CP&L

Carolina Power & Light Company

SERIAL: LAP-83-291

July 29, 1983

Director of Nuclear Reactor Regulation
Attention: Mr. D. B. Vassallo, Chief
Operating Reactors Branch No. 2
Division of Licensing
United States Nuclear Regulatory Commission
Washington, DC 20555

BRUNSWICK STEAM ELECTRIC PLANT, UNIT NOS. 1 AND 2
DOCKET NOS. 50-325 AND 50-324
LICENSE NOS. DPR-71 AND DPR-62
IE BULLETIN 80-11, MASONRY DESIGN
REQUEST FOR ADDITIONAL INFORMATION

Dear Mr. Vassallo:

By letter dated August 2, 1982, you requested Carolina Power & Light Company (CP&L) to provide additional information needed to complete your review of our responses to IE Bulletin 80-11, Masonry Design, for the Brunswick Steam Electric Plant Unit Nos. 1 and 2. The attached submittal addresses the questions outlined in your August 2, 1982 request for additional information (RAI) with the exception of RAI 13. RAI 13 requests CP&L to provide detailed drawings and a current status of proposed repairs. Carolina Power & Light Company has designed some fixes to be implemented in 1983. The design for the remaining fixes will be completed in 1984. Detailed drawings are not available at this time, but will be provided as the fixes are implemented.

If you have any questions concerning this information, please contact us.

Yours very truly,

S. R. Zimmerman
Manager
Licensing & Permits

CEH/pgp (7264CEH)
Attachment

cc: Mr. D. O. Myers (NRC-BSEP)
Mr. S. D. MacKay (NRC)
Mr. J. P. O'Reilly

IE11
1/4

RESPONSE

TO

NRC

REQUEST FOR ADDITIONAL INFORMATION

MASONRY WALL DESIGN

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 AND 2

PREPARED BY: J. J. Ucciferro 3-11-83
J. J. Ucciferro Date

Joseph Katz 3/10/83
J. Katz Date

APPROVED BY: L. R. Scott 3/15/83
L. R. Scott Date

REQUEST FOR ADDITIONAL INFORMATION
MASONRY WALL DESIGN
BRUNSWICK STEAM ELECTRIC PLANT, UNITS 1 AND 2

Technical Evaluation

A technical evaluation was conducted based on the Licensee's response to IE Bulletin 80-11. In general, the Licensee's response is satisfactory, but some information is still required to facilitate a proper final evaluation. It is noted that sufficient information has not been provided to justify certain damping values and increase factors for allowable stresses at Brunswick Units 1 and 2. More information is also required regarding the energy balance technique and arching theory. Before a final technical evaluation report can be issued, the Licensee is requested to provide the following information:

RAI 1a - Indicate whether the walls have stack bond or running bond.

RAI 1b - If any stack bond wall exists, provide sample calculations to obtain moment and shear stress of a typical wall.

RAI 2 - Indicate how frequency variations due to uncertainties in mass, materials and other parameters were considered.

RAI 3 - Describe how in-plane interstory drift was considered.

RAI 4 - Indicate if cracking of sections was given proper consideration in the analysis.

RAI 5a - Indicate whether the block pullout was considered in the evaluation.

RAI 5b - If yes, provide sample calculations of block pullout analysis.

RAI 6a - In Reference 3, the Licensee indicated that loads and load combinations are based on NRC Standard Review Plan for the elastic design method. The Licensee is requested to clarify whether they are consistent with Plant Final Safety Analysis Report (FSAR). If any deviations exist, justification should be given.

RAI 6b - With reference to load combinations, the Licensee is requested to provide justification for the stress factors of 1.5 for dead plus live plus abnormal temperature loads and 1.1 for dead plus live plus DBE seismic plus abnormal temperature loads.

Technical Evaluation (cont'd)

RAI 6c - In Reference 3, the Licensee indicated that impulsive and impactive loads were considered. Describe types of these loads (pipe rupture, missile impact, etc.) Also provide a sample calculation illustrating how these loads were treated in the analysis.

RAI 7 - Show, by sample calculation, how the effect of higher modes of vibration was considered in the analysis.

RAI 8a - Indicate whether the construction practice for the masonry walls at the Brunswick plant was in conformance with the provisions specified for the special inspection category in ACI 531-79 [8].

RAI 8b - If not, explain and justify the use of allowable stresses.

RAI 9a - With respect to Tables A-2 and A-3 [3], justify the use of the following increase for factored loads (the increase factors allowed in the SEB criteria [6] are shown in parentheses)

shear in flexural members	1.5 (1.3)
tension normal to the bed joints	1.67(1.3)
tension parellel to the bed joints	1.67(1.5)

RAI 9b - If the Licensee intends to use any existing test data to justify these factors, the Licensee is requested to discuss the applicability of those tests to the masonry walls at the plant to the following areas:

- nature of loads
- boundary conditions
- material used
- size of test walls

RAI 10a- In Reference 3, the Licensee indicates that the energy balance technique and arching theory have been used to qualify some masonry walls. The NRC, at present, does not accept the application of these techniques 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 each of these techniques and provide the resulting stresses and displacements.

RAI 10b--The following areas need technical verification before any conclusion can be made about these techniques:

1. Energy Balance Technique

Technical Evaluation (cont'd)

RAI 10b - 1. Energy Balance Technique

- For the walls which were analyzed using the energy balance technique, provide technical basis to insure that the ductile mode of failure will take place (if they fail).

- Provide justification and test data (if available) to validate the applicability of the energy balance technique to the masonry structures at Brunswick Units 1 and 2 with particular emphasis on the following areas:

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

RAI 10c - The following areas need technical verification before any conclusion can be made about these techniques:

2. Arching Theory -

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

- Provide justification and test data (if available) to validate the applicability of the arching theory to the masonry structures at Brunswick Units 1 and 2 with particular emphasis on the following areas:

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

- If the hinges are formed in the walls, the capability of the structure 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 effect of this phenomenon can be accurately determined.

Technical Evaluation (cont'd)

RAI 11 - Regulatory Guide 1.61 allows 4% damping for an OBE and 7% damping for an SSE. Provide justification for using 10% damping for unreinforced walls in the arching action analysis.

RAI 12a - With reference to multiple wythes, clarify whether the collar joint strength was used in the analysis. If so, justify the allowable stresses of the collar joint.

RAI 12b - Also, provide sample calculations illustrating the analysis of multiple wythe walls.

RAI 13a - Provide detailed drawings and current status of proposed repairs

RAI 13b - Also, provide sample calculations to illustrate that the modified walls will be qualified under the working stress design condition.

RAI 1a - Indicate whether the walls have stack bond or running bond.

RESPONSE

There are no stack bond walls, all walls have running bond construction.

RAI 1b - If any stack bond wall exists, provide sample calculations to obtain moment and shear stress of a typical wall.

RESPONSE

There are no stack bond walls, therefore, there are no sample calculations.

RAI 2 - Indicate how frequency variations due to uncertainties in mass, materials and other parameters were considered.

RESPONSE

Variations in frequency were conservatively accounted for by varying the modulus of elasticity, E_m, between the following limits:

A. Hollow masonry: 1,000 f'_m - 600 f'_m

B. Solid or grouted masonry: 1,200 f'_m - 800 f'_m

This conservative variation in material properties is deemed sufficient to account for the variation in parameters such as mass, materials, etc. Frequency variations for unreinforced and reinforced masonry walls are given below.

A. Unreinforced Concrete Masonry Walls

The seismic acceleration for unreinforced concrete walls with rigid supports is selected as the maximum response between the following frequency range:

1. One-Way Behavior

a. Hollow masonry

$$K\sqrt{E_m\ I} \geq f \geq K\sqrt{0.6\ E_m\ I}$$

b. Solid or grouted masonry

$$K\sqrt{1.2\ E_m\ I} \geq f \geq K\sqrt{0.8\ E_m\ I}$$

where

I = the uncracked moment of inertia.

RAI 2

RESPONSE (cont'd)

A. Unreinforced Concrete Masonry Walls (cont'd)

2. Two-Way Behavior

a. Hollow masonry

$$K\sqrt{D} \geq f \geq K\sqrt{0.6\,D}$$

b. Solid or grouted masonry

$$K\sqrt{1.2\,D} \geq f \geq K\sqrt{0.8\,D}$$

B. Reinforced Concrete Masonry Walls

1. One-Way Behavior

The seismic acceleration for reinforced concrete masonry walls with rigid supports in which one-way behavior is considered is selected as the maximum response between the following frequency range:

$$K\sqrt{1.2\,E_m\,I_t} \geq f \geq K\sqrt{0.8\,E_m\,I_t}$$

where

I_t = uncracked transformed moment of inertia

If the applied moment (M_a) due to all loads in a load combination exceeds the uncracked moment capacity (M_{cr}), the wall should be considered to be cracked. In this situation, the effective moment of inertia (I_e) may be calculated as follows:

RAI 2

RESPONSE

B. Reinforced Concrete Masonry Walls (Con't)

1. One-Way Behavior (Cont'd)

$$I_e = \left\{\frac{M_{cr}}{M_a}\right\}^3 I_t + \left\{1 - \left[\frac{M_{cr}}{M_a}\right]^3 I_{cr}\right\}$$

$$M_{cr} = f_r \left\{\frac{I_t}{Y}\right\}$$

Where

M_{cr} = Uncracked moment capacity

M_a = Applied maximum moment on the wall

I_t = Moment of inertia of transformed section

I_{cr} = Moment of inertia of the cracked section

f_r = Modulus of rupture

y = Distance of neutral plane from tension face

The maximum seismic response is then selected between the following frequency range:

$$K \sqrt{1.2\, E_m\, I_e} \geq f \geq K \sqrt{0.8\, E_m\, I_e}$$

If the use of I_e results in an applied moment M_a which is less than M_{cr}, then the wall may be verified for M_{cr}.

2. Two Way Behavior

The seismic acceleration for reinforced concrete walls in which two-way behavior is considered is selected as the maximum response between the following frequency range:

RAI 2

RESPONSE

B. Reinforced Concrete Masonry Walls (Con't)

2. Two Way Behavior (Cont'd)

a. For uncracked walls ($M_a \leq M_{cr}$)

$$K \sqrt{1.2 \frac{I_t}{I} D} \geq f \geq K \sqrt{0.8 \frac{I_t}{I} D}$$

b. For cracked walls ($M_a \geq M_{cr}$)

$$K \sqrt{1.2 \frac{I_e}{I} D} \geq f \geq K \sqrt{0.8 \frac{I_e}{I} D}$$

Where

I = The moment of inertia of the uncracked concrete masonry alone (non-transformed section).

RAI 3 Describe how in-plane interstory drift was considered.

RESPONSE

In-plane interstory drift was considered by comparing the in-plane strain induced in the wall (relative displacement between the top and bottom of wall divided by the wall height) to the following in-plane strain limits. This criteria was conservatively established for in-plane effects in order to provide adequate margin for out of plane effects.

For unconfined Walls - not bounded by an adjacent steel or concrete primary structure:

$$\alpha = 0.0001$$

For Confined Walls - at a minimum, bounded top and bottom or bounded on three sides:

$$\alpha = \frac{1 + \left[\frac{B}{H}\right]^2}{2000 \frac{B}{H}}$$

Where:

B = Wall width

H = Wall height

All masonry walls at Brunswick respond within the above limits.

All masonry walls are non-structural and do not contribute to the global response of the building. Masonry walls are used for shielding or infills, etc. The design basis earthquake consisted of one horizontal and one vertical component. When the maximum in-plane strain is applied to the wall little if any out of plane forces are present. This conservatively established criteria for masonry

RAI 3 Describe how in-plane interstory drift was considered.

RESPONSE (cont'd)

walls with both in-plane and out of plane loading was conservatively applied in the evaluation of masonry walls with essentially in-plane loading.

RAI 4 - Indicate if cracking of sections was given proper consideration in the analysis.

RESPONSE

Cracking is not permitted in unreinforced masonry walls. For reinforced masonry walls cracking was properly accounted for in both frequency and strength calculations. Frequency variations which account for cracking are described in the answer to RAI 2. Cracking is conservatively accounted for in strength calculations by assuming the masonry takes no tension (reinforced masonry).

RAI 5a - Indicate whether the block pullout was considered in the evaluation.

RESPONSE

Block pullout was considered. Prelimary calculations were made to determine the "Pull-out Strength" of one 8" x 8" x 16" block in both a reinforced and unreinforced wall. See Attachment RAI 5a for sample calculations. Field surveys of the various attachments to the masonry walls at the Brunswick Plant were reviewed.

There were no attachments supports to individual blocks with loadings in excess of the "Pull-out Strength".

SAMPLE CALCULATIONS

9527-088

SHEETS 1 & 2

(2 PAGES)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 5a

GENERAL COMPUTATION SHEET

united engineers & constructors inc

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
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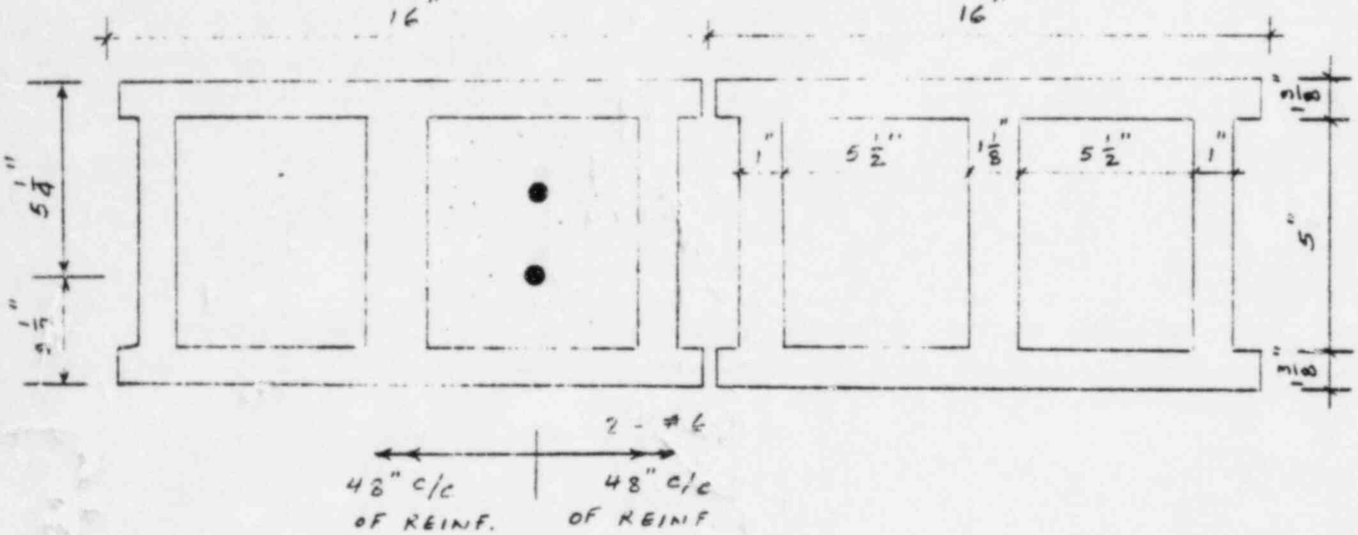
NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT MASONRY WALLS

PULL-OUT STRENGTH OF ONE BLOCK -



(1) UNREINFORCED BLOCK - NOMINAL SIZE 8 x 8 x 16

PULL OUT STRENGTH OF BLOCK = LOAD ALLOWED BY SHEAR STRESS IN MORTAR JOINTS

ASSUME MORTAR ONLY ON FLANGES (CONSERVATIVE)

TOP OF BLOCK -	2 x 16 x 1.375	=	44 in^2
BOTT OF BLOCK -	2 x 16 x 1.375	=	44 in^2
LEFT SIDE -	2 x 8 x 1.375	=	22 in^2
RIGHT SIDE	2 x 8 x 1.375	=	22 in^2
	TOTAL AREA		132 in^2

ALLOWABLE SHEAR STRESS $= 1.1\sqrt{f_m}$

$= 1.1\sqrt{1820} = 47$ psi

PULL-OUT STRENGTH = $132^{in^2} \times 47^{psi} = 6204^{\#}$

SAY $6200^{\#}$

5007 REV. 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc.

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NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT MASONRY WALLS

PULL-OUT STRENGTH OF REINF BLOCK -

(1) MORTAR JOINT (FROM SHT #1) = 6204#

(2) 2-#6 BARS - 2 x 2 x 0.44 in² x (0.4 x 60000 psi) = 42240#
(DOUBLE SHEAR)

(3) FILLED CELL AROUND REINF.
2 x 5" x 5.5" x 47 psi = 2585#
(DOUBLE SHEAR)

TOTAL 51029#

SAY 51,000#

RAI 5b - If yes, provide sample calculations of block pullout analysis.

RESPONSE

See response to RAI 5a.

RAI 6a - In Reference 3, the Licensee indicated that loads and load combinations are based on NRC Standard Review Plan for the elastic design method. The Licensee is requested to clarify whether they are consistent with Plant Final Safety Analysis Report (FSAR). If any deviations exist, justification should be given.

RESPONSE

Loads and load combinations are consistent with those in the Plant Final Safety Analysis Report (FSAR).

RAI 6b - With reference to load combinations, the Licensee is requested to provide justification for the stress factor of 1.5 for dead plus live plus abnormal temperature loads and 1.1 for dead plus live plus DBE seismic plus abnormal temperature loads.

RESPONSE

Thermal stresses were not relied upon to reduce the resulting tension, and increases for load combinations which included temperature were not taken. The increases were included in the criteria because thermal loads are secondary and self relieving in nature. Stress increases are normally taken in design for load equations involving temperature. The masonry walls evaluated in this program were not subjected to postulated temperature gradients through the thickness. Therefore the walls would not experience thermal induced flexural or shear stresses.

RAI 6c - In Reference 3, the Licensee indicated that impulsive and impactive loads were considered. Describe types of these loads (pipe rupture missile impact, ect).

RESPONSE

The impulsive load provisions pertain to masonry walls which separate the diesel generators in the Diesel Generator Building. Each side of the masonry walls is protected by ¼" steel plate attached by through bolting with 3/4" diameter bolts. Two air compressors, which contain compressed air at 350 psig, exist in each of the diesel generator rooms between the steel plate protected masonry walls.

The following are commitments related to potential missiles generated by the air receivers:

CASE 1 - A two (2) inch diameter plug of weight 1.38 lbs which becomes loose and is propelled by exhausting air.

CASE 2 - The air receiver is punctured and becomes a jet propelled missile.

CASE 3 - The air receiver explodes into fragments. A fragment is idealized as a 2 inch diameter circular disc.

RAI 6d - Also, provide a sample calculation illustrating how these loads were treated in the analysis.

RESPONSE

The sample calculation, Attachment RAI 6d, describes how impactive effects were considered. Both local and overall response was accounted for.

The sample calculation relates to Case 1, See RAI 6c, (2 inch diameter plug of 1.38 lb weight). This was the most severe missile impact case and was shown to envelop that associated with Case 3. For Case 2 a puncture in the most severe location was postulated and it was shown that attachments and supports of the air receivers were adequate to prevent impact on the masonry walls. Therefore, no impact calculations were performed for Case 2.

SAMPLE CALCULATIONS

9527-088

SHEETS 1 to 3

(3 PAGES)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 6d

07 REV. 7/79

GENERAL COMPUTATION SHEET

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(LINE)

NAME OF COMPANY: CP&L - BSEP UNIT/S: 1, 2

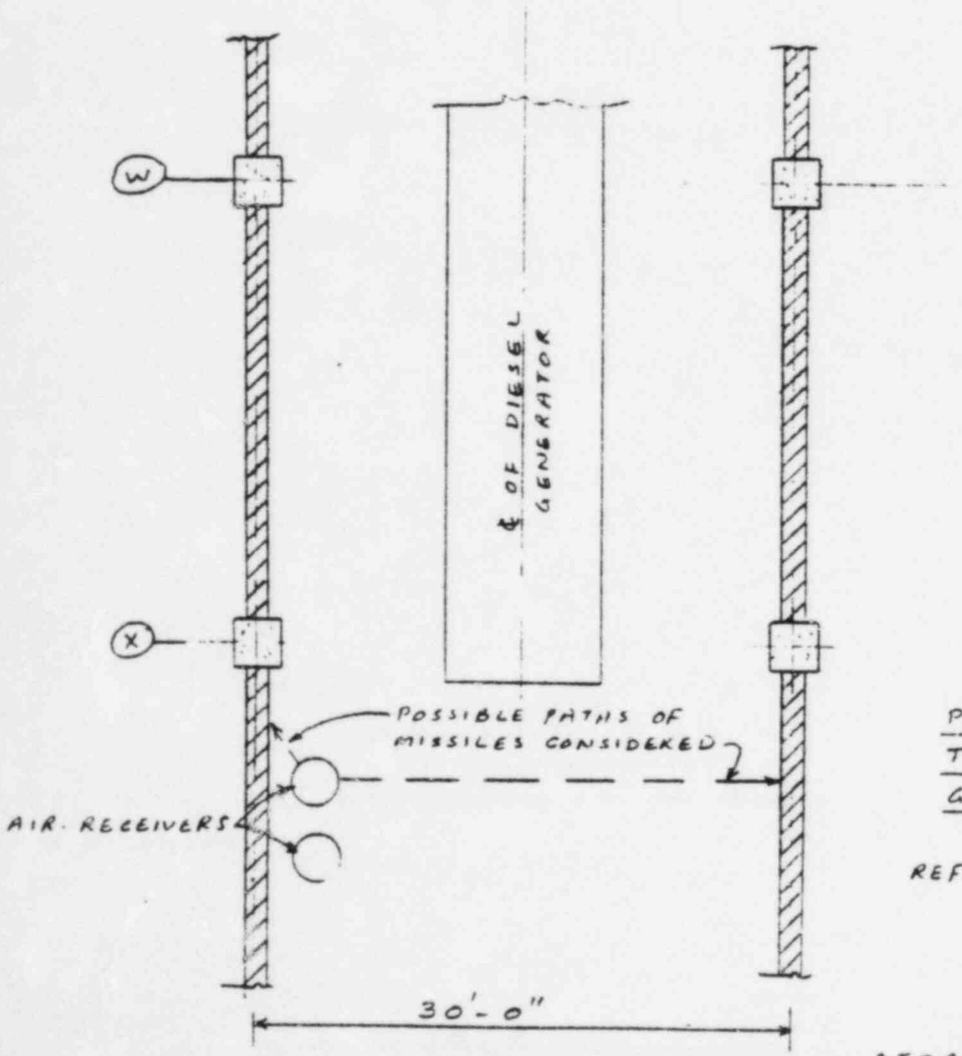
SUBJECT: DIESEL GEN BLDG - BLOCK WALLS

WALLS 7a, b, c, d — SAMPLE CALCULATIONS

THESE WALLS SEPARATE THE DIESEL GENERATORS. IN EACH ROOM THERE ARE TWO AIR RECEIVERS WHICH CONTAIN COMPRESSED AIR AT 350 PSI (GAUGE) OPERATING PRESSURE USED FOR STARTING THE DIESEL GENERATORS.

THREE TYPES OF MISSILES HAVE BEEN POSTULATED.

1. A PLUG OF WT 1.38 LBS, 2 IN DIA BECOMES LOOSE AND IS PROPELLED BY EXHAUSTING AIR
2. THE AIR RECEIVER IS PUNCTURED AND BECOMES A JET-PROPELLED MISSILE.
3. THE AIR RECEIVER EXPLODES INTO FRAGMENTS, A FRAGMENT IS IDEALIZED AS 2 IN DIA CIRCULAR DISC.



PART PLAN OF TYPICAL DIESEL GEN ROOM

REF. DWG. 9527-F-2214

ATTACHMENT RAI 6d-1

07 REV. 7/79

GENERAL COMPUTATION SHEET

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SHEET 2 OF 3			DATE	DATE
J.O. 9527-088				

NAME OF COMPANY CP&L - BSEP UNIT/S 1, 2

SUBJECT DIESEL GEN. BLDG. - BLOCK WALLS

SAMPLE CALCULATIONS

WALLS 7a, b, c, d - CONT'D.

CHECK THICKNESS OF PROTECTIVE PLATE -
USE BRL FORMULA -

$$T^{3/2} = \frac{0.5 m v^2}{17400 K^2 D^{3/2}}$$ WHERE

T = PENETRATION THICKNESS
m = MASS OF MISSILE, SLUG
V = VELOCITY OF MISSILE, FT/SEC
K = 1
D = DIA. OF MISSILE, INCH

$m = \frac{1.38^{\#}}{32.2} = 0.04286$ SLUGS

V = 310 FT/SEC, D = 2"

$$T^{3/2} = \frac{0.5 \times 0.04286 \times 310^2}{17400 \times 1 \times 2^{3/2}} = 0.0418$$

T = 0.12 < 0.25" THICKNESS PROVIDED O.K

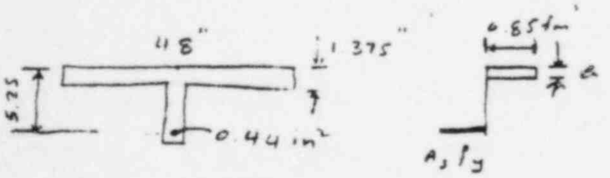
CHECK OVERALL RESPONSE OF WALL -

KINETIC ENERGY of MISSILE = ENERGY ABSORBED BY THE WALL DURING DEFORMATION

KINETIC ENERGY OF MISSILE $= \frac{1}{2} m v^2$

$= \frac{1}{2} \times \frac{1.38}{32.2} \times 310^2 \times 12 = 24712$ "#

- CALCULATE ULTIMATE STRENGTH OF WALL -



$A_s f_y = 0.85 f'_m a b$

$0.44 \times 60000 = 0.85 \times 1820 \times a \times 48$

$a = \frac{0.44 \times 60000}{0.85 \times 1820 \times 48} \approx 0.36 < 1.375"$

$m.R. = 0.9 \times 0.44 \times 60000 \left(5.25 - \frac{0.36}{2}\right) \times \frac{1}{12} \times \frac{1}{4} = 2510$ '#/,

ATTACHMENT RAI 6d-2

SAMPLE CALCULATIONS

WALLS 7a, b, c, d - CONT'D.

