LIC-96-0121

ATTACHMENT 5

Question 8

Calculation FC06011
and
Summary of Calculation for Blockwall No. 9

COPY

SARGENT & LUNDY ENGINEERS	PROJECT NAME: Fort Calhoun UNIT NO.: 1 PROJECT NO.: 7751-27 CLIENT: OPPD CALC. NO.: FC06011	COMMENT NO.	QA SERIAL #
DESIGN CONTROL SUMMARY DESIGN VERIFICATION E PAGE 1	PURPOSE: Seismic Qualification of Diesel Fuel Tanks ☒ SAFETY RELATED ☐ NON SAFETY RELATED		

SIGNATURE & DATE FOR REV.			IDENTIFICATION OF PAGES ADDED/REVISED/SUPERSEDED/VOIDED & REVIEW METHOD	COMMENT NO.	QA SERIAL #
REV. 0	PREPARER	Adam AliDabbagh 9/28/92	Added pages: 1 thru 49 59 thru 68		
	REVIEWER	M. Amin 9/28/92			
	APPROVER	Shih-Lung Chu 9-28-92	REVIEW METHOD Detailed Review		
REV. 0	PREPARER	[signature] 9-25-92	Added pages: 50 thru 58		
	REVIEWER	Adam AliDabbagh 9/25/92			
	APPROVER	Shih-Lung Chu 9-28-92	REVIEW METHOD Detail Review		
REV. __	PREPARER				
	REVIEWER				
	APPROVER		REVIEW METHOD		
REV. __	PREPARER				
	REVIEWER				
	APPROVER		REVIEW METHOD		

FORM SAS-22.1 REV.2 (11-18-91)

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	PROJECT NO.: 7751-27 PAGE NO.: 2
DESIGN CONTROL SUMMARY DESIGN AREA CALCULATION INDEX	CLIENT: OPPD
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REVISION

FORM SAS-22.3 REV.2 (11-18-91)

SARGENT & LUNDY ENGINEERS	PROJECT NAME: Fort Calhoun REV. NO.: 0	REVISION
DESIGN CONTROL SUMMARY PURPOSE AND SCOPE	PROJECT NO.: 7751-27 PAGE NO.: 3 CLIENT: OPPD CALC. NO.: FC06011	

The purpose of this calculation is to seismically qualify the Diesel Fuel Oil Tank (FO-1). This tank is about 23' long and 12' in diameter used as diesel Fuel underground storage tank. The tank has been in service since plant construction.

This calculation includes the engineering analyses and stress evaluation of the tank under various loading condition including seismic. If the stress evaluations of the tank are acceptable, this calculation will be the support for the tank seismic qualification. Since Tank FO-10 is identical in size, material, thickness, and overburden, the qualification of FO-1 will automatically qualify FO-10.

FORM SAS-22.4 REV.2 (11-18-91)

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DESIGN CONTROL SUMMARY DESIGN INPUT - DESCRIPTION OF STRUCTURAL SYSTEMS	PROJECT NO.: 7751-27 PAGE NO.: 4
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The fuel storage tank is 23'-0" long, 12'-0" in diameter, and made of 5/16" A285C steel. The tank is anchord to a reinforced concrete foundation by two hold down bands. The top of the foundation is at elev. 989'-0". The tank top is 2'-6" below Grade (Elev. 1004'-6"). A concrete slab of about 4" cover the ground above the tank. The tank has an 18"φ manhole, 2-3"φ nozzles and one 4"φ nozzle which is used to transfer diesel oil to the generator.

FORM SAS-22.5 REV.2 (11-18-91)

FORM SAS-22.6 REV.3 (11-18-91)

SARGENT & LUNDY ENGINEERS	DESIGN CONTROL SUMMARY DESIGN INPUT - DESIGN PARAMETERS	PROJECT NAME: Fort Calhoun PROJECT NO.: 7751-27 CLIENT: OPPD CALC. NO.: FC06011	REV. NO.: 0 PAGE NO.: 5	REVISION

NO.	SOURCE OF DESIGN INPUT: TITLE	REVISION	DATE	DESCRIPTION OF INPUT	REMARKS	REVISION
I-1	Fuel Oil Tank Drawing by Eaton Metal Products Corp. Drawing no. 802381	3	5-5-89	Tank size, weight, thickness, material type, and penetration details	Tank no. 69-533, Ref. no. 002381	
I-2	OPPD, Fort Calhoun Station Unit 1, drawing no. 11405-S-418	6	1-29-75	Tank Reinforced Concrete foundation details		
I-3	Eaton Metal Products Corp. Drawing no. 02346	1	8-5-69	Hold down detail for FO-1 tank	File # 11395-144	
I.4	Piping Isometric for Oil Piping, OPPD Drawing no D-4612 sheet 1 of 2	0	—	Piping connected to tank		
I-5	Updated Safety Analysis Report_ Fort Calhoun Station Unit 1, Vol. 6, Appendix F	—	7/87	Design Earthquake: 0.08 g horizontal, .053 g vertical Max. Hyp. Eq.: 0.17 g horiz., 0.17 g vertical.		
I.6	ASCE Standard ASCE 4-86, Standard for Seismic Analysis of Safety Related Nuclear Structures	—	Sept. 1986	Wave velocities	See Section 3.5.2.1. See note below	

Note: For uniform soil layer overlying rock, the propagation velocity of seismic waves along a buried structure is found to be strongly infuenced by the wave speed in rock. The plant site consists of fine sand layer over lime stone bed rock (Appendix C & D of USAR). In such cases the apparent wave velocities are generally greater than the speed of waves in the bed rock. Hence conservatively apparent shear wave velocity of 3000 ft/sec. and apparent compression wave velocity of 6000 ft

FORM SAS-22.7 REV.3 (11-18-91)

SARGENT & LUNDY ENGINEERS	DESIGN CONTROL SUMMARY DESIGN INPUT MATERIAL PARAMETERS/SYMBOLS			PROJECT NAME: Fort Calhoun REV. NO.: 0 PROJECT NO.: 7751-27 PAGE NO.: 6 CLIENT: OPPD CALC. NO.: FC06011	REVISION
DESCRIPTION	PARAMETERS			DESCRIPTION OF USE	
	SYMBOL	VALUE	UNITS		
Steel A 285C	σ_y	30	ksi	To obtain allowable tensile stress.	
	S_m	13.7 *	ksi	allowable stress intensity	
Fill Material	γ	130	psf	calculate soil overburden	

LEGEND OF SPECIAL SYMBOLS AND NOTATIONS:

σ_y = minimum yield strength

γ = weight density of backfill

S_m = Allowable Stress Intensity

* From 1986 ASME Boiler & Pressure Vessel Code Section III, Div. 1, Table I-7.1.

SARGENT & LUNDY ENGINEERS	PROJECT NAME: Fort Calhoun	REV. NO.: 0	REVISION
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1. Density of backfill & overburden is assumed to be 130 psf. This density is reasonable for compacted wet soil.

2. The ambient temp. is assumed to be 70°F. The soil temperature is assumed to be about 50°F. Therefore a differential temperature Δt of -20°F is considered.

3. The base slab width is 15'. It is assumed that the ditch width is also 15'.

4. The maximum water table level is assumed to be at the top of the tank or 2.5' below grade. Per Section 2.7.2.1 of USAR, the water table ranges from 2 to 17 feet below the surface. The assumed level is conservative.

5. Effect of seismic waves is conservatively considered in the longitudinal direction of the tank, but not in the lateral direction. Based on the design basis ground motion spectra, the period of seismic waves is about 0.25 sec. Assuming an apparent propagation velocity of 3000 ft/sec, the corresponding wave length is 750 feet. This wave length is over sixty times the tank diameter. Therefore, wave effect of seismic motion can be ignored. A similar argument applies to the longitudinal direction. However, conservatively, this effect is considered in the longitudinal direction.

