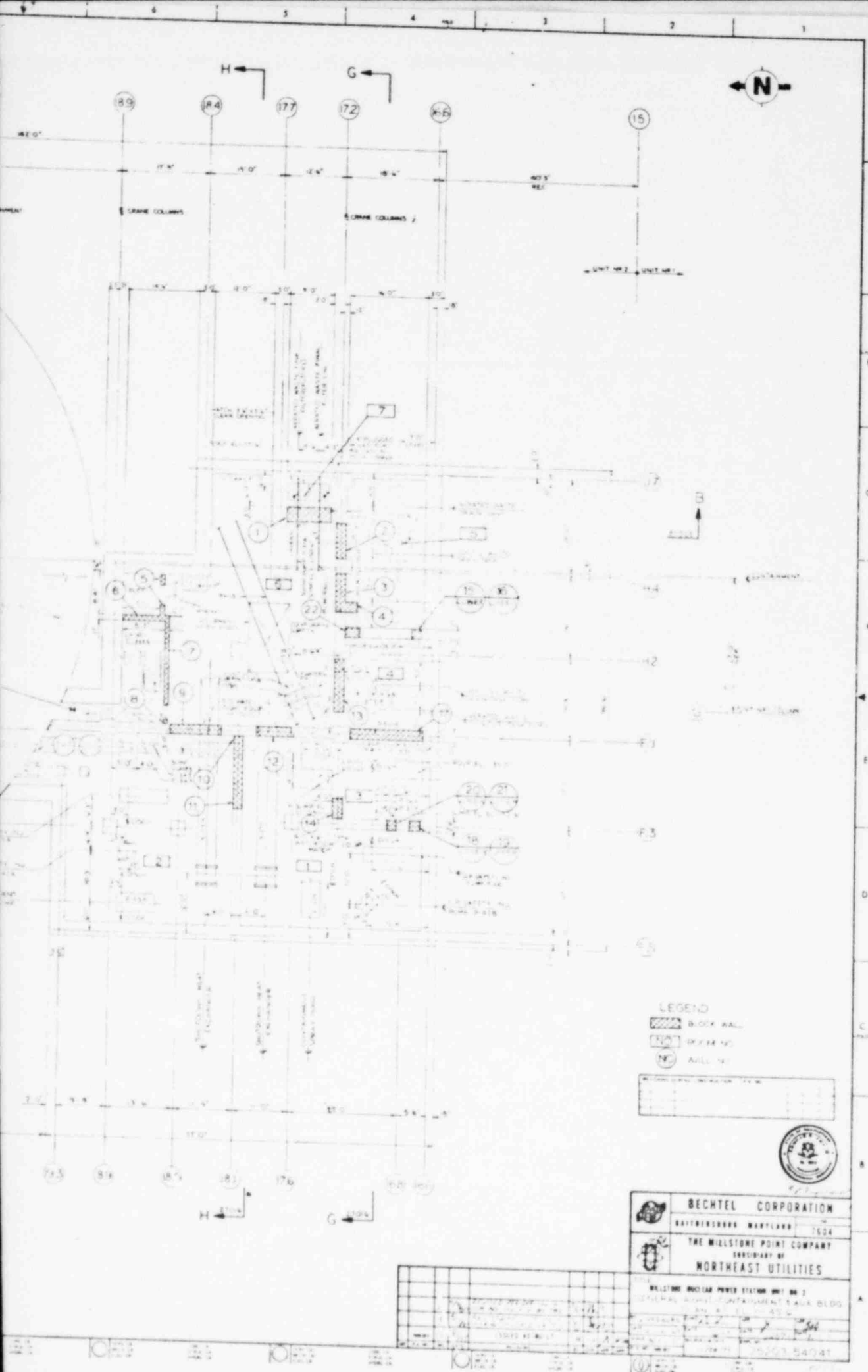

CONTAINMENT
TENDON ACCESS
TENDON ACCESS
TENDON ACCESS
TENDON ACCESS
REACTOR
REACTOR COOLANT PUMP
LOOP 1
LOOP 2
STEAM GENERATOR
TRANSFER TUBE
FUEL HANDLING MACHINE
REGENERATIVE HEAT EXCHANGER
PIPING PENETRATION ROOM
FAN DUCT
FAN DUCT
CONTAINMENT AIR COOLERS
PLATE EL 74'-0"
EL 113'-0"
EL 100'-0"
A
B
A

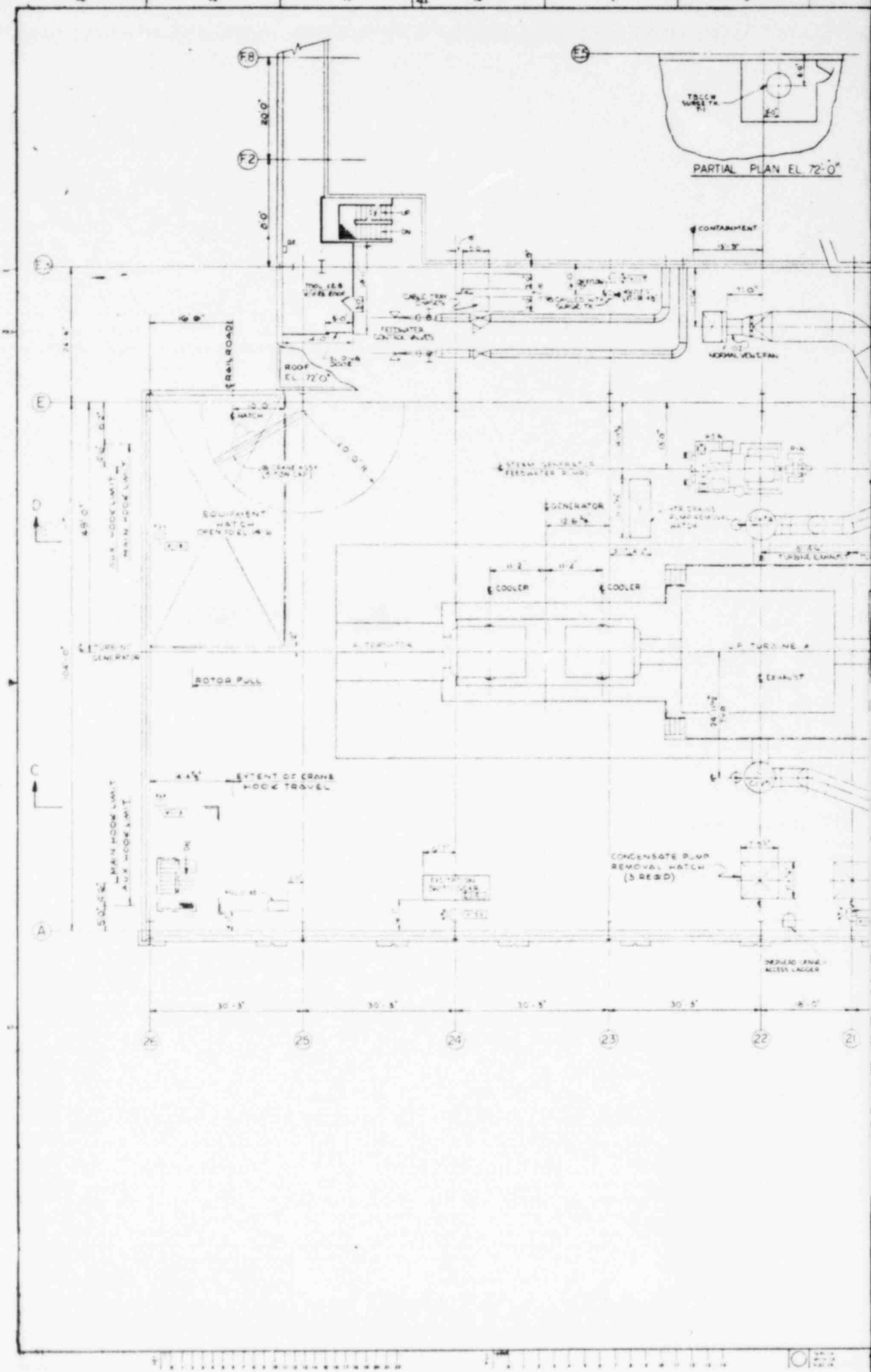

N
SPENT FUEL POOL FILTER
UNIT 2
UNIT 1
PIPEWAY
CONCENTRATES PUMP P-93
SPENT FUEL POOL
RECOVERED BORIC ACID STORAGE TANKS
BORIC ACID TANKS
CONTAINMENT
BLOCK WALL
WALL NO
ROOM NO
DEMINERALIZER
RELEASED FROM NPR CONTROL
BECHTEL CORPORATION
GAITHERSBURG, MARYLAND
7604
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES
MILLSTONE NUCLEAR POWER STATION UNIT NO 2
BLDG
25203-54039