- CALCULATE DEFLECTION OF WALL AT YIELD - (Δ_y)

$$\Delta_y = \frac{m_y L^2}{12 E I} \qquad L = 23.25' \times 12 = 279'' \qquad I = 35.4 \text{ (USE } I_{CR}\text{)}$$

$$= \frac{2510^{\#} \times 12 \times 279^2}{12 \times 1.82 \times 10^6 \times 35.4} \approx 3''$$

- CALCULATE MOMENT DUE TO MISSILE IMPACT

ASSUME KINETIC ENERGY DUE TO MISSILE IMPACT IS ABSORBED BY 6' WIDTH OF WALL -

ENERGY PER FT WIDTH = $\frac{24712}{6} = 4119''^{\#}$

$$\frac{m^2 L}{6 E I} = 4119$$

$$m^2 = \frac{6 \times 1.82 \times 10^6 \times 35.4 \times 4119}{279}$$

$$m = 75545''^{\#} \times \frac{1}{12} = 6295 \text{ FT LBS}$$

- CALCULATE ULTIMATE WALL DEFLECTION -

$$\Delta_e = \frac{m L^2}{12 E I}$$

$$= \frac{6295 \times 12 \times 279^2}{12 \times 1.82 \times 10^6 \times 35.4} = 7.6''$$

- CALCULATE DUCTILITY RATIO -

$$\frac{\Delta_y}{\Delta_e} = \frac{1}{\sqrt{2\mu - 1}}$$

$$\frac{1}{2\mu - 1} = \left(\frac{3}{7.6}\right)^2 = 0.156$$

$$2\mu - 1 = 6.41$$

$$\mu = \frac{7.41}{2} = 3.7 < 10 \quad \underline{\text{O.K.}}$$

ATTACHMENT RAI 6d-3

RAI 7 - Show, by sample calculation, how the effect of higher modes of vibration was considered in the analysis.

RESPONSE

A study contained in the "Recommended Guidelines for the Reassessment of Safety Related Masonry Walls" dated October 6, 1980 prepared by Owners and Engineering Firms Informal Group on Concrete Masonry Walls contains a study which demonstrates that the first mode contributes to over 99% of the total flexural response (see Attachment RAI7). Similar results are expected for shear at the boundary. For this reason higher modes were not accounted for in the calculation of stresses. For the cases where amplification was predicted, loads were conservatively applied since peak accelerations were assumed to exist uniformly over the entire wall instead of the actual distribution which is more sinusoidal in nature.

RECOMMENDED GUIDELINES

FOR THE

REASSESSMENT OF

SAFETY RELATED CONCRETE MASONRY WALLS

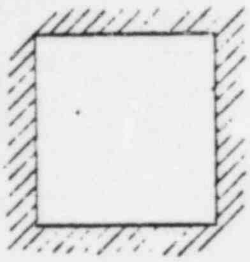
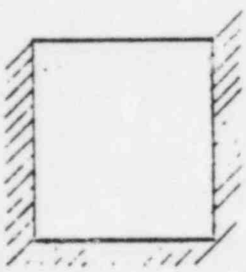
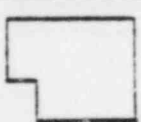
PAGE 3 - 22

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 7

Support Case	h/L	Modes	Contribution from First Mode Moment for SRSS of All Mode Moments	
	1.0	1 thru 8	99.30%	
	.67			
	.54	1 thru 8	99.86%	
	1.0	1 thru 5	99.82%	
	0.67	1 thru 5	99.78%	
	0.50	1 thru 5	99.70%	
 FIXED	1.0	1 thru 8	99.75%	R1

COMPARISON OF FIRST MODE MOMENT TO SRSS MOMENT

RAI 8a - Indicate whether the construction practice for the masonry walls at the Brunswick plant was in conformance with the provisions specified for the special inspector category in ACI-531-79 [8].

RESPONSE

The construction practice, for the masonry walls at the Brunswick plant, was not in conformance with the provisions specified for the special inspection category in ACI-531-79 [8].

Construction practices utilized in the laying up of these walls were the normally accepted practices for masonry work at the time the plant was constructed.

Because the masonry work was being done for a nuclear plant, a higher level of daily inspections by the Superintendents for the Sub-Contractor, Contractor and Owner was performed during the construction of the masonry walls.

RAI 8b - If not, explain and justify the use of allowable stresses.

RESPONSE

Allowable stresses utilized in the design of the masonry walls at the Brunswick plant were those normally for masonry work at the time the plant was constructed and in conformance with:

Specification No.	Description	Rev.	Date
9527-01-29-1	Specification for Masonry and Caulking	3	4/31/72
NCMA	Specification for the Design & Construction of Load Bearing Concrete Masonry		
REPORT 3357	Pittsburgh Testing Laboratory		5/5/72
REPORT 3911	Pittsburgh Testing Laboratory		5/24/72

RAI 9a - With respect to Tables A-2 and A-3 [3], justify the use of the following increase for factored leads (the increase factors allowed in the SEB criteria [6] are shown in parentheses)

shear in flexural members	1.5	(1.3)
tension normal to the bed joints	1.67	(1.3)
tension parallel to the bed joints	1.67	(1.5)

RESPONSE

Increase factors for both flexural members and shear walls have been established based on tests for shear walls.

RAI 9b - If the Licensee intends to use any existing test data to justify these factors, the Licensee is requested to discuss the applicability of those tests to the masonry walls at the plant to the following areas:

- nature of loads
- boundary conditions
- material used
- size of test walls

RESPONSE

SHEAR (Reinforced)

Two major test programs have evaluated the shear strength on concrete block masonry walls. The first was performed by Schneider and his test results were used as the basis for developing the UBC, NCMA and ACI code allowable stresses for reinforced masonry.

A more recent and extensive test program has been performed at the University of California, Berkeley and these results will be used as a comparison with the code allowables. The test results are shown in Attachment RAI 9b-1, lower bound values are indicated for reinforcement taking all the shear and masonry taking all the shear. These are compared to the allowables recommended for unfactored and factored loads in Attachment RAI 9b-2.

The discussion and related test data shown in Attachment RAI 9b-3 & 4 demonstrate the suitability of the designated stress increase factors for use in the re-evaluation program. The test data which is presented represents the majority of existing test data on which the allowable stresses in governing Codes are based. This data was carefully reviewed

RAI 9b

RESPONSE (cont'd)

in order to establish the allowable stresses for both the service and factored load conditions.

The allowable stresses in the governing Codes which are based on this test data are generally accepted for all masonry walls regardless of the nature of loads, boundary conditions, materials, and size of walls. Therefore, since the same data was used in establishing the allowable stresses for factored loads (with a suitable margin of safety) the allowable stresses for the factored condition are suitable for the range of materials, geometry, boundary conditions, etc. which exist at Brunswick.

RECOMMENDED GUIDELINES

FOR THE

REASSESSMENT OF

SAFETY RELATED CONCRETE MASONRY WALLS

FIGURE 2

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CAROLINA POWER & LIGHT COMPANY

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UNITS 1 & 2

ATTACHMENT RAI 9b-1

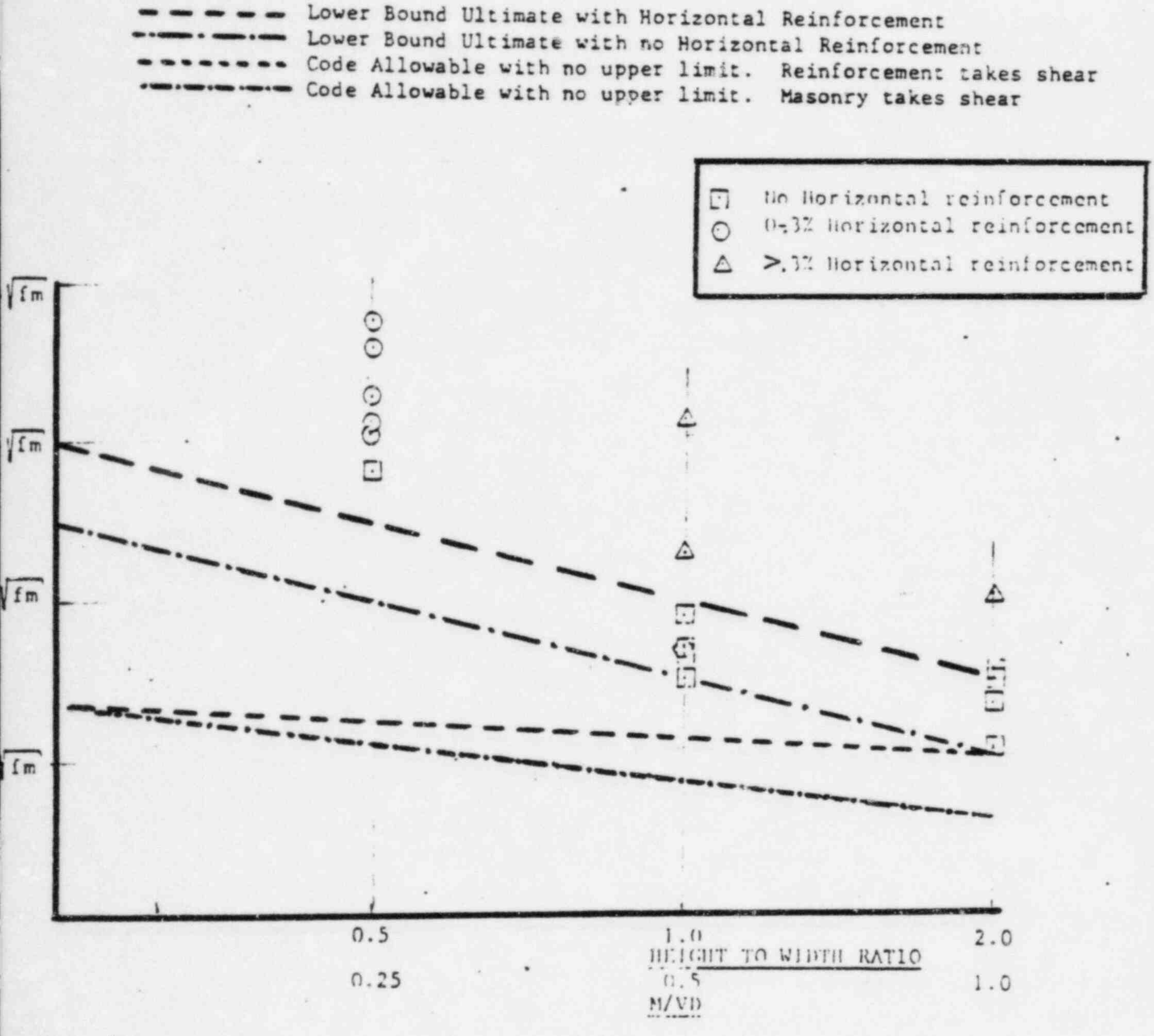
Lower Bound Ultimate with Horizontal Reinforcement
Lower Bound Ultimate with no Horizontal Reinforcement
Code Allowable with no upper limit. Reinforcement takes shear
Code Allowable with no upper limit. Masonry takes shear
No Horizontal reinforcement
0-.3% Horizontal reinforcement
>.3% Horizontal reinforcement
0.5
1.0
2.0
HEIGHT TO WIDTH RATIO
0.25
0.5
1.0
M/VD

Figure 2

RECOMMENDED GUIDELINES

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UNITS 1 & 2

For the unfactored loads the factor of safety varies from 2.22 to 3.0. For the factored loads the factor of safety varies from 1.20 to 1.76. The ductility indicator associated with stress levels for the factored loads is of the order of 3 which provides an added factor of safety.

Table 4: Comparison of Test Restuls and Code Allowables

Description	S	U	Test Results	Tests/S	Tests/U
Masonry Takes Shear					
M/Vd = 1	$0.9\sqrt{f'_m}$	$1.5\sqrt{f'_m}$	$2\sqrt{f'_m}$	2.22	1.33
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$5\sqrt{f'_m}$	2.50	1.47
Reinforcement Takes Shear					
M/Vd = 1	$1.5\sqrt{f'_m}$	$2.5\sqrt{f'_m}$	$3\sqrt{f'_m}$	2.0	1.20
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$6\sqrt{f'_m}$	3.0	1.76

RECOMMENDED GUIDELINES

FOR THE

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(14 SHEETS)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 9b-3

2.C5. SHEAR (Unreinforced)

INTRODUCTION

The present literature on shear strength capability varies greatly on the approach used to determine acceptable values and to some extent, the controversey over these approaches and interpretation of the results. Debate, on the applicability of model or full size tests and the effects of monotonic versus cyclic loading further seems to complicate this resolution.

Much of the effort to define a permissible in-plane shear stress may be somewhat academic, in that the normal case for unreinforced walls being used in nuclear plant structures, the nature of the shear is one of being forced on the structural panel as a result of being confined by the building frame and not one of depending on the panel to transmit building shear forces. This forced drift or displacement results in shear stresses and strains, but because of the complex interaction between the panel and the confining structural elements strain or displacement is a more meaningful index for qualifying the in-plane performance of the panel. The area of in-plane strains is being addressed in another committee report.

The most extensive review on shear strength literature appears to have been done by Mayes, et al,[1] and published in Earthquake Engineering Research Center Report EERC No. 75-15 which was done for both brick and masonry block.

This report attempts to summarize some of the findings that appear to be pertinent towards defining permissible shear stress values that can be used for reevaluation of the non reinforced concrete masonry.

SUMMARY

The shear value of 0.9 $\sqrt{f'_m}$ provided by the ACI-531-79[2,3] code for reinforced masonry appear to be reasonable basis on which to proceed with the reevaluation program. This value appears to conservatively bound the actual expected shear strength of concrete

block masonry. A summary of several different sources for shear stress-design values is shown by Table 5. An increase in these allowable values for the re-evaluation program of $1.35\sqrt{f'_m}$ for severe loading conditions appears warranted. Any further increase at this time without further substantiation and review is not seen as advisable.

DISCUSSION

A number of tests have been identified as being the primary basis for permissible shear stress values in both National Concrete Masonry Association (NCMA) "Specification for the Design and Construction of Load-Bearing Concrete Masonry" [4,5] and the American Concrete Institute Standard "Building Code Requirements for Concrete Masonry Structures" (ACI-531-79). [2,3] No apparent tests are traceable to the origin of the Uniform Building Code (UBC) chapter 24 on "Masonry."[6]

Those tests performed to substantiate the NCMA values are primarily performed by the National Bureau of Standards (NBS) on full size (4 ft by 8 ft, and 8 ft by 8 ft) test panels. These tests were performed by Whittemore, et al and Fishburn[10] within the period 1939 to 1961. The Whittemore tests were done, as usual in that period, utilizing a hold down detail and thereby introducing a clamping or compressive stress within the assemblage. A number of studies have shown that compressive stresses affect the shear strength significantly. The Fishburn tests, utilize a racking configuration with the testing being performed on the panel in its original laid up position. A load setting up principal tension stress causing failure is an accepted measure of shear stress determination by the American Society of Testing Material for brickwork.[11] The test results from the above references used by NCMA are shown on Table 6.

The principal tests that seem to formulate the ACI 531 basis are the tests performed on concrete masonry piers for Masonry Research of Los Angeles, by Schneider.[12] These tests had a system for removing the compressive load on the specimen being loaded by

shear and were set up to vary the a/d (M/Vd) ratio and measure this effect on a parametric basis.

The two predominant failure modes of a masonry panel under shear are diagonal tension (causing a "splitting" failure) and shear bond (causing a "joint separation" failure) or some combination of these two effects. The theory behind these were elaborated on by Yokel et al.[13] The parameter of normal stress and its effects on a shear strength, which was also reviewed by Yokel[13] and Mayes[1,14], has been demonstrated to be consequential on the determination of actual shear stress capability. This parameter is not identified, today, by any of the codes[2,4,6,15,16] shown in Table 5.

It is expected that under zero or small compressive loads the predominate shear failure will be by the shear bond mode of failure. Tests which have been done with regard to the determination of joint separation were performed by Copeland and Saxer,[17] as well as Hamid, et al.[18] These tests are, by their nature, extremely sensitive to normal stress and consequently do relate the effects of normal stress on permissable shear values. This relationship is shown on Table 5. It is of interest that there appears to be good correlation between these tests on the shear strength with zero normal stress.

The Applied Technology Council (ATC) is presently reviewing a formulation for increasing the shear stress as a function of normal stress. This formulation is developed to coincide with their present permissible shear stress of 12 psi and is consistent with the UBC's fundamental direction as a <u>design</u> code, forcing reinforcing for seismicly designed masonry structures.

As a practical matter, walls subject to the conditions of confinement will experience large compressive loads - although these are difficult to determine. Compressive loads for the most part, imparted by boundary conditions and behavior of the building frame are ignored in the evaluation of the masonry panel. If these normal stresses are added the shear resistance would be increased. This implies a conservatism on the allowable shear value when one assumes this value as chosen on the basis of zero

normal stress. On this basis, and the tests results discussed, the shear value of $0.9\sqrt{f'_m}$ chosen by the ACI code appears to be justified and should be established as a reasonable basis by which to proceed with the re-evaluation.

Out of plane, or so called flexural shear is defined by the code as equalling $1.1\sqrt{f'_m}$. The derivation of this value is analogous to be permissible shear value of concrete, disregarding any reinforcement, of $1.1\sqrt{f'_c}$. Although this is somewhate different (there is no tension steel by which to determine the appropriate j distance), the actual value is a mute point since tension will be the critical value for determining out-of-plane acceptability of a flexural member.

Because of the nature of the stresses, however, and the various concerns with regard to the correctness of interpretation of the effects on boundary conditions as well as such conditions as: actual mortar properties; absorbtivity of the mortar; confinement or lack of it on the test specimen during test; arrangement and effect of actual load, it does not seem warranted to increase these stresses beyond a value of $1.35\sqrt{f'_m}$ (1.5×0.9 f'_m). This value is consistent with an adequate margin of safety for both the full panel wall test specimens referenced and the shear bond values observed by test. Any additional increase in the shear stress values for nonreinforced masonry under extreme environmental loads is not recommended at this time.

urce	Date	Shear Stress	Remarks
:I-531 [2]	79	$0.9\sqrt{f'm} \leq 34$	$M/VD \geq 1$
CA [4]	79	34 23	Type M or S Motor Type N Mortar Based on NBS tests (circa 1939-1961)
C [6]	79	12/10*	Type M or S/N Mortar *12 psi for solid units
C 3-06 [15]	78	12 $*12 + 0.20\sigma c \leq 30$	Lightweight units limited to 85 percent shear value *being proposed for compressive stresses between 0 and 120 psi
sonry [16] ociety	Proposed	$1.0\sqrt{f'm} \leq 35$	$a/1 \leq 1$ May be increased by 0.20 c (due to dead load)
mid, et al [18]	79	$76 + 1.07\sigma c$	Ultimate value based on type S mortar
peland/Saxen [17]	64	$70 + 9\sqrt{\sigma c}$ (fitted)	Ultimate value based on 2630 compressive mortar strength

) Values based on inspected workmanship
σc = compressive stress.

TABLE 6 RACKING TEST DATA--NONREINFORCED CONCRETE MASONRY WALLS (1)

Construction	Mortar Type	Ultimate Racking Load, psi, Net Mortar Bedded Area	S.F. Act./Allow	Ref.
" Hollow Units	N	66	2.87	7
	N	58	2.52	7
	N	57	2.48	7
" 3-Core Hollow	N	69	3.00	8
	N	62	2.70	8
	N	78	3.39	8
" Hollow Units	N	79	3.43	10
	N	79	3.43	10
	N	73	3.17	10
	N	119	5.17	10
	N	129	5.61	10
	N	109	4.74	10
	S	132	3.88	10
	S	139	4.09	10
	S	129	3.79	10
	S	159	4.68	10
	S	132	3.88	10
	S	159	4.68	10
-2-4 Cavity Wall f Hollow Units	M	103	3.03	9
	M	108	3.18	9
	M	102	3.00	9

Avg = 3.65

Range = 2.48 - 5.61

(1) From Reference 5

LIST OF REFERENCES
FOR SHEAR (Unreinforced)

1 Mayes and Clough, "Literature Survey - Compressive, Tensile, Bond and Shear Strength of Masonry," Earthquake Engineering Research Center, University of California, 1975.

2 ACI Standard, "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

3 Commentary on "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

4 "Specification for the Design and Construction of Load-Bearing Concrete Masonry" - NCMA - 1979.

5 Research Data and Discussion Relating to "Specification for the Design and Construction of Load Bearing Concrete Masonry" - NCMA - 1970.

6 Uniform Building Code, Chapter 24 "Masonry" - 1979.

7 Whittemore, Stang, and Parsons "Structural Properties of Six Masonry Wall Constructions," Building Materials and Structures Report No. 5., NBS - 1938.

8 Whittemore, Stang, and Parsons "Structural Properties of Two Buch-Concrete Block Constructions and a Concrete Block Wall Construction Sponsored by the National Concrete Masonry Association," Building Materials and Structures Report.

9 Whittemore, Stang, and Parsons, "Structural Properties of Concrete Block Cavity Wall Construction" Building Materials and Structures Report 21, NBS 1939.

10 Fishburn, "Effect of Motar Strength and Strength of Unit on the Strength of Concrete Masonry Walls," Monograph 36, NBS, 1961.

11 ASTM Standard Specification for Brick and Applicable Standard Testing Methods for Units and Masonry Assemblages - May 1975.

12 Schneider, "Shear in Concrete Masonry Piers," California State Polytechnic College, Pomona, California.

13 Yokel and Fattal "Failure Hypothesis for Masonry Shear Walls" - Journal of the Structural Division, March 1976.

14 "A State of the Art Review - Masonry Design Criteria" - Computech - 1980.

15 "Tentative Provisions for the Development of Seismic Regulations for Buildings" - Applied Technology Council Chapter 12 A - ATC 3-06-1978.

16 The Masonry Society Standard Building Code Requirements for Masonry Construction, First Draft.

17 Copeland and Saxer, "Tests of Structural Bond of Masonry Mortars to Concrete Block" - Journal of the Structural Division - November 1964.

18 Hamid, Drysdale, and Heidebrecht, "Shear Strength of Concrete Masonry Joints," Journal of the Structural Division - July 1979.

C6. TENSION (Unreinforced)

A. Normal to the Bed Joint

A summary of the static monotonic tests performed to determine code allowable stress for tension normal to the bed joint was given in the NCMA Specifications.

Stresses for tension in flexure are related to the type of mortar and the type of unit (hollow or solid). Research used to arrive at allowable stresses for tension in flexure in the veritcal span (i.e. tension perpendicular to the bed joints) consisted of 27 flexural tests of uniformly-loaded single-wythe walls of hollow units. These monotonic tests were made in accordance with ASTM E 72. Table 7 summarizes the test results.

From Table 7 the average modulus of rupture for walls built with Types M and S mortar is 93 psi on net area. For Type N mortar, the value is 64 psi. Applying a safety factor of four (4) to these values results in allowable stresses for hollow units as follows:

Mortar Type	Allowable Tension in Flexure
M&S	23 psi
N	16 psi

These values are consistent with those published in the 1970 ACI Committee 531 Report and which have been only slightly altered in ACI 531-79 Code.

Based upon these tests the minimum factors of safey for each mortar type are:

Mortar Type	Factor of Safety
M	3.87
S	2.60
N	2.81

To establish allowable tensile stresses for walls of solid units, the 8-inch composite walls in Table 8 were used. These walls, composed of 4-inch concrete brick and 4-inch hollow block, were greater than 75% solid, and thus were evaluated as solid masonry

construction. Modulus of rupture (gross area) for these walls averaged 157 psi, giving an allowable stress of 39 psi when a safety factor of 4 is applied. The composite wall tests in Table 8 used Type S mortar. To establish allowable stresses for solid units with Type N mortar, the mortar influence established previously for hollow units was used:

$$\frac{23}{16} : \frac{39}{f} ; f = 27 \text{ psi}$$

The minimum factor of safety for these tests for Type S mortar was 2.33.

Recent dynamic tests have been performed at Berkeley and the values of tension obtained at cracking at the mid-height of the walls are as follows: 13 psi; 20 psi; 23 psi; 27 psi.

The recommended values have a factor of safety of 2.8 with respect to the lower bound of the static tests for the unfactored loads and are towards the lower limit of the initiation of cracking for the dynamic tests. An increase of 1.67 appeared reasonable for factored loads based on the static tests.

TABLE 7
FLEXURAL STRENGTH--SINGLE WYTHE WALLS OF HOLLOW UNITS--
UNIFORM LOAD--VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	10
M	108	NCMA
M	102	10
M	97	10
M	95	NCMA
S	94	NCMA
M	91	NCMA
M	89	NCMA
N	88	4
S	84	10
S	83	NCMA
S	81	10
S	75	NCMA
S	69	NCMA
N	67	4
N	62	4
S	60	10
N	58	4
N	45	4
O	60	10
O	41	4
O	36	4
O	36	4
O	33	4
O	32	4
O	30	10
O	27	4

TABLE 8 FLEXURAL STRENGTH, VERTICAL SPAN CONCRETE MASONRY WALLS FROM TESTS AT NCMA LABORATORY

ASTM Mortar Type*	Wall				
				Modulus of Rupture	
	Nominal Thickness in.	Max. Uniform Load psf.	Net Section Modulus in 3/ft	Gross Area, psi	Net Mortar Bedded Area, psi
Monowythe Walls of Hollow Units					
M	8	85.15	80.97	61.74	88.73
M	8	87.10	80.97	63.15	90.76
M	8	91.00	80.97	65.97	94.82
M	8	103.35	80.97	74.93	107.69
S	8	62.40	80.97	45.24	69.47
S	8	72.15	80.97	52.31	75.18
S	12	183.3	164.64	57.11	93.94
S	12	161.2	164.64	50.22	82.62
Composite Walls of Concrete Brick & Hollow CMU					
S	8	222.3	103.82	161.16	180.67
S	8	219.7	103.82	159.29	178.55
S	8	187.2	78.16	135.72	202.09
S	8	228.8	103.82	165.88	185.95
S	8	218.4	78.16	158.34	235.77
S	8	223.6	78.16	162.11	241.38
S	12	171.6	139.83	53.46	103.55
S	12	150.8	139.83	46.98	91.00
S	12	156.0	139.83	48.60	94.14
S	12	213.2	139.83	66.42	128.66
Cavity Walls					
S	10	98.8	50.36	158.62	165.55
S	10	156.0	50.36	250.44	261.38
S	10	88.4	48.16	141.91	154.88
S	10	119.6	50.36	192.01	200.40
S	10	114.4	50.36	183.66	191.69
S	10	109.2	48.16	175.30	191.32
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(6-2-4)	135.2	77.80	127.38	146.63
S	12(6-2-4)	119.6	77.80	112.68	129.70

* Mortar type by proportion requirements.

B. Tension Parallel to Bed Joints

Values for allowable tension in flexure for walls supported in the horizontal span are established by doubling the allowables in the vertical span. While it is recognized that flexural tensile strength of walls spanning horizontally is more a function of unit strength than mortar, it is conservative to use double the vertical span values. Table 9 lists a summary of all published tests and indicates an average safety factor of 5.3 for the 43 walls containing no joint reinforcement and 5.6 for the 15 walls containing joint reinforcement.

It is important to note that the factor of safety for those walls loaded at the quarter points, Reference (6), have an average factor of safety of 2.02 with a minimum value of 1.22, while those loaded at the center had an average factor of safety of 6.08 with a minimum value of 3.59. However, it should be noted that the values tested at the ¼ points were also tested at 15 days.

The results associated with the early date of testing and the use of quarter point loading are difficult to explain other than to state they are at variance with all other test results.

An increase in the allowable by a factor of 1.67 is recommended for factored loads. The committee believes that the recommended values could be increased because of the larger factors of safety in the test results; however the value of 1.67 was chosen to be compatible with the increase in other stresses for unreinforced masonry.