FORM SAS-22.8 REV.2 (11-18-91)

Analytical Methods:

Calculate individual stresses due to the loading described in this DCS using the information provided in this section. Combine the stresses based on the load combinations presented in the DCS. Note that vertical seismic loads are combined with one horizontal seismic (longitudinal or lateral) loads to obtain the effect of the seismic loading in any specific loading combination. The acceptance criteria presented as part of this DCS shall be used to determine the calculated stresses acceptability.

1. Tank Thickness (provided $t = 0.3125'' = 5/16''$)
   Based on Reference 3, the required thickness is 0.302" which is less than 0.3125".

2. Fill Soil

   The fill soil is assumed to be compacted with a wet density, γ, of 130 lb/ft^3 (a conservative density)

3. The under ground water level is assumed to be at the top of the tank. The wet density of the fill material will be used to calculate the over burden pressure effects.

4. Maximum Load on Tank Due to overburden:

   Ref. 1, Section 24.5

   $$W_e = C_d \gamma B_d^2$$

   where

   W_e = load on tank in pounds per foot length of the tank

   B_d = horizontal width of excavation ditch = 15 feet

$$C_d = \frac{1 - e^{-2K\mu' \frac{h}{B_d}}}{2K\mu'}$$

From Ref. 1, Figure 24-3, for granular material $K\mu' = 0.1924$ (See Note below),

h = Fill height above tank = 2.5'

$$\frac{h}{B_d} = \frac{2.5}{15} = 0.1667$$

$$C_d = \frac{1 - e^{-2 \times .1924 \times .1667}}{2 \times .1924} = 0.1614$$

$$W_e = 0.1614 \times 130 (15)^2 = 4721 \text{ lb/foot}$$

Note From Figure 24-3 of Reference 1, For a small $h/B_d < .3$ the value of Cd does not change for different soil conditions.

5. Full Tank Average Internal Pressure.

The diesel fuel density is slightly less than that of water. Conservatively the density of water is used.

$$\text{Average pressure} = \frac{62.4 \times 12/2}{144} = 2.6 \text{ psi}$$

6. Seismic Effects

a. Effects Due to Seismic Waves:

Particle Velocity:

Per Reference (7) Article 7.2, the

particle velocity for competent soil conditions is assumed to be 48 in/sec/g.

Peak velocities are:

Design Earthquake (OBE)

$V_h = 48 \times .08 = 3.84$ in/sec

$V_v = \frac{2}{3} \times 3.84 = 2.56$ in/ses

Maximum Hypothetical Earthquake (SSE)

$V_h = 48 \times 0.17 = 8.16$ in/sec

$V_v = \frac{2}{3} \times 8.16 = 5.44$ in/sec.

Compression and Shear wave velocities:

The apparent shear wave velocity is conservatively assumed to be 3000 ft/sec (C_s), {Input Ref. I.6}.

The apparent compression wave velocity is assumed to be 6000 ft/sec. (C_p)

Strain in Tank Due to Seismic Waves

Axial strain in the tank

$$\epsilon_a = \frac{V}{C_p} \qquad \text{(Ref. 4)}$$

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Bending Strain

$$\epsilon_b = \frac{R \cdot a}{C_s^2} \qquad (\text{Ref.4})$$

R = tank radius = 72"

a = maximum particle acceleration

= 0.08g = 30.91 in/sec (OBE)

= 0.17g = 65.69 in/sec (SSE)

For OBE Conditions:

$$\epsilon_a = \frac{V_h}{C_p} = \frac{3.84}{6000 \times 12} = 5.333 \times 10^{-5}$$

$$\epsilon_b = \frac{R \cdot a}{C_s^2} = \frac{72 \times 30.91}{(3000 \times 12)^2} = 1.717 \times 10^{-6}$$

For SSE Conditions

$$\epsilon_a = \frac{8.16}{6000 \times 12} = 1.133 \times 10^{-4}$$

$$\epsilon_b = \frac{72 \times 65.69}{(3000 \times 12)^2} = 3.649 \times 10^{-6}$$

The bending strains are small and will be ignored.

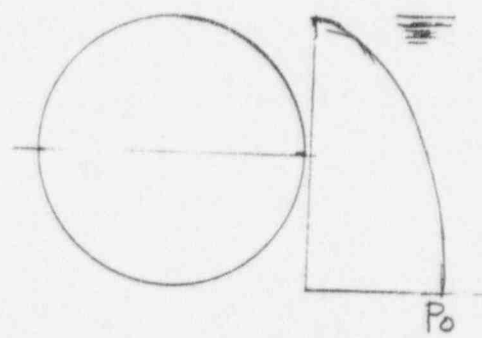
$$\sigma_a|_{OBE} = \epsilon_a E = 5.333 \times 10^{-5} \times 29 \times 10^{6} = 1546 \text{ psi}$$

$$\sigma_a|_{SSE} = \epsilon_a E = 1.133 \times 10^{-4} \times 29 \times 10^{6} = 3286 \text{ psi}$$

FORM SAS-22.9 REV.4 (11-18-91)

b. Effects Due to Underground Water

Per Ref. 8, Equation (40), the maximum pressure at the bottom of the tank is P_0 which is



$P_0 = C\,a\,h$

$C = 0.026$ ton/ft^3
$= 52$ lb/ft^3

$a =$ acceleration in g's

$h = 12'$

For OBE conditions

$P_0 = 52 \times 0.08 \times 12' \approx 50$ psf

For SSE condition

$P_0 = 52 \times 0.17 \times 12 = 106$ psf

Assuming eliptical pressure distribution,

Pressure Force $P = \frac{\pi}{4} \times D P_0$

For OBE $P_0 = 50$ psf

$P = \frac{\pi}{4} \times 12 \times 50 = 471$ lb/linear Foot

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For SSE $P_0 = 106$ psf

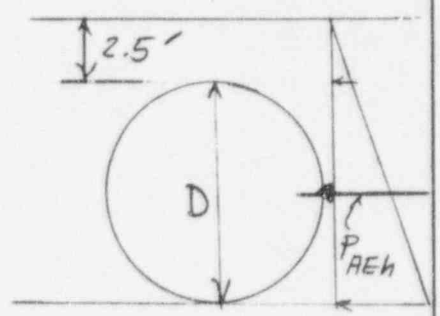
$\&$ $P = \frac{\pi}{4} \times 12 \times 106 = 999$ lb/linear foot

c. Effect Due To lateral Soil Pressure:

Per Ref. 10 pages 114 thru 118, the following dynamic soil force: may be expected:

$$P_{AEh} \simeq \gamma \left(\frac{D+2.5+2.5}{2}\right) D \cdot K_{AE}$$

$$= 0.5\gamma(D+5)D \cdot K_{AE}$$



From Figure 12(b) of Ref. 10
$\delta = 0°$

for horiz. acc. $k_h = 0.08g$ (OBE)

$K_{AE} = 0.32$

For horiz. acc. $k_h = 0.17g$ (SSE)

$K_{AE} = 0.37$

$P_{AEh} = 0.5 \times 0.32\, \gamma D (D+5)$

$= 0.16\, \gamma D(D+5)$ OBE

$= 0.16 \times 130 \times 12 \times 17$

$P_{AEh} =$ 4243 lb/linear foot (OBE)

$P_{AEh} = 0.185\, \gamma D(D+5)$ SSE

$P_{AEh} =$ 4906 lb/linear foot (SSE)

The above calculated forces include the static and the dynamic effects.