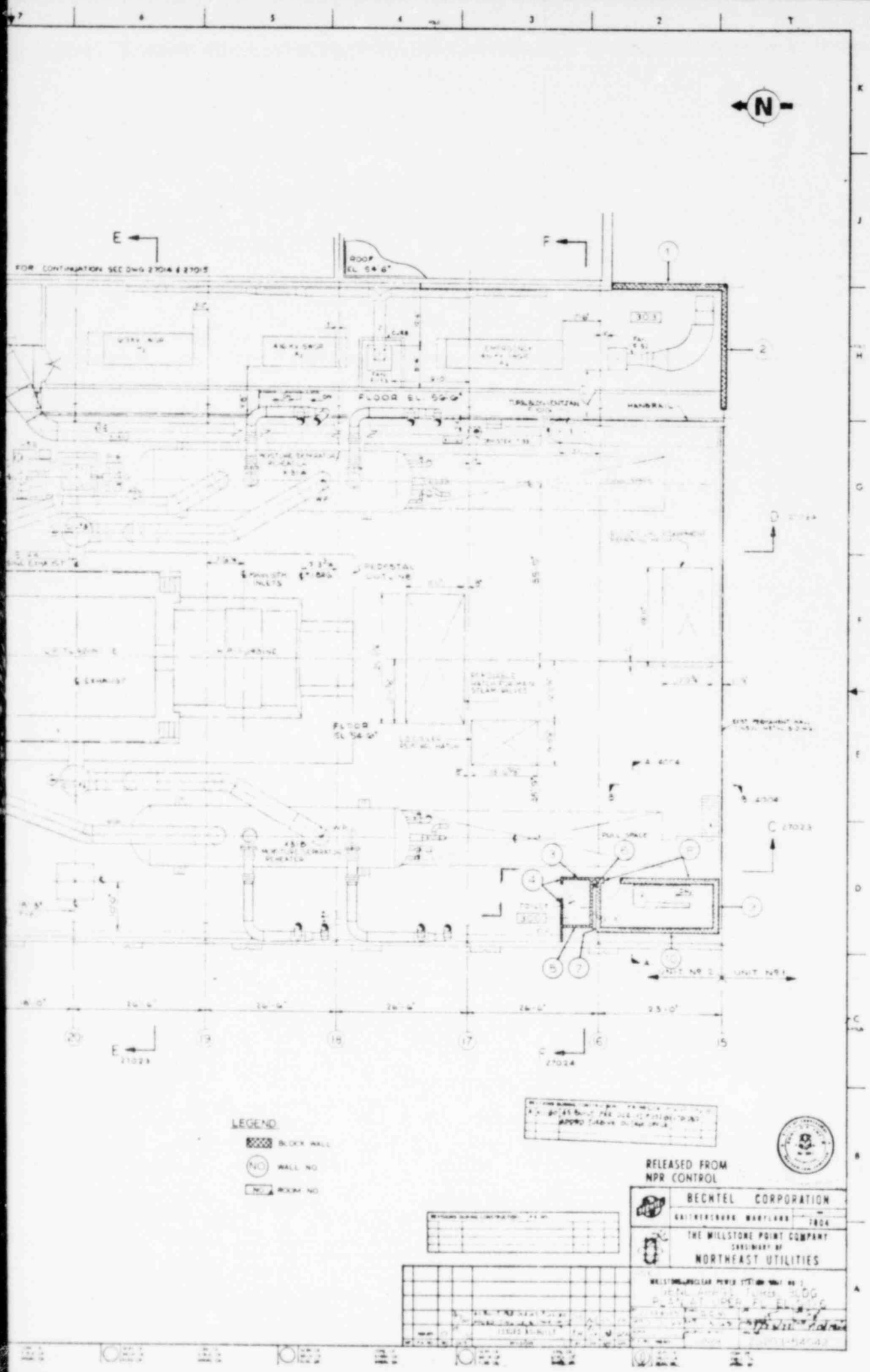

TENDON ACCESS
LADDER
EL.+24'-0"
℄ PRIMARY DRAIN TANK
℄ QUENCH TANK COOLER
PUMP
RAISED SLAB
EL.+21'-5"
PRIMARY DRAIN TANK
QUENCH TANK
HEAT EXCHANGER
REACTOR CAVITY ACCESS
EL.+127'-6"
CONTAINMENT
LADDER
TENDON ACCESS
LADDER
A
B
A

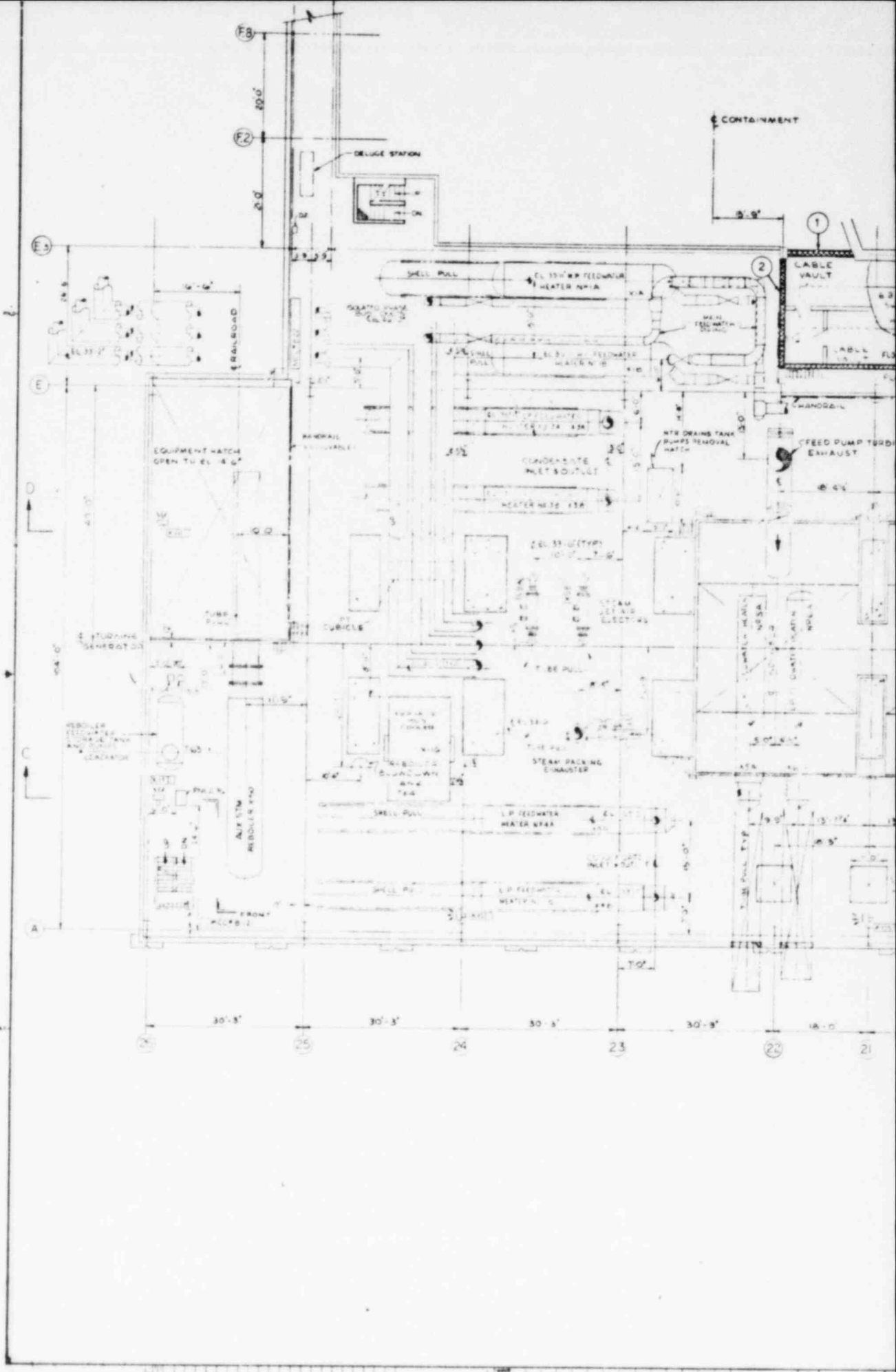

N
BECHTEL CORPORATION
GAITHERSBURG MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