The values recommended for stack bonded construction although at variance with current building codes (which allow zero) are thought to be reasonable values for a reevaluation program. In a test program performed by PCA(1) a horizontally spanning stack bonded wall had 1/3 the capacity of an equivalent wall laid in running bond. The recommended values are in accordance with this test data. i.e. two-thirds of the value normal to the bed joint is equivalent to 1/3 the values recommended for parallel to the bed joint.

Reference:

[1]) Portland Cement Association, "Load Tests of Patterened Concrete Masonry Walls, " Trowel Talk an aid to Masonry Industry, 1963.

TABLE 9 FLEXURAL STRENGTH, HORIZONTAL SPAN, NONREINFORCED CONCRETE MASONRY WALLS

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
Monowythe 8"	N	Uniform	127	1.2	4.13	4
Hollow, 3-Core	N	"	136	141	4.41	4
	N	"	127	132	4.13	4
	N	"	159	176	5.50	4
	N	"	173	180	5.63	4
	O	"	123	128	4.00	4
	O	"	158	164	5.13	4
Monowythe 8"	N	"	149	155	4.84	4
Hollow, Joint	N	"	160	166	5.19	4
Reinf. @ 16 in.cc	N	"	193	201	6.28	4
	O	"	150	156	4.88	4
	O	"	186	193	6.03	4
Monowythe 8"	N	"	203	211	6.59	4
Hollow Joint	N	"	196	204	6.38	4
Reinf. @ 8 in.cc	O	"	202	210	6.56	4
	O	"	195	203	6.34	4
Monowythe 8"	N	1/4 pt	56	58	1.81	6
Hollow	N	"	38	39	1.22	6
	N	"	61	63	1.97	6
	N	"	60	62	1.94	6
	N	"	69	71	2.22	6
	N	"	93	96	3.00	6
8" Monowythe	M	Center	199	217	4.72	26
Hollow, 2-Core	M	"	176	192	4.17	26
	M	"	151	165	3.59	26
4-2-4 Cavity	M	"	111	210	4.57	26
Wall, Hollow	M	"	135	255	5.54	26
Units	M	"	95	180	3.91	26
8" Monowythe	M	"	159	173	3.76	26
Hollow 2-Core	M	"	159	173	3.76	26
Joint Rc. @ 8"oc	M	"	191	208	4.52	26
4-2-4 Cavity of	M	"	159	300	6.52	26
Hollow Units Tied	M	"	159	300	6.52	26
w/Joint Rc. @ 8"oc	M	"	159	300	6.52	26

TABLE 9 (Continued)

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
4" Hollow Monowythe	N	Center	138	365	11.41	25
	N	"	157	415	12.97	25
	N	"	101	268	8.38	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	314	237	5.15	25
	M	"	314	237	5.15	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	314	237	7.41	25
	N	"	314	237	7.41	25
8" Hollow Monowythe	O	"	259	195	6.09	25
	O	"	277	210	6.56	25
	O	"	277	210	6.56	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	297	224	4.87	25
	M	"	277	210	4.56	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	259	195	6.09	25
	N	"	297	224	7.00	25
8" Hollow Monowythe	O	"	360	271	8.45	25
	O	"	297	224	7.00	25
	O	"	268	202	6.31	25
12" Hollow Monowythe	N	"	352	142	4.44	25
	N	"	314	127	3.97	25
	N	"	333	134	4.19	25

RAI 10a - In Reference 3, the Licensee indicates that the energy balance technique and arching theory have been used to qualify some masonry walls. The NRC, at present, does not accept the application of these techniques 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 each of these techniques and provide the resulting stresses and displacements.

RESPONSE

See Attachment RAI 10a1 "List of Safety Related Walls Analyzed by Arching Theory" and Attachment RAI 10a2 "List of Safety Related Walls Analyzed by Energy Balance Technique".

LIST OF SAFETY RELATED WALLS

ANALYZED BY

ENERGY BALANCE TECHNIQUE

MASONRY WALL DESIGN

(IEB 80-11)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 10a2

J.O. 7453-150

CP&L - BSEP

MASONRY WALLS

LIST OF SAFETY RELATED WALLS
ANALYSED BY ARCHING THEORY

Wall No.	Wall Dimensions Length	Height	Thick	Block Type	Elevation Floor	Ceiling	Design Accel.	Load Lbs/Ft.	Capacity Lbs/Ft.	$\frac{L}{C}$	Bldg.	Remarks
2a	44'-6"	15'-3"	12"	Hollow	50'-0"	63'-3"	0.47g	96	193	0.50	Diesel Gener.	
2b	41'-2½"	15'-3"	12"	Hollow	50'-0"	63'-3"	0.47g	96	193	0.50	Diesel Gener.	
2c	41'-2½"	15'-3"	12"	Hollow	50'-0"	63'-3"	0.47g	96	193	0.50	Diesel Gener.	
2d	41'-2½"	15'-3"	12"	Hollow	50'-0"	63'-3"	0.47g	96	193	0.50	Diesel Gener.	
4a	120'-7"	17'-0"	8"	Hollow	50'-0"	67'-0"	0.55g	33	41	0.8	Diesel Gener.	
9a	31'-0½"	13'-8"	12"	Hollow	23'-0"	36'-8"	0.25g	12	240	0.05	Control Bldg.	
9b	31'-0½"	12'-10"	12"	Hollow	23'-0"	35'-10"	0.25g	12	240	0.05	Control Bldg.	
9c	31'-0½"	12'-10"	12"	Hollow	23'-0"	35'-10"	0.25g	12	240	0.05	Control Bldg.	
9d	31'-0½"	13'-8"	12"	Hollow	23'-0"	36'-8"	0.25g	12	240	0.05	Control Bldg.	

LIST OF SAFETY RELATED WALLS

ANALYZED BY

ARCHING THEORY

MASONRY WALL DESIGN

(IEB 80-11)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 10a1

J.O. 7453-150

CP&L - BSEP

MASONRY WALLS

LIST OF SAFETY RELATED WALLS
ANALYSED BY ENERGY BALANCE TECHNIQUE

DIESEL GENERATOR BLDG.

Wall No.	Length	Height	Thick	Vertical Reinf.	Horizontal Reinf. ⊕	Design Acceleration OBE	Design Acceleration DBE	Elevation Floor	Elevation Ceiling	Deflection Δy	Deflection Δe	μ	Type	Remarks
4a	120'-7"	17'-0"	8"	2 - #7 @ 4'-0" O.C.	Std. Dur-O-Wall	1.60g	2.12g	50'-0"	67'-0"	2.1"	2.2"	1.05	**	

μ = Ductility Ratio
Δy = Wall Deflection at Yield
Δe = Wall Deflection for Elastic Behavior

⊕ = Hori. Reinf. in Every Second Course

** = Hollow Blocks Filled at Reinf. Only

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

JAN 25 1983

Docket Nos: 50-255	50-250/251
50-277/278	50-213
50-269/270/287	50-245/336
50-317/318	50-244
50-346	50-309
50-237/249	50-271
50-254/265	50-325/324

FACILITIES: Palisades	Turkey Point 3/4
Peach Bottom 2/3	Haddam Neck
Oconee 1/2/3	Millstone 1/2
Calvert Cliffs 1/2	Ginna
Davis-Besse 1	Maine Yankee
Dresden 2/3	Vermont Yankee 1
Quad Cities 1/2	Brunswick 1/2

LICENSEES: Consumers Power	Florida Power & Light
Philadelphia Electric	Connecticut Yankee Atomic Power
Duke Power	Northeast Nuclear Energy
Baltimore Gas & Electric	Rochester Gas & Electric
Toledo Edison	Maine Yankee Atomic Power
Commonwealth Edison	Vermont Yankee Nuclear Power
	Carolina Power & Light

SUMMARY OF MEETING HELD ON JANUARY 20, 1983 RE: MASONRY WALL DESIGN (IEB 80-11). USE OF "ENERGY BALANCE TECHNIQUE" AND ARCHING ACTION" FOR MASONRY WALL QUALIFICATION.

On January 20, 1983, the NRC staff, its contractor and consultants met with representatives of the utilities identified above and others as shown in Attachment 1 for further discussion of the use of the energy balance technique and arching action for masonry wall qualification. Copies of viewgraphs presented at the meeting are contained in Attachment 2.

The purpose of the meeting was to allow the utilities an opportunity to present a response to an earlier meeting (November 3, 1982) in which NRC stated that the validity of the two analysis techniques should be substantiated by appropriate masonry testing.

Based on the technical information presented, it was the conclusion of the group that additional testing to demonstrate the applicability of the energy balance technique or the arching action technique as applied in the specific cases for each facility is not warranted. Further technical information on each technique was provided at the conclusion of the technical presentation (see Attachments 3 and 4) but was not explicitly discussed at the meeting.

The NRC staff, contractor and consultants will be reviewing the information provided at the meeting to decide on a resolution of this issue. The possibility of plant-specific reviews, wherein each specific application of these techniques is reviewed in detail, was discussed.

In providing the information and response to the NRC at this meeting, the utility group considers its function as a group to be completed.

Charles M. Trammell

Charles M. Trammell
Lead Engineer
Masonry Wall Design

Attachments:
1) Attendance list
2) Viewgraphs
3) Arching Theory Rebuttal
4) Energy Balance Technique Rebuttal

cc: Attendees plus
see next page

SUMMARY OF MEETING

JANUARY 20, 1983

MASONRY WALL DESIGN

(IEB 80-11)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 10a

RAI 10b - The following areas need technical verification before any conclusion can be made about these technique:

1. Energy Balance Technique

   - For the walls which were analyzed using the energy balance technique, provide technical basis to insure that the ductile mode of failure will take place (if they fail).

   - Provide justification and test data (if available) to validate the applicability of the energy balance technique to the masonry structures at Brunswick Units 1 and 2 with particular emphasis on the following areas:

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

RESPONSE

See Response to RAI 10a.

RAI 10c - The follwoing areas need technical verification before any conclusion can be made about these techniques:

2. Arching Theory

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

- Provide justification and test data (if available) to validate the applicability of the arching theory to the masonry structures at Brunswick Units 1 and 2 with particular emphasis on the following areas:

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

- If the hinges are formed in the walls, the capability of the structure 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

See Response to RAI 10a.

RAI 11 - Regulatory Guide 1.61 allows 4% damping for an OBE and 7% damping for an SSE. Provide justification for using 10% damping for unreinforced walls in the arching action analysis.

RESPONSE

See RESPONSE to RAI 10a.

RAI 12a - With reference to multiple wythes, clarify whether the collar joint strength was used in the analysis. If so, justify the allowable stresses of the collar joint.

RESPONSE

The analysis of multiple wythe walls was performed based on no-shear transfer in the collar joint. Composite action was not utilized in qualifying the wall elements and each wythe was conservatively assumed to act independently. Hence, the allowable stresses in the collar joint are not applicable.

RAI 12b - Also, provide sample calculations illustrating the analysis of multiple wythe walls.

RESPONSE

Calculation data package 9527-1-GP-MW-02-F, Attachment 12b, illustrates the analysis of multiple wythe walls.

CALCULATION SET NO.

9527-1-GP-MW-02-F

SHEETS 1 to 31

(35 PAGES)

CAROLINA POWER & LIGHT COMPANY

BRUNSWICK STEAM ELECTRIC PLANT

UNITS 1 & 2

ATTACHMENT RAI 12b

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PRELIM		0	R P U	CR
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SHEET 1 OF 51			DATE	DATE
JO 9527-088				

NAME OF COMPANY C P & L UNIT/S 1 2

SUBJECT REACTOR BLDG - MASONRY WALLS

Wall 1 Unit #1 & #2 EL. -17'-4"

Ref DWG. F-1275 & F-12075

wall is 8'-8" L × 16'-4" H × 2'-0" Thick (3 wythes @ 8") unit #1
8'-8" L × 16'-4" H × 2'-0" Thick (2 wythes @ 12") unit #2

since multiple wythes are not positively connected together, assume each wythe as an individual wall -

$\frac{H}{L} = \frac{16.33'}{8.67'} = 1.88 > 1.5$ wall should span horizontally.

walls constructed of std. weight solid blocks (144 PCF). all cores filled mortar. Unreinforced walls.

consider single 8" wythe - $\left(M_{cr} = \frac{93 \times 443}{12 \times 3.81} = 901'^{\#} \right)$

$I = \frac{12'' \times 7.625^3}{12} = 443\ in^4$ $wt. = 144^{PCF} \times \frac{8''}{12} = 96\ PSF \times \frac{1}{12} = 8^{\#}/"$

$P = \sqrt{\frac{8 \times 10,000,000,000}{386 \times 1,820,000 \times 443}} \times \frac{144\ L^2}{157,000} = 0.000465 L^2$

P = 0.000465 × 8.67² = 0.035 sec. → f = 28.6 CPS.
frequency spread 1.095 → 0.895

f_{high} = 1.095 × 28.6 = 31.3 CPS → P = 0.032 sec

f_{low} = 0.895 × 28.6 = 25.6 CPS → P = 0.039 sec

From computer printouts.

	OBE 2%	DBE 4%	
accel. =	DON'T HAVE	DON'T HAVE	EL.(-)17'-4" N-S response
accel. =	0.28	0.435	EL. 20'-0" N-S response

Accel. @ EL. 20 should be max ∴ use.

Design accel. = 1.05 × 0.28 = 0.29g - OBE
1.05 × 0.435 = 0.46g - DBE

$Ratio = \frac{0.46}{0.29} = 1.59$

$W_H = 0.29 \times 96^{\#/'} \times 8.67' = 241^{\#} \times \frac{1}{2} = 121^{\#} = R = V$

$M = 241^{\#} \times \frac{8.67'}{8} = 261'^{\#}$

wall 1 - cont. -

$f_m = \frac{261^{1\#} \times 12 \times 3.8'}{443} = 27$ psi < 75 O.K. ($1.5\sqrt{m_o} = 1.5 \times 50 = 75$ psi)

$\frac{f}{F} = \frac{27}{75} = 0.36$

$v = \frac{121^{\#}}{7.625 \times 12} = 1.3$ psi < 47 O.K. $\frac{f}{F} = \frac{1.3}{47} = 0.03$

For DBE. $f_m = 27 \times 1.59 = 43$ psi < 125 O.K. ($2.5\sqrt{2500} = 125$ psi)

$v = 1.3 \times 1.59 = 2$ psi < 72 O.K. $\frac{f}{F} = \frac{43}{125} = 0.34$

$\frac{f}{F} = \frac{2}{72} = 0.03$

check 12" wythe

$I = \frac{12 \times 11.625^3}{12} = 1571$ in^4 wt. $= 144^{pcf} \times \frac{1}{12} = 12$ #/in

$P = \sqrt{\frac{12 \times 10,000,000,000}{386 \times 1,820,000 \times 1571}} \times \frac{144 L^2}{157,000} = 0.000302 L^2$

$\left(M_{cr} = \frac{93 \times 1571}{12 \times 5.81} = 2095^{1\#}\right)$

$P = 0.000302 (8.67)^2 = 0.023$ sec → 44 CPS. > 33 CPS

frequency spread = 1.095 → 0.895

$f_{high} = 1.095 \times 44 = 48.18$ CPS → P = 0.02 sec.

$f_{low} = 0.895 \times 44 = 39.38$ CPS → P = 0.025 sec.

From computer printouts.

	OBE 2%	DBE 4%		
accel =	DON'T HAVE	DON'T HAVE	EL. -17'-4"	N-S response
accel =	0.27	0.47	EL. 20'-0"	N-S response

Accel @ EL. 20' should be max. Use.

accel ratio $= \frac{0.47}{0.27} = 1.74$

$W_H = 0.27 \times 144^{\#} \times 8.67' = 337^{\#} \times \frac{1}{2} = 169^{\#} = R = V$

$M = 337 \times \frac{8.67'}{8} = 365'^{\#}$

$f_m = \frac{365^{1\#} \times 12 \times 5.81'}{1571} = 16$ psi < 75 O.K. $\frac{f}{F} = \frac{16}{75} = 0.21$

$v = \frac{169^{\#}}{11.625 \times 12} = 1.2$ psi < 47 O.K. $\frac{f}{F} = \frac{1.2}{47} = 0.03$

For DBE,

$f_m = 16 \times 1.74 = 28$ psi < 125 O.K. $\frac{f}{F} = \frac{28}{125} = 0.22$

$v = 1.2 \times 1.74 = 2$ psi < 72 O.K. $\frac{f}{F} = \frac{2}{72} = 0.03$

wall is acceptable as per new criteria.

Check for In-plane shear strain criteria -

wall in E-W direction and is confined

∴ shear strain O.K see sheet # 31

007 REV 7/79

GENERAL COMPUTATION SHEET

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CALC. SET NO.		REV	COMP BY	CHK'D BY
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JO 9527-088				

(LINE)

NAME OF COMPANY C P & L UNIT/S 1, 2

SUBJECT REACTOR BLDG - MASONRY WALLS

Wall 2 - cont -

check shear stress in collar joints -
2'-0" Thick wall

$I = \frac{12 \times 24^3}{12} = 13824\ in^4$ $wt = 2' \times 144^{pcf} \times 288 \times \frac{1}{12} = 24\ \#/in$

$P = \sqrt{\frac{24 \times 10.000.000.000}{386 \times 1.820.000 \times 13824}} \times \frac{144 L^2}{157.000} = 0.000144 L^2$

$P = 0.000144 (8.67)^2 = 0.011 \rightarrow f = 92\ cps > 33\ cps$
frequency spread $= 1.095 \rightarrow 0.895$

$f_{high} = 1.095 \times 92 = 100.7\ cps \rightarrow P = 0.01\ sec$
$f_{low} = 0.895 \times 92 = 82.3\ cps \rightarrow P = 0.012\ sec$

From computer printouts.

	OBE 2%	DBE 4%	
accel.	DON'T HAVE	DON'T HAVE	EL. 17'-4 N-S response
accel.	0.27	0.43	EL 20'-0" N-S response

accel at EL 20' should be max. ∴ Use.

accel ratio $= \frac{0.43}{0.27} = 1.60$

$w_H = 0.27 \times 288^{\#} \times 8.67' = 674 \times \frac{1}{2} = 337^{\#} = R = V$
$m = 674 \times \frac{8.67}{8} = 730'^{\#}$

$f_m = \frac{730 \times 12 \times 12}{13824} = 8\ psi < 75$ O.K. $\frac{f}{F} = \frac{8}{75} = 0.11$

$v = \frac{337^{\#}}{12 \times 24} = 1.2\ psi < 47$ O.K. $\frac{f}{F} = \frac{1.2}{47} = 0.03$

$\frac{f}{F} = \frac{2}{8} = 0.25$

shear stress in collar joint $= 1.5 \times 1.2^{psi} = 2\ psi < 8$ O.K.

For DBE Loads -

$f_m = 8^{psi} \times 1.6 = 12.8^{psi} < 125$ O.K. $\frac{f}{F} = \frac{12.8}{125} = 0.10$

$v = 1.2^{psi} \times 1.6 = 2^{psi} < 72$ O.K. $\frac{f}{F} = \frac{2}{72} = 0.03$

shear stress in collar joint $= 2^{psi} \times 1.6 = 3.2^{psi} < 12$ O.K.

$\frac{f}{F} = \frac{3.2}{12} = 0.27$

Wall 2a Unit # 1 only EL. 20'-0"

Ref. DWG - F-12075, F-12030, F-12033

unreinforced, std. wt. blocks (144 pcf), all cores filled

wall is 18'-0" L x 14'-4" H x 44" Thick (4 - 8" wythes & 1 - 12" wythe)

There is no support at top of wall. ∴ wall spans horizontally

clear span = 15'-0"

check single 8" wythe. (neglect the effect of opening)

$P = 0.000465 (15)^2$ (see sheet #1/RPU)

$= 0.105$ sec. → $f = 9.52$ cps.

frequency spread = 1.095 → 0.895

$f_{high} = 9.52 \times 1.095 = 10.4$ cps → $P = 0.096$ sec.

$f_{low} = 9.52 \times 0.895 = 8.5$ cps → $P = 0.117$ sec

wall is running N-W to S-E direction, ∴ use max accel. response in E-W or N-S direction

	2% OBE	4% DBE
EL. 20'-0"	0.40	0.60

wall does not extend upto next EL. 50'-0"

∴ use only floor response

Design accel. = 0.40 x 1.05 = 0.42g

$W_H = 0.42 \times 96^{psf} \times 15' = 605^{\#} \times \frac{1}{2} = 303^{\#} = R = V$

$M = 605 \times \frac{15}{8} = 1134^{\#}$

$f_m = \frac{1134'^{\#} \times 12 \times 3.81}{443} = 117$ psi > 75 **N.G** ←

NOTE: Bending stress would be even higher if the effect of opening was considered.

∴ 8" wythe overstressed in horizontal span -

check the composite wall for one way behavior in horizontal span.

Wall 2a Unit # 1 only EL. 20'-0"

Ref. DWG - F-12075, F-12030, F-12033

unreinforced, std. wt. blocks (144 PCF), all cores filled

wall is 18'-0" L x 14'-4" H x 44" Thick (4 - 8" wythes & 1 - 12" wythe)

There is no support at top of wall. ∴ wall spans horizontally
clear span = 15'-0"

check single 8" wythe. (neglect the effect of opening)

$P = 0.000465(15)^2$ (see sheet #1/RPU)
= 0.105 sec. → f = 9.52 cps.
frequency spread = 1.095 → 0.895

f_{high} = 9.52 x 1.095 = 10.4 cps → P = 0.096 sec.
f_{low} = 9.52 x 0.895 = 8.5 cps → P = 0.117 sec

wall is running N-W to S-E direction, ∴ use max accel. response in E-W or N-S direction

	2% OBE	4% DBE
EL. 20'-0"	0.40	0.60

wall does not extend upto next EL. 50'-0"
∴ use only floor response

Design accel. = 0.40 x 1.05 = 0.42g

w_H = 0.42 x 96 PSF x 15' = 605# x 1/2 = 303# = R = V
M = 605 x 15/8 = 1134 #

$f_m = \frac{1134'^{\#} \times 12 \times 3.81}{443}$ = 117 psi > 75 N.G —

NOTE: Bending stress would be even higher if the effect of opening was considered.
∴ 8" wythe over-stressed in horizontal span -

check the composite wall for one way behavior in horizontal span.

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NAME OF COMPANY C P & L UNIT/S 1

SUBJECT REACTOR BLDG. - MASONRY WALLS

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Wall 2a Unit # 1 only EL 20'-0"

Assume one way behavior - Horizontally spanning.

Ref. DWGS: - F-12075, F-12030, F-12033

Wall is 15'-0" L x 14'-0" H x 44" thick (4-8" wythes & 1-12" wythe)

No support at top of wall, ∴ wall spans horizontally.

Consider composite wall 44" thick.

$I = \frac{12 \times 44^3}{12} = 85{,}184 \ in^4$ wt. $= 144^{pcf} \times \frac{44"}{12} = 528^{\#/'} \times \frac{1}{12} = 44^{\#}/in$

$P = \frac{1}{1.57}\sqrt{\frac{44 \times 180^4}{386 \times 1{,}820{,}000 \times 85184}} = 0.0177\ sec. \rightarrow f = 56.5\ cps > 33\ cps$

frequency spread = 1.095 → 0.895

$f_{high} = 1.095 \times 56.5 = 61.87\ cps \rightarrow P = 0.016\ sec$

$f_{low} = 0.895 \times 56.5 = 50.57\ cps \rightarrow P = 0.0198\ sec$

For frequency calcs the effect of opening is neglected

From computer printouts,

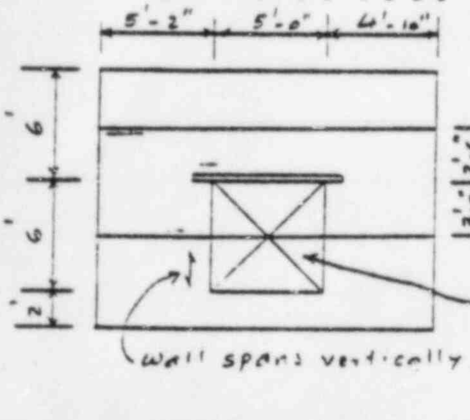
	2% OBE	4% DBE	
accel. =	0.27	0.43	N-S direction (EL. 20')
accel. =	0.29	0.47	E-W direction (EL. 20')

walls is in N-W to S-E direction - use ave accel. for design

Design accel = 0.28 OBE & 0.45 for DBE

accel. ratio = $\frac{0.45}{0.28} = 1.60$

$w = 0.28 \times 528^{psf} = 147.8^{psf}$



Conservative assume load distributed on width = $\frac{span}{6} = \frac{15'-0"}{6} = 2'-6"$

width of strip = 3'-0" + 2'-6 = 5'-6

$w = 147.8^{psf} \times \frac{5.5'}{2.5'} = 325^{\#/'}$

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GENERAL COMPUTATION SHEET

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SHEET 6 OF 31			DATE	DATE
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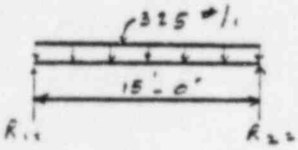
NAME OF COMPANY C P & L UNIT/S 1

SUBJECT Electrical Bldg. - Masonry Walls

Wall 2a - cont. -



$R_1 = R_2 = 325^{\#/'} \times 7.5 = 2438^{\#}$

$m = \frac{325 \times 15^2}{8} = 9141'^{\#}$

$f_m = \frac{9141'^{\#} \times 12 \times 22"}{85,184} = 29$ psi < 75 psi O.K. $\frac{f}{c} = \frac{29}{75} = 0.39$

$v = \frac{2438^{\#}}{12 \times 44} = 4.6$ psi < 47 O.K. $\frac{f}{c} = \frac{4.6}{47} = 0.10$

shear stress in collar joint = $1.5 \times 4.6^{psi} = 6.9^{psi}$ < 8 O.K.

$\frac{f}{c} = \frac{6.9}{8} = 0.86$

For DBE loads -

$f_m = 29 \times 1.6 = 46.4$ psi < 125^{psi} O.K. $\frac{f}{c} = \frac{46.4}{125} = 0.37$

$v = 4.6^{psi} \times 1.6 = 7.4$ psi < 72^{psi} O.K. $\frac{f}{c} = \frac{7.4}{72} = 0.10$

shear stress in collar joint = $1.6 \times 6.9^{psi} = 11^{psi}$ < 12 O.K.

$\frac{f}{c} = \frac{11}{12} = 0.92$

wall is acceptable as per committee criteria

check for inplane shear strain criteria -

wall is confined ∴ O.K. (see sheet #30)

REV 7/79

GENERAL COMPUTATION SHEET

united engineers & constructors inc

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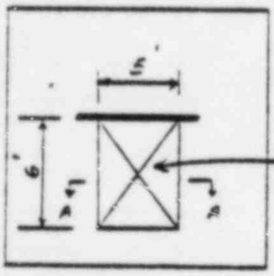
J.O. 9527.022				

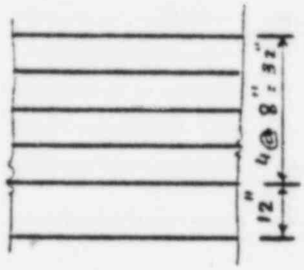
NAME OF COMPANY: CPEL UNIT/S 1

SUBJECT: REACTOR BLDG. - MASONRY WALLS

Wall 2a - cont.