The average pressure is

$p_{AEh} = \frac{4243}{12} = 354$ psf for static plus OBE

$= \frac{4906}{12} = 409$ psf for static plus SSE

To calculate average static pressure, use Figure 12b of Ref. 10 at 0 acceleration

$$K_{AE} = 0.27$$

$$p_{AEh} = 0.5\,\gamma\,(D+S)\,K_{AE}$$

$$= 0.5 \times 130 \times 17 \times 0.27$$

$$= 298 \text{ psf}$$

<u>Summary of Lateral Soil Pressure:</u>

Ave. Soil Pressure = 298 psf

Ave Soil Pressure (OBE) = 354 psf

Ave Soil Pressure (SSE) = 409 psf

d. Vertical Seismic Force Due to Overburden

OBE:

$$a_v = 0.08 \times \frac{2}{3} = 0.053 \ g$$

From Item 4

$$\Delta W_e = 4721 \times .053 = 250 \text{ lb/linear foot}$$

SSE:

$$a_v = 0.17 \times \frac{2}{3} = 0.113 \ g$$

$$\Delta W_e = 4721 \times 0.113 = 533 \text{ lb/linear foot}$$

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
ANALYTICAL METHODS

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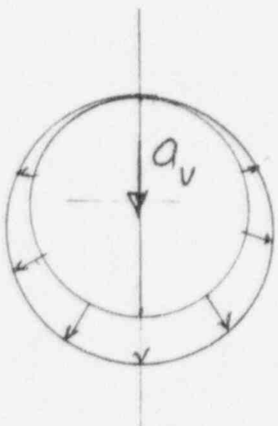
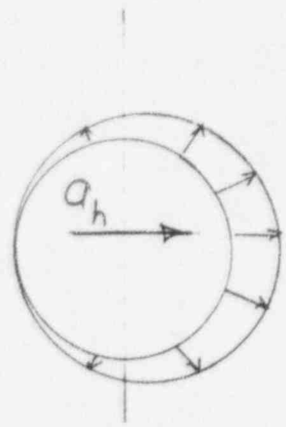
e. Seismic Pressure Due to Diesel Fuel (Full Tank assumed)





$$P_{S,V\,max.} = \frac{\gamma D}{144} a_v \qquad P_{S,h\,max.} = \frac{\gamma D}{144} a_h$$

Max. Seismic pressure due to vertical & horizontal lateral seismic

$$P_{S,max.} = \frac{\gamma D}{144} \sqrt{a_v^2 + a_h^2}$$

For OBE $a_v = \frac{2}{3} a_h$, $a_h = 0.08\,g$

$$P_{S,max} = \frac{1.20 \gamma D a_h}{144}$$

$$= \frac{1.20 \times 62.4 \times 12 \times .08}{144} = 0.5 \text{ psi}$$

For SSE $a_v = \frac{2}{3} a_h$, $a_h = 0.17\,g$

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$$P_{s,max.} = \frac{1.20 \times 62.4 \times 12 \times 0.17}{144} = 1.06 \text{ psi}$$

Maximum Seismic Pressures due to vertical & horizontal longitudinal seismic

$$P_{s,max.} = P_{s,v} = \frac{\gamma D}{144} \cdot a_v$$

For OBE $a_v = 0.053\,g$

$$P_{s,max.} = \frac{62.4 \times 12}{144} \times 0.053 = 0.276 \text{ psi}$$

For SSE $a_v = 0.113\,g$

$$P_{s,max.} = \frac{62.4 \times 12}{144} \times 0.113 = 0.589 \text{ psi}$$

Tank Length = 23' = L

Seismic Pressure acting on tank head

$$P_{s,l} = \frac{\gamma L}{144} a_h$$

For OBE, $a_h = 0.08\,g$

$$P_{s,l} = \frac{62.4 \times 23}{144} \times .08 = 0.797 \text{ psi}$$

For SSE, $a_h = 0.17\,g$

$$P_{s,l} = \frac{62.4 \times 23}{144} \times 0.17 = 1.694 \text{ psi}$$

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7. Compression Governing Loading:
Tank Empty is critical

7.1 Hoop Stress

a. Top horizontal Load Due to over-burden & Vertical Seismic

$W_e = 4721$ lb/linear foot (Item 4)

$\Delta W_e|_{OBE} = 250$ lb/ linear foot (Item 6d)

$\Delta W_e|_{SSE} = 533$ lb/ linear foot (Item 6d)

$W_{OBE} = 4721 + 250 = 4971$ lb/lin. foot

$W_{SSE} = 4721 + 533 = 5254$ lb/ lin. foot

b. Side Vertical Load

$P_{AEh}|_{OBE} = 4243$ lb/ lin. foot

$P_{AEh}|_{SSE} = 4906$ lb / lin foot

$P_w|_{OBE} = 471$ lb/ lin. foot

$P_w|_{SSE} = 999$ lb/ lin. foot

$W_{OBE} = 4243 + 471 = 4714$ lb/lin. foot

$W_{SSE} = 4906 + 999 = 5905$ lb/lin. foot

c. Governing Loads

$W_{OBE} = 4971$ lb/lin. foot

$W_{SSE} = 5905$ lb/lin foot

d. Stresses

$$\sigma_{\theta,OBE} = \frac{4971}{24t} = \frac{4971}{24 \times .3125} = 663 \text{ psi}$$

$$\sigma_{\theta,SSE} = \frac{5905}{24 \times .3125} = 787 \text{ psi}$$

7.2 Longitudinal Stress

a. Stress due to Seismic wave

$\sigma_{a,OBE}$ = 1546 psi (Item 6a)

$\sigma_{a,SSE}$ = 3286 psi (Item 6a)

b. Stress due to external soil & underground water pressure

Ave Soil Pressure (static + Dynamic OBE)
= 354 psf (Item 6C)

Ave soil Pressure (static + Dynamic SSE)
= 409 psf (Item 6C)

Ave water Pressure $= \frac{1}{2} \gamma D = \frac{1}{2} \times 62.4 \times 12 = 374$ psf

Ave Seismic water pressure (OBE) $= \frac{471}{12} = 39$ psf (Item 6b)

Ave Seismic water pressure (SSE) $= \frac{999}{12} = 83$ psf (Item 6b)

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Total Pressure

= Soil Pressure + Water Pressure + Seismic effects

$$P_{OBE} = 354 + 374 + 39 = 767 \text{ psf}$$

$$P_{SSE} = 409 + 374 + 83 = 866 \text{ psf}$$

$$\sigma_{p,OBE} = \frac{Pr}{2t} = \frac{767 \times 72}{144 \times 2 \times .3125} = 614 \text{ psi}$$

$$\sigma_{p,SSE} = \frac{866 \times 72}{144 \times 2 \times .3125} = 693 \text{ psi}$$

c. Total Stresses

$$\sigma_{\phi,OBE} = 1546 + 614 = 2160 \text{ psi}$$

$$\sigma_{\phi,SSE} = 3286 + 693 \approx 3980 \text{ psi}$$

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Loads

1. Soil overburden pressure, We
2. Diesel oil internal pressure, P
3. Thermal loading due to $-20°F$ differential, T
4. Seismic Loads (OBE & SSE)
   a. Soil & underground water
   b. Seismic wave stresses

Loading Combinations

1. $W_e + T^{**} + OBE^{*}$
2. $W_e + T + P + OBE$
3. $W_e + T + SSE^{*}$
4. $W_e + T + P + SSE$

* Includes all seismic loading from vertical seismic plus one horizontal seismic (lateral or longitudinal)

** Tension stresses due to thermal are ignored when compression stresses are evaluated.

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Allowable Compression Stresses:

$$\left(\frac{D}{r}\right)^2 = \left\{\frac{2R}{\sqrt{\frac{bt^3}{12bt}}}\right\}^2 = \left\{4\sqrt{3}\frac{R}{t}\right\}^2 = 48\left(\frac{R}{t}\right)^2$$