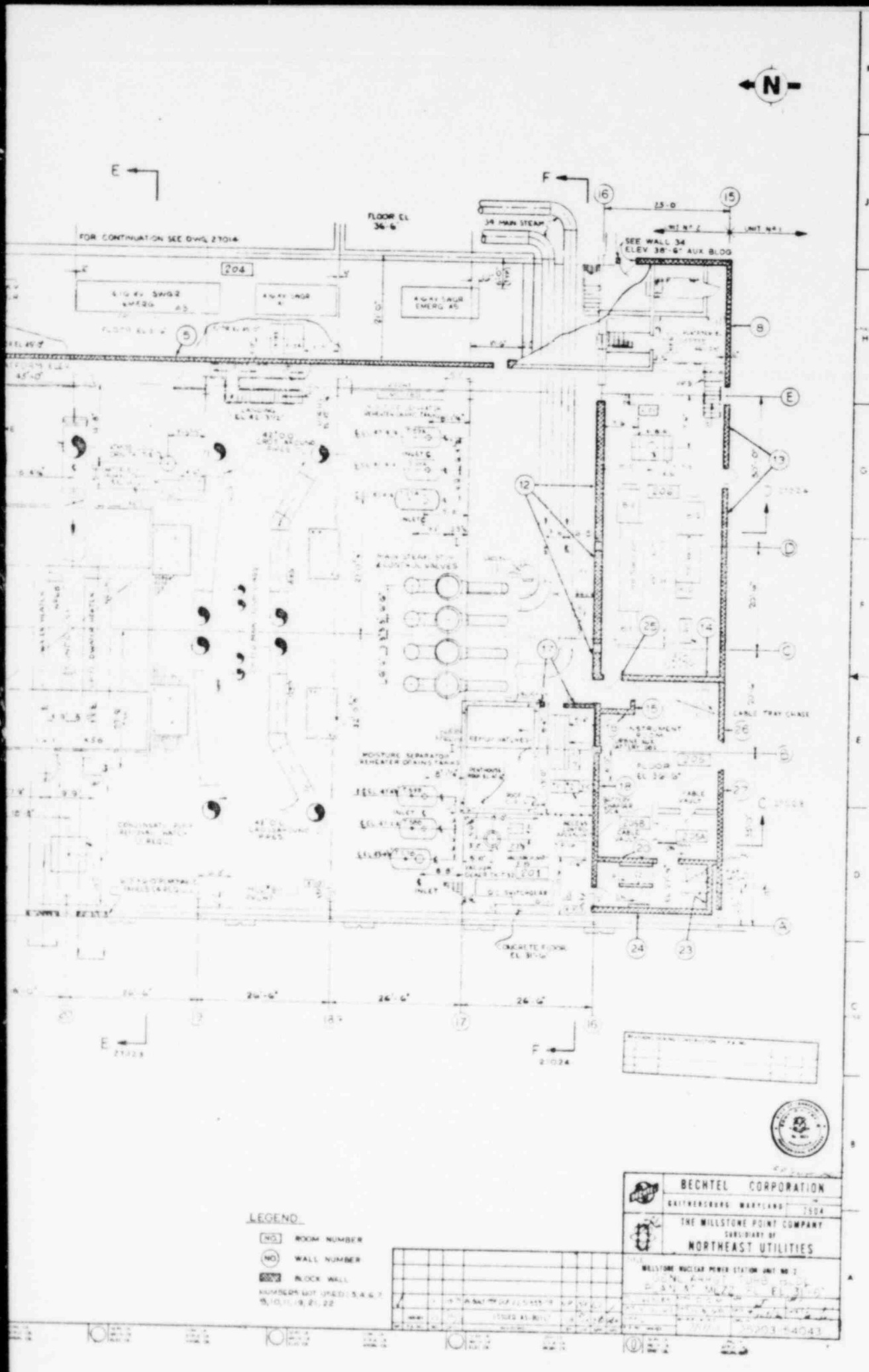

A
B CONTAINMENT
OUTSIDE OF CONTAINMENT MAT
B
TENDON ACCESS
A

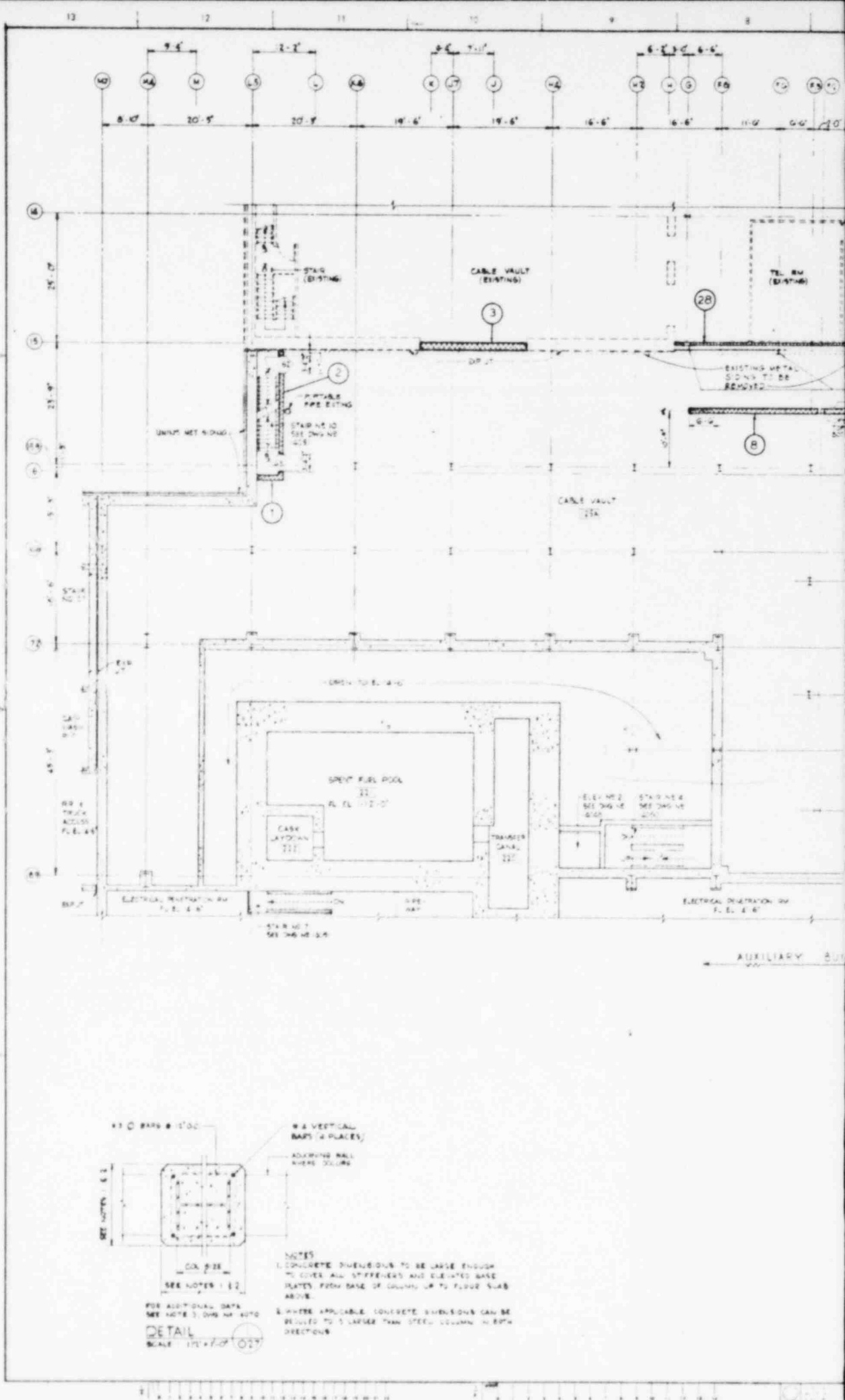

H
G
N
B
LEGEND
BECHTEL CORPORATION
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES


PARTIAL PLAN EL. 72'-0"
CONTAINMENT
ROOF EL. 72'-0"
EQUIPMENT HATCH
ROTOR PULL
EXTENT OF CRANE HOOK TRAVEL
CONDENSATE PUMP REMOVAL HATCH (3 REQ'D)
COOLER
COOLER
GENERATOR
EXHAUST
MAIN HOOK LIMIT
AUX HOOK LIMIT
30'-5"
30'-5"
30'-5"
30'-5"
25
24
23
22
21


N
E
F
FOR CONTINUATION SEE DWG 27014 & 27015
ROOF EL 54'6"
FLOOR EL 59'6"
FLOOR EL 54'6"
PEDESTAL OUTLINE
PEAK SPACE
UNIT NO 2
UNIT NO 1
LEGEND
BLOCK WALL
WALL NO
ROOM NO
RELEASED FROM NPR CONTROL
BECHTEL CORPORATION
GAITHERSBURG MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES


F8
F2
E.5
E
D
C
A
DELUGE STATION
CONTAINMENT
CABLE VAULT
RAILROAD
EQUIPMENT HATCH
OPEN TO EL 4'-6"
SHELL PULL
CONDENSATE INLET & OUTLET
CHANDRAIL
FEED PUMP TURBI
EXHAUST
STEAM PACKING EXHAUSTER
AUX STM REBOILER END
FRONT
30'-3"
30'-3"
30'-6"
30'-3"
18'-0"
26
25
24
23
22
21