ELEVATION



SEC. A-A

The opening is closed with loose stacked blocks. These blocks should be prevented from falling inside and damaging any safety related item. To accomplish this, use solid mortar joints for 12" wythe and keep four 8" wythes as loose stacked blocks.

Check 12" wythe wall spanning 5'-0' horizontally -

Design accel. OBE = 0.28 g, DBE = 0.45 g } see sht # 5

$I = \frac{12 \times 11.625^3}{12} = 1571 \text{ in}^4$ Accel. ratio = $\frac{0.45}{0.28} = 1.60$

weight = $144^{PCF} \times 3.67' = 528^{PSF}$

$w = 528^{PSF} \times 0.28 = 148^{PSF}$ $V = 148 \times \frac{5}{2} = 370^{\#}$

$m = 148 \times \frac{5^2}{8} = 463'^{\#}$

$f_m = \frac{463'^{\#} \times 12 \times 5.81''}{1571} = 21^{psi} < 75$ O.K. $\frac{f}{c} = \frac{21}{75} = 0.28$

$v = \frac{370^{\#}}{12'' \times 11.625''} = 2.7 \text{ psi} < 47$ O.K. $\frac{f}{c} = \frac{2.7}{47} = 0.06$

For DBE Loads,

$f_m = 21^{psi} \times 1.60 = 34 \text{ psi} < 125^{psi}$ O.K. $\frac{f}{c} = \frac{34}{125} = 0.27$

$v = 2.7^{psi} \times 1.60 = 4.3 \text{ psi} < 72$ O.K. $\frac{f}{c} = \frac{4.3}{72} = 0.06$

wall is acceptable.

ACTION: SEE PM 80-256, MWR-6A FOR FIX.

9527-1-SF-NW-02-F 82A

DOT REV 7/74

GENERAL COMPUTATION SHEET

(LINE)

united engineers & constructors inc

NAME OF COMPANY CP&L BSEP UNIT/S 1

SUBJECT Reactor Building

CALC SET NO		REV	COMP BY	CHK'D BY
PRELIM		1	R PU	[illegible]
FINAL			DATE 2-6-81	DATE 2-9-81
VOID				
SHEET 1 OF 3			DATE	DATE
JO 9527-028				

Wall 2a

(1) Provide steel grating to prevent loose blocks from falling on air lock.

Accel = 0.28g OBE, 0.45g DBE (see sheet # 5)

wt. of wall = 144 PCF × 3.67' = 528 PSF

w = 0.28 × 528 PSF = 148 PSF - OBE
w = 0.45 × 528 PSF = 238 PSF - DBE

span for Grating = 7'-0" ±

Assume friction force = 25 %

Design load for Grating = 0.75 × 238 = 180 PSF

From Morris-Wheeler catalog, Pg 43, use steel bar Grating $1\frac{1}{2}" \times \frac{3}{16}"$ bearing bars @ $\frac{15}{16}"$ o.c.

∴ Allowable load on 7' span = 191 PSF > 180 O.K

(2) Design of steel angles -
Load on angle = $180^{PSF} \times 3' = 540^{\#/\prime}$

$m = \frac{0.54 \times 2.33^2}{8} = 0.37^{\prime K}$

$S_{req.} = \frac{0.37 \times 12}{22} = 0.20\,in^3$

Use L 3×2×3/8 (LLV) $S = 0.371\,in^3 > 0.20$ O.K

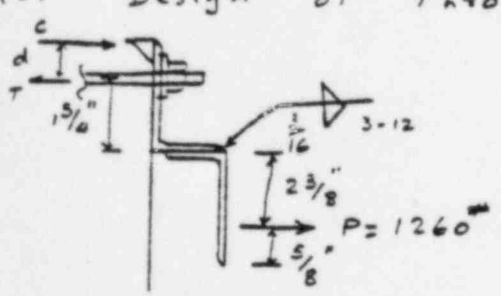
check bending in Leg of L3×2×3/8

$w = 2.33'$ spacing $\times 540^{\#/\prime} = 1260^{\#}$

$M = 1260^{\#} \times 4\frac{1}{8}" = 5200^{"\#}$ $S = \frac{bd^2}{6} = \frac{(12)(\frac{3}{8})^2}{6} = 0.281\,in^3$

$f = \frac{5200}{0.281} = 18{,}500$ psi O.K.

(3) Design of Through Rod -



$P = 540^{\#/\prime} \times 2.33' = 1260^{\#}$

$e = 2\frac{3}{8}" + 1\frac{3}{4}" = 4\frac{1}{8}"$

$m = P \cdot e = 1.260^{K} \times 4.125" = 5.20^{"K}$

$d = 0.87 \times 1.25" = 1.09"$ (approx.)

Wall 2a - cont.

$T = 1.260^k + \frac{5.20''^k}{1.09} = 6.03^k$

wt of Grating = $15^{PSF} \times 7' = 105$ #/l
wt. of angles 12 #/l
117 #/l

v = shear on bolt = 117 #/l × 2.33' = 273# = 0.28^k

Try $\frac{3}{4}''\phi$ ASTM A-307 bolt

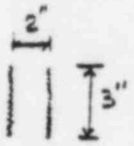
$T_a = 8.8^k$ $V_a = 4.4^k$ (From AISC manual, Eighth Edition.)

$\frac{6.03}{8.8} + \frac{0.28}{4.4} = 0.75 < 1$ O.K.

Use $\frac{3}{4}''\phi$ A-307 Through bolt 4'-0" Long with nut and washer at both ends.

(4) Design weld between two angles -

Try 3" weld at 12" o.c.



$S = 2 \times 3 = 6 \text{ in}^2$
$L_w = 3''$
$w = 540$ #/l.
$f_v = \frac{540}{2 \times 3''} = 90$ #/in

$m = 540^{\#} \times 2.375'' = 1283''^{\#}$

$f_t = \frac{1283}{2 \times 3} = 214$ #/in

$f_r = \sqrt{90^2 + 214^2} = 233$ #/in

Try $\frac{3}{16}''$ weld.

$f_{allow} = 0.707 \times \frac{3}{16} \times 21{,}000 = 2784$ #/in > 233 O.K

Use $\frac{3}{16}''$ weld. 3" @ 12" o.c.

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

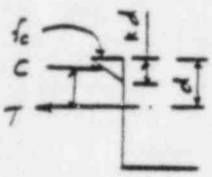
CALC. SET NO.		
PRELIM.		
FINAL	X	
VOID		
SHEET 3 OF 3		
J.O. 9527-088		
REV	COMP. BY	CHK'D BY
0	RPU DATE 3-27-81	[illegible] DATE [illegible]
	DATE	DATE

NAME OF COMPANY C P & L ESEP UNIT/S 1

SUBJECT REACTOR BUILDING — unit 1

wall 2a — cont

check bearing pressure on Blocks —



$f_c = 0.62 f_m' = 1820 \times 0.62 = 1128$ psi say 1100 psi (allowable)

$d = 1.25"$

$m = 5200"\#$ (from sheet #1)

$C = \frac{1}{2} \times 1100 \times kd \times 28" = 19250k$ (d = 1.25")

$m = C \text{ (or T)} \times \left(d - \frac{kd}{3} \right)$

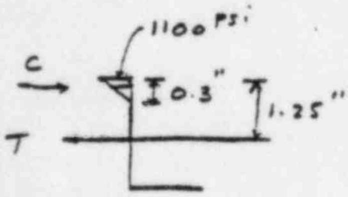
$5200"\# = 19250k \left(1.25 - \frac{1.25k}{3}\right)$

$5200 = 24062.5k - 8020.83k^2$

$k^2 - 3k + 0.648 = 0$

$k = 0.235$

$kd = 0.235 \times 1.25 = 0.30"$



GENERAL COMPUTATION SHEET

united engineers & constructors inc

NAME OF COMPANY: C P & L UNIT/S: 1

SUBJECT: REACTOR BLDG - MASONRY WALLS

CALC. SET NO.			REV	COMP BY	CHK'D BY
PRELIM			0	RFU	-
FINAL	X			DATE 10/23/80	DATE
VOID					
SHEET 7 OF 31				DATE	DATE
10 ACT - 077					

Wall 2E - Unit # 1 only Ref DWG F-12075

Wall is 8'-8" L × 14'-4" H × 36" Thick (3 - 8" wythes & 1-12" wythe)

$\frac{H}{L} = \frac{14.33'}{8.67'} = 1.65 > 1.5$ wall should span horizontally

Horizontal span = 8'-8" (one way behavior)

Consider single 8" wythe –

$I = 443\ in^4$, $w = 96$ PSF , $P = 0.000465L^2$ (see sht. # 1)
$P = 0.000465 \times 8.67^2 = 0.035$ sec → $f = 28.6$ CPS
frequency spread = 1.095 → 0.895
$f_{high} = 1.095 \times 28.6 = 31.3$ CPS → $P = 0.032$ sec
$f_{low} = 0.895 \times 28.6 = 25.6$ CPS → $P = 0.039$ sec
From computer printouts,

	2% OBE	4% DBE	
accel.	0.29	0.48	EL. 20' - N.S OR EW response
accel.	0.32	0.50	El 50' - N S OR EW response

Design accel $= \frac{0.29 + 0.32}{2} \times 1.05 = 0.32$ - OBE

$= \frac{0.48 + 0.50}{2} \times 1.05 = 0.51$ - DBE

accel. ratio $= \frac{0.51}{0.32} = 1.59$

$W_H = 0.32 \times 96^{PSF} \times 8.67' = 266^{\#} \times \frac{1}{2} = 133^{\#} = R = V$
$M = 266^{\#} \times \frac{8.67'}{8} = 288'^{\#}$

$f_m = \frac{288'^{\#} \times 12 \times 3.81}{443} = 30$ psi < 75 O.K. $\frac{L}{C} = \frac{30}{75} = 0.40$

$v = \frac{133^{\#}}{7.625 \times 12} = 1.5$ psi < 47 O.K. $\frac{L}{C} = \frac{1.5}{47} = 0.03$

For DBE

$f_m = 30^{psi} \times 1.59 = 48^{psi} < 125$ O.K. $\frac{L}{C} = \frac{48}{125} = 0.38$

$v = 1.5 \times 1.59 = 2.4$ psi < 72 O.K. $\frac{L}{C} = \frac{2.4}{72} = 0.033$

Wall 26 - Unit # 1 only - cont.

Consider 12" wythe -

$I = 1571 \text{ in}^4$, $w = 144^{psf}$, $P = 0.000302 L^2$ (see sht # 2)

$P = 0.000302 \times 8.67^2 = 0.023$ sec → 44 cps > 33 cps.
frequency spread = 1.095 → 0.895

$f_{high} = 1.095 \times 44 = 48.18$ cps → $P = 0.02$ sec
$f_{low} = 0.895 \times 44 = 39.38$ cps → $P = 0.025$ sec.

From computer printouts,

	2% OBE	4% DBE	
accel =	0.29	0.47	EL 20 N-S or E-W Response
accel =	0.31	0.50	EL 50 N-S or E-W Response

Design accel - OBE = 0.30 { 1.05 factor not used
DBE = 0.49 since f > 33 }

accel ratio = $\frac{0.49}{0.30} = 1.63$

$w_H = 0.30 \times 144 \times 8.67 = 375^{\#} \times \frac{1}{2} = 188^{\#} = R = V$
$m = 375 \times \frac{8.67}{8} = 406'^{\#}$

$f_m = \frac{406 \times 12 \times 5.81}{1571} = 18$ psi < 75 O.K. $\frac{L}{C} = \frac{18}{75} = 0.24$

$v = \frac{188^{\#}}{11.625 \times 12} = 1.4$ psi < 47 O.K. $\frac{L}{C} = \frac{1.4}{47} = 0.03$

For DBE

$f_m = 18^{psi} \times 1.63 = 30$ psi < 125 O.K. $\frac{L}{C} = \frac{30}{125} = 0.24$

$v = 1.4^{psi} \times 1.63 = 2.3$ psi < 72 O.K. $\frac{L}{C} = \frac{2.3}{72} = 0.03$

Wall is acceptable as per new criteria.

Check inplane shear strain criteria -
wall is confined. OK see sheet # 30

007 REV 7/79 GENERAL COMPUTATION SHEET

(LINE) united engineers & constructors inc

NAME OF COMPANY C P & L UNIT/S 1

SUBJECT REACTOR BLDG - MASONRY WALLS

CALC SET NO		REV	COMP BY	CHK'D BY
PRELIM		0	R P U	
FINAL	X		DATE 11/1/80	DATE
VOID				
SHEET 9 OF 31			DATE	DATE
JO 9527.088				

Wall 26 - cont. Unit # 1 only

check shear in collar joints -

wall is 36" thick

$I = \frac{12 \times 36^3}{12} = 46,656 \text{ in}^4$ $wt = 144^{pcf} \times 3' = 432^{\#/'} \times \frac{1}{12} = 36^{\#/in}$

$P = \frac{1}{1.57}\sqrt{\frac{36 \times 104^4}{386 \times 1220000 \times 46656}} = 0.0072 \text{ sec} \rightarrow f = 138 \text{ cps} > 33 \text{ cps}$

Period spread is 0.913 → 1.12

$P = 0.913 \times 0.0072 = 0.0066$ sec.
$P = 1.12 \times 0.0072 = 0.008$ sec.

From computer printouts,

	2% OBE	4% DBE	
accel. =	0.29	0.47	EL 20' EW OR NS response
accel. =	0.31	0.50	EL 50' EW OR NS response

Design accel. OBE = 0.30
Design accel. DBE = 0.49 ratio = $\frac{0.49}{0.30} = 1.63$

$W_H = 0.30 \times 432^{\#/'} \times 8.67' = 1124^{\#} \times \frac{1}{2} = 562^{\#} = R = V$
$m = 1124^{\#} \times \frac{8.67}{8} = 1218'^{\#}$

$f_m = \frac{1218^{'\#} \times 12 \times 18''}{46,656} = 6 \text{ psi} < 75$ O.K. $\frac{L}{C} = \frac{6}{75} = 0.08$

$v = \frac{562^{\#}}{36'' \times 12''} = 1.30 < 47$ O.K. $\frac{L}{C} = \frac{1.30}{47} = 0.03$

shear stress in collar joint = $S_c = 1.5 \times 1.30^{psi} = 2 \text{ psi} < 8$ O.K. $\frac{L}{C} = \frac{2}{8} = 0.25$

For DBE Loads.

$f_m = 1.63 \times 6 \text{ psi} = 9.8^{psi} < 125$ O.K.
$v = 1.63 \times 1.30^{psi} = 2.1^{psi} < 72$ O.K.
$S_c = 1.63 \times 2^{psi} = 3.3 \text{ psi} < 12$ O.K. $\frac{L}{C} = \frac{3.3}{12} = 0.28$

007 REV 7/79

GENERAL COMPUTATION SHEET

(LINE)

united engineers & constructors inc

CALC. SET NO.			REV	COMP BY	CHK'D BY
PRELIM			0	RPU	
FINAL	X			DATE 10/25/80	DATE
VOID					
SHEET 10 OF 31				DATE	DATE
10 9527-088					

NAME OF COMPANY C P & L UNIT/S 1, 2

SUBJECT REACTOR BLDG - MASONRY WALLS

Wall 3 EL. 20'-0"

Ref DWGs. - F-1275 , F-12075

Unit #1 - 11'-4" L x 10'-0" H x 32" Thick (4 - 8" wythes)
Unit #2 - 11'-4" L x 10'-0" H x 32" Thick (1 - 8" wythe & 2 - 12" wythes)

One way behavior - vertical span

consider 8" wythe - ($I = 443 \text{ in}^4$ $w = 96^{PSF}$ $P = 0.000465 L^2$)

$P = 0.000465 \times 10^2 = 0.0465 \text{ sec} \rightarrow f = 21.5 \text{ cps}$

frequency range = 1.095 → 0.895

$f_{high} = 1.095 \times 21.5 = 23.5 \text{ cps} \rightarrow P = 0.04 \text{ sec.}$

$f_{low} = 0.895 \times 21.5 = 19.2 \text{ cps} \rightarrow P = 0.052 \text{ sec}$

From computer printouts.

	OBE 2%	DBE 4%	
accel	0.30	0.49	EL. 20 E-W response
accel	0.32	0.51	EL 50 E-W response

Design accel - OBE = 0.31 × 1.05 = 0.33

DBE = 0.50 × 1.05 = 0.53

accel. ratio = $\frac{0.53}{0.33} = 1.6$

$W_H = 0.33 \times 96^{PSF} \times 10' = 317^{\#} \times \frac{1}{2} = 159^{\#} = R = V$

$M = 317 \times \frac{10'}{8} = 396'^{\#}$

$f_m = \frac{396'^{\#} \times 12 \times 3.81}{443} = 41 \text{ psi} < 75$ O.K. $\frac{f}{F} = \frac{41}{75} = 0.55$

$v = \frac{159^{\#}}{7.625 \times 12} = 1.74 \text{ psi} < 47$ O.K. $\frac{f}{F} = \frac{1.74}{47} = 0.04$

For DBE,

$f_m = 41^{psi} \times 1.60 = 66 \text{ psi} < 125$ O.K $\frac{f}{F} = \frac{66}{125} = 0.52$

$v = 1.74 \times 1.60 = 2.80 < 72$ O.K. $\frac{f}{F} = \frac{2.8}{72} = 0.04$

consider 12" wythe -

$I = 1571 \text{ in}^4$ $w = 144^{PSF}$ $P = 0.000302 L^2$ (see sht #2)

$P = 0.000302 \times 10^2 = 0.03 \text{ sec} \rightarrow f = 33 \text{ cps}$

Wall 3 - cont.

frequency spread = 1.095 — 0.895
f_{high} = 1.095 × 33 = 36.5 cps ⟶ P = 0.027 sec
f_{low} = 0.895 × 33 = 29.8 cps ⟶ P = 0.034 sec

From computer printouts -

		2% OBE	4% DBE	
accel	=	0.30	0.48	EL 20' E-W response
	=	0.30	0.50	EL 50' E-W response

Design accel = 0.30 × 1.05 = 0.32 OBE
= 0.49 × 1.05 = 0.52 DBE

accel. ratio = $\frac{0.52}{0.32}$ = 1.63

w_H = 0.32 × 144 × 10' = 461# × ½ = 231# = R = V

m = 461 × $\frac{10'}{8}$ = 576'#

f_m = $\frac{576'^{\#} \times 12 \times 5.81}{1571}$ = 26 psi < 75 O.K. $\frac{L}{C}$ = $\frac{26}{75}$ = 0.34

v = $\frac{231^{\#}}{11.625 \times 12}$ = 1.66 psi < 47 O.K. $\frac{L}{C}$ = $\frac{1.66}{47}$ = 0.04

For DBE Loads.

f_m = 1.63 × 26 psi = 43 psi < 125 O.K. $\frac{L}{C}$ = $\frac{43}{125}$ = 0.34

v = 1.66 psi × 1.63 = 2.7 psi < 72 O.K. $\frac{L}{C}$ = $\frac{2.7}{72}$ = 0.04

Wall is acceptable as per new criteria.

Check Inplane shear strain -

wall in N-S direction and confined
wall is OK see sheet # 30

5007 REV 7/79

GENERAL COMPUTATION SHEET

united engineers & constructors inc

CALC SET NO		REV	COMP BY	CHK'D BY
PRELIM		0	R P W	
FINAL	X		DATE	DATE
VOID				
SHEET 12 OF 31			DATE	DATE
JO 9527-089				

NAME OF COMPANY C P E L UNIT/S 1, 2

SUBJECT REACTOR BLDG – MASONRY WALLS

Wall 3 – cont.

check shear in collar joints – wall 32" thick

$I = \frac{12 \times 32^3}{12} = 32768 \text{ in}^4 \quad w = 144^{pcf} \times \frac{32''}{12} = 384^{psf} \times \frac{1}{12} = 32 \#/\text{in}$

$P = \frac{1}{1.57}\sqrt{\frac{32 \times 120^4}{386 \times 1820000 \times 32768}} = 0.0108 \text{ sec} \rightarrow f = 92.6 \text{ cps} >> 33 \text{ cps}$

Period spread = 0.913 → 1.12

$P_{Low} = 0.0108 \times 0.913 = 0.01 \text{ sec}$

$P_{High} = 1.12 \times 0.0108 = 0.012 \text{ sec}$

accel. from computer printouts.

	2% OBE	4% DBE	
EL. 20'-0"	0.29	0.47	E-W response
EL. 50'-0"	0.31	0.50	E-W response

Design accel OBE = 0.30 and DBE = 0.49

accel ratio = $\frac{0.49}{0.30}$ = 1.63

$w_H = 0.30 \times 384^{psf} \times 10' = 1152^{\#} \times \frac{1}{2} = 576^{\#} = R = V$

$M = 1152^{\#} \times \frac{10'}{8} = 1440'^{\#}$

$f_m = \frac{1440'^{\#} \times 12 \times 16''}{32768} = 8.5 \text{ psi} < 40 \text{ psi} \quad \text{O.K}$

$v = \frac{576^{\#}}{12 \times 32} = 1.5 \text{ psi} < 47 \quad \text{O.K.}$

shear stress in collar joint $= S_c = 1.5^{psi} \times 1.5 = 2.25^{psi} < 8 \quad \text{O.K.}$

$\frac{f}{F} = \frac{2.25}{8} = 0.28$

For DBE Loads,

$f_m = 1.63 \times 8.5^{psi} = 13.9^{psi} < 67 \quad \text{O.K.}$

$v = 1.63 \times 1.5^{psi} = 2.4 \text{ psi} < 72 \quad \text{O.K.}$

$S_c = 1.63 \times 2.25^{psi} = 3.7 \text{ psi} < 12 \quad \text{O.K.} \quad \frac{f}{F} = \frac{3.7}{12} = 0.31$

GENERAL COMPUTATION SHEET

united engineers & constructors inc

NAME OF COMPANY: CP & L   UNIT/S: 1, 2

SUBJECT: REACTOR BUILDING MASONRY WALLS

CALC. SET NO		REV	COMP BY	CHK'D BY
PRELIM		0	R. [illegible]	
FINAL	X		DATE 10-30-80	DATE
VOID				
SHEET 13 OF 31			DATE	DATE
JO 9527-CEE				

WALL 56   UNIT #1 & #2

REFERENCE DWG F-12075, F-25001

L = LENGTH = 5'-1"

H = HEIGHT = 9'-6"   EL. 20'-0"

UNIT 1 = 4 WYTHES @ 8" = 32" UNIT 2 = 2 WYTHES @ 12" + ONE WYTHE @ 8 = 32

SINCE MULTIPLE WYTHES ARE NOT POSITIVELY CONNECTED TOGETHER, ASSUME EACH WYTHE AS AN INDIVIDUAL WALL

$\frac{H}{L} = \frac{9.5}{5.08} = 1.87 > 1.5$ WALL SHOULD SPAN HORIZONTALLY

WALL CONSTRUCTED OF STD WEIGHT SOLID BLOCKS (144 PCF)
ALL CORES FILLED MORTAR. UNREINFORCED WALLS

CONSIDER SINGLE 8" WYTHE

$I = \frac{12 \times 7.625^3}{12} = 443 \text{ IN}^4$ $WT = 144 \times \frac{8}{12} = 96 \text{ PSF} \times \frac{1}{12} = 8\text{#/IN}$

P = PERIOD OF WALL

$P = \frac{2}{\pi}\sqrt{\frac{w l^4}{EIg}} = \frac{1}{1.57}\sqrt{\frac{8 \times (61)^4}{386 \times 443 \times 1820000}} = 0.012 \text{ SEC.}$

FREQUENCY = 83 CPS > 33 CPS

FREQUENCY SPREAD 1.095 ⟶ 0.895

$f_{high} = 1.095 \times 83 = 91 \text{ CPS}$ $p = 0.0109$

$f_{low} = 0.895 \times 83 = 74 \text{ CPS}$ $p = 0.0135$

FROM COMPUTER PRINT OUT

	OBE 2%	DBE 4%	
ACC^n	0.31	0.50	EL. 50'-0" N-S RESPONSE.
	0.27	0.42	EL. 20'-0" N-S RESPONSE

DESIGN ACC^n = 0.46 (DBE)

DESIGN ACC^n = 0.29 (OBE)

$RATIO = \frac{0.46}{0.29} = 1.59$

$WH = 0.29 \times 96 \times 5.08 = 142\text{#} \times \frac{1}{2} = R = V = 71\text{ #}$

$MOMENT = \frac{142 \times 5.08}{8} = 90'\text{#}$

WALL 56 CONTINUED

$\frac{L}{C} = \frac{9.3}{75} = 0.12$ COMP $= \frac{9.3}{600} = 0.016$

$f_m = \frac{90 \times 12 \times 3.81}{443} = 9.30$ PSI < 75 PSI ($1.5\sqrt{m_o} = 1.5 \times 50 = 75$ PSI)

STRESS PARALLEL TO JOINT

$V = \frac{71}{7.625 \times 12} = 0.78$ psi < 47 ok ($1.1\sqrt{f'm} = 47$)

$\frac{L}{C} = \frac{0.78}{47} = 0.017$

$f'm = 1820$ psi

FOR DBE

$\frac{L}{C} = \frac{15}{125} = 0.12$

$f_m = 9.3 \times 1.59 = 15$ psi < 125 psi ok ($2.5\sqrt{2500} = 125$ psi)

$v = 0.78 \times 1.59 = 1.24$ psi < 72 ok ($1.7\sqrt{f'm} = 72$)

COMP $= \frac{15}{1547} = 0.01$ $\frac{L}{C} = \frac{1.24}{72} = 0.017$

CHECK 12" WYTHE

$I = \frac{12 \times 11.625^3}{12} = 1571$ in^4 WT $= 144 \times \frac{1}{12} = 12$ #/in

PERIOD $P = \frac{1}{1.57}\sqrt{\frac{WL^4}{EI g}} = \frac{1}{1.57}\sqrt{\frac{12 \times 61^4}{1571 \times 1820000 \times 386}} = 0.0078$

FREQUENCY $= \frac{1}{.0078} = 128$

FREQUENCY SPREAD $1.095 \rightarrow 0.895$

$f_{high} = 1.095 \times 128 = 140$ $p = 0.0071$

$f_{low} = 0.895 \times 128 = 115$ $p = 0.0086$

FROM COMPUTER PRINTOUTS

	OBE 2%	DBE 4%	
ACC'N	0.31	0.50	EL. 50'-0" N-S. RESPONSE
	0.27	0.42	EL. 20'-0" N-S RESPONSE

RATIO $= \frac{DBE}{OBE} = \frac{0.46}{0.29} = 1.59$

$WH = 0.29 \times 144 \times 5.05 = 212$ #

$R = V = 106$#

MOMENT $= 212 \times \frac{5.08}{8} = 135$ 'lb

$\frac{L}{C} = \frac{6}{75} = 0.08$ $f_m = \frac{135 \times 12 \times 5.81}{1571} = 6.0 < 75$ PSI