D = Tank diameter

r = radius of Gyration of tank skin section

R = Tank Radius = D/2

t = tank thickness

b = unit length of tank

$$\frac{R}{t} = \frac{72''}{5/16} = 230.4$$

$$\frac{D}{r} = 230.4 \times 4\sqrt{3} = 1596$$

From Reference 5 page 91, for $\frac{D}{r} > 500$ the ultimate hoop compression in the wall for $\frac{D}{r} > 500$ is

$$\sigma_\theta = \frac{4.93 \times 10^9}{(D/r)^2} = 1935 \text{ psi}$$

Considering the use of a factor of safety of 2.0 as recommended by Reference 5, for normal operating conditions

$$\sigma_{\theta,all.} = \frac{1935}{2} = 967 \text{ psi}$$

For Safe shut down earthquake

FORM SAS-22.12 REV.2 (11-18-91)

$\sigma_{\theta, all.} = 967 \times 1.5 = 1450$ psi

From Reference 6 Article -1712.1.1 and 1511., the allowable longitudinal Stress is determined as follows:

From Article 1511

(1) $\alpha_{\phi L} = 1.52 - 0.473 \log_{10} \frac{R}{t}$

$= 1.52 - 0.473 \log_{10} 230.4 = 0.403$

(2) $M = \frac{l}{\sqrt{Rt}}$

where l = distance between lines of support

$= 19'-8'' = 236''$

$M = \frac{236}{\sqrt{72 \times \frac{5}{16}}} = 49.75 > 10$

$\alpha_{\phi L} = 0.207$ for $M > 10$

The larger of (1) & (2) above is

$\alpha_{\phi L} = 0.403$

Except that from Figure 1511-1 of Ref. 6 for steel with $\sigma_y = 30{,}000$ psi

$\alpha_{\phi L} = 0.31$ (Use)

From 1712.1.1

$\sigma_{\phi eL} = C_\phi \frac{Et}{R}$ Theortical Buckling stress

$= 0.605 \frac{\times 29 \times 10^6}{72} \times 5/16$ $C_\phi = 0.605$ for $M > 1.73$.

$= 76150$ psi

The actual buckling stress will be affected by imperfection through the use of the factor $\alpha_{\phi L} = 0.31$.

$\sigma_\phi = 76150 \times 0.31 = 23607$ psi

For normal Operating conditions (Factor of Safety = 2.0)

$\sigma_{\phi, all.} = \frac{23607}{2} = 11803$ psi

For Safe shutdown earthquak

$\sigma_{\phi, all} = 11803 \times 1.5 = 17,705$ psi

From Ref. 6 Article 1712.1.1(b).(2) and 1511, the allowable hoop stress is determined as follows:

From 1511, $\alpha_{\theta L} = 0.80$

From 1712.1.1.(b).(2)

$$\sigma_{\theta eL} = \sigma_{heL} = C_{\theta h} \frac{Et}{R}$$

$$M_\phi = \frac{l}{\sqrt{Rt}} = \frac{236'}{\sqrt{72 \times \frac{5}{16}}} = 49.75 > 3.5$$

$$1.65 \frac{R}{t} = 1.65 \times 72 \times \frac{16}{5} = 380.16 > M_\phi$$

$$C_{\theta h} = \frac{0.92}{M_\phi - 0.636} = \frac{0.92}{49.75 - 0.636}$$

$$C_{\theta h} = 0.01873$$

$$\sigma_{\theta eL} = \sigma_{heL} = 0.01873 \times \frac{29 \times 10^6}{72} \times \frac{5}{16}$$

$$= 2358 \text{ psi}$$

The actual buckling stress will be affected imperfection through the use of the factor $\alpha_{\theta L} = 0.80$.

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$$\sigma_\theta = 2358 \times 0.80$$
$$= 1886 \text{ psi}$$

This value is close to the value calculated earlier of 1935 psi. We will use σ_θ as 1886 psi since the interaction equation of Ref. 6 will be used in this calculation.

Considering a factor of safety of 2.0 for normal operating conditions

$$\sigma_{\theta, all.} = \frac{1886}{2} = 943 \text{ psi}$$

For safe shutdown earthquake, use a factor of safety of 1.34

$$\sigma_{\theta, all} = \frac{1886}{1.34} = 1407 \text{ psi}$$

FORM SAS-22.9 REV.4 (11-18-91)

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Allowable Tension Stresses:

$$\sigma_y = 30{,}000 \text{ psi}$$

Per 1986 ASME Boiler & Pressure Vessel Code Section III Division 1 Table I-7.1 the allowable stress values S = 13,700 psi

$$\sigma_{\theta,all} = \sigma_{\phi,all} = S = 13700 \text{ psi}$$

For safe shut down earthquake

$$\sigma_{\phi,all} = \sigma_{\theta,all} = 13700 \times 1.5$$

$$= 20550 \text{ psi}$$

FORM SAS-22.12 REV.2 (11-18-91)

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In addition to the individual stresses (hoop or longintudinal) satisfying the limits based on the above allowables, the interaction equation of Article 1713.1.1(b) shall also be satisfied when both stresses are in compression. This equation is statd below:

$$\frac{\sigma_{\phi S} - 0.5\,\sigma_{heL}}{\sigma_{\phi eL} - 0.5\,\sigma_{heL}} + \left(\frac{\sigma_{\theta S}}{\sigma_{heL}}\right)^2 \leq 1.0$$

In the above equation,

σ_{heL} = 2358 psi

$\sigma_{\phi eL}$ = 76 150 psi

$$\sigma_{\phi S} = \frac{\sigma_{\phi} \cdot FS}{\alpha_{\phi L}}$$

$$\sigma_{\theta S} = \frac{\sigma_{\theta} \cdot FS}{\alpha_{\theta L}}$$

$\sigma_{\phi}, \sigma_{\theta}$ are actual longitudinal & hoop stresses

FS Factor of Safety, = 2.0 for OBE,
= 1.34 for SSE

$\alpha_{\phi L}$ = 0.31

FORM SAS-22.12 REV.2 (11-18-91)

$\alpha_{\theta L} = 0.80$

For all other stress combinations of σ_ϕ & σ_θ (other than both compression), use the following criteria:

$$|\sigma_\phi| + |\sigma_\theta| \leq S'$$

where

$S' = S_m = 13{,}700$ psi for OBE conditions

$S' = 1.5S_m = 20{,}550$ psi for SSE conditions

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NO.	COMMENT	REVISION
	1. Conservative approach is taken in Calculating the combined stresses to assure the adequacy of the tank under seismic conditions.	
	2. The thermal stresses are considered only in the calculation of maximum tensile stresses. They are ignored otherwise.	
	3. In calculating tension stresses the surrounding soil pressure is ignored.	

FORM SAS-22.13 REV.0 (11-18-91)

FORM SAS-22.15 REV.0 (11-18-91)

SARGENT & LUNDY ENGINEERS	DESIGN CONTROL SUMMARY REFERENCES			PROJECT NAME: Fort Calhoun PROJECT NO.: 7751-27 CLIENT: OPPD CALC. NO.: FC06011 REV. NO.: 0 PAGE NO.: 34	REVISION
NO.	**TITLE/IDENTIFICATION**	**REVISION**	**DATE**	**TYPE**	**DESCRIPTION OF DESIGN INFORMATION**
1	"Soil Engineering", by M.G. Spangler, 2nd Edition International Textbook Co.	Second Edition	—	Text book	Chapter 24 "Loads on Underground Conduits" Calculate soil overburden load on tank
2	Updated Safety Analysis Report, Fort Calhoun Station Unit no. 1, Appendix F	Current	—	SAR	Seismic Accelerations. Seismic Forces Combination.
3	UL 58, "Standard For Steel Underground Tanks For Flammable and Combustible Liquids", Approved as ANSI/UL 58-1985, Oct. 22, 1985	8th Edition	April 15, 1986	Standard	Allowable Tank thickness
4	Seismic Analysis Committee of ASCE Nuclear Structures & Material Committee, "Seismic Response of Buried Pipes & Structural Components," 1983.	—	1983	ASCE Publication	Equations for Calculating strain in tank due to seismic waves
5	"Handbook of Steel Drainage & Highway Construction Products", Published by AISI	2nd Edition	1971		Chapter 3, Section 8.4 Compression stress in large pipe.