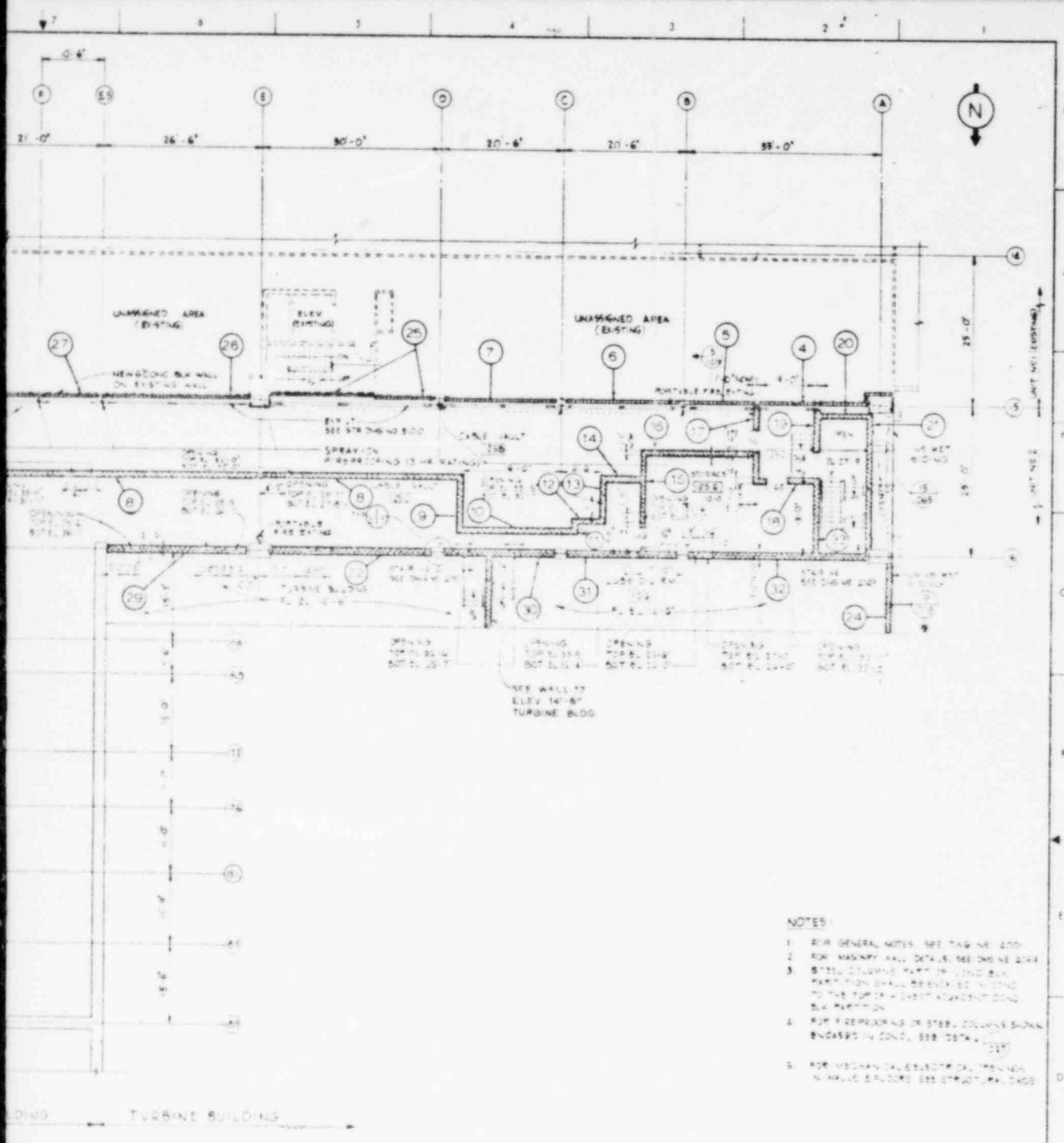

N
FOR CONTINUATION SEE DWG. 27014
FLOOR EL 36'-6"
34 MAIN STEAM
SEE WALL 34 ELEV 38'-6" AUX BLDG
204
25'-0"
26'-6"
26'-6"
26'-6"
MOISTURE SEPARATOR REHEATER DRAIN TANKS
CONCRETE FLOOR EL 9'-6"
LEGEND
ROOM NUMBER
WALL NUMBER
BLOCK WALL
BECHTEL CORPORATION
GAITHERSBURG MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES
MILLSTONE NUCLEAR POWER STATION UNIT NO 2



NOTES
1. CONCRETE DIMENSIONS TO BE LARGE ENOUGH TO COVER ALL STIFFENERS AND ELEVATED BASE PLATES FROM BASE OF COLUMN UP TO FLOOR SLAB ABOVE.
2. WHERE APPLICABLE CONCRETE DIMENSIONS CAN BE REDUCED TO 3 LARGER THAN STEEL COLUMN WIDTH DIMENSIONS

FOR ADDITIONAL DATA SEE NOTE 3, DWG NO. 4070

DETAIL
SCALE: 1"=1'-0" (027)



NOTES:

1. FOR GENERAL NOTES, SEE DWG NO. 6001.
2. FOR MASONRY WALL DETAILS, SEE DWG NO. 6044.
3. STEEL COLUMNS PART OF CONC BLK PARTITION SHALL BRACE AT EXISTING CONC SLAB OVERHEAD WITH STEEL ANGLES AT CONC BLK PARTITION.
4. FOR REINFORCING OF STEEL COLUMNS SHOWN ENCASED IN CONC, SEE DETAIL 2 ON DWG 6047.
5. FOR MECHANICAL & ELECTRICAL PENETRATIONS IN WALLS & FLOORS SEE STRUCTURAL DWGS.

LEGEND:

- WALL NO.
- ROOM NO.
- WALL NO.