$\frac{L}{C} = \frac{0.8}{47} = 0.017$ $v = \frac{106}{11.625 \times 12} = 0.80$ PSI < 47 PSI

DBE LOADS

$f_m = 6 \times 1.59 = 9.5 < 125$ psi

$v = 0.8 \times 1.59 = 1.3 < 72$ PSI

$\frac{L}{C} = \frac{9.5}{125} = 0.08$

$\frac{L}{C} = \frac{1.3}{72} = 0.018$

WALL IS OK AS PER NEW CRITERIA

WALL 5b CONTINUED

CHECK SHEAR IN COLLAR JOINTS —
THICKNESS OF WALL = 2'-8"

$I = \frac{12 \times 32^3}{12} = 32768\ in^4$ $wt = 2.66 \times 144\ psf = \frac{383}{12} = 32\#/in$

$P = \frac{2}{\pi}\sqrt{\frac{WL^4}{EIg}} = \frac{1}{1.57}\sqrt{\frac{32 \times 61^4}{386 \times 1820000 \times 32768}} = 0.0028$

FREQUENCY = 357
FREQUENCY SPREAD 1.095 → 0.895

$f_{high} = 1.095 \times 357 = 391\ CPS$ $p = 0.0025\ SEC$
$f_{low} = 0.895 \times 357 = 319\ CPS$ $p = 0.0031\ SEC$

FROM COMPUTER PRINTOUTS

	OBE 2%	DBE 4%	
Acc^n	0.31	0.50	EL 50'-0" N-S RES...
	0.27	0.42	EL 20'-0" N-S RESPONSE

DESIGN ACC^n = 0.29 (OBE)
DESIGN ACC^n = 0.46 (DBE)

$RATIO = \frac{DBE}{OBE} = \frac{0.46}{0.29} = 1.59$

$W_H = 383 \times 0.29 \times 5.08 = 564\ lb$ $R = V = 282\ lb$

$Moment = \frac{564 \times 5.08}{8} = 358'lb$

$f_m = \frac{358 \times 12 \times 16}{32768} = 2.0\ PSI < 75\ psi$

$V = \frac{282}{32 \times 12} = 0.73 < 47\ psi$

DBE LOADS
$f_m = 2.0 \times 1.59 = 3.2 < 125\ psi$
$f_v = 0.73 \times 1.59 = 1.2 < 72\ psi$
WALL IS OK AS PER NEW CRITERIA

9527-1-GP-NW-02-F

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

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PRELIM		0	R. PRASAD	[illegible]
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SHEET 16 OF 31			DATE	DATE
J.O. 9527-OBE				

NAME OF COMPANY C.P. & L UNIT/S 1, 2,

SUBJECT REACTOR BUILDING MASONRY WALL

WALL 56 CONTINUED

CHECK STRESS FOR COLLAR JOINTS

OBE LOAD

SHEAR STRESS $= 1.5 \times 0.73 = 1.1$ psi < 8 psi OK.

$$L/c = \frac{1.1}{8} = 0.14$$

FOR DBE LOAD

SHEAR STRESS $= 1.59 \times 1.1 = 1.75$ psi < 12 psi OK $L/c = \frac{1.75}{12} = 0.146$

HENCE THE WALL IS OK
FOR NEW DESIGN CRITERIA

CHECK FOR INPLANE SHEAR STRAIN
WALL IN EAST-WEST DIRECTION
WALL IS OK SEE SHT NO-30

GENERAL COMPUTATION SHEET

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NAME OF COMPANY C P & L UNIT/S 1, 2

SUBJECT REACTOR BLDG. - MASONRY WALLS

Walls 6a, b & c - Units

walls 6a, b - 5'-0" L x 8'-0" H x 3'-0" Thick (3-8" wythes & 1-12" wythe - #1)
(3-12" wythes for Unit #2)

wall 6c - 5'-0" L x 9'-4" H x 4'-0" Thick (6-8" wythes - Unit #1)
4-12" wythes - Unit #2)

walls should span horizontally - 5'-0" span ($\frac{H}{L} > 1.5$)

check 8" wythe -

$I = 443 \text{ in}^4$ $w = 96 \text{ PSF}$ $P = 0.000465 L^2$ (see sht. #1)

$P = 0.000465 (5)^2 = 0.012 \text{ sec} \longrightarrow f = 83 \text{ cps} > 33 \text{ cps}$

frequency spread = 1.095 — 0.895

$f_{high} = 1.095 \times 83 = 91 \text{ cps} \rightarrow P = 0.011 \text{ sec}$

$f_{low} = 0.895 \times 83 = 74.3 \text{ cps} \rightarrow P = 0.013 \text{ sec}$

- accel. from computer printouts.

	2% OBE	4% DBE	
EL. 50'	0.30	0.49	E-W response
EL 80'	0.33	0.53	E-W response

Design accel. OBE = 0.32, DBE = 0.51 (Conservative since walls do not extend to EL 80')

accel. ratio = $\frac{0.51}{0.32} = 1.60$

$W_H = 0.32 \times 96^{PSF} \times 5' = 154^{\#} \times \frac{1}{2} = 77^{\#} = R = V$

$m = 154^{\#} \times \frac{5'}{8} = 97'^{\#}$

$f_m = \frac{97 \times 12 \times 3.81}{443} = 10 \text{ psi} < 75$ O.K. $\frac{L}{C} = \frac{10}{75} = 0.13$

$v = \frac{77^{\#}}{7.625 \times 12} = 0.8 \text{ psi} < 47$ O.K. $\frac{L}{C} = \frac{0.8}{47} = 0.017$

For DBE Loads,

$f_m = 10^{psi} \times 1.60 = 16 \text{ psi} < 125$ O.K. $\frac{L}{C} = \frac{16}{125} = 0.13$

$v = 0.8^{psi} \times 1.6 = 1.3 \text{ psi} < 72$ O.K. $\frac{L}{C} = \frac{1.3}{72} = 0.018$

9527-1-GP-MW-02-F

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GENERAL COMPUTATION SHEET

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NAME OF COMPANY: G P E L   UNIT/S: 1, 2

SUBJECT: REACTOR BLDG – MASONRY WALLS

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walls 6a, b & c – cont.

check 12" wythe –

$I = 1571\ in^4$ $w = 144\ PSF$ $P = 0.000302\,L^2$ (see sht. #2)

$P = 0.000302(5)^2 = 0.0076\ sec \rightarrow f = 131.6\ cps > 33\ cps$

Frequency spread: $1.095 \rightarrow 0.895$

$f_{high} = 1.095 \times 131.6 = 144\ cps \rightarrow P = 0.007\ sec.$

$f_{low} = 0.895 \times 131.6 = 117.8\ cps \rightarrow P = 0.0085\ sec.$

accel. from computer printouts.

	2% OBE	4% DBE	
EL. 50'-0"	0.30	0.49	E-W response
EL. 80'-0"	0.33	0.53	E-W response

Design accel. OBE = 0.32 (conservative)

DBE = 0.51

accel ratio $= \frac{0.51}{0.32} = 1.60$

$w_H = 0.32 \times 144^{PSF} \times 5' = 230^{\#} \times \frac{1}{2} = 115^{\#} = R = V$

$M = 230^{\#} \times \frac{5'}{8} = 144'^{\#}$

$f_m = \frac{144 \times 12 \times 5.81}{1571} = 7\ psi < 75$ O.K. $\frac{L}{C} = \frac{7}{75} = 0.10$

$v = \frac{115^{\#}}{12 \times 11.625} = 0.8\ psi < 47$ O.K. $\frac{L}{C} = \frac{0.8}{47} = 0.02$

For DBE Loads,

$f_m = 7^{psi} \times 1.6 = 11\ psi < 125\ psi$ $\frac{L}{C} = \frac{11}{125} = 0.09$

$v = 0.8^{psi} \times 1.6 = 1.3\ psi < 72\ psi$ $\frac{L}{C} = \frac{1.3}{72} = 0.018$

wall is acceptable as per new criteria

check walls for Inplane shear strain –

walls in N-S direction and are confined.

walls O.K. see sheet # 30

Walls 6 a. b cont. -

wall 36" thick - spans 5'-0" horizontally.

check shear in collar joints -

$I = \frac{12 \times 36^3}{12} = 46{,}656 \text{ in}^4$ $w = 144^{pcf} \times 3' = 432^{\#/'} \times \frac{1}{12} = 36^{\#/"}$

$P = \frac{1}{1.57}\sqrt{\frac{36 \times 60^4}{386 \times 1{,}820{,}000 \times 46{,}656}} = 0.0024 \text{ sec} \rightarrow f = 416 \text{ cps} >> 33 \text{ cps}$

Period spread 0.913 → 1.12

$P_{low} = 0.913 \times 0.0024 = 0.0022 \text{ sec}$
$P_{high} = 1.12 \times 0.0024 = 0.0027 \text{ sec}$

accel. from computer printouts.

	2% OBE	4% DBE	
EL. 50'-0"	0.30	0.49	E-W response
EL. 80'-0"	0.33	0.53	E-W response

Design accel. OBE = 0.32g , DBE = 0.51g

accel. ratio = $\frac{0.51}{0.32} = 1.60$

$w_u = 0.32 \times 432^{\#} \times 5' = 692^{\#} \times \frac{1}{2} = 346^{\#} = R = V$
$m = 692^{\#} \times \frac{5'}{8} = 433'^{\#}$

$f_m = \frac{433'^{\#} \times 12 \times 18}{46{,}656} = 2 \text{ psi} < 75$ O.K

$v = \frac{346^{\#}}{12 \times 36} = 0.8 \text{ psi}$ Negligible.

$S_c = 1.5 \times 0.8^{psi} = 1.2^{psi} < 8$ O.K $\frac{f}{F} = 0.15$

For DBE.

$f_m = 2 \times 1.60 = 3.2^{psi} < 125$ O.K
$v = 0.8 \times 1.6 = 1.3 \text{ psi} < 72$ O.K.
$S_c = 1.2 \times 1.6 = 2 \text{ psi} < 12$ O.K $\frac{f}{F} = \frac{2}{12} = 0.17$

Wall 6C

check for Tension in collar joints -

Wall 48" thick, spans 5'-0" Horizontally.

$I = \frac{12 \times 48^3}{12} = 110592 \text{ in}^4$ $w = 144^{pcf} \times 4' = 576^{\#/'} \times \frac{1}{12} = 48^{\#/''}$

$P = \frac{1}{1.57}\sqrt{\frac{48 \times 60^4}{386 \times 1820000 \times 110592}} = 0.0018 \text{ sec} \rightarrow f = 555 \text{ cps} >> 33 \text{ cps}$

Period spread = 0.913 → 1.12

$P_{Low} = 0.0018 \times 0.913 = 0.0016$
$P_{High} = 0.0018 \times 1.12 = 0.0020$

accel. from computer printouts.

	2% OBE	4% DBE	
EL. 50'	0.30	0.49	E-W response
EL. 80'	0.33	0.53	E-W response

Design accel. OBE = 0.32 g, DBE = 0.51 g
accel ratio = $\frac{0.51}{0.32} = 1.60$

$w_H = 0.32 \times 576^{\#} \times 5' = 922^{\#} \times \frac{1}{2} = 461^{\#} = R = V$

$M = 922^{\#} \times \frac{5'}{8} = 576'^{\#}$

$f_m = \frac{576'^{\#} \times 12 \times 24}{110592} = 1.5 \text{ psi} < 75$ O.K.

$v = \frac{461^{\#}}{12 \times 48} = 0.8 \text{ psi}$ Negligible.

$s_c = 1.5 \times 0.8 = 1.2^{psi} < 8$ O.K. $\frac{L}{C} = \frac{1.2}{8} = 0.15$

For DBE,

$f_m = 1.5^{psi} \times 1.6 = 2.4 \text{ psi} < 125$ O.K.
$v = 0.8 \times 1.6 = 1.3 \text{ psi} < 72$ O.K.
$s_c = 1.2 \times 1.6 = 2 \text{ psi} < 12$ O.K.

$\frac{L}{C} = \frac{2}{12} = 0.17$

FORM 500? REV 7/79 GENERAL COMPUTATION SHEET
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JO 9527-086			DATE	DATE

NAME OF COMPANY CP & L UNIT/S 1, 2

SUBJECT REACTOR BUILDING MASONRY WALL

WALL 8CEEB UNIT #1 & #2 REF DWG - 12075
DWG - 1275

WALL IS 7'-0" H × 12'-0" L × 4'-0" THICK (6 WYTHES @ 8") UNIT #1
7'-0 H × 12'-0" L × 4'-0" THICK (4 WYTHES @ 12") UNIT #2

SINCE MULTIPLE WYTHES ARE NOT POSITIVELY CONNECTED TOGETHER ASSUME EACH WYTHE AS AN INDIVIDUAL WALL

$\frac{L}{H} = \frac{12}{7} = 1.7 > 1.5$ WALL SHOULD SPAN VERTICALLY

WALL CONSTRUCTED OF Std WEIGHT SOLID BLOCKS (144 PCF)
ALL CORES FILLED MORTAR. UNREINFORCED WALL
TYPE O MORTAR USED

CONSIDER SINGLE 8" WYTHE

$f'_m = 500$ psi
BY EXTRAPOLATION
TABLE 4.3 ACI 531-79
$E = 1000 \times 500$
$= 500,000$ PSI

MOMENT OF INERTIA $= I = \frac{12 \times 7.625^3}{12} = 443$ IN4

$WT = 144 \times \frac{8}{12} = 96$ PSF $\times \frac{1}{12} = 8$ #/IN

$P = \frac{1}{1.57}\sqrt{\frac{wL^4}{EIg}} = \frac{1}{1.57}\sqrt{\frac{8 \times 84^4}{386 \times 500{,}000 \times 443}} = 0.0435$

FREQUENCY $= \frac{1}{0.0435} = 23$

FREQUENCY SPREAD 1.095 → 0.895
$f_{high} = 1.095 \times 23 = 25$ CPS P = 0.04 SEC
$f_{low} = 0.895 \times 23 = 21$ CPS P = 0.048 SEC

FROM COMPUTER PRINTOUTS

	OBE 2%	DBE 4%
Acc^n	0.30	0.48 EL. 20'-0" E-W RESPONSE
Acc^n	0.32	0.507 EL 50'-0 E-W RESPONSE

AVERAGE $Acc^n = 1.05 \times \frac{0.30 + 0.32}{2} = 0.325$ (OBE)

AVERAGE $Acc^n = 1.05 \times \frac{0.48 + 0.507}{2} = 0.495$ (DBE)

$\frac{DBE}{OBE} = \frac{0.495}{0.33} = 1.5$

$WH = 0.325 \times 96 \times 7 = 218.4$ $R = V = 109.2$

$M = \frac{218.4 \times 7}{8} = 191.0$ 'lb

Wall 80.3 Contd

TYPE O MORTAR

$f_m = \frac{191.0 \times 12 \times 3.81}{443} = 19.7$ PSI > 18.7 PSI $(1.0\sqrt{m_o} = 18.7 \perp$ TO BED JOINT
(slightly overstressed ok) $m_o = 350$ psi

$\frac{L}{C} = \frac{19.7}{18.7} = 1.05$ $\quad$ COMP $= \frac{19.7}{165} = 0.11$

$v = \frac{109.2}{7.625 \times 12} = 1.12 < 25$ PSI OK

$1.1\sqrt{f_m} = 25$ psi $\quad \frac{L}{C} = \frac{1.12}{25} = 0.044$

FOR DBE LOAD

$f_m = 1.5 \times 19.7 = 29.5 < 31$ PSI OK $\quad \frac{L}{C} = \frac{29.5}{31} = 0.95$

$v = 1.5 \times 1.12 = 1.68$ PSI < 38 PSI OK $\quad \frac{L}{C} = \frac{1.68}{38} = 0.044$

COMP $= \frac{29.5}{425} = 0.07$

CHECK 12" WYTHE

$I = \frac{12 \times 11.625^3}{12} = 1571\ IN^3$

$WT = 144 \times \frac{1}{12} = 12$ #/IN

THICKNESS OF WALL = 4'-0"

$P = \frac{1}{1.57}\sqrt{\frac{WL^4}{EIg}} = \frac{1}{1.57}\sqrt{\frac{12 \times 84^4}{386 \times 500000 \times 1571}} = 0.028$

frequency $= \frac{1}{0.028} = 35.71 > 33$ CPS

FREQUENCY SPREAD 1.095 → 0.895

$f_{high} = 1.095 \times 35.71 = 39.0 \quad P = 0.026$ sec

$f_{low} = 0.895 \times 35.71 = 32 \quad P = 0.031$ SEC

From Computer Printouts

	OBE 2%	DBE 4%	
Accn	0.29	0.47	EL. 20'-0" (E-W RESPONSE)
Accn	0.31	0.50	EL 50'-0" (E-W RESPONSE)

AVERAGE Accn $= \frac{0.29 + 0.31}{2} = 0.30$ (OBE)

AVERAGE Accn $= \frac{0.47 + 0.50}{2} = 0.485$ (DBE)

Accn RATIO $= \frac{DBE}{OBE} = \frac{0.485}{0.30} = 1.61$

$WH = 0.30 \times 144 \times 7 = 303$ # $\quad R = V = 151.5$ lb

$M = 303 \times \frac{7}{8} = 265$ '#

Y = 5.81 (AS PER PAGE 13 BY DALE)

Wall 20.b contd

$f_m = \frac{265 \times 12 \times 5.81}{1544} = 12 < 18.7$ PSI OK. $\frac{L}{C} = \frac{12}{18.7} = 0.64$

$v = \frac{151.5}{11.625 \times 12} = 1.08 < 25$ psi OK $\frac{L}{C} = \frac{1.08}{25} = 0.043$

CHECK FOR DBE LOAD

$f_m = 12.0 \times 1.61 = 19$ PSI < 31 PSI OK. $\frac{L}{C} = \frac{19}{31} = 0.61$

$v = 1.04 \times 1.61 = 1.7$ PSI < 38 PSI OK $\frac{L}{C} = \frac{1.7}{38} = 0.044$

WALL IS ACCEPTABLE AS PER NEW DESIGN CRITERIA

CHECK SHEAR IN COLLAR JOINTS

WALL THICKNESS 4'-0"

$I = \frac{12 \times 48^3}{12} = 110592 \text{ IN}^4$ $WT = 144 \times 4 \times \frac{1}{12} = 48$ #/IN

$P = \frac{1}{1.57}\sqrt{\frac{WL^4}{EIg}} = \frac{1}{1.57}\sqrt{\frac{48 \times 84^4}{386 \times 110592 \times 500000}} = 0.0067$ SEC.

Frequency $= \frac{1}{0.0067} = 149$ CPS. > 33 CPS

Frequency Spread $- 1.095 \rightarrow 0.895$

$f_{low} = 1.095 \times 149 = 163$ cps. $P = 0.0061$ SEC.

$f_{high} = 0.895 \times 149 = 133$ CPS $P = 0.0075$ SEC.

From Computer printouts

	OBE 2%	DBE 4%	
Acc^n	0.289	0.469	EL 20'-0" E-W RESPONSE
Acc^n	0.304	0.496	EL 50'-0" E-W RESPONSE

AVERAGE $ACC^n = \frac{0.289 + 0.304}{2} = 0.29$ (OBE)

AVERAGE $ACC^n = \frac{0.469 + 0.491}{2} = 0.48$ (DBE)

RATIO $= \frac{DBE}{OBE} = \frac{0.48}{0.296} = 1.65$

Wall East contd

WH = 0.29 x 576 x 7 = 1169 lb.   R = V = 1169/2 = 585 lb.

MOMENT = 1169 x 7/8 = 1023 'lb

$fm = \frac{1023 \times 12 \times 24}{110592} = 2.6$ psi < 18.7 psi OK $L/c = \frac{2.6}{18.7} = 0.14$

$v = \frac{584.5}{48 \times 12} = 1.02$ psi < 25 psi OK $L/c = \frac{1.02}{25} = 0.04$

DBE LOAD

fm = 2.6 x 1.65 = 4.29 psi < 31 psi OK $L/C = \frac{4.29}{31} = 0.14$

v = 1.02 x 1.65 = 1.68 psi < 38 psi $L/c = \frac{1.68}{38} = 0.04$

COLLAR JOINTS

FOR OBE LOAD

SHEAR STRESS = 1.5 x 1.02 = 1.53 < 8 psi

$L/c = \frac{1.53}{8} = 0.19$

FOR DBE LOAD

SHEAR STRESS = 1.65 x 1.53 = 2.52 psi < 12 psi

$L/c = \frac{2.52}{12} = 0.21$

CHECK FOR INPLANE SHEAR STRAIN

WALL IN N-S DIRECTION CONFINED

WALL IS OK SEE SHT NO-30

Walls 10b & 10c   Ref DWG - 9527-F-12076
F-1276

Original calcs - sheet #22/Dale

Assume that the bond and friction between block and concrete slab will provide support at the top of walls.

walls span vertically.

Properties of 8" Hollow unreinforced walls.

$I = 365\ in^4$ $w = 32\ ^{PSF}$ $P = 0.000296L^2$ (see sht. # 7/Dale)

$P = 0.000296(9.33)^2 = 0.026\ sec \longrightarrow f = 38.8\ cps > 33\ cps$

$\left(M_{cr} = \frac{93 \times 365}{12 \times 3.81} = 742'^{\#}\right)$

Frequency Spread = 1.0 → 0.775

$f_{high} = 38.8 \times 1.0 = 38.8\ cps \longrightarrow P = 0.026\ sec.$

$f_{low} = 38.8 \times 0.775 = 30.1\ cps \longrightarrow P = 0.033\ sec.$

		2% OBE	4% DBE	
EL. 20'	accel. =	0.30	0.48	E-W response for wall 10c
EL. 20'	accel. =	0.28	0.44	N-S response for wall 10a

Since the walls do not go upto next Level, use only floor response

check the walls for max. response in E-W direction.

$w_H = 0.30 \times 32 \times 9.33' = 90^{\#} \times \frac{1}{2} = 45^{\#} = R = V$

$m = 90 \times \frac{9.33'}{8} = 105'^{\#} < M_{cr} = 742'^{\#}$ ∴ uncracked

$f_m = \frac{105'^{\#} \times 12 \times 3.81}{365} = 13\ psi < 25$ O.K. $\frac{L}{C} = \frac{13}{25} = 0.52$

$v = \frac{90^{\#}}{2 \times 12 \times 1.375} = 3\ psi < 47$ O.K. $\frac{L}{C} = \frac{3}{47} = 0.06$

For DBE Loads

accel. ratio $= \frac{0.48}{0.30} = 1.60$

$f_m = 13^{psi} \times 1.6 = 21\ psi < 42$ O.K $\frac{L}{C} = \frac{21}{42} = 0.50$

$v = 1.6 \times 3^{psi} = 5\ psi < 72$ O.K. $\frac{L}{C} = \frac{5}{72} = 0.07$

walls are acceptable as per new criteria.

Check inplane shear strain criteria -

Wall 10b - in E-W direction O.K

Wall 10c - in N-S direction - confined O.K } see sheet #30

5OC7 REV 7/79 GENERAL COMPUTATION SHEET

(PLINE)

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PRELIM		0	F. F U	~I
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SHEET 26 OF 31			DATE	DATE
JO 9527.088				

NAME OF COMPANY C D & L UNIT/S 1, 2

SUBJECT REACTOR BLDG - MASONRY WALLS

Wall 12 a - EL 20'-0"

Ref DWG 9527-F-1273

wall is unreinforced - Bond beam is @ EL - 41-4"

wall Dimensions - 12'-4" L x 21'-4" H x 8" hollow blocks

$\frac{H}{L} = \frac{21'-4"}{12'-4"} = 1.73 > 1.5$ ∴ wall spans horizontally.

wall properties - see sht # 12 / Dale

$I = 341 \text{ in}^4$ $w = 32 \text{ PSF}$ $P = 0.000305 L^2$

$P = 0.000305 (12.33)^2 = 0.046 \text{ sec} \rightarrow f = 21.6 \text{ cps}$

frequency range = 1.0 → 0.775

$f_{high} = 1.0 \times 21.6 = 21.6 \text{ CPS} \rightarrow P = 0.046 \text{ sec.}$

$f_{low} = 0.775 \times 21.6 = 16.7 \text{ CPS} \rightarrow P = 0.06 \text{ sec}$

accel. from computer printouts.

	2% OBE	4% DBE	
EL. 20'	0.29	0.46	N-S response
EL. 50'	0.33	0.53	N-S response.

Design accel OBE $= \frac{0.29 + 0.33}{2} \times 1.05 = 0.33$

DBE $= \frac{0.46 + 0.53}{2} \times 1.05 = 0.52$

accel ratio $= \frac{0.52}{0.33} = 1.58$ $\left(M_{cr} = \frac{93 \times 341}{12 \times 3.81} = 693'^{\#} \right)$

$w_H = 0.33 \times 32^{PSF} \times 12.33' = 130^{\#} \times \frac{1}{2} = 65^{\#} = R = V$

$M = 130^{\#} \times \frac{12.33}{8} = 200'^{\#}$

$f_m = \frac{200'^{\#} \times 12 \times 3.81}{341} = 27 \text{ psi} < 50$ O.K. $\frac{L}{C} = \frac{27}{50} = 0.54$

$v = \frac{65^{\#}}{2 \times 12 \times 1.375} = 2 \text{ psi} < 47$ O.K. $\frac{L}{C} = \frac{2}{47} = 0.04$

For DBE Loads

$f_m = 27^{psi} \times 1.58 = 43^{psi} < 84$ O.K. $\frac{L}{C} = \frac{43}{84} = 0.51$

$v = 2^{psi} \times 1.58 = 3.2^{psi} < 72$ O.K. $\frac{L}{C} = \frac{3.2}{72} = 0.044$

wall is acceptable as per new criteria

5007 REV 7/79

GENERAL COMPUTATION SHEET

united engineers & constructors inc

(DISCIPLINE)

CALC SET NO.		REV	COMP BY	CHK'D BY
PRELIM		0	R F U	
FINAL	X		DATE 10/2/80	DATE
VOID				
SHEET 27 OF 31			DATE	DATE
JO 9527-088				

NAME OF COMPANY CP&L UNIT/S 1, 2

SUBJECT REACTOR BLDG - MASONRY WALLS

wall 12 b EL. 50'-0" Ref. DWG. - 9527-F-1273.

wall is unreinforced - Bond beam is @ EL. 71'-4"

wall dimension: - 10'-4" L × 21'-4" H 8" hollow blocks

$\frac{H}{L} = \frac{21'-4"}{10'-4"} = 2.06 > 1.5$

wall spans horizontally

wall properties - see sht. #12 / Dale.

$I = 341 in^4$ $w = 32$ PSF $P = 0.000305 L^2$

$P = 0.000305 (10.33)^2 = 0.033$ sec → $f = 30.3$ cps

frequency range = 1.0 → 0.775

$f_{high} = 1.0 \times 30.3 = 30.3$ cps → $P = 0.033$ sec.

$f_{low} = 0.775 \times 30.3 = 23.5$ cps → $P = 0.042$ sec.

accel from computer printouts.