FORM SAS-22.15 REV.0 (11-18-91)

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NO.	**TITLE/IDENTIFICATION**	**REVISION**	**DATE**	**TYPE**	**DESCRIPTION OF DESIGN INFORMATION**	**REVISION**
6.	Cases of ASME Boiler & Pressure Vessel Code, Case N-284	—	Valid to: 7-30-92 Approved: Aug. 25, 80	Code Case	Cylindrical shell buckling stresses taking into consideration imperfection effects	
7.	"Development of Criteria for Seismic Review of selected Nuclear Power Plants", NUREG-CR-0098 prepared for the NRC, May 1978	—	May, 1978	Regulatory Guide	Particle velocity	
8	"Water Pressures on Dams During Earthquake", ASCE Transactions, Vol-98, Oct. 1933, PP418-433, Discussion PP 434-472	—	Oct. 1933	Journal Paper	Dynamic Seismic Water pressure	
9	Welding Research Council Bulletin 107/1968	3rd printing	1972	Bulletin	Local membrane stresses in Cylindrical Shell due to nozzle Loads and pressure.	
10	"Design of Earth Retaining Structures for Dynamic Loads," by H. Bolton Seed & R.V. Whitman, 1970 ASCE specialty Conference on Lateral Stresses in the Ground and Design of Earth Resisting Structures", June 22-24, 1970, Ithaca, N.Y.	—	June, 1970	Journal Paper	Soil static and dynamic pressure during seismic event.	

FORM SAS-22.16 REV.0 (11-18-91)

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NO.	SOURCE OF DESIGN INPUT			DESCRIPTION OF DESIGN INPUT	AFFECTED CALCULATIONS			VERIFICATION STATUS			REVISION
	TITLE	REVISION	ISSUE DATE		CALC. REFERENCE	REVISION	PAGE NO(s)	SOURCE	REVISION	ISSUE DATE	
			No unverified input/assumptions are used								

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1.0 EVALUATION

1. Tank thickness

Per Table 3.1 of Reference 3, the required thickness for 12001 to 20000 gallons tank with a diameter of 144" (12') is 0.302". This is the required thickness for uncoated carbon steel tank which is the case for this tank.

No corrosion allowance is considered in this section. Therefore the full thickness of 0.3125" (5/16") is used.

2. Evaluation of stresses (Tank Empty)

This case provides a check for the stress status in the tank when the diesel fuel is low and the tank is under high compression due to the surrounding soil.

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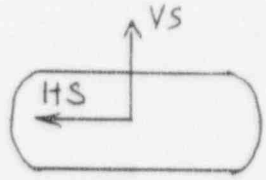
a. Tank Empty & OBE Conditions.

- Consider vertical and longitudinal seismic:



σ_θ = 663 psi (Section 7.1.d)

< 943 psi allowable o.k.

σ_ϕ = 2160 psi (Section 7.2.c)

< 11 803 psi allowable o.k.

Check Compression Interaction Equation

$\sigma_{heL} = 2358$ psi

$\sigma_{\phi eL} = 76\,150$ psi

$$\sigma_{\phi S} = \frac{\sigma_\phi \cdot FS}{\alpha_{\phi L}} = \frac{2160 \times 2}{0.31} = 13935 \text{ psi}$$

$$\sigma_{\theta S} = \frac{\sigma_\theta \cdot FS}{\alpha_{\theta L}} = \frac{663 \times 2}{0.8} = 1658 \text{ psi}$$

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$$\frac{\sigma_{\phi s} - 0.5\sigma_{heL}}{\sigma_{\phi eL} - 0.5\sigma_{heL}} + \left(\frac{\sigma_{\theta s}}{\sigma_{heL}}\right)^2$$

$$= \frac{13935 - 0.5 \times 2358}{76150 - 0.5 \times 2358} + \left(\frac{1658}{2358}\right)^2$$

$$= 0.665 < 1.0 \qquad \text{o.k.}$$

- Consider Vertical and lateral Seismic

The vertical seismic effects produce a maximum hoop stress of 663 psi. The latiral seismic effects produce a hoop stress lower than 663 psi, therefore the hoop stress stays at 663 psi. The longitudinal stress σ_ϕ is considerably lower than 2160 psi since it does not include the stress due to seismic strain. Therefore we can conclude that this case does not govern.

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b. Tank Empty & SSE Conditions:

- Consider vertical and Longitudinal Seismic

$\sigma_\theta = 787$ psi (Section 7.1-d)

< 1407 psi allowable o.k.

$\sigma_\phi = 3980$ psi (Section 7.2.C)

$< 17{,}705$ psi allowable o.k.

Check compression interaction equation:

$$\sigma_{\phi S} = \frac{\sigma_\phi . FS}{\alpha_{\phi L}} = \frac{3980 \times 1.34}{0.31} = 17204 \text{ psi}$$

$$\sigma_{\theta S} = \frac{\sigma_\theta \cdot FS}{\alpha_{\theta L}} = \frac{787 \times 1.34}{0.8} = 1318 \text{ psi}$$

$$\frac{\sigma_{\phi S} - 0.5\,\sigma_{heL}}{\sigma_{\phi eL} - 0.5\,\sigma_{heL}} + \left(\frac{\sigma_{\theta S}}{\sigma_{heL}}\right)^2$$

$$= \frac{17204 - 0.5 \times 2358}{76150 - 0.5 \times 2358} + \left(\frac{1318}{2358}\right)^2$$

$$= 0.526 < 1.0 \quad \text{o.k.}$$

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- For the same reason given in (a) above, the vertical plus lateral seismic case does not govern.

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3. Evaluation of Stresses (Tank Full)

This evaluation case primarely provides check for the maximum tensile stresses in the tank.

Include effects of $\Delta T = -20^\circ F$ in this evaluation

$$\sigma_{\phi,T} = \sigma_{\theta,T} = \alpha . \Delta T . E$$

$$= 6.5 \times 10^{-6} \times 20 \times 29 \times 10^{6}$$

$$= 3770 \text{ psi}$$

Internal liquid pressure stresses

$P_{av.} = 2.6$ psi (Section 5, DSC Analytical method)

$$\sigma_{\phi,P} = \frac{PR}{2t} = \frac{2.6 \times 72}{2 \times 0.3125} = 300 \text{ psi}$$

$$\sigma_{\theta,P} = \frac{PR}{t} = \frac{2.6 \times 72}{0.3125} = 600 \text{ psi}$$

a. OBE evaluation

Vertical plus longitudinal seismic.

- Stress due to seismic strain

$$\sigma_{\phi,sl} = 1546 \text{ psi}$$

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- Liquid additional pressure & associated stresses due to vertical seismic

$P_{s,max.} = 0.5$ psi (DCS, Analytical Methods Section 6.e)

$\sigma_\phi = \frac{0.5 \times 72}{2 \times 0.3125} = 58$ psi

$\sigma_\theta = 2\sigma_\phi = 116$ psi

- Liquid additional pressure & associated stress due to longitudinal seismic

$P_{s,max.} = 0.797$ psi (DCS, Analytical methods, Section 6.e)

$\sigma_\phi = \frac{P_{s,max.} R}{2t} = \frac{0.797 \times 72}{2 \times .3125} = 92$ psi

- Total Stresses

$\sigma_\phi = 3770 + 1546 + 300 + 58 + 92$

$= 5766$ psi

$\sigma_\theta = 3770 + 600 + 116$

$= 4486$ psi

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$$\sigma_\phi + \sigma_\theta = 5766 + 4486$$

$$= 10252 \text{ psi} < 13{,}700 \text{ psi}$$

o.k.