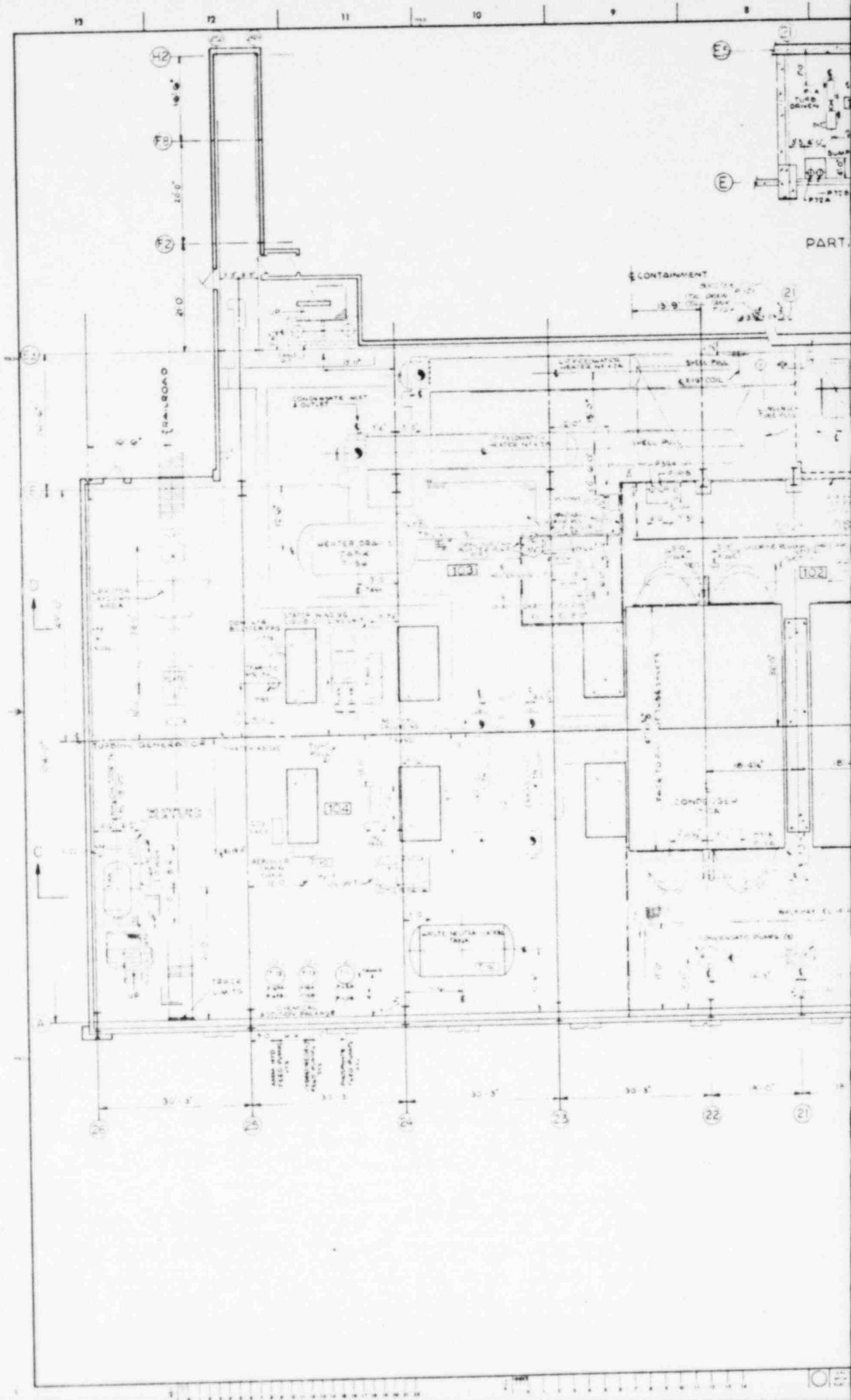
NUMBER NOT USED 1 25

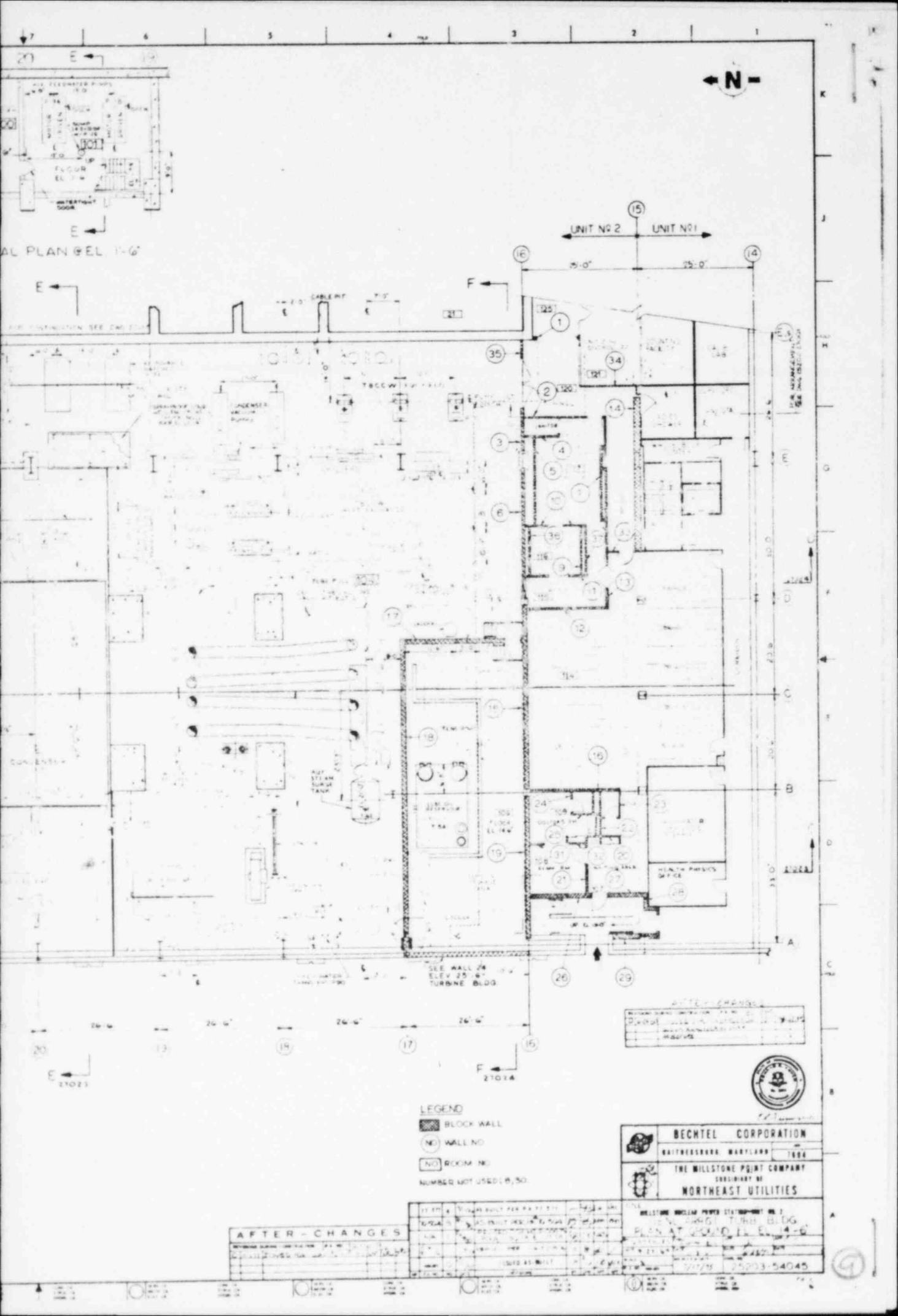
BECHTEL CORPORATION
GAITHERSBURG, MARYLAND 7604

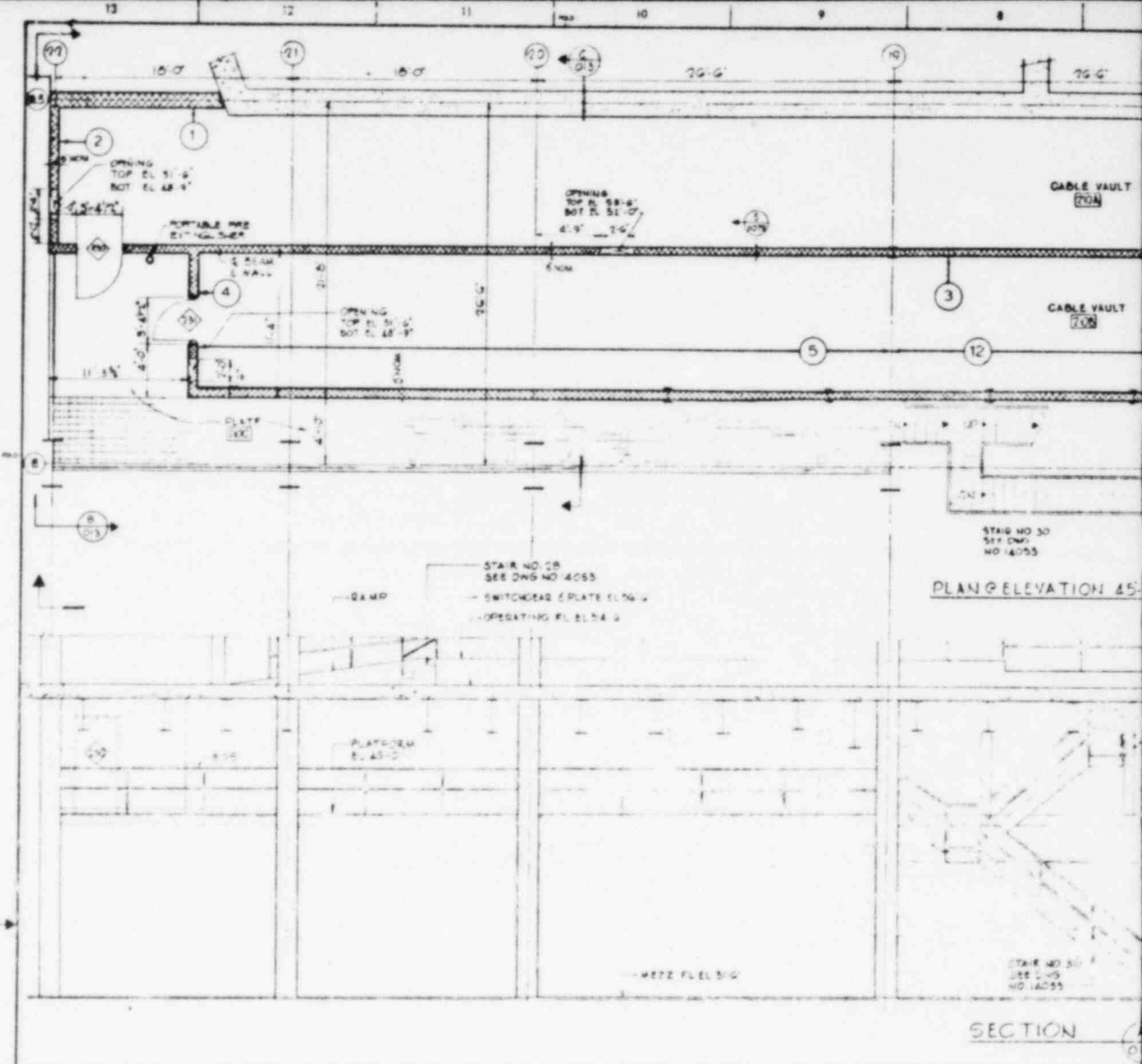
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

MILLSTONE NUCLEAR POWER STATION UNIT NO. 3
TURBINE & AUXILIARY BUILDINGS
CABLE VAULT PLAN AT EL. 19'-6"

9

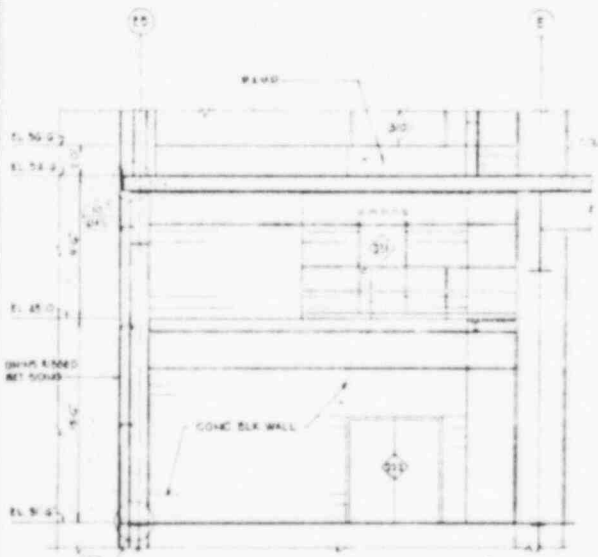

CONTAINMENT
103
102
104
TURBINE GENERATOR
CONTROL ROOM
30'-5"
30'-5"
30'-5"