	2% OBE	4% DBE	
EL. 50'-0"	0.32	0.51	N-S response
EL. 80'-0"	0.41	0.66	N-S response

Design accel $OBE = \frac{0.32 + 0.41}{2} \times 1.05 = 0.38$

$DBE = \frac{0.51 + 0.66}{2} \times 1.05 = 0.61$

accel ratio $= \frac{0.61}{0.38} = 1.61$

$w_H = 0.38 \times 32 \times 10.33' = 126^{\#} \times \frac{1}{2} = 63^{\#} = R = V$

$M = 126^{\#} \times \frac{10.33'}{8} = 163'^{\#}$

$f_m = \frac{163'^{\#} \times 12 \times 3.81}{341} = 22$ psi < 50 O.K. $\frac{f}{c} = \frac{22}{50} = 0.44$

$v = \frac{63^{\#}}{2 \times 12 \times 1.375} = 2$ psi < 47 O.K. $\frac{f}{c} = \frac{2}{47} = 0.04$

For DBE Loads,

$f_m = 22^{psi} \times 1.61 = 36$ psi < 84 O.K. $\frac{f}{c} = \frac{36}{84} = 0.43$

$v = 2^{psi} \times 1.61 = 3.2$ psi < 72 O.K. $\frac{f}{c} = \frac{3.2}{72} = 0.04$

wall is acceptable as per new criteria

		REV	COMP BY	CHK'D BY
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GENERAL COMPUTATION SHEET

united engineers & constructors inc

(LINE)

NAME OF COMPANY C P & L UNIT/S 1 2

SUBJECT REACTOR BLDG - MASONRY WALLS

Wall 12C EL. 50' ELEVATOR SHAFT

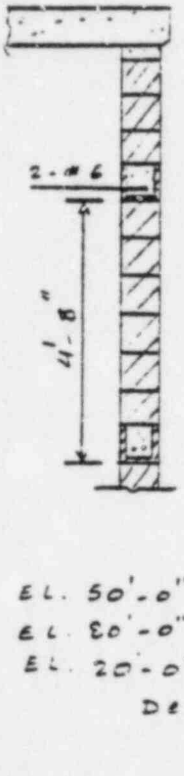
Ref DWGS - F-1273, F-12073

Original calcs by Dale sheet # 32, 33, 34 & 12

Bond beams are 4'-8" o.c. - wall spans → ally.

Wall properties (8" Hollow block wall)

w = 15.3 #/in (see sheet #32/Dale) I_{cr} = 145 in⁴



(Hori span) $I_g = \frac{56 \times 7.625^3}{12} - \frac{48 \times 4.875^3}{12} = 1605\ in^4$

I_g per ft width $= \frac{1605}{4.67} = 343.8\ in^4/ft$

$$P = \sqrt{\frac{15.3 \times 10.000.000.000}{386 \times 1.820.000 \times 343.8} \times \frac{144\,(9)^2}{157.000}}$$

$= 0.059$ sec → f = 16.9 cps.

Frequency spread = 1.095 → 0.895

f_{high} = 16.9 × 1.095 = 18.5 cps → P = 0.054 sec

f_{low} = 16.9 × 0.895 = 15.1 cps → P = 0.066 sec

accel. from computer printouts.

	4% OBE	7% DBE	
EL. 50'-0"	0.32	0.51	E-W response
EL. 80'-0"	0.35	0.56	E-W response
EL. 20'-0"	0.30	0.50	E-W response

Design accel. OBE $= \frac{0.32 + 0.35}{2} \times 1.05 = 0.35g$

DBE $= \frac{0.51 + 0.56}{2} \times 1.05 = 0.56g$

accel. ratio $= \frac{0.56}{0.35} = 1.6$ $\left(m_{cr} = \frac{343.8 \times 93}{12 \times 3.81} = 699'^{\#}\right)$

w_H = 0.35 × 183.7 #/' × 9 = 579# × ½ = 290# = R = V

$m = 579 \times \frac{9}{8} = 652'^{\#} < m_{cr} = 699'^{\#}$ ∴ uncracked.

$f_m = \frac{652'^{\#} \times 12 \times 1.03"}{145} = 56$ psi < 600 O.K. $\frac{L}{C} = \frac{56}{600} = 0.10$

$f_s = \frac{652 \times 12 \times 4.22" \times 15.9}{145} = 3620$ psi < 24.000 O.K.

$\frac{L}{C} = \frac{3620}{24.000} = 0.15$

Wall 12c cont.

$v = \frac{290^{\#}}{7.625 \times 5.25''} = 7.3$ psi < 47 O.K. $\frac{L}{C} = \frac{7.3}{47} = 0.15$

$u = \frac{290^{\#}}{2.4 \times .93 \times 5.25} = 25$ psi < 140 O.K. $\frac{L}{C} = \frac{25}{140} = 0.18$

For DBE Loads,

$f_m = 56^{psi} \times 1.6 = 90$ psi < 1547 psi $\frac{L}{C} = \frac{90}{1547} = 0.06$

$f_s = 3620^{psi} \times 1.6 = 5792 < 54,000$ $\frac{L}{C} = \frac{5792}{54,000} = 0.11$

$v = 7.3^{psi} \times 1.6 = 12$ psi < 72 O.K $\frac{L}{C} = \frac{12}{72} = 0.17$

$u = 25^{psi} \times 1.6 = 40$ psi < 186 O.K $\frac{L}{C} = \frac{40}{186} = 0.22$

Check 6 blocks between bond beams for bending ⊥ to bed joints - see sketch on sht #18

$w_H = 0.35 \times 32^{\#} \times 4' = 45^{\#} \times \frac{1}{2} = 23^{\#}$

$m = 45^{\#} \times \frac{4'}{8} = 23'^{\#}$

$f_m = \frac{23'^{\#} \times 12 \times 3.81}{365} = 3$ psi < 25 O.K $\frac{L}{C} = \frac{3}{25} = 0.12$

since wall at EL 50'-0" is acceptable, the wall 12c at lower level (EL 20'-0") is also acceptable by inspection.

check walls for Inplane shear strain -

wall 12b - in E-W direction
wall 12c - in N-S direction and confined.

walls o.k. see sheet #30

...M 5007 REV 7/70 GENERAL COMPUTATION SHEET

...CIPLINE)

united engineers & constructors inc.

NAME OF COMPANY C P & L UNIT/S 1, 2

SUBJECT REACTOR BLDG. - MASONRY WALLS

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
PRELIM		0	R. P. U.	CR
FINAL	X		DATE 11/3/80	DATE 11/.../80
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SHEET 30 OF 31			DATE	DATE
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IN-PLANE SHEAR STRAIN CRITERIA

As per 0.6 committee report section #5, acceptable shear strain for confined walls is 0.001 and for unconfined walls is 0.0001.

The max. relative displacements are as per memo from SA.YGn. of Structural Analysis Group dated 10/24/80

For DBE

E-W Direction, between EL. 20'-0" & 50'-0"

max relative displacement = 0.03386"

$\gamma = \frac{\Delta}{H} = \frac{0.03386''}{30 \times 12} = 0.0000941 < 0.001$

< 0.0001

O.K for confined and unconfined walls.

E-W Direction, between EL. 50'-0" & 80'-0"

$\gamma = \frac{0.05035''}{30' \times 12} = 0.00014 < 0.001$

$\simeq 0.0001$

O.K. for confined and unconfined walls.

N-S Direction, between EL. 20'-0" and 50'-0"

0.125 $\gamma = \frac{0.1252''}{12 \times 30} = 0.0003478 < 0.001$

> 0.0001

O.K for confined walls

N.G. for unconfined walls.

N-S Direction, between EL. 50'-0" and 80'-0"

0.134 $\gamma = \frac{0.1339''}{12 \times 30} = 0.0003719 < 0.001$

> 0.0001

O.K for confined walls

N.G. for unconfined walls.

check all unconfined walls in N-S direction.

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NAME OF COMPANY C P & L UNIT/S 1, 2

SUBJECT REACTOR BLDG - MASONRY WALLS

IN-PLANE SHEAR STRAIN CRITERIA — cont.

E-W Direction, between EL.(-)17'-0" and 20'-0"

$$\gamma = \frac{\Delta}{H} = \frac{0.05464''}{12 \times 37} = 0.00012 \quad < 0.001, \quad \geq 0.0001$$

O.K for confined & unconfined walls.

N-S Direction, between EL.(-)17'-0" and 20'-0"

$$\gamma = \frac{0.09926}{12 \times 37} = 0.0002236 \quad < 0.001, \quad > 0.0001$$

O.K for confined walls
N.G. for unconfined walls.

RAI 13a Provide detailed drawings and current status of proposed repairs.

RESPONSE

CP&L has designed some fixes to be implemented in 1983. The design for remaining fixes will be completed in 1984. Design drawings will be submitted as fixes are implemented.

RAI 13b - Also, provide sample calculations to illustrate that the modified walls will be qualified under the working stress design condition.

RESPONSE

Three sets of sample calculations, as listed below, show the proposed modifications to correct the wall deficiencies.

WALL NO.	LOCATION	NUMBER OF PAGES	
4a	Diesel Generator Bldg	Pgs. 26 to 43B	(20)
6a	" " "	Pgs. 48 to 54A	(8)
5a	" " "	Pgs. 46 to 47A	(3)

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DISCIPLINE

GENERAL COMPUTATION SHEET

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CALC. SET NO.		
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J.O. 9527 0??		
REV	COMP. BY	CHK'D BY
0	GR DATE 11-6-80	R.P DATE 11-21-80
	DATE	DATE

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG. - BLOCK WALLS

Wall 20 El. 50'-0"

8" ll sol. Ref. Dwg.: 9527-F-1926, 1927
reinforced fin'd wall
with 2-#7 @ 4'-0" crs. - Seismic load plus air pressure of 1.8 ins of water

$f'_m = 1820$

$I_g = 363.40 in^4/ft.$ (See sht. 48/RPU)
$I'_{cr} = 46\ in^4/ft.$ (See sht. 12/DPLE)
$w = 42\ lbs/ft^2 \rightarrow 3.5\ lb/in.$
$l = 17'0" \rightarrow 204\ ins.$
$g = 386\ in/sec^2$
$E = 1,820,000\ lb/in^2$

$$P = \frac{2}{\pi}\sqrt{\frac{wl^4}{gEI}}$$
$$= \frac{2}{\pi}\sqrt{\frac{3.5 \times 204^4}{386 \times 1820000 \times 363.4}}$$
$$= 0.098\ secs.$$

PERIOD SPREAD: 0.11 sec. → 0.09 secs PERIOD SPREAD: 0.098 → 0.078

ACCN. RESPONSE:

		OBE (4%)	DBE (7%)
El. 50'0"	E-W	0.26g	0.47g
El. 63'0"	E-W	0.42g	0.70g

OBE LOADS

$$\text{DESIGN ACCN.} = 1.05 \times \frac{0.26+0.42}{2} g = 0.36g$$
$$\text{AIR PRESSURE} = \frac{1.8}{12} \times 62.5 \approx 10\ psf$$

$W = 0.36 \times 42 \times 17 + 10 \times 17 = 427\ lbs.$

$M_o = \frac{427 \times 17}{8} = 907\ lb.ft. > M_{cr}$ $M_{cr} = \frac{93 \times 363.4}{3.81 \times 12} = 739\ lb.ft$

∴ SECTION IS CRACKED.

CALCULATE MA BASED ON MAX. ACCN. :- $M_a = (1.60 \times 42 \times 17 + 10 \times 17) \frac{17}{8} = 2789\ lb.ft.$

ACC: $1.05 \times \frac{1.38+1.66}{2} = 1.60g$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left\{1 - \left(\frac{M_{cr}}{M_a}\right)^3\right\} I_{cr} -$$
$$= \left(\frac{739}{2789}\right)^3 \times 363.4 + \left\{1 - \left(\frac{739}{2789}\right)^3\right\} \times 46$$
$$= 6.76 + 45.1 = 51.9\ in^4$$

REVISED SPREAD, $P = 0.098 \times \sqrt{\frac{363.4}{51.9}}$
$= 0.259\ secs.$

PERIOD SPREAD: 0.29 sec → 0.24 secs

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(DISCIPLINE)

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG - BLOCK WALLS

CALC. SET NO.		
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0	CR DATE 11-6-80	R.P. DATE 11-21-80
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Wall 4a (Contd.)

REVISED ACCN. RESPONSE :

	OBE (4%)	DBE (7%)
El. 50'0" E-W	1.38	1.84
El. 67'0" E-W	1.66	2.20

OBE LOADS

DESIGN ACCN. = $1.05 \times \frac{1.38+1.66}{2}\, g = 1.60g$

AIR PRESSURE = 10 psf

$W = 1.60 \times 43 \times 17 + 10 \times 17 = 1312$ lbs.

$M_a = 1312 \times \frac{17}{8} = 2788$ lb.ft. }

$V = 1312/2 = 656$ lbs. }

$f_m = \frac{2788 \times 12 \times 1.26}{46} = 917$ psi > 600 psi N.G.

$f_s = \frac{2788 \times 12 \times 0.89 \times 15.9}{46} = 46,141$ psi > 24,000 N.G.

$v = \frac{656 \times 4}{7.625 \times 5.25} = 66$ psi > 47 psi N.G.

$u = \frac{656 \times 4}{2.75 \times 0.92 \times 5.25} = 198$ psi > 140 psi N.G.

WALL OVERSTRESSED, USE OPERABILITY CRITERIA

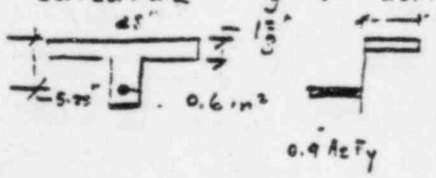
USE DBE LOADS

DESIGN ACCN. : $1.05 \times \frac{1.84+2.20}{2} g = 2.12g$

$W = 2.12 \times 43 \times 17 + 10 \times 17 = 1684$ lbs.

$M_a = 1684 \times \frac{17}{8} = 3579$ lb.ft.

- Calculate M_y :- 4 ft. width



$A_s F_y = 0.85 f'_m a\, b.$

$0.6 \times 60,000 = 0.85 \times 1820 \times a \times 48$

$a = \frac{0.6 \times 60000}{0.85 \times 1820 \times 48} = 0.48$ in.

$\therefore M_y = 0.9 A_s f_y \left(d - \frac{a}{2}\right) = 0.9 \times 0.6 \times 60,000 \left(5.25 - \frac{0.48}{2}\right)$

$= 13527 \div 4 = 3382$ lb ft / ft. width

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united engineers & constructors inc.

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	DATE	DATE

NAME OF COMPANY CP&L — BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG — BLOCK WALLS

Wall 40 (Contd)

– $\Delta_y = \frac{5}{48} \frac{M\ell^2}{EI}$.

M_y = 3382 lb ft x12 = 40584 lb. ins.
I = 46 in⁴ (USE FULLY CRACKED SECTION)
E = 1,820,000 lb/in²
ℓ = 17'x12 = 204 in.

$$\Delta_y = \frac{5}{48} \times \frac{40584 \times 204^2}{1,820,000 \times 46}$$

= 2.1 ins

– Calculate defln assuming elastic behaviour

$$\Delta_e = 2.1 \times \frac{3570}{3382} = 2.2''$$

– We have $\frac{\Delta_y}{\Delta_e} = \frac{1}{\sqrt{2\mu-1}}$ where $\mu = \frac{\Delta}{\Delta_y}$

$$\therefore \frac{1}{2\mu-1} = \left(\frac{2.1}{2.2}\right)^2 = 0.91$$

$$\mu = \frac{1+1.1}{2} = 1.05$$

$$\therefore \Delta = 1.05 \times 2.1 = 2.2''$$

– $\Delta < 3\Delta_y$ ∴ OPERABILITY CRITERIA SATISFIED.

VERIFY THAT DEFL^N OF 2.2" WILL NOT AFFECT SAFETY-RELATED SYSTEMS ATTACHED TO OR ADJACENT TO WALL; PROVIDE LTF.

IN-PLANE CRITERIA : WALL RUNNING N-S UNCONFINED OK

ACTION: SEE PM 80-232, MWR-1

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GENERAL COMPUTATION SHEET

united engineers & constructors inc.

NAME OF COMPANY: CP&L — SCSP   UNIT/S ____

SUBJECT: DIESEL GEN. BLDG - BLOCK WALLS

CALC. SET NO.			REV	COMP BY	CHK'D BY
PRELIM			0	C.K.	R.P.
FINAL	X			DATE ·2-80	DATE 12-21-80
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SHEET 29 OF 95				DATE	DATE
J.O. 9527.022					

Wall 4a (Contd.)

TOP LATERAL SUPPORT FOR WALL 4a IS INTERMITTENT CONSIDER WALL BEHAVING AS A CANTILEVER

$I_g = 363.4$
$w = 3.5$ lb/in.
$l = 206$ in
$g = 386$ in/sec²
$E = 1,820,000$ lb/in²

PERIOD OF WALL, $P = \frac{2\pi}{3.52}\sqrt{\frac{wl^4}{gEI}}$

$= \frac{2\pi}{3.52}\sqrt{\frac{3.5 \times 206^4}{386 \times 1.82 \times 10^6 \times 363.4}}$

$= 0.275$ Sec.

PERIOD SPREAD: 0.31 sec ⟶ 0.25 sec.

ACCN. RESPONSE :-

		DBE (7%)
El. 286'	E-W	1.84 g

DESIGN ACCN. = 1.05 × 1.84g = 1.93g ; AIR PRESSURE = 10 psf.

∴ OVERTURNING MOMENT $= \frac{33}{140} \times 1.93 \times 42 \times 17^2 + \frac{10 \times 17^2}{2} = 6,967$ lb. ft.

RESISTING MOMENTS :—

(1) DUE TO TENSILE STRENGTH @ BASE

$M = f_r \frac{I}{y} = \frac{93 \times 363.4}{3.81 \times 12} = 739$ lb. ft

(2) DUE TO SELF WT. OF WALL

$M = \left(1 - \frac{2}{3} \times 1.84\right) \times 17 \times 42 \times \frac{3.81}{12} = -51$ lb. ft

∴ TOTAL RESISTING MOMENT = 739 − 51 = 688 lb. ft
<< 6,967 lb. ft.

∴ WITHOUT LATERAL SUPPORT AT THE TOP, WALL WOULD TRY TO OVERTURN DURING AN EARTHQUAKE [illegible]

[illegible] = 17×12 − [illegible] = 2?3.25
[illegible] −0.75 [illegible] = 2??.34
[illegible] FOUL [illegible]
WALL WILL NOT [illegible]

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GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

NAME OF COMPANY	CPSL — BTGP	UNIT/S	2
SUBJECT	Generator Bldg – Block Walls		

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM.		0	SF	RPU
FINAL	X		DATE	DATE 12/1/80
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J.O. 9527.020				

WALL 6a (CONT)

WALL ROTATED, SINCE JOINT FILLER WOULD HAVE HARDENED WALL WILL BIND.

$f_c' = 0.25 f_m' = 0.85 \times 1820 = 1547$ psi

$f_t = 6\sqrt{f_m'} = 6 \times 42.67 = 256$ psi

$a = 1''$

$t = 7.625''$

$k = \frac{3.125}{16} = 0.2$

$f_t = \{ f_c' \cdot a + f_t (t - ak) \cos 45° \} k$

$= 0.2 \{ 1547 \times 1 + 256 (7.625 - 0.2) \times 0.707 \}$ per inch width

$= 578$ lb/in. width

$\therefore H/ft = \frac{578 \times 12}{1.5} = 4624$ lb.

For gapped arching

$H = \frac{P\ell^2}{4t}$

$\therefore P = \frac{4Ht}{\ell^2}$

$= \frac{4 \times 4624 \times 0.635}{17^2} = 41$ lb/ft. CAPACITY.

DETERMINE REVISED ACCN. RESPONSE:

$u = \frac{HL}{2AE}$

$= \frac{4624 \times 17 \times 12}{2 \times 42 \times 1.82 \times 10^6} = 0.00617$

$\therefore \Delta = \frac{Lu}{2t}$

$= \frac{17 \times 12 \times 0.00617}{2 \times 7.625}$

$= 0.083''$ $\left(< \frac{7.625}{2} = 2.5'' \therefore \text{DEFL}^{N} \text{ O.K.} \right)$

Period $= 2\pi \sqrt{\frac{\Delta}{g}} = 2\pi \sqrt{\frac{0.083}{386}}$

$= 0.092$ secs.

Period Spread: 0.10 secs → 0.084 secs.

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(DISCIPLINE)

united engineers & constructors inc

NAME OF COMPANY: [illegible] UNIT/S 1&2

SUBJECT: [illegible]

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM		0	[illegible] DATE 11-25-80	RPU DATE 12/1/80
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J.O. 9527.028				

Wall 4a (Cont'd.)

ACC'N RESPONSE :

	DBE (10%)
El. 50'-0" EW	0.43g
El. 68'-0" EW	0.62g

∴ DESIGN ACCN. = 1.05 × $\frac{0.43+0.62}{2}$ = 0.55 g.

∴ APPLIED LOAD = 0.55 × 42 + 10 (AIR PRESSURE) = 33 lb/ft² < 41 lb/ft² CAPACITY

∴ WALL HAS STRUCTURAL INTEGRITY.

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GENERAL COMPUTATION SHEET

united engineers & constructors inc

CALC. SET NO.		REV	COMP. BY	CHK'D BY
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(DISCIPLINE)

NAME OF COMPANY CP&L - BSEP UNIT/S

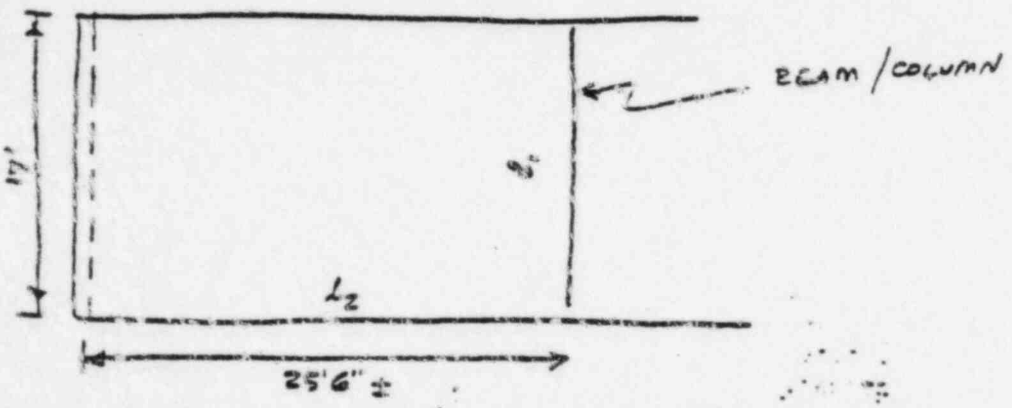
SUBJECT [illegible]

Wall 4a (Contd.)

DESIGN OF SUITABLE FIX FOR Wall 4a

ADD CHANNELS ON BOTH SIDES OF WALL TO CREATE ADDITIONAL SUPPORTS THAT WOULD MAKE THE WALL SPAN TWO-WAY.

Ref. Dwg. - 9527-F-4093 For Equipment



BEAM SHOULD HAVE SUFFICIENT STIFFNESS RELATED TO WALL TO MAKE THE WALL SPAN HORIZONTALLY AS WELL.

∴ MAXIMUM HORIZONTAL SPACING = $17 \times 1.5 = 25'6"$

MINIMUM STIFFNESS REQD. BY BEAM IS GIVEN BY (USING ACI 318-71, SECTION 13.3.4.4)

$$\alpha_1 l_2 / l_1 = 1 \text{ , WHERE } \alpha_1 = \frac{I_b}{I_s}$$

WHERE I_b .. MI of bm.
I_s M.I. of slab l_2 wide

MOMENT OF INERTIA OF WALL / FT. WIDTH = 363.4 in^4

$$l_2 = 25'6", \quad l_1 = 17'$$

$$\therefore \quad \alpha_1 \cdot \frac{25.5}{17} = 1$$

$$\text{i.e.} \quad \frac{I_b}{25.5 \times 363.4} \times \frac{25.5}{17} = 1$$

$$\text{i.e.} \quad I_b = 1 \times 17 \times 363.4 = \underline{\underline{6178 \text{ in}^4}}$$

5007 REV 7/79

(DISCIPLINE)

GENERAL COMPUTATION SHEET

united engineers & constructors inc

NAME OF COMPANY CPXL UNIT/S

SUBJECT [illegible] WALL

CALC. SET NO.		REV	COMP. BY	CHK'D. BY
PRELIM		0	CK	RPU
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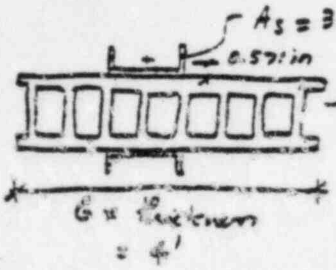
SHEET 33 OF 95			–	–
J.O. 9527.???			DATE	DATE

Wall 4a (Contd)

CALCULATE MOMENT OF INERTIA OF A PAIR OF CHANNELS BOLTED TOGETHER. TRY C9×11.5. ALSO ASSUME 6× THICKNESS OF WALL IS EFFECTIVE AS PART OF BEAM



$I = 363.4 in^4/ft$. $m = 15.9$

$I_{total} = 4 \times 363.4 + 15.9 \times [1.32 + 3.38 \times (4.38)^2] \times 2$

$= 1454 + 2104 = 3558 in^4 < 6178 in^4$ N.G.

TRY USING A SINGLE COLUMN ON ONE SIDE

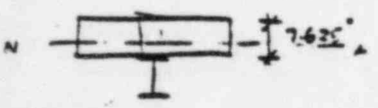
(1) WITHOUT USING INTERCONNECTION

$I_{cw} = \frac{6178}{15.9} = 389 in^4$ → SECTION TOO HEAVY

(2) WITH INTERCONNECTION TRY W 8×31

CALCULATE EQ. AREA – $\frac{Ah^2}{12} = 4 \times 363.4$

$\therefore A = \frac{12 \times 4 \times 363.4}{7.625^2} = 300 in^2$



W8×31
$I_{xx} = 110 in^4$
$A = 9.13 in^2$

$300 \times \frac{7.625}{2} + 9.13 \times 11.625 \times 15.9 = (300 + 15.9 \times 9.13)\bar{x}$

$\bar{x} = \frac{1144 + 1688}{445.2} = 6.36"$

$I_{comp} = 4 \times 363.4 + 300 \times 2.55^2 + 15.9 \times 110 + 15.9 \times 9.13 \times 5.265^2$

$= 1454 + 1951 + 1749 + 4024 = 9178 in^4 > 6178 in^4$

Ok

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JO 9527.035				

NAME OF COMPANY CP&L UNIT/S

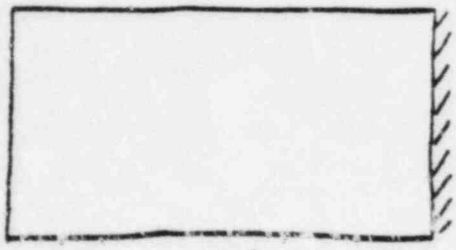
SUBJECT DIESEL GEN. BLDG. – BLOCK WALL

Wall 4a (Contd.)