The vertical plus lateral seismic case does not govern.

b. SSE Evaluation (Vertical plus Longit. Seismic)

- Stress due to seismic strain

$= 3286$ psi

- Liquid additional pressure & associated stresses due to vertical seismic

$P_{s,max.} = 1.06$ psi (DCS, Analytical Methods Section 6.e)

$$\sigma_\phi = \frac{1.06 \times 72}{2 \times .3125} = 122 \text{ psi}$$

$$\sigma_\theta = 2\sigma_\phi = 244 \text{ psi}$$

- Liquid additional pressure & associated stress due to longitudinal seismic

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$P_{S,max.} = 1.694 \text{ psi}$ (DCS, Analytical methods Section 6.e)

$$\sigma_\phi = \frac{1.694 \times 72}{2 \times .3125} = 195 \text{ psi}$$

- Total stresses

$$\sigma_\phi = 3770 + 3286 + 300 + 122 + 195$$
$$= 7673 \text{ psi}$$

$$\sigma_\theta = 3770 + 600 + 244 = 4614 \text{ psi}$$

$$\sigma_\phi + \sigma_\theta = 7673 + 4614$$
$$= 12287 < 20{,}550 \text{ psi allowable}$$

- The vertical plus lateral seismic case does not govern.

2.0 Estimate of Tank Thickness Margin

Assume that tank thickness is reduced by 0.03 or 9.6%,

New $t = 0.3125 - 0.03 = 0.2825''$

From previous calculations it appears that the OBE conditions with tank assumed empty is the governing case. We will re-calculate stresses, allowables and interaction using the new thickness of 0.2625".

$$\sigma_{\theta,OBE} = 663 \times \frac{0.3125}{0.2825} = 733 \text{ psi}$$

$$\sigma_{\phi} = 1546 + \frac{767 \times 72}{144 \times 2 \times 0.2825} = 2225 \text{ psi}$$

Recalculate allowable and buckling stresses using the new thickness:

$$M = \frac{l}{\sqrt{Rt}} = \frac{236}{\sqrt{72 \times .2825}} = 52.3283 > 10$$

$\alpha_{\phi L} = 0.207$

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$$\alpha_{\phi L} = 1.52 - 0.473 \log_{10} \frac{R}{t}$$

$$= 1.52 - 0.473 \log_{10} \frac{72}{0.2825} = 0.382$$

$$\alpha_{\phi L} = 0.382$$

Except that from Figure 1511-1 of Ref. 6, for steel of $\sigma_y = 30000\,psi$, $\alpha_{\phi L} = 0.31$

Use $\alpha_{\phi L} = 0.31$

$$\sigma_{\phi eL} = C_\phi \frac{Et}{R}$$

$$= 0.605 \times \frac{29 \times 10^6 \times 0.2825}{72} = 68,840 \ psi$$

$$\sigma_\phi = \alpha_{\phi L} \cdot \sigma_{\phi eL} = 0.31 \times 68,840 = 21,340 \ psi$$

$$\sigma_{\phi, all., OBE} = \frac{\sigma_\phi}{2} = \frac{21340}{2} = 10,670 \ psi$$

$$\sigma_{\phi, all., SSE} = 10,670 \times 1.5 = 16005 \ psi$$

$$\alpha_{\theta L} = 0.8$$

$$\sigma_{\theta eL} = \sigma_{heL} = C_{\theta h} \frac{Et}{R}$$

$$M = M_\phi = 54.29$$

$$1.65 \frac{R}{t} = 1.65 \times \frac{72}{.2875} = 420.5 > M_\phi$$

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$$C_{\theta h} = \frac{0.92}{M_{\phi} - 0.636} = \frac{0.92}{52.3283 - 0.636} = 0.0178$$

$$\sigma_{\theta eL} = \sigma_{heL} = 0.0178 \times \frac{29 \times 10^{6} \times 0.2825}{72}$$

$$= 2025 \text{ psi}$$

Adjust for imperfection

$$\sigma_{\theta} = 0.8\, \sigma_{\theta eL}$$

$$= 0.8 \times 2025 = 1620 \text{ psi}$$

$$\sigma_{\theta, all., OBE} = \frac{\sigma_{\theta}}{2} = \frac{1620}{2} = 810 \text{ psi}$$

$$\sigma_{\theta, all., SSE} = \frac{\sigma_{\theta}}{1.34} = 1210 \text{ psi}$$

Actual hoop stress

$$\sigma_{\theta} = 733 \text{ psi} < 810 \text{ psi}$$

Actual longitudinal stress

$$\sigma_{\phi} = 2225 \text{ psi} < 10{,}670 \text{ psi}$$

Check iteraction equation.

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$$\sigma_{\phi S} = \frac{\sigma_{\phi} . FS}{\alpha_{\phi L}} = \frac{2225 \times 2}{0.31} = 14,355 \text{ psi}$$

$$\sigma_{\theta S} = \frac{\sigma_{\theta} . FS}{\alpha_{\theta L}} = \frac{733 \times 2}{0.8} = 1833 \text{ psi}$$

$$\frac{\sigma_{\phi S} - 0.5\,\sigma_{heL}}{\sigma_{\phi eL} - 0.5\,\sigma_{heL}} + \left(\frac{\sigma_{\theta S}}{\sigma_{heL}}\right)^2$$

$$\frac{14,355 - 0.5 \times 2025}{68,840 - 0.5 \times 2025} + \left(\frac{1833}{2025}\right)^2$$

$$= 1.016 \simeq 1.0$$

The above checks indicates that the thickness of 0.2825" is adequate.

Margin on thickness is

$$= \frac{0.3125 - 0.2825}{0.3125} \times 100 = 9.6\%$$

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3.0 FORCES AT NOZZLES DUE TO PIPING

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NOZZLE LOADS FOR PENETRATION CHECKS

To cover all Inlet, outlet and Vent pipes the following diameter pipes are evaluated

2" sch 40

1½" sch 40

3" sch 40

See Drawing No

ISO No FO-100051 1 of 1 Dated 11-18-69 ✓

ISO No FO-90061 1 of 1 Dated 10-7-69 ✓

ISO No FO-90051 1 of 1 Dated 10-6-69 ✓

ISO No FO-10100041 1 of 1 Dated 10-3-69 ✓

SEISMIC FORCES FROM PIPES INSIDE TANK

The following conservative assumptions are made

- Pipe lengths = 12' Supported at top

(If the pipes are small seismic forces will be small as peak of ground seismic response spectra is used, without freq. calculation)

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- Pipes are assumed to be filled with oil and tank allmost empty (This gives largest seismic force)

- Peak of Response spectra for 0.5% damping is used (For cantilever type of simple system it has only one mode response, hence 1.5 factor for equivalent static method is not used)

Pipe	inside Dia in	Inside Area in^2 (flow)	wt of metal lb/Ft	wt of oil* $\gamma A \times 1$	Total wt lb/Ft
1½" Sch 40	1.61	2.036	2.72	0.88	3.6
2" Sch 40	2.067	3.355	3.65	1.45	5.1
3" Sch	3.068	7.392	7.58	3.20	10.78

* Oil density [conservatively] same as water density of 62.4 lb/cft is used.