N
UNIT NO 2
UNIT NO 1
AL PLAN @EL 1'-6"
SEE WALL 26
ELEV 25'-6"
TURBINE BLDG
LEGEND
BLOCK WALL
(NO) WALL NO
NO ROOM NO
NUMBER NOT USED 8,30
AFTER - CHANGES
BECHTEL CORPORATION
GAITHERSBURG MARYLAND
THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES
25203-54045

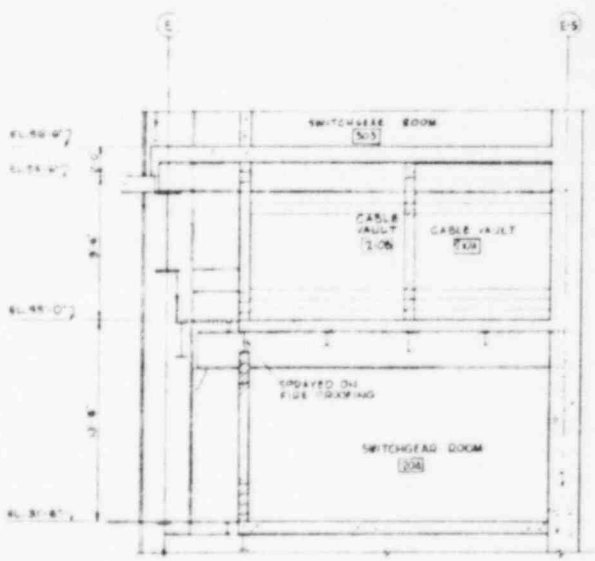


PLAN & ELEVATION 45-

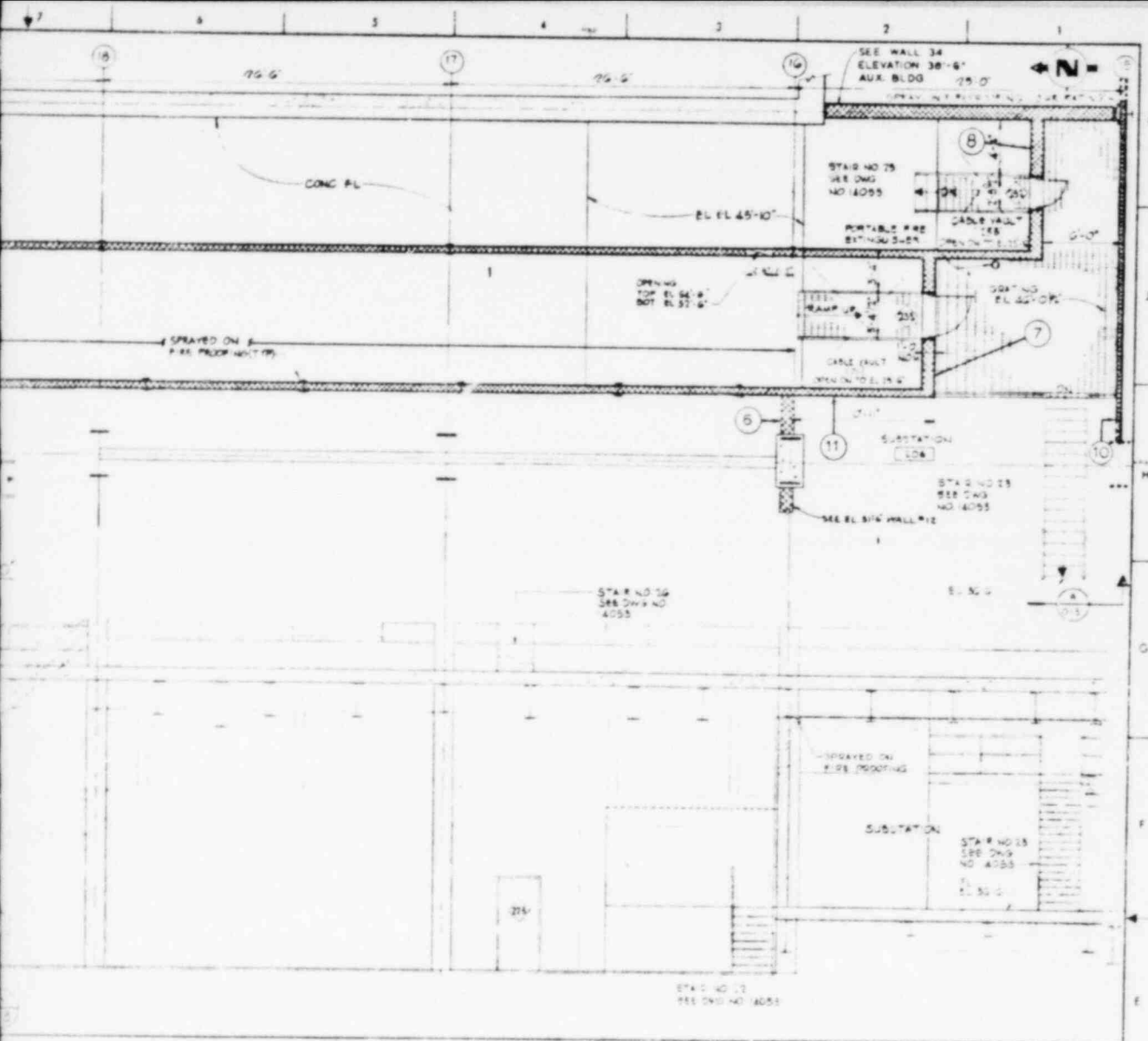
SECTION



PARTIAL SECT @ COL LINE 22



SECTION



NOTES

1. FOR GENERAL NOTES SEE DWG NO 4001
2. FOR MASONRY WALL DETAILS SEE DWG NO 4024 & SEE DWG NO 4179

LEGEND

NO ROOM NUMBER

(NO) WALL NUMBER

BLOCK WALL
NUMBER NOT USED

RELEASED FOR NRC CONTROL

BECHTEL CORPORATION
GAITHERSBURG MARYLAND 1974

THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

MILLSTONE NUCLEAR POWER STATION UNIT NO 2

TURBINE BLDG CABLE VAULT EL 45'-0"
PLAN & ELEVATIONS

DWG 4046

10

ATTACHMENT B

'SAFETY-RELATED WALL SUMMARY'