Derived from "Structural Handbook" – By Gaylord & Gaylord Pg. 7-22

DETERMINE WALL FREQUENCY

17'



25'6"

$$\omega_n = \frac{15.57}{a^2}\sqrt{\frac{D}{\bar{m}}}$$

where $D = \frac{EI}{1-\mu^2}$, I = moment of inertia / unit width

$\bar{m}$ = mass / unit area

a = short length

$$I_{av} = \frac{363.4 + 341}{2} = 352.2 \rightarrow 29.4\ in^4/in$$

$$D = \frac{1.82\times10^6 \times 29.4}{1-0.15^2} = 54.74\times10^6$$

$$\bar{m} = \frac{42}{144\times386} = 0.000756$$

$$a = 17\times12 = 204\ in$$

$$\omega_n = \frac{15.57}{204^2}\times\sqrt{\frac{54.74\times10^6}{0.000756}} = 100.7\ rad/sec$$

$$\text{Period} = \frac{2\pi}{\omega_n} = \frac{2\pi}{100.7} = 0.062\ secs.$$

Period spread : 0.069 secs → 0.057 secs. Frequency spread: [illegible] — [illegible]

ACCN. RESPONSE :–

		OBE (4%)	DBE (7%)
El. 50'-0"	E–W	0.20 g	0.40 g
El. 62'-0"	E–W	0.26 g	0.49 g

OBE LOADS

$$\text{DESIGN ACCN.} = 1.05\times\frac{0.2+0.26}{2}g = 0.24\ g.$$

$$\therefore w = 0.24\times42 + \underbrace{\frac{1.8\times62.5}{12}}_{\text{air pressure}} = 19.5\ psf$$

ASSUME ONE-WAY BEHAVIOR FOR STRESSES

$$\therefore M_a = \frac{19.5\times17^2}{8} = 704.4\ lb.ft < M_{cr} - 739\ lb.ft$$

∴ UNCRACKED

$$V = \frac{19.5\times17}{2} = 166\ lbs.$$

FORM 5007 REV 7/79

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VOID				
SHEET 35 OF 95			DATE	DATE
J.O. 9527.028				

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG. - BLOCK WALLS

Wall 4a. (Contd)

			b/c
$f_m = \frac{704.4 \times 12 \times 1.26}{46} =$	232 psi < 600 psi		0.39
$f_s = \frac{704.4 \times 12 \times 3.99 \times 15.9}{46} =$	11,658 psi < 24,000 psi		0.49
$v = \frac{166 \times 4}{7.625 \times 5.25} =$	17 psi < 47 psi		0.36
$u = \frac{166 \times 4}{2.75 \times 0.92 \times 5.25} =$	50 psi < 140 psi		0.36

OK

DBE LOADS

DESIGN ACCN. = $1.05 \times \frac{0.4 + 0.49}{2} g = 0.47g$

RATIO $\frac{DBE}{OBE} = \frac{0.47 \times 42 + 9.4}{19.5} = 1.5$

$M_a = 1.5 \times 704.4 = 1057$ lb.ft > M_{cr} (739 lb-ft)

∴ CRACKED

HENCE REVISE PERIOD ASSUMING CRACKING.
CONVERGENCE WAS OBTAINED FOR THE FOLL. MAX ACCN. RESPONSE

			DBE (7%)
PERIOD, 0.15 Sec	El. 50'-0"	E-W	0.67g
	El. 68'-0"	E-W	0.87g

DESIGN ACCN. = $1.05 \times \frac{0.67 + 0.87}{2} = 0.81g$.

∴ $w = 0.81 \times 42 + \underbrace{\frac{1.8}{12} \times 62.5}_{\text{AIR PRESS.}} = 43.4$ psf

RATIO $\frac{DBE}{OBE} = \frac{43.4}{19.5} = 2.22$

$M_a = 2.22 \times 704.4 = 1564$ lb.ft

CALCULATE I_e

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left\{1 - \left(\frac{M_{cr}}{M_a}\right)^3\right\} I_{cr}$$

$$= \left(\frac{739}{1564}\right)^3 \times 363.4 + \left\{1 - \left(\frac{739}{1564}\right)^3\right\} \times 46$$

$$= 38.3 + 41.1 = 79.4 \text{ in}^4$$

$$D_{REVISED} = D \times \frac{I_e}{I} = \frac{79.4}{363.4} \times 54.74 \times 10^6 = 11.96 \times 10^6$$

Wall 4a (Contd.)

$$\omega_n = \frac{15.57}{a^2}\sqrt{\frac{D_{Rew}}{\bar{m}}}$$

$$= \frac{15.57}{204^2} \times \sqrt{\frac{11.96\times10^6}{0.000756}}$$

$$= 47 \text{ rad/s}$$

∴ Period = $\frac{2\pi}{\omega_n} = \frac{2\pi}{47} = 0.134$ secs

Period spread : 0.15 secs → 0.122 sec.

→ O.K.

∴ M_a = 1566 lb.ft
$V = 43.4 \times \frac{17}{2} = 369$ lb. } RATIO = 2.23

				L/C
f_m = 2.23 × 232	=	517 psi	< 1547 psi	0.33
f_s = 2.23 × 11,658	=	26,000 psi	< 54,000 psi	0.48
v = 2.23 × 17	=	38 psi	< 73 psi	0.52
u = 2.23 × 50	=	112 psi	< 186 psi	0.60

} OK

∴ WALL O.K. WITH W8×31 COL. @ 25'6" c/c MINIMUM TO MAKE THE WALL BEHAVE TWO-WAY

FORM 5007 REV 7/79

(DISCIPLINE)

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J.O. 9-27.088				

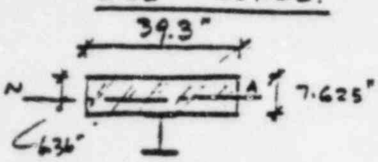
NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG - BLOC. WALLS

Wall 4a (Contd.)

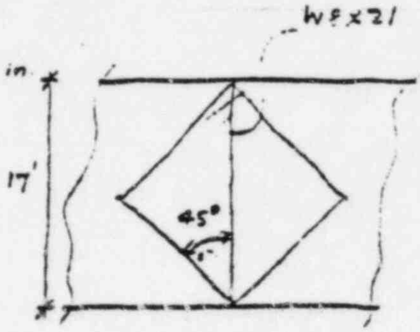
CHECK STRENGTH OF W8x31 COMP. BEAM

OBE LOADS.



EQ. WIDTH = $\frac{300}{7.625}$ = 39.3 in.

I_{comp} = 9178 in⁴



LOAD CARRIED BY BEAM

Case I EQ. ACTS EAST

$W = (12\times12)\times\{0.24\times42\times9.4\}$
= 2805 lb.

$M_{max} = \frac{WL}{6} = \frac{2805\times17}{6}$ = 7948 lb. ft.

V_{max} = 1403 lb

$f_m = \frac{7948\times12\times6.36}{9178}$ = 66 psi < 600 psi OK

$f_s = \frac{7948\times12\times9.265\times15.9}{9178}$ = 1531 psi < 0.6×36^k = 21,600 psi OK

CHECK INTERCONNECTION BETWEEN MASONRY & STEEL
PROVIDE 2-½"φ THROUGH BOLTS @ 16" c/c.

$\bar{y}$ = -3.81 + 6.36
= 2.55

MAX. BOLT SHEAR (PER BOLT) = $\frac{VA\bar{y}}{I}\times16\times\frac{1}{2}$

$= \frac{1403\times300\times2.55}{9178}\times16\times\frac{1}{2}$

= 936 lbs. < 4400 lbs. FOR A307 BOLTS
O.K.

CHECK BEARING BETWEEN MASONRY & BOLTS

BEARING STRESS = $\frac{936}{1.375\times0.75}$ = 907 psi. > $.25f'_m$
= .25×3220
= 805 psi
N.G.

Wall 4a (Contd.)

CONSIDER FRICTION BETWEEN BLOCK WALL & STEEL COLUMN DUE TO BOLT TENSION TO REDUCE BEARING STRESS

SAY TENSION IN EACH BOLT = 1^K CONSERVATIVELY

$\therefore$ NORMAL STRESS = $\frac{1 \times 2 \times 1000}{16 \times 8} = 16$ psi

REDUCTION IN "SHEAR STRESS" = $\underset{\text{COEFFT. OF FRICTION}}{0.3} \times 16 \approx 5$ psi

$\therefore$ REVISED BOLT SHEAR PER BOLT = $\left\{\frac{1403 \times 300 \times 2.55}{9178 \times 8} - 5\right\} \times 16 \times \frac{1}{2} \times 8$

= 616 lbs. < 4400 lbs. O.K.

REVISED BEARING BETWEEN MASONRY & BOLTS, f_m' = 3200 (wall 0.25 f_m')

BEARING STRESS = $\frac{616}{1.375 \times 0.75}$ = 597 psi < 805 psi

O.K.

Case II EQ. ACTS WEST

MIN. AIR PRESSURE IS ZERO

$\therefore$ W = 12×12 × 0.24×42

= 1452 lbs.

V = 726 lbs.

$M_{max} = \frac{Wl}{6} = 1452 \times \frac{17}{6} = 4114$ lb.ft.

$f_m = \frac{4114 \times 12 \times 6.36}{9178} = 34$ psi > 25 psi N.G.

$f_s = \frac{4114 \times 12 \times 9.265 \times 15.9}{9178} = 792$ psi << 21,600 psi OK (Eq. 20.1)

DBE LOADS

Case I EQ. ACTS EAST

W = (12×12) × {0.47×42 + 9.4}

= 4196 lbs.

$M_{max} = \frac{Wl}{6} = 4196 \times \frac{17}{6} = 11,889$ lb ft.

V_{max} = 4196/2 = 2098 lb.

$f_m = \frac{11,889 \times 12 \times 6.36}{9178} \approx 99$ psi < 1547 psi

$f_s = \frac{11,889 \times 12 \times 9.265 \times 15.9}{9178} = 2290$ psi < 32,400 psi (0.9×36000 =)

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J.O. 9527 OEP					

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG - BLOCK WALLS

Wall 4a (G.H)

INTERCONNECTION BETWEEN MASONRY & STEEL,

BOLT SHEAR = $\left\{ \frac{2092 \times 300 \times 2.55}{9.73 \times 2} - 5 \right\} \times 16 \times 8 \times \frac{1}{2}$

= 1079 lbs. < 4400 lbs. OK f'_m 3000 masonry unit

BEARING STRESS = $\frac{1079}{1.375 \times 0.75}$ = 1046 psi < 2000 psi ($0.67 f'_m$) O.K.

Case II E.Q. ACTS WEST

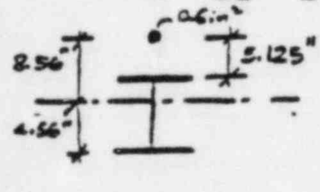
$W = 0.47 \times 42 \times 12 \times 12 = 2843$ lbs.

$M_{max} = \frac{Wl}{6} = 2843 \times \frac{17}{6} = 8055$ lb. ft

$f_m = \frac{8055 \times 12 \times 6.36}{9178}$ = 67 psi > 42 psi N.G.

$\frac{f_s}{n} = \frac{8055 \times 12 \times 9.265 \times 15.9}{9178}$ = 1551 psi << 1.5 × 21,600 psi OK

IN THE ABOVE CALCULATION PRESENCE OF REINFORCING STEEL HAS BEEN IGNORED. SUPPOSE WE CONSIDER ALL MASONRY TO BE CRACKED WE GET THE FOLL. COMP. SECTION:



$9.13 \times 4 + 13.125 \times 0.6 = (9.13 + 0.6)\bar{x}$

$\bar{x} = \frac{44.395}{9.73} = 4.56$ in.

$I_{n.a.} + A\bar{x}^2 = 110 + 9.13 \times 4^2 + 0.6 \times 13.125^2$

i.e. $I_{n.a} + 9.73 \times 4.56^2 = 110 + 146.1 + 103.4$

$I_{n.a.} = 359.5 - 202.3 = 157.2$ in⁴

OBE LOADS:

$f_{st} = \frac{4114 \times 12 \times 8.56}{157.2}$ = 2688 psi < 24,000 psi O.K.

$f_{sc} = \frac{4114 \times 12 \times 4.56}{157.2}$ = 1432 psi << 21,600 psi OK

SHEAR IN BOLTS NEGLECTING FRICTION

$= \frac{VA\bar{y}}{I} \times 16 \times \frac{1}{2}$

$= \frac{726 \times 0.6 \times 8.56}{157.2} \times 16 \times \frac{1}{2}$ = 190 lbs.

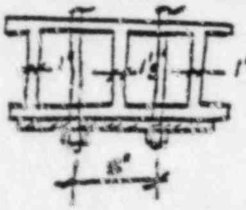
O.K. BY INSPECTION

DBE LOADS: O.K. BY INSPECTION

Wall 4a (Contd.)

CHECK COMPRESSIVE STRESS IN WEBS OF CONC. BLOCK DUE TO BOLT TENSION

- ONLY ONE BLOCK WOULD BE AFFECTED
- PROVIDE ½" THICK PLATE x 1'-2" Lg. TO DISTRIBUTE THE COMP. LOAD ON ALL THE WEBS



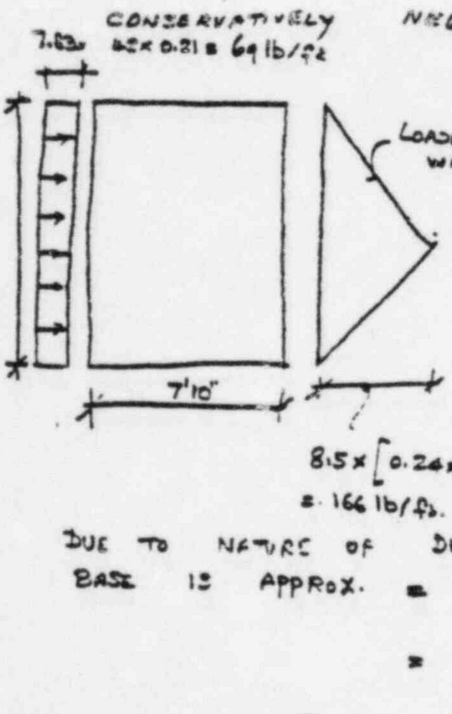
TOTAL COMP = 2 x 1k = 2000 lbs.

TOTAL COMP. AREA = 2 x (1 + 1½ + 1) = 25 in²

Comp STRESS = 2000 / 25 = 80 psi < 708 psi O.K.

ALLOWABLE max. = 0.22 f_m = 0.22 x 3220 = 708 psi (for block mat.)

CHECK ADEQUACY OF WALLS 4b,c TO CARRY LOADS IN PLANE



8.5 x [0.26 x 42 + 9.4] = 166 lb/ft.

By INSPECTION ACCN. RESPONSE:

	OBE (4%)	DBE (7%)
El. 58'-0" EW	0.20g	0.38g
El. 68'-0" EW	0.22g	0.43g

DESIGN ACC = 0.21g.

∴ TOTAL LOAD ON WALL = ½ x 166 x 17 + 69 x 17

= 1411 + 1173 = 2584 lb.

DUE TO NATURE OF DEFLECTED SHAPE OF CANTILEVER, MOMENT AT BASE IS APPROX. = $\frac{33}{140}$ Wℓ

= $\frac{33 \times 2584 \times 17}{140}$ = 10,355 lb ft.

$I = \frac{1}{12} \times 1.375 \times 94^3 \times 2 = 190,342\ in^4$

$A = 2 \times 1.375 \times 94 = 259\ in^2$

SELF WT OF WALL = 17 x 7.83 x 42 = 5591 lb.

∴ $f_a = \left[1 - \frac{2}{3} \times 0.21\right] \times \frac{5591}{259} = 18.5$ psi

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J.O. 9527.OPS					

NAME OF COMPANY: SP&L - BSEP UNIT/S

SUBJECT: DIESEL GEN. BLDG. - BLOCK WALLS

Wall 40 (contd.)

$$f_m = \frac{10,355 \times 12 \times 47}{190,342}$$

$$= 30.7 \text{ psi}$$

$\therefore$ $f_{comp} = 30.7 + 24.6 = 55 \text{ psi} < 600 \text{ psi}$ OK

$f_{tension} = -30.7 + 18.5 = 12 \text{ psi} < 25 \text{ psi}$ OK.

FOR DBE LOADS.

DESIGN ACCN. = $\frac{0.38 + 0.43}{2}$ = 0.41 g.

TOTAL LOAD ON WALL = $\frac{1}{2} \times 248 \times 17 + 135 \times 17$ = 2108 + 2295 = 4403 lbs.

$M_{base} = \frac{33}{140} W\ell = \frac{33}{140} \times 4403 \times 17 = 17,643$ lb.ft

$f_a = \left[1 - \frac{2}{3} \times 0.41\right] \times \frac{5591}{259} = 15.7$ psi or

$\left[1 + \frac{2}{3} \times 0.41\right] \times \frac{5591}{259} = 27.5$ psi

$$f_m = \frac{17643 \times 12 \times 47}{190,342}$$

$$= 52.2 \text{ psi}$$

$f_{comp} = 52.2 + 27.5 \cong 80 \text{ psi} < 1527 \text{ psi}$ OK

$f_{tens} = 52.2 - 15.7 = 37 \text{ psi} < 42 \text{ psi}$ OK.

$\therefore$ USE W8x31 @ 25'6" MAX. SPACING WITH 2-3/4"ϕ BOLTS @ 16" CTS.

CHECK HORIZONTAL MOMENT IN WALL:

ALTHOUGH FOR STRENGTH ANALYSIS ONLY VERTICAL SPAN IS CONSIDERED, CONSERVATIVELY, FOR INTEGRITY OF WALLS FOR POSTULATED 2-WAY BEHAVIOUR HORIZ. FLEXURAL STRESSES MUST BE WITHIN ALLOWABLE

OBE $M_H = 0.031 \times 19.5 \times 25.5^2 = 393$ lb.ft/ft. (USE 2 WAY SLAB COEFFTS ACI 318-63)

$\therefore f_{tension} = \frac{393 \times 12 \times 3.81}{341} = 53 \text{ psi} \approx 50 \text{ psi}$ ALLOWABLE OK

DBE $f_{tension} = 2.23 \times 53 = 118 \text{ psi} > 82 \text{ psi}$ N.G.

HENCE CONSIDER DUR-O-WALL REINFORCING IN WALL.

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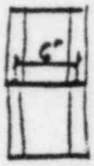
Wall 6c (contd)

CALCULATE CAPACITY OF 8" WALL WITH DUR-O-WALL (EXTRA HEAVY) REINFORCEMENT SPANNING HORIZONTALLY:



$d \cong 7.625 - \left(\frac{7.625-6}{2}\right) = 6.8''$

EFF. $A_s = \frac{.069}{2} = 0.0345\ in^2/8''$ width $\rightarrow$ $0.052\ in^2/ft$ width.

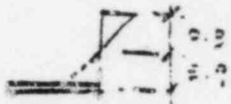
$p = \frac{A_s}{bd} = \frac{0.052}{12\times6.8} = 0.000637$ $n = 15.9$

$np = 15.9\times.000637 = 0.01$

$k = \sqrt{(np)^2+2np} - np = \sqrt{0.01^2+2\times0.01} - 0.01 = 0.131$

$kd = 0.131\times6.8 \cong 0.9'' < 1.375''$ O.K.

$j = 1 - \frac{0.131}{3} = 0.96$



$\frac{f_s/n}{5.9} = \frac{f_m}{0.9}$

$\frac{30,000/15.9}{5.9} = \frac{f_m}{0.9}$

$\therefore f_m = 0.9\times\frac{30\,000}{15.9}\times\frac{1}{5.9} = 288\ psi < 600\ psi$ ∴ STEEL GOVERNS

$\therefore$ M.R. $= A_s f_y \left(6.8 - \frac{0.9}{3}\right) = 0.0345\times30,000\times\frac{6.5}{12}$

$= 561$ lb.ft. FOR OBE LOADS > 392 lb.ft

FOR DBE STRESSES:

$F_y = 75,000$ psi

$\frac{67,500/15.9}{5.9} = \frac{f_m}{0.9}$

$f_m = 0.9\times\frac{67,500}{15.9}\times\frac{1}{5.9} = 648\ psi < 1547\ psi$ ∴ STEEL GOVERNS

M.R. $= 0.0345\times67500\times\frac{6.5}{12} = 1261$ lb.ft. $< 2.23\times393 = 876$ lb.ft O.K.

∴ HORIZONTAL MOMENT IN WALL LESS THAN CAPACITY OF WALL WITH DUR-O-WALL REINFORCEMENT. O.K.

ACTION: SEE PM 80-232, MWR-9

FORM 5007 REV 7/79

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J.O. 9227.055				

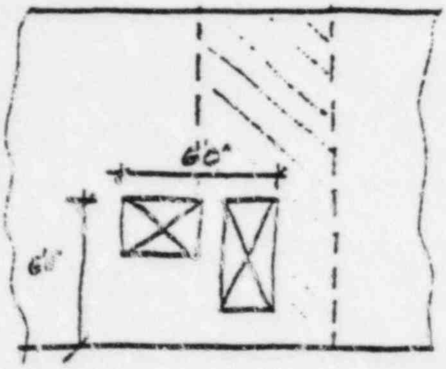
NAME OF COMPANY: CP&L - BSEP UNIT/S

SUBJECT: DIESEL GEN. BLDG. - BLOCK WALLS

Wall 4a (Cont'd)

EFFECT OF OPENINGS

MOST CRITICAL OPENING OCCURS BETWEEN (B3) & (E1)

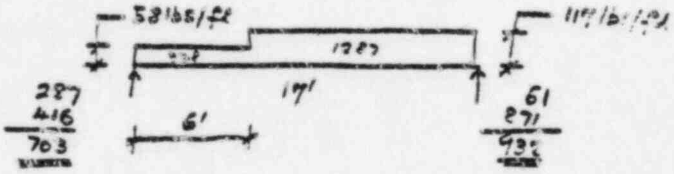


EFF. WIDTH = $\frac{SPAN}{5} = \frac{17'0"}{6'} \approx 3'0"$

OBE LOADS

$w_1 = 3\{0.24 \times 42 + \frac{1.8}{12} \times 62.5\} = 58 \text{ lbs/ft}$

$w_2 = 6\{0.24 \times 42 + \frac{1.8}{12} \times 62.5\} = 117 \text{ lbs/ft}$



$M_{crit} = 703 \times 6 - 58 \times \frac{6^2}{2} = 4218 - 1044 = 3174 \text{ lb.ft}$

$\therefore M/ft = 1058 \text{ lb.ft/ft}$
$V/ft = 234 \text{ lb.}$

				L/C
$f_m = \frac{1058 \times 12 \times 1.26}{46}$	=	348 psi < 600 psi		0.58
$f_s = \frac{1058 \times 12 \times 3.99 \times 15.9}{46}$	=	17,510 psi < 24,000 psi		0.73
$v = \frac{234 \times 4}{7.625 \times 5.25}$	=	23 psi < 47 psi		0.49
$u = \frac{234 \times 4}{2.75 \times 0.92 \times 5.25}$	=	71 psi < 140 psi		0.51

DBE LOADS

RATIO $\frac{DBE}{OBE} = 2.23$

				L/C
$f_m = 348 \times 2.23$	=	776 psi	< 1547 psi	0.5
$f_s = 17,510 \times 2.23$	=	39,047 psi	< 54,000 psi	0.72
$v = 23 \times 2.23$	=	51 psi	< 73 psi	0.70
$u = 71 \times 2.23$	=	158 psi	< 186 psi	0.85

$\therefore$ WALL O.K.

IN PLANE SHEAR STRAIN CRITERIA: WALL RUNNING NS UNCONFINED O.K.

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM		1	RPU	JCG
FINAL	X		DATE 2-5-81	DATE 2-5-81
VOID				
SHEET 43A OF 95			DATE	DATE
J.O. 9527-088				

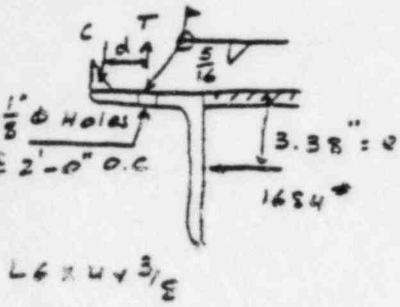
NAME OF COMPANY CP&L BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG.

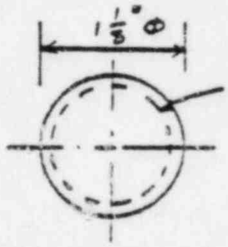
MASONRY WALL REPAIR - Wall 4a

Design of weld for Restraint Angles -

Force at top of wall = 842 #/' (DBE Loads)



$e = \frac{6 - 0.75}{2} + 0.75 = 3.38"$



Try 5/16" weld

Force on weld = 842# x 2' = 1684#

$C_w = \pi \times \frac{31}{32}" = 3.04"$ $d = 0.87 \times 2" = 1.74"$ (approx.)

$m = 3.38" \times 1684 = 5692"\#$

$f_v = \frac{1684}{3.04} = 554 \#/in$

$f_t = \frac{5692"\#}{1.74 \times (3.04 \times 0.5)} = 2152 \#/in$

(Use Half of C_w)

$f_r = \sqrt{554^2 + 2152^2} = 2222 \#/in$

$f_{allowable} = \frac{5}{16} \times 0.707 \times 21{,}000 = 4640 \#/in > 2222$ O.K.

Form 5007 Rev. 3-77

GENERAL COMPUTATION SHEET

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united engineers & constructors inc.

CALC. SET NO.		
PRELIM.		
FINAL	X	
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SHEET 43B OF 95		
J.O. 9527-088		
REV	COMP. BY	CHK'D BY
0	RGU DATE 2-23-81	JRB DATE 2-25-81
	DATE	DATE

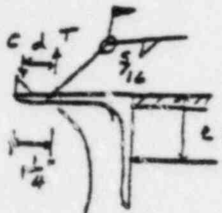
NAME OF COMPANY C P & L BSEP UNIT/S

SUBJECT DIESEL GENERATOR BLDG.

MASONRY WALL REPAIR — WALL 4a

Design of weld for Restraint Angle:—

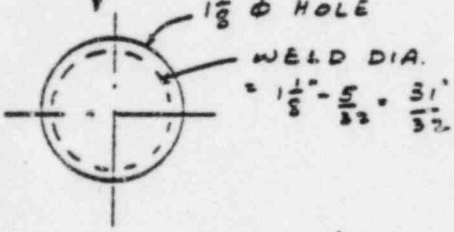
Force @ Top of wall = 842#/' (DBE)



L 6×6×1/2

$e = \frac{6 - 0.75}{2} + 0.75 = 3.38"$

Try $\frac{5}{16}$" weld



Force on weld = 842#/' × 2' = 1684#

$C_w = \pi \times \frac{31}{32}" = 3.04"$

$d = 0.87 \times 1.25" = 1.09$ (APPROX.)