Pipe Damping = 0.5% of critical

for Design Basis Earthquake and Max^m Hypothetical Earthquake

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(From TABLE F-2 of Appendix F of USAR)

Max^m .5% damped Spectral Acceleration

	Horizontal (g)	Vertical = 2/3 Horizontal (g)
Design Basis Earthquake (DBE)	0.36	.24
Max^m Hypoth Earthquake (MHE)	0.80	.53

(From Figures F-1 an F-2 of Appendix F of USAR)

Seismic Forces Design Basis Earthquake

Pipe	Vertical Axial (lb) g W/L L	Horizontal Shear (lb)	Moment (lbF) L/2
1½" Sch. 40	.24 × 3.6 × 12 = 10. (DBE)	.36 × 3.6 × 12 = 16. (DBE)	16 × 6 = 96. (DBE)
	.53 × 3.6 × 12 = 23. (MHE)	.80 × 3.6 × 12 = 35. (MHE)	210. (MHE)
2" Sch. 40	.24 × 5.1 × 12 = 15. (DBE)	.36 × 5.1 × 12 = 22. (DBE)	22 × 6 = 132. (DBE)
	.53 × 5.1 × 12 = 33. (MHE)	.80 × 5.1 × 12 = 49 (MHE)	294 (MHE)
3" Sch. 40	.24 × 10.78 × 12 = 31. (DBE)	.36 × 10.78 × 12 = 47. (DBE)	47 × 6 = ~~279~~ 282 (DBE)
	.53 × 10.78 × 12 = 68. (MHE)	.80 × 10.75 × 12 = 103. (MHE)	618 (MHE)

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SEISMIC FORCE FROM OUTSIDE OF TANK

The Inlet and outlet pipe rise about 9" from the tank top and then they have several bends. Since the straight pipes are very close to the tank, no relative displacement is expected between the other end of the pipe and the tank penetration end. Even if one considers the seismic strain in soil, the max^{m} force which can be transmitted to the pipe is the friction force from the soil.

3" sch 40 Vent pipe

length of pipe in ground = 2.5' (Soil cover = 2.5')

horizontal soil pressure at rest

$= k_o \gamma h = .4 \times 130 \times \frac{2.5}{2}$ (at mid point

$k_o = .4$ is a reasonable value.

$p = 65 \text{ lb/sqFt} = .45 \text{ lb/in}^2$

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Friction Force per unit length of pipe

$= \mu \pi d p = .35 \times \pi \times 3.5 \times .45$ ← outside dia

$= 1.73$ lb/in ✓

Total Friction Force $= 1.73 \times \underbrace{2.5 \times 12}_{\text{length}} = 52$ lbs ✓

Let us increase this by a factor of 2 for a conservative assessment to account for uncertainities such as in the ko and μ values.

Use Axial Force of 100 lbs ✓

For 2" and 1½" (inlet, outlet pipes which have only 9" straight portion above the tank conservatively use the same length as above.

Conservatively Estimated Axial Force due to Friction:

← Diam Ratio

1½" pipe $= 100 \times \frac{1.9}{3.5} = 54$ lb ✓

2" pipe $= 100 \times \frac{2.375}{3.5} = 68$ lb ✓

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Total Nozzle Forces At penetration

From pipe inside tank + Pipe Outside tank

Design Basis Earthquake

$1\frac{1}{2}$" Pipe (sch 40)

Axial = 10 + 54 = 64 lbs. ✓

Shear = 16 lbs ✓

Moment = 96 lb Ft = 1152 lb in (Longitudinal & hoop)

2" Pipe

Axial = 15 + 68 = 83 lbs ✓

Shear = 22 lbs ✓

Moment = 132 lb Ft = 1584 lb in

3" Pipe Axial = 31 + 100 = 131 lbs ✓

Shear = 47 lbs ✓

Moment = ~~279~~ 282 x12 = ~~3348~~ 3384 lb in

Max^m Hyp. Earthquake

1½" Pipe

Axial = 23 + 54 = 77 lbs

Shear = 35 lbs

Moment = 210 lb Ft = 2520 lb in

2" Pipe

Axial = 33 + 68 = 101 lb Say 100 lbs

Shear = 49 lbs

Moment = 294 lb Ft = 3528 lb in

3" Pipe

Axial = 68 + 100 = 168 lbs

Shear = 103 lbs

Moment = 618 lb Ft = 7416 lb in

For Temperature change, the maximum Forces are the upperbound friction Force ie 54 lbs, 68 lbs and 100 lbs for 1½ inch, 2 inch and 3 inch pipes respectively.

Note that above forces are displacement

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related. Hence if the pipe connection at the tank is allowed to move, the forces will decrease. Since the tank walls are very flexible (large diameter/thickness ratio), the above forces are conservatively estimated with the assumption of restrained at the tank connection.

Form GO-3.08.1 Rev. 2

4.0 Tank Stress Check At Vicinity of Penetrations (Nozzles)

Per Drawing no. 90057 Rev-2 for Job no. 4157 issued by Eaton Metal Products Corporation, the tank has 3 - 2"ϕ Nozzles, 2 - 3"ϕ Nozzles and one 4"ϕ nozzle. The piping associated with these nozzles are 1½"ϕ, 2"ϕ, and 3"ϕ respectively.

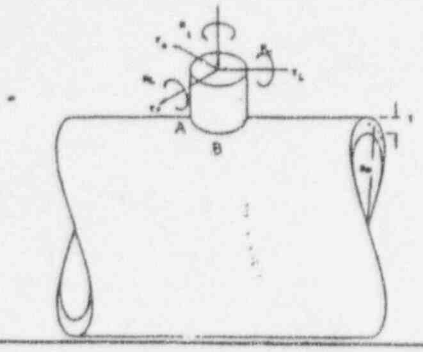
To simplify the penetrations check, a conservative approach is used as follows:

1. Check the 2"ϕ nozzle only
2. Use forces corresponding to the 3"ϕ piping
3. Obtain these forces from Appendix "A" for Maximum Hypothetical Earthquake

4. For tank pressure stresses, use the maximum σ_θ & σ_ϕ obtained for SSE condition. Mainly $\sigma_\theta = 4614$ psi, $\sigma_\phi = 7673$ psi.

Reference 9 is used to calculate the maximum local membrane stress. The allowable stress intensity of 13700 psi is used in verifying the stress acceptability. The allowable stress used is 1.5 times the stress intensity.

LOCAL MEMBRANE STRESSES IN A CYLINDRICAL SHELL DUE TO NOZZLE LOADS AND PRESSURE



Project Name: Fort Calhoun	
Project No: 7751-27	Date: 9/23/92
~~Spec.~~ Calc. No: FC06011	Page: 61
Equip. #	Pipe Line #
Equipt. Description: Diesel Fuel Tank	
Nozzle Description:	
~~Subsystem #~~	~~Code #~~	EIPSYS
Internal Pressure P_o =	Psig

APPLIED NOZZLE LOADS		
AXIAL LOAD	P = 168	lbs
SHEAR LOAD	V_c = 0	lb.
SHEAR LOAD	V_L = 103	lbs
TORSIONAL MOMENT	M_T = 0	in-lbs
CIRCUMFERENTIAL MOMENT	M_C = 0	in-lbs
LONGITUDINAL MOMENT	M_L = 7416	in-lbs

GEOMETRY		
VESSEL MEAN RADIUS	R_m = 72	in.
VESSEL THICKNESS	T = 0.3125"	in.
NOZZLE RADIUS	$r_o = \frac{2.375}{2} = 1.1875$	in.