SCALING RULES

BUILDING	ELEVATION	WALL NO.	TYPE	I/1	S.E.				ORIGINATOR	CHECKER	ELASTIC ANALYSIS PASS/FAIL NON-COMPOS.	ELASTIC ANALYSIS PASS/FAIL COMPOSITE	REPAIR YES/NO	REPAIR TYPE	INELASTIC ANALYSIS PASS/FAIL	COMMENTS	BUILDING	ELEVATION
WAREHOUSE	58'-6"	1.73 (A)	W	X		1	U		T.C.	R.A.B.	PASS	N/A	NO	N/A	N/A			
↓	↓	1.9	W	X		1	R	2	T.C.	T.G., R.M.	FAIL	N/A	YES	ANGLES AT TOP	PASS	ENERGY BALANCE A+13	AUXILIARY	(-) 5'-0"
		1.11	W	X		1	R	1	Y.G./T.C.	T.C./T.M.	PASS	N/A	NO	N/A	N/A		↓	↓
		1.12	W	X		1	R	2	T.C.	T.L./T.W.	FAIL	N/A	YES	ANGLES AT TOP	PASS	ENERGY BALANCE A+13, R/W [illegible]		
↓		1.15	W	X		1	R	2	T.C.	H.E.M.	FAIL	N/A	YES	ANGLES AT TOP	PASS	ENERGY BALANCE A.R.10		
WAREHOUSE		1.16	W	X		1	R		R.W.K./T.C.	[illegible]	FAIL	N/A	YES	REDUCE SPAN	FAIL		AUXILIARY	(-)5'-0"
AUXILIARY		1.18	W	X	X		U		R.W.K.	T.G.	PASS	N/A	NO	N/A	N/A			
AUXILIARY	↓	1.20	W	X	X		U		R.W.K.	T.G.	PASS	N/A	NO	N/A	N/A		AUXILIARY	(-)25'-6"
AUXILIARY	58'-6"	1.21	W	X	X	4	U	2	T.S.S.	T.L.	FAIL	PASS	NO	N/A	PASS		↓	↓
AUXILIARY	58'-6"	1.23	W	X	X	1	R	2	T.C.	R.E.B.	FAIL	N/A	YES	REDUCE SPAN	FAIL			
↓	↓	1.24	W		X	1	R		H.R.	H.E.M.	PASS	N/A	NO	N/A	N/A			
		1.25	W	X	X	4	U	5	R.E.B.	T.C.	PASS	N/A	NO	N/A	N/A	(R/W 1.26)	↓	↓
		1.30	B	X		4	U	2	R.W.K.	T.C.	FAIL	PASS	NO	N/A	PASS		AUXILIARY	(-)25'-6"
		1.31	B		X	4	U		R.E.B./H.E.M.	T.C.	FAIL	PASS	NO	N/A	PASS			
↓		1.32	W	X	X		U	1	T.H.	Y.G./T.P.	FAIL	N/A	YES	ANGLE AT TOP & REDUCE SPAN	FAIL	TORNADO LOADS	AUXILIARY	(-)25'-6"
AUXILIARY		1.34	W	X	X	1	R	1	T.S.S.	R.E.M.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		1.35	B	X	X	7	U	5	R.E.B.	T.C.	PASS	N/A	NO	N/A	N/A	(R/W 1.36)	AUXILIARY	(-)25'-6"
↓		1.36	B	X	X	7	U		T.C.	T.H.	FAIL	FAIL	NO	N/A	PASS	(R/W 1.35)	AUXILIARY	(-)25'-6"
		1.38	B	X	X	4	U	1	T.C.	T.G.	PASS	N/A	NO	N/A	N/A	(R/W 1.39)		
WAREHOUSE		1.39	B	X	X	4	U	1	T.C.	T.G.	PASS	N/A	NO	N/A	N/A	(R/W 1.38 & 1.54)	AUXILIARY	(-)25'-6"
AUXILIARY		1.41	B	X		7	U	3	T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A		↓	↓
↓		1.42	B	X		4	U	2	R.E.B.	F.W.	PASS	N/A	NO	N/A	N/A	(R/W 1.45)		
		1.43	B	X		7	U		T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A		AUXILIARY	(-)25'-6"
		1.45	B	X		4	U	2	T.L.	T.P.S.	PASS	N/A	NO	N/A	N/A			
		1.46 / 1.47	B	X	X	4	U		T.C.	R.E.B.	PASS	N/A	NO	N/A	N/A	COMMON WALL	AUXILIARY	(-)45'-6"
		1.48	W	X	X	3	U	1	T.C./R.E.B.	R.E.M./T.G.	PASS	N/A	YES	ANGLES AT TOP	N/A		↓	↓
		1.49	B	X	X	7	U	1	R.E.B.	T.C.	FAIL	PASS	NO	N/A	PASS	(R/W 1.50)		
↓		1.50	B	X		7	U	1	R.W.K./T.C.	R.E.B./M.L.	FAIL	FAIL	NO	N/A	PASS			
AUXILIARY		1.51	B	X	X	7	U	2	T.C.	P.H.P.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		1.52	B	X		7	U	4	T.G.	H.E.M.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		1.53	B	X		4	U	4	R.E.B.	T.C.	PASS	N/A	NO	N/A	N/A	(R/W 1.54)		
WAREHOUSE	↓	1.54	B	X		4	U	2	T.C.	T.P.S.	PASS	N/A	NO	N/A	N/A			
AUXILIARY	58'-6"	1.59	B	X		7			R.E.B.	T.C.	FAIL	PASS	NO	N/A	PASS	(R/W 1.58 & 1.38)		
WAREHOUSE	14'-6"	2.6	B	X		3	U	1	T.C.	R.E.M.	PASS	N/A	NO	N/A	N/A	(R/W 2.7)	↓	↓
↓	↓	2.7	B	X	X	6	U	1	T.C.	Y.G., T.P., M.L.	FAIL	PASS	NO	N/A	PASS		AUXILIARY	(-)45'-6"
		2.8	B	X	X	7	U	1	T.C.	T.P.S./T.H.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		2.9	B	X	X	4	U	1	T.C.	W.W.C.	PASS	N/A	NO	N/A	N/A	(R/W 2.7)	TURBINE	54'-6"
AUXILIARY		[illegible]	W	X		1	U	2	[illegible]	[illegible]	PASS	N/A	YES	ANGLES AT TOP	N/A		TURBINE	64'-6"
↓		2.15	W	X	X	1	U	1	T.C.	H.E.M.	PASS	N/A	YES	ANGLES AT TOP	N/A			
		2.16	W	X	X	1	U	1	H.R.	H.E.M.	PASS	N/A	NO	N/A	N/A		TURBINE	51'-6"
		2.21	W	X	X	1	U	1	R.E.M.	W.W.C.	PASS	N/A	YES	ANGLES AT TOP	N/A	FACADE ON UNIT 1	↓	↓
		2.22	W	X	X	1	U	2	R.E.M.	T.H.	PASS	N/A	YES	L AT TOP	N/A	(R/W 2.33)		
		2.23	W	X	X	1	U	1	T.H.	R.E.M.	PASS	N/A	YES	L AT TOP	N/A			
		2.24	W	X	X	1	U	1	R.E.M.	W.W.C.	PASS	N/A	YES	ANGLES AT TOP	N/A	FACADE ON UNIT 1		
		2.26	B	X		7	U	3	T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A			
		2.28	B	X		4	U		R.E.M.	R.E.M.	PASS	N/A	NO	N/A	N/A		TURBINE	51'-6"
		2.29	B	X	X	4	U	5	R.E.M.	R.E.M.	PASS	N/A	YES	[illegible] SUPPORT	N/A			
		2.33	W	X		1	U	2	T.H.	R.E.M.	PASS	N/A	YES	L AT TOP	N/A			
		2.41	B	X		4	U	4	H.E.M.	T.L.	PASS	N/A	NO	N/A	N/A			
		2.35	B	X		4	U	5	R.E.B.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.36)		
		2.36	B	X		4	U	5	R.E.B.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.35)	TURBINE	85'-6"
																	↓	↓
		2.59	B	X		4	U		T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A			
		2.60	B	X	X	4	U		T.L.	H.E.M.	PASS	N/A	NO	N/A	N/A			
		2.64	B	X		2	U		T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A	(R/W [illegible])		
		2.65	B	X	X	4	U		T.C.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.74 & 2.68)		
		2.66	B	X	X	4	U		T.C.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.65)		
		2.67	B	X		4	U		T.L.	T.C.	PASS	N/A	NO	N/A	N/A			
↓		2.68	B	X		4	U		T.L.	R.E.M.	PASS	N/A	NO	N/A	N/A			
AUXILIARY		2.69	B	X		4	U	1	T.L.	R.E.B.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		2.70	B	X		5	U	2	H.E.M.	T.C.	PASS	N/A	NO	N/A	N/A			
WAREHOUSE		2.71	B	X	X	5	U		T.C.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.74 & 2.[illegible])		
WAREHOUSE		2.75	B	X		2	U		H.E.M.	T.L.	PASS	N/A	NO	N/A	N/A			
AUXILIARY		2.78	B	X		6	U		H.E.M.	R.E.B.	PASS	N/A	NO	N/A	N/A		↓	↓
AUXILIARY	↓	2.79	B		X	2	U		H.E.M.	T.L.	PASS	N/A	NO	N/A	N/A		TURBINE	85'-6"
AUXILIARY	14'-6"	2.80	B		X	4	U		H.E.M.	T.L.	PASS	N/A	NO	N/A	N/A	(R/W 2.29)		
		[illegible]	W	X		1	U		T.H.	T.C.	PASS	N/A	NO	N/A	N/A		TURBINE	14'-6"
AUXILIARY	(-)16'-0"	5.1	W	X		6	U		R.E.M.	R.E.B./W.W.C.	PASS	N/A	NO	N/A	N/A		↓	↓
↓	↓	5.5	W	X		7	U	1	H.E.M.	T.L.	PASS	N/A	NO	N/A	N/A			
		5.6	W	X	X	6	U	1	R.E.B.	R.E.M.	FAIL	PASS	YES	[illegible] THROUGH BOLT WALL	N/A		TURBINE	14'-6"
		5.7	B	X	X	6	U		T.H.	W.W.C.	PASS	N/A	NO	N/A	N/A			
		5.8	W	X		7	U	4	R.E.B.	T.C.	PASS	N/A	NO	N/A	N/A		TURBINE	45'-0"
		5.9	W	X		8	U	2	R.E.M.	H.E.M./T.C.	PASS	N/A	NO	N/A	N/A		↓	↓
		5.10	W	X		8	U	2	R.E.M.	H.E.M./T.C.	FAIL	PASS	YES	GROUT [illegible] ADJ. WALL	PASS			
		5.11	W	X		8	U	2	R.E.M.	H.E.M./T.C.	PASS	N/A	NO	N/A	N/A			
		5.12	W	X		8	U	2	R.E.M.	H.E.M./T.C.	FAIL	PASS	YES	GROUT [illegible] ADJ. WALL	PASS			
		5.13	W	X		8	U	2	R.E.M.	H.E.M./T.C.	PASS	N/A	NO	N/A	N/A			
		5.14	W	X		8	U	2	R.E.M.	H.E.M./T.C.	PASS	N/A	NO	N/A	N/A			
		5.17	W	X	X	6	U	4	R.E.B.	T.C.	FAIL	PASS	YES	THROUGH BOLT WALL	N/A			
		5.18	W	X	X	6	U	2	R.E.B.	T.C.	FAIL	PASS	YES	THROUGH BOLT WALL	N/A		TURBINE	45'-0"
		5.19	B	X	X	6	U	2	R.E.M.	R.E.B.	FAIL	PASS	YES	THROUGH BOLT WALL	N/A			
		5.20	W	X		1	U	4	H.R.	R.E.B.	PASS	N/A	NO	N/A	N/A			
		5.21	W	X	X	5	U		T.C.	[illegible]	FAIL	[illegible]	YES	COLUMN [illegible]	N/A	BEAM [illegible]		
		5.22	W	X	X	5	U		T.C.	T.L.	PASS	N/A	NO	N/A	N/A			
		5.23	W	X	X	5	U		T.H.	R.E.B./R.E.M.	FAIL	PASS	NO	N/A	PASS			
		5.24	W	X		8	U		T.H./T.C.	R.E.M.	PASS	N/A	NO	N/A	N/A			
↓	↓	5.25	W	X	X	8	U		T.H./T.C.	R.B.M.	FAIL	PASS	YES	REDUCE SPAN	FAIL			
AUXILIARY	(-)8'-0"	5.26	B	X		14	U		T.C.	R.E.B.	PASS	N/A	NO	N/A	N/A			