$M = 1684^{\#} \times 3.38 = 5692"^{\#}$

$f_v = \frac{1684}{3.04} = 554$ #/in

$T = \frac{M}{d} = \frac{5692"^{\#}}{1.09"} = 5222^{\#}$

Assume 20% increase in tension due to Prying action

$f_t = \frac{1.2 \times 5222^{\#}}{(3.04 \times 0.5)} = 4123^{\#}$

Use Half C_w

$f_r = \sqrt{4123^2 + 554^2} = 4160$ #/in

$f_{allow} = \frac{5}{16} \times 0.707 \times 21000 = 4640$ #/in > 4160 O.K.

8016 Rev. 11/74

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

(DISCIPLINE)

CALC. SET NO.		
PRELIM.		
FINAL	X	
VOID		
SHEET 46 OF 95		
J.O. 9527.088		
REV	COMP. BY	CHK'D BY
0	CK DATE 11/5/80	R.P. DATE 11-21-80
	DATE	DATE

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG. - BLOCK WALLS

Walls 5a,b,c,d El. 50'0"

Ref. Dwg. 9527-F-1926, 1927

12" lt wt. conc. block reinf with 2-#8 @ 4'-0"

$f'_m = 1820$

$I_g = 1088$ in⁴/ft. (See SHT. #70/RPU)

Calculate properties of reinforced section:

(See Fig. 9/DALE)

n = 15.9

48 × 1.375 × (dn − 0.6875) = 0.79 × 15.9 × (9.125 − dn)

66 dn − 45.375 = 114.6 − 12.56 dn

78.56 dn = 114.6 + 45.375 ≈ 160

dn ≈ 2"

$$I_{cr} = \frac{48 \times 2^3}{3} - \frac{48 \times 0.625^3}{3} + 0.79 \times 15.9 \times 7.125^2$$

$$= 762 \text{ in}^4 \div 4 = 191 \text{ in}^4/\text{ft. width}$$

Wt. of block = 58 psf (See sht. 82/DALE).

PERIOD OF WALL $= \frac{2}{\pi}\sqrt{\frac{w\ell^4}{gEI}}$

w = 58/12 = 4.8 lb/in
ℓ = 14×12 = 168 in.
E = 1,820,000 psi
I_g = 1088 in⁴
g = 386 in/sec²

$$= \frac{2}{\pi}\sqrt{\frac{4.8 \times 168^4}{386 \times 1820000 \times 1088}} = 0.045 \text{ sec.}$$

FREQUENCY RANGE: 1.095 → 0.895

∴ PERIOD RANGE: 0.050 → 0.041 Sec.

ACCN. RESPONSE:

	OBE (4%)	DBE (7%)
El. 50'0" E-W	0.24 g	0.42 g
El. 68'0" E-W	0.25 g	0.45 g

OBE LOADS

$$\text{DESIGN ACCN.} = 1.05 \times \frac{0.24 + 0.25}{2} g = 0.26 g$$

$M_x = 0.26 \times 58 \times \frac{14^2}{8} = 370$ lb-ft < M_{cr} → $M_{cr} = \frac{93 \times 1088}{5.81 \times 12} = 1452$ lb-ft

∴ UNCRACKED

$V = 0.26 \times 58 \times \frac{14}{2} = 106$ lbs

Form 5015 Rev. 11/74

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc.

CALC. SET NO.	
PRELIM.	
FINAL	Y
VOID	

J.O. 9527.085

REV	COMP. BY	CHK'D BY
0	SF DATE 11-5-80	PP DATE 11-21-80
	DATE	DATE

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GEN. BLDG - BLOCK WALLS

Walls 5 a,b,c,d (Contd)

$f_m = \frac{370 \times 12 \times 2}{191} = 47 \text{ psi} < 60 \text{ psi OK}$ $\frac{L}{C} = 0.08$

$f_s = \frac{370 \times 12 \times 7.125 \times 15.9}{191}$

$= 2634 \text{ psi} < 24{,}000 \text{ psi}$ $\frac{L}{C} = 0.11$

$v = \frac{106 \times 4}{7.625 \times 9.125} = 6 \text{ psi} < 47 \text{ psi}$ $\frac{L}{C} = 0.13$

$u = \frac{106 \times 4}{3.14 \times 0.92 \times 9.125} = 16 \text{ psi} < 140 \text{ psi}$ $\frac{L}{C} = 0.12$

DBE LOADS

DESIGN ACC^n = $1.05 \times \frac{0.47 + 0.45}{2} g = 0.46 g$

RATIO : $\frac{DBE}{OBE} = \frac{0.46}{0.26} = 1.77$

$M_a = 1.77 \times 376 = 666 \text{ ft-lb} < 1469 \text{ ft-lb} = M_{cr}$ ∴ UNCRACKED

$f_m = 47 \times 1.77 = 83 \text{ psi} < 1547 \text{ psi}$ $\frac{L}{C} = 0.05$

$f_s = 2634 \times 1.77 = 4662 \text{ psi} < 54000 \text{ psi}$ $\frac{L}{C} = 0.09$

$v = 6 \times 1.77 = 11 \text{ psi} < 73 \text{ psi}$ $\frac{L}{C} = 0.15$

$u = 16 \times 1.77 = 28 \text{ psi} < 186 \text{ psi}$ $\frac{L}{C} = 0.15$

ANGLES AT THE TOP OF THE WALL MISSING

EFFECT OF OPENINGS -

WALL 5d HAS A 3'4" (L) × 4'4" (H) OPENING. O.K. BY INSPECTION

IN-PLANE CRITERIA :- WALL IS CONFINED, RUNNING N-S OK

ACTION: SEE PM 80-232, MWR-7

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers

9527-1-GP-MW-02-F   15HA

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM		1	RPU	JRB
FINAL	X		DATE 10-16-81	DATE 10-21-81
VOID				
SHEET 47A OF 95			DATE	DATE
10 9527-088				

NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GENERATOR BLDG.

REF. MWR-7 REV. C AND MWR-17

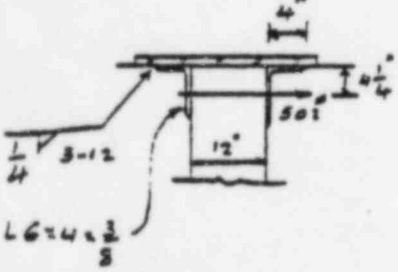
Walls	Shear Force at Top of wall	
5a, 5b, 5c, 5d	188#/FT	SUPC. DEFIN. MWR-7 REV. B DCN 9527-088-006

Through bolts are at 2'-8" o.c.

Force/bolt = 188#/FT x 2.67' = 502#

Use $\frac{1}{2}$" ϕ A-307 Bolt. P_{allow} = 0.142 in² x 20,000 psi = 2840#

> 502# O.K.



check weld between L6x4 & insert PL

Length of weld in 32"

= 3 x $\frac{32}{12}$ = 8" (on each angle)

d = 0.87 x 4" = 3.48"

L_w = 2 x 8 = 16"

$f_1 = \frac{502}{16}$ = 32#/in $f_2 = \frac{502 \times 4.25}{8 \times 3.48}$ = 77#/in

$f_R = (32^2 + 77^2)^{1/2}$ = 84#/in < 3712 O.K.

check Torque on $\frac{1}{2}$" ϕ bolt -

Bolt Tighten to a torque of 20 FT.LBS.
Bolt stress = 20,000 psi
20,000 x 0.142 in² = 2840#

Bearing stress on block = $\frac{2840}{6" \times 6"}$ = 79 psi

Assume (6" x 6")   < 0.62 f'm = 0.62 x 1820
= 1128 psi
O.K.

SEE MWR-7 REV C ON DCN 004
OF PAI 80-33_

Form 5007 Rev. 3-77

GENERAL COMPUTATION SHEET

united engineers & constructors inc.

(DISCIPLINE)

NAME OF COMPANY: C P E L UNIT/S:

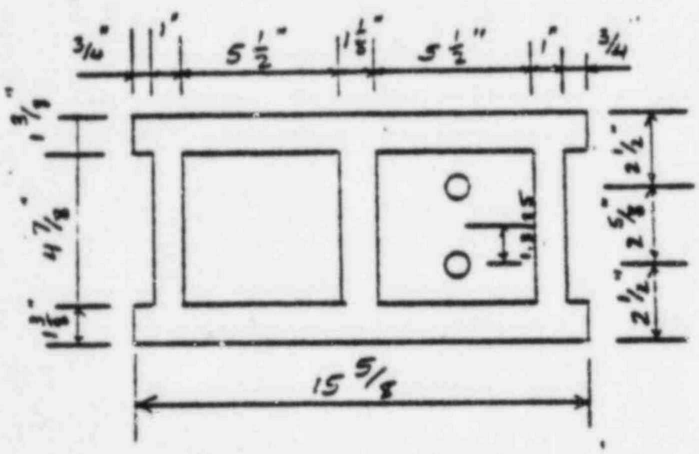
SUBJECT: DIESEL GENERATOR BLDG. – MASONRY WALLS

CALC. SET NO.		
PRELIM.		
FINAL	X	
VOID		
SHEET 48 OF 95		
J.O. 9527.098		
REV.	COMP. BY	CHK'D BY
0	RPU DATE 10/3/80	CR DATE 10-9-80
	DATE	DATE

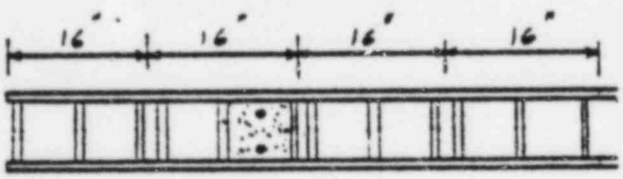
walls 6a. b El. 23'-0"

Ref. – calcs by Dale sheet # 49.
Ref. DWGS – F-1926, F-1927

calculate I_g – uncracked gross moment of inertia



Reinf. 2 - #6 @ 4'-0"



$I_1 = \frac{48 \times (7.625)^3}{12} = +1773.29$

$I_2 = \frac{38.62 \times (4.875)^3}{12} = -372.87$

$I_3 = \frac{5.5'' \times (4.875)^3}{12} = +53.10$

$I_g = 1453.52 \times \frac{1}{4} = 363.40 \text{ in}^4/\text{ft}$

f_r = 93 psi from 0-6 committee report Page 2-24

$M_{cr} = f_r \left\{ \frac{I_g}{y} \right\} = \frac{93 \times 363.40}{12 \times 3.81} = 739'^{\#}$

$I_{g\,Hori} = \frac{12 \times 7.625^3}{12} - \frac{12 \times 4.875^3}{12} = 327 \text{ in}^4/ = 27.3 \text{ in}^4/\text{in}$

FORM 5007 REV 7/79 **GENERAL COMPUTATION SHEET**

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CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM.		0	RPU	CF.
FINAL	X		DATE 11/8/80	DATE 11/23/80
VOID				
SHEET 49 OF 95			DATE	DATE
10 9527-088				

NAME OF COMPANY C P & L UNIT/S

SUBJECT DIESEL GEN. BLDG. - MASONRY WALLS

walls 6a 6b - cont.

$$P = \sqrt{\frac{3.5 \times 10,000,000,000}{386 \times 1,820,000 \times 363.4}} \times \frac{144L^2}{157,000} = 0.00034L^2 \quad \text{(one way behavior)}$$

$P = 0.00034(23.5)^2 = 0.19$ sec. — wall 6a
$P = 0.00034(24.5)^2 = 0.204$ sec. — wall 6b

Period spread = 0.91 → 1.11
$P_{Low} = 0.204 \times 0.91 = 0.186$ sec.
$P_{High} = 0.204 \times 1.11 = 0.226$ sec

From computer printouts,

	4% OBE	7% DBE	
Accel. @ EL. 23'	0.93	1.28	E-W response
Accel. @ EL. 50'	1.38	1.84	E-W response

Design accel. $= \frac{0.93 + 1.38}{2} \times 1.05 = 1.21g$ — OBE

$= \frac{1.28 + 1.84}{2} \times 1.05 = 1.64g$ — DBE

$M_a = 1.21 \times 42^{psf} \times \frac{24.5^2}{8} = 3813'^{\#} > M_{cr} = 739'^{\#}$ ∴ cracked

$\frac{M_{cr}}{M_a} = \frac{739}{3813} = 0.19$

$I_e = 0.19^3 \times 363.4 + (1 - 0.19^3)\,35.4 = 37.6 \text{ in}^4/\text{ft}$

$$P = \frac{1}{1.57}\sqrt{\frac{3.5 \times 294^4}{386 \times 1,820,000 \times 37.6}} = 0.63 \text{ sec.}$$

Period range = 0.91 → 1.11
$P_{Low} = 0.63 \times 0.91 = 0.573$ sec.
$P_{High} = 0.63 \times 1.11 = 0.70$ sec.

From computer printouts,

	4% OBE	7% DBE	
Accel. @ EL. 23'	0.39	0.65	E-W response
Accel. @ El. 50'	0.43	0.71	E-W response

Design accel. $= 0.41 \times 1.05 = 0.43$ — OBE
$= 0.68 \times 1.05 = 0.71$ — DBE

accel. ratio $= \frac{0.71}{0.43} = 1.65$

9527-1-GP-MW-CZ-F

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

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united engineers & constructors inc.

CALC. SET NO.			REV	COMP BY	CHK'D BY
PRELIM.			0	F. F U	
FINAL	X			DATE 11/3/80	DATE
VOID					
SHEET 50 OF 95				DATE	DATE
J.O. 9527-083					

NAME OF COMPANY C P & L UNIT/S

SUBJECT DIESEL GEN BLDG - MASONRY WALLS

walls 6a, b - cont

$W_H = 0.43 \times 42^{PSF} \times 24.5' = 442^{\#} \times \frac{1}{2} = 221^{\#} = R = V$

$m = \dfrac{442^{\#} \times 24.5'}{8} = 1354'^{\#}$

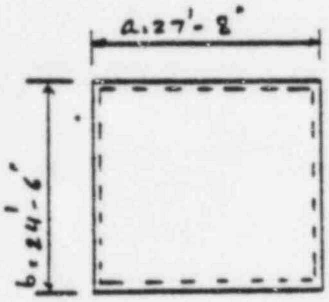
$f_m = \dfrac{1354'^{\#} \times 12 \times 1.1}{35.4} = 505\text{ psi} < 600$ O.K.

$f_s = \dfrac{1354'^{\#} \times 12 \times 4.15'' \times 15.9}{35.4} = 30,286\text{ psi} > 24,000$ N.G.

check walls for Two-way behavior -

wall 6a - 26'-7" L × 23'-6" H $\frac{L}{H} = \frac{26'-7''}{23-6''} = 1.13$ say 1.1

wall 6b - 27'-8" L × 24'-6" H $H/L = \frac{27'-8''}{24-6''} = 1.13$ say 1.1



Assume wall supported (simple supports) on four sides -

(Ref - Theory and Analysis of plates by Rudolph Szilard)

$\omega_{mn} = \pi^2\left(\frac{m^2}{a^2} + \frac{n^2}{b^2}\right)\sqrt{\frac{D}{\bar{m}}}$ (Ref. Pg. 695)

$D = \dfrac{EI}{(1-\mu^2)} = \dfrac{1,920,000 \times 352}{(1-0.15^2)} = 6.55 \times 10^8$

$I_x = 363\text{ in}^4$
$I_y = 341\text{ in}^4$
$I_{av.} = 352\text{ in}^4$

$\bar{m} = \dfrac{42^{PSF}}{144} \times \dfrac{1}{386} = 0.000756$

$\omega_{11} = \pi^2\left(\dfrac{1}{332^2} + \dfrac{1}{294^2}\right)\sqrt{\dfrac{6.55 \times 10^8}{0.000756}} = 190$

$P = \dfrac{2\pi}{\omega_{11}} = \dfrac{2\pi}{190} = 0.033\text{ sec.} \longrightarrow f = 30.2\text{ cps.}$

Period spread = 0.91 → 1.11

$P_{low} = 0.033 \times 0.91 = 0.030\text{ sec.}$

$P_{high} = 0.033 \times 1.11 = 0.0366\text{ sec.}$

Accel from computer printouts,

	4% OBE	7% DBE	
EL. 23'	0.19	0.34	E-W response -
EL 50'	0.24	0.42	E-W response -

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers

NAME OF COMPANY C P E L UNIT/S

SUBJECT DIESEL GEN. BLDG. - MASONRY WALLS

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM.		0	R F U	CF
FINAL	X		DATE 11/9/80	DATE
VOID				
SHEET 51 of 95			DATE	DATE
J.O. 9527-082				

walls 6a, 6 - cont.

Design accel = $\frac{0.19 + 0.24}{2} \times 1.05 = 0.23\ g$ - OBE

accel = $\frac{0.34 + 0.42}{2} \times 1.05 = 0.40$ - DBE

accel ratio = $\frac{0.40}{0.23} = 1.74$

Assume one way behavior for stress calculation:-

$W_H = 0.23 \times 42^{psf} \times 24.5 = 237^{\#} \times \frac{1}{2} = 119^{\#} = R = V$

$M = 237^{\#} \times \frac{24.5'}{8} = 726'^{\#} < M_{cr} = 739'^{\#}$ ∴ uncracked

$f_m = \frac{726'^{\#} \times 12 \times 1.1}{35.4} = 271^{psi} < 600$ O.K. $\frac{L}{C} = \frac{271}{600} = 0.45$

$f_s = \frac{726 \times 12 \times 4.15 \times 15.9}{35.4} = 16,239$ psi $< 24,000$ O.K.

$\frac{L}{C} = \frac{16,239}{24,000} = 0.68$

$v = \frac{119^{\#} \times 4'}{7.625'' \times 5.25''} = 11.9$ psi < 47 O.K. $\frac{L}{C} = \frac{11.9}{47} = 0.25$

$u = \frac{119^{\#} \times 4'}{2.4 \times 0.93 \times 5.25} = 40.6$ psi < 140 O.K. $\frac{L}{C} = \frac{40.6}{140} = 0.29$

For DBE Loads

$M_a = 726'^{\#} \times 1.74 = 1263 > M_{cr} = 739$ ∴ cracked.

$\frac{M_{cr}}{M_a} = \frac{739}{1263} = 0.59$

$I_e = 0.59^3 \times 363 + (1 - 0.59^3) 35.4 = 102.7\ in^4$

$\frac{I_e}{I} = \frac{102.7}{363} = 0.28$

$\omega = \pi^2 \left(\frac{1}{33.2^2} + \frac{1}{294^2} \right) \sqrt{\frac{0.28 \times 6.55 \times 10^8}{0.000756}} = 100.3$

$P = \frac{2\pi}{100.3} = 0.063$ sec.

period spread = 0.07 — 0.057

Accel. response, for DBE.

		7%
EL. 23'-0"	E-W	0.40
EL. 50'-0"	E-W	0.41

Design accel = 1.05 × 0.405 = 0.42g

wall 6 a.6 - cont. -

$m_a = 0.42 \times 42 \times \frac{24.5^2}{8} = 1324'^{\#} > 1263'^{\#}$ (assumed value)

convergence at period = 0.082 sec.

Accel. response,

		7% DBE
EL 23'-0"	E-W	0.522
EL 50'-0"	E-W	0.435

Design accel = $1.05 \times \frac{(0.522 + 0.435)}{2} = 0.50g$

$m_a = 0.50 \times 42 \times \frac{24.5^2}{8} = 1576'^{\#}$

$\frac{m_{cr}}{m_a} = \frac{739}{1576} = 0.47$

$\left(\frac{m_{cr}}{m_a}\right)^3 = 0.10$

$I_e = 0.10 \times 363.4 + 0.9 \times 35.4 = 68.2\ in^4$

$\frac{I_e}{I} = \frac{68.2}{363.4} = 0.19$

$\omega = \pi^2 \left(\frac{1}{232^2} + \frac{1}{294^2}\right) \sqrt{\frac{0.19 \times 6.55 \times 10^8}{0.000756}} = 82.7$

$P = \frac{2\pi}{82.7} = 0.076$ sec.

period spread = 0.085 → 0.069

Accel response,

		7% DBE
23'-0"	E-W	0.52
50'-0"	E-W	0.44

Design accel. = $1.05 \times 0.48 = 0.50g$

$m_a = 0.50 \times 42 \times \frac{24.5^2}{8} = 1576'^{\#} = 1576'^{\#}$ (assumed value)

FORM 3007 REV 7/79

GENERAL COMPUTATION SHEET

(DISCIPLINE)

united engineers & constructors inc

NAME OF COMPANY C P & L UNIT/S

SUBJECT DIESEL GEN. BLDG. - MASONRY WALLS

CALC. SET NO		REV	COMP BY	CHK'D BY
PRELIM.		0	RPU	CR
FINAL	X		DATE 11/10/80	DATE 11/12/80
VOID				
SHEET 53 OF 95			DATE	DATE
J.O. 9527-028				

Wall 6 a. 6 - Cont.

Accel. ratio = $\frac{0.50}{0.23}$ = 2.17

f_m = 271 psi × 2.17 = 588 psi < 1547 O.K. $\frac{L}{C}$ = 0.38

f_s = 16,239 psi × 2.17 = 35,239 psi < 54,000 O.K. $\frac{L}{C}$ = 0.65

v = 11.9 psi × 2.17 = 26 psi < 72 O.K. $\frac{L}{C}$ = 0.36

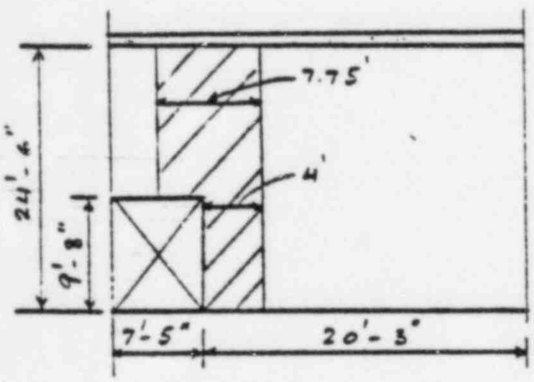
u = 40.6 psi × 2.17 = 88 psi < 186 O.K. $\frac{L}{C}$ = 0.47

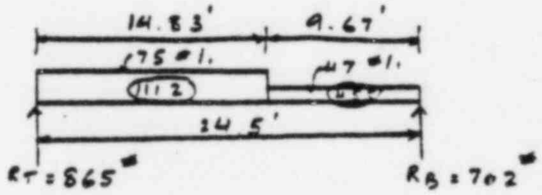
check for effect of opening -

width of critical strip = $\frac{24'-6"}{6}$ = 4'

w_1 = 0.23 × 42 psf × 7.75' = 75 #/'

w_2 = 0.23 × 42 × 4' + 0.23 × 10 psf × 3.75' = 47 #/'





M_{cr} = 702# × 9.67' - 47 × $\frac{9.67^2}{2}$ = $\frac{4591}{4}$ = 1148'#

f_m = $\frac{1148 \times 12 \times 1.1}{35.4}$ = 428 < 600 O.K.

f_s = $\frac{1148 \times 12 \times 4.15 \times 15.9}{35.4}$ = 25,678 psi > 24,000 psi

(1) conservatively load is considered only in vertical direction
(2) 2 - #6 extra rebars are provided in vertical span @ door frame - see DWG. F-1927, sec. 10-10 & sec. 11-11

∴ 7% overstress in reinf. steel is O.K.

v = $\frac{702\#}{7.625 \times 5.25}$ = 18 psi < 42 O.K.

u = $\frac{702\#}{2.4 \times 0.93 \times 5.25}$ = 60 psi < 140 O.K.

FORM 5007 REV 7/79 GENERAL COMPUTATION SHEET

(DISCIPLINE) united engineers

NAME OF COMPANY C P & L UNIT/S

SUBJECT DIESEL GEN. BLDG. - MASONRY WALLS

CALC. SET NO		REV	COMP BY	CHK'D BY
PRELIM		0	RPU	CR
FINAL	X		DATE 11/10/80	DATE 11/20/80
VOID				
SHEET 54 OF 15			DATE	DATE
JO 9527-088				

Walls 6a. b - cont.

For DBE Loads.

accel. ratio = 2.17 (conser.)

$f_m = 428^{psi} \times 2.17 = 929 < 1547$ O.K

$f_s = 25.678^{psi} \times 2.17 = 55721^{psi} \approx 54,000$ O.K. (3% overstressed)

$v = 18^{psi} \times 2.17 = 39 psi < 72$ O.K

$u = 60^{psi} \times 2.17 = 130 psi < 186$ O.K.

NOTES:

Wall 6b — 'AS-BUILT' sketch shows solid mortar joints on sides. Wall is O.K. for two way behavior.

Wall 6a — Lateral support angles called for in construction drawings are missing and must be installed. 'AS BUILT' sketch shows caulked joints on sides. Field verification is requested to determine if sides are caulked or solid mortar joints. If sides are caulked vertical angles should be installed to get two way behavior.

In plane shear strain criteria -

walls run N-S and confined. O.K.

ACTION: SEE PM 80-232, MWR-7 for wall 6a.

FORM 5007 REV 7/79

GENERAL COMPUTATION SHEET

united engineers & constructors inc

(DISCIPLINE)

CALC. SET NO.		REV	COMP BY	CHK'D BY
PRELIM			RPU	JRG
FINAL	X		DATE 10-20-81	DATE 10-2-81
VOID				
SHEET	54A OF 95		DATE	DATE
JO	9527-088			

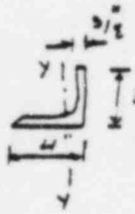
NAME OF COMPANY CP&L - BSEP UNIT/S

SUBJECT DIESEL GENERATOR BLDG.

REF. MWR-17 & TELECOPY TC-656 UEC 1564

WALL 6A SHEAR FORCE AT TOP = 258#/FT

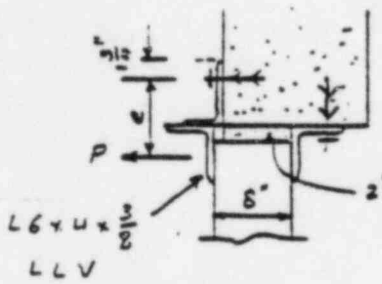
Restraining L6x4x3/8 is notched to clear existing concrete beam. check bending stress in notched angle.



$S_y = 1.46\ in^3$ load = 258 #/FT span = 2'-6" ±

$f_b = \frac{258 \times 2.5^2}{8 \times 1.46} = 138$ psi << 21,600 psi O.K.

check concrete wedge anchors - WS3470 @ 2'-0" O.C.



P = 258 #/' x 2' = 516#

e = 2" + 2" (GROUT) + 4 1/4" = 8.25"

$T = 516^{\#} + \frac{516 \times 8.25"}{0.87 \times 1.75"} = 3312$ #/Anchor

$> \frac{12405}{4} = 3101^{\#}$

REDUCE SPACING TO 22" O.C.

SEE MWR-17 ON DCN 004 OF PM 80-232

WELD BETWEEN ANGLES:

M = 5" x 258 = 1290 LB IN

WELD SHEAR = 1290 ÷ 3" = 430 LB / WELD 3" LG.

CAPACITY = 3 x 1/4 x .707 x 21 = 11,130 LB