GEOMETRIC PARAMETERS	
$\gamma = \frac{R_m}{T}$	$= \frac{72}{0.3125} = 230$
$\beta = (0.875)\frac{r_o}{R_m}$	$= .875 \times \frac{1.1875}{72} \simeq 0.015$

	MULTIPLY AND ENTER ABSOLUTE VALUE OF RESULT		STRESSES (AT LOCATIONS A,B)	
	READ CURVES FOR	CALCULATE	A	B
CIRCUMFERENTIAL STRESSES	$\frac{N_\phi}{P/R_m}$ = 40	$\frac{P}{R_m T}$ = 7.47	299	299
	$\frac{N_\phi}{M_c/R_m^2\beta}$ = 2.5	$\frac{M_c}{R_m^2 \beta T}$ =		—
	$\frac{N_\phi}{M_L/R_m^2\beta}$ = 9.0	$\frac{M_L}{R_m^2 \beta T}$ = 305.2	2861	
	Circumferential Pressure Stress, $P_o R_m/T$		4614	4614 *
	Sum of Circumferential Stresses σ_ϕ =		(7774)	(4913)
LONGITUDINAL STRESSES	$\frac{N_x}{P/R_m}$ = 45	$\frac{P}{R_m T}$ = 7.47	336	336
	$\frac{N_x}{M_c/R_m^2\beta}$ = 2.5	$\frac{M_c}{R_m^2 \beta T}$ = 305.2		—
	$\frac{N_x}{M_L/R_m^2\beta}$ = 2.5	$\frac{M_L}{R_m^2 \beta T}$ =	795	
	Longitudinal Pressure Stress, $P_o R_m/2T$		7673	7673 *
	Sum of Longitudinal Stresses σ_x =		(8804)	(8009)
SHEAR STRESSES	Due to Torsion M_T	$\frac{M_T}{2\pi r_o^2 T}$	—	—
	Due to Load V_c	$\frac{V_c}{\pi r_o T}$	—	
	Due to Load V_L	$\frac{V_L}{\pi r_o T}$		88
	Sum of Shear Stresses τ =		(0)	(88)
Maximum Stress	$= 1/2\left[(\sigma_\phi + \sigma_x) + \sqrt{(\sigma_\phi - \sigma_x)^2 + 4\tau^2}\right]$ =		8804	8011

Maximum stress should be less than or equal to 1.5S where S is the allowable stress of shell material.

* From page 45 (maximum stresses)

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$S_m = 13{,}700$ psi

$1.5 S_m = 20{,}550$ psi

Calculated maximum stress

$= 8804$ psi $<< 20{,}550$ psi

Form GQ-3.08.1 Rev. 2

M3599.001 6-90 PF01001

SARGENT & LUNDY ENGINEERS

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5.0 Sofety Check Against Tank Floatation

The tank weight = 13,937 #

The tank is seated on reinforced concrete foundation which is in turn supported by four piles. The tank is anchored to the foundation by using two steel straps of 3" x 5/8" which are connected to 1½" φ rod. The rods are connected to 1½" φ anchor rods by turnbuckles. The anchor bars are embedded in the concrete 12" vertically and 6" horizontally. See Eaton Metal Products Corporation Drawing file no. 11395-14A R1 and OPPD drawing no. 11405-S-418 Rev.6 for tanks weight and strap & and anchor details.

4/8 13

Form GQ-3.08.1 Rev. 2

M3599.001 6-90 PF01001

Anchor Allowable force :

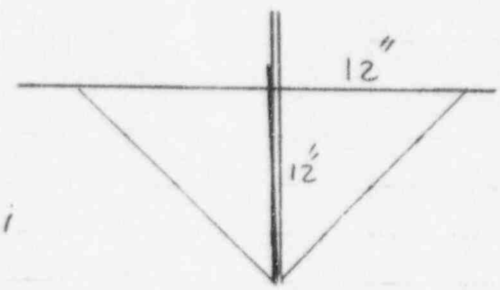
Assume A36 steel with $F_y = 36$ ksi, $F_u = 58^k$

Ultimate anchor Resistance

$= 58 (0.75^2 \pi)$

$\doteq 102.5$ kips

Determine anchor force based on Concrete pull out :



Use Concrete ultimate Strength $f_c' = 3500$ psi

$F_c = 4 \phi \sqrt{f_c'} (A_c)$

Use $\phi = 0.65$ Reduction Factor

$= 4 \times 0.65 \sqrt{3500} (12^2 \pi)$

$= 69,586$ lb $= 69.6$ kips < 102.5 kips

Use for anchor allowable force 69.6 kips each

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Total anchor resistance

$= 4 \times 69.6 = 278.4$ kips

Tank Capacity 18000 gallons

Tank Volume $\simeq \frac{18000}{7.4805} = 2406\ ft^3$

Upward force due to submerged condition

$= 2406 \times 62.4 = 150,150$ lbs.

Tank weight = 13,937 lbs

wt. of soil above tank $= 2.5' \times 23' \times 12' \times 100^{*}\ lb/ft^3$

$= 69000$ lbs

* 100 psf is used here to calculate conservative factor of safety.

Resistance Force $= 278.4 + 13.937 + 69.0$

$= 361.3$ kips

Margin $= \frac{361.3}{150.15} = 2.41$ o.k.

Form GO-3.08.1 Rev. 2

M3599.001 6-90 PF01001

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Margin of Safety against floatation $= \dfrac{361.3}{150.15} = 2.41$

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6.0 Conclusions

The tank has been evaluated in Section 1.0 for the loads described in the DCS, including the Design Basis Earthquake and the Maximum Hypothetical Earthquake. The as provided thicknes of 5/16" is used. The criteria used in the evaluation is described in the DCS. In Section 2.0, an evaluation is made to show that the thickness of 5/16" has a margin of about 10% (0.03").

In Section 3.0 the tank stresses in the vicinity of the nozzles are calculated and found to be well within the allowables. In Section 4.0 the tank safety margin against floatation was calculated to be 2.41. Based on above evaluation results, it is concluded that the tank is seismically qualified, and it

Form GQ-3.08.1 Rev. 2

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can suffer a thickness loss of 0.03"
without exceeding the acceptance criteria.

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SUMMARY OF CALCULATION FOR BLOCK WALL NO. 9

1.0 PURPOSE

The motor control centers MCC-4A2 and MCC-3C2 are located at elevation 1007'-0" adjacent to Block Wall No. 9. The portion of Wall No. 9 which may potentially interact with the motor control centers during a seismic event extends from about 12' east of row line Q to about 12' west of row line Q (see Figures 1 & 2). The purpose of this report is to summarize the evaluation for this portion of the Wall No. 9. This summary is prepared by extracting relevant information from the original Calculation No. S775305A, which includes original evaluations for several block walls, including Wall No. 9, which were performed in response to Generic Letter GL 80-11.

2.0 PHYSICAL DESCRIPTION OF THE WALL

As shown in Figure-1, Wall No. 9 runs east west and it starts from row line C and continues west to about 12' west of row line Q. It is located about 13' south of column line 8a. The wall is 6" thick and extends vertically from floor at elevation 1007'-0" to the bottom of slab at elevation 1025'-0". For a small portion of the wall, at 6'-8" above the floor level, 2'-8" of the wall height is reduced in thickness to 4" and a structural facing tile is constructed to fill the 2" gap. This arrangement runs for only 11'-2", starting at about 2' east of row line Q and extending toward row line P. Figure-5 shows the cross section of the wall within this zone.

As a result of I.E. Bulletin No. 80-11, Wall No. 9 was reinforced along with several other block walls in the area. The reinforcement included the installation of two $W10 \times 60$ lateral supports which are attached to the wall, to the ceiling, and to the reinforced concrete wall on column line 7a. Figures 2 through 4 show the plan and details of the reinforcement. In addition, the top of the wall is anchored to the slab above using 3"×3" angle and expansion anchors.

3.0 CALCULATION SUMMARY