8 7 6 5

SIZE E

Wall No.	Originator	Checker	Elastic Analysis Pass/Fail Non-Composite	Elastic Analysis Pass/Fail Composite	Repair Yes/No	Repair Type	Inelastic Analysis Pass/Fail	Comments
.28	R.W.K.	R.E.M.	FAIL	NOT INVESTIGATED	YES	PROVIDE SUPPORT [illegible]	N/A	
.29	T.L.	T.H.	FAIL	PASS	YES	[illegible]	PASS	
.30	TSS/REM	REM/TSS	FAIL	PASS	YES	[illegible]	PASS	
.31	T&B	R.E.M.	FAIL	NOT INVESTIGATED	YES	[illegible]	PASS	
.35	RWK/REB	REM/RWK	PASS	N/A	NO	N/A	N/A	
[illegible]	MH/JC	TC/KLB	FAIL	PASS	NO	N/A	PASS	
[illegible]	R.W.C.	J.C.	FAIL	PASS	YES	COLUMNS TO CUT [illegible] SPAN	NOT INVESTIGATED	FLAT ANALYSIS BY HAND
[illegible]	R.W.C.	R.E.M.	PASS	N/A	NO	N/A	N/A	FLAT PLATE ANALYSIS
[illegible]	M.H.	REB/REM	FAIL	PASS	NO	N/A	N/A	[illegible]
[illegible]	TSS/REM	REM/REM	FAIL	FAIL	YES	REDUCE SPAN	NOT INVESTIGATED	
[illegible]	R.E.B.	J.C.	PASS	N/A	NO	N/A	N/A	
[illegible]	M.H.	R.E.M.	FAIL	FAIL	YES	REDUCE SPAN	FAIL	
[illegible]	RWC/JC	JC/REM	FAIL	PASS	YES	[illegible]	PASS	
[illegible]	REM/JC	[illegible]	FAIL	PASS	YES	PROVIDE SUPPORT [illegible]	FAIL	[illegible]
[illegible]	REM	T.L.	FAIL	PASS	YES	[illegible]	FAIL	[illegible]
[illegible]	REB	T.B.	PASS	N/A	NO	N/A	N/A	
[illegible]	REM	REB	PASS	N/A	NO	N/A	N/A	(N.W. [illegible])
[illegible]	TSS/RW	RWC	PASS	N/A	YES	REPAIR HANGERS [illegible]	N/A	[illegible]
[illegible]	RWK	J.C.	PASS	N/A	NO	N/A	N/A	[illegible]
[illegible]	RWC	T.L.	PASS	N/A	NO	N/A	N/A	
[illegible]	J.H.	RWC	PASS	N/A	NO	N/A	N/A	
[illegible]	J.H.	REM	FAIL	FAIL	YES	[illegible]	FAIL	
[illegible]	J.C.	R.E.M.	FAIL	FAIL	YES	[illegible]	FAIL	[illegible] WALL
[illegible]	RWM	RWC	FAIL	[illegible]	YES	[illegible]	PASS	
[illegible]	REM	T.L.	FAIL	PASS	YES	[illegible]	FAIL	
[illegible]	J.C.	T.L.	PASS	N/A	NO	N/A	N/A	
[illegible]	REM	J.C.	FAIL	FAIL	YES	ADD COLUMN TO REDUCE SPAN	NOT APPLICABLE [illegible]	
[illegible]	T.G.	YPS/JT	PASS	N/A	NO	N/A	N/A	
[illegible]	T.G.	J.C.	PASS	N/A	NO	N/A	N/A	
[illegible]	T.G.	YPS	PASS	N/A	NO	N/A	N/A	[illegible]
[illegible]	T.G.	REM	PASS	N/A	NO	N/A	N/A	
[illegible]	REM	T.L.	PASS	N/A	NO	N/A	N/A	[illegible] LOADS
[illegible]	[illegible]	REM	PASS	N/A	YES	REDUCE SPAN	N/A	[illegible] LOADS
[illegible]	REM	RWC	PASS	N/A	NO	N/A	N/A	
[illegible]	J.C.	[illegible]	FAIL	N/A	YES	PROVIDE SUPPORT [illegible]	N/A	[illegible] LOADS
[illegible]	RWC	TSS	FAIL	N/A	YES	REDUCE SPAN	N/A	[illegible] LOADS
[illegible]	TSS	YPS	FAIL	N/A	YES	REDUCE SPAN	N/A	
[illegible]	REM	MRL	N/A	N/A	NO	N/A	N/A	[illegible]
[illegible]	RWM	J.C.	PASS	N/A	NO	N/A	N/A	
[illegible]	RWM	REM	PASS	N/A	NO	N/A	N/A	
[illegible]	RWM	REB	PASS	N/A	NO	N/A	N/A	[illegible] 1 SIDE
[illegible]	RWM	REM	PASS	N/A	YES	[illegible]	N/A	[illegible] 1 SIDE
[illegible]	J.H.	REM	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	RWK	T.G.	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	RWC	REM	PASS	N/A	NO	N/A	N/A	
[illegible]	TC/RWK	[illegible]	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	[illegible]	J.H.	PASS	N/A	YES	[illegible] CELLS	N/A	[illegible] LOADS
[illegible]	RWP	[illegible]	PASS	N/A	NO	N/A	N/A	[illegible] 1 SIDE
[illegible]	J.H.	J.C.	PASS	N/A	NO	N/A	N/A	[illegible] 1 SIDE
[illegible]	J.H.	[illegible]	PASS	N/A	NO	N/A	N/A	[illegible] 1 SIDE
[illegible]	T.P.	J.C.	PASS	N/A	NO	N/A	N/A	[illegible] 1 SIDE
[illegible]	RWK	REM	PASS	N/A	YES	[illegible] CELLS	N/A	[illegible] LOADS
[illegible]	RWK	KLB	PASS	N/A	NO	N/A	N/A	[illegible] LOADS
[illegible]	J.H.	T.L.	FAIL	N/A	YES	REDUCE SPAN	N/A	
[illegible]	J.H.	REM	PASS	N/A	NO	N/A	N/A	
[illegible]	J.H.	REM	PASS	N/A	NO	N/A	N/A	
[illegible]	J.H.	REM	FAIL	N/A	YES	PROVIDE SUPPORT [illegible]	N/A	
[illegible]	T.L.	J.C.	PASS	N/A	NO	N/A	N/A	
[illegible]	T.L.	REM	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	REB	REM	FAIL	N/A	YES	PROVIDE SUPPORT [illegible] ANGLES	FAIL	[illegible] LOADS
[illegible]	MH/JC	TC/REM	PASS	N/A	NO	N/A	N/A	
[illegible]	MP/JC	TC/PHP	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	RPH/JC	REP	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	RWC/TC	TC/REM	PASS	N/A	YES	ANGLES AT TOP	N/A	
[illegible]	RWC		FAIL	N/A	YES	REDUCE SPAN	FAIL	[illegible] LOADS

LEGEND:
- ◤ APPROVED CALC.
- ◸ UNDER REVIEW
- U UNREINFORCED MASONRY WALL
- R REINFORCED MASONRY WALL
- W WALL
- B BLOCKOUT
- 2/1 TWO OVER-ONE SITUATION
- SR WALL OR BLOCKOUT HAS SAFETY-RELATED ATTACHMENTS

BECHTEL CORPORATION
GAITHERSBURG, MARYLAND

THE MILLSTONE POINT COMPANY
SUBSIDIARY OF
NORTHEAST UTILITIES

TITLE
MILLSTONE NUCLEAR POWER STATION UNIT NO. 2
I.E. BULLETIN 80-11
MASONRY WALL STATUS

ATTACHMENT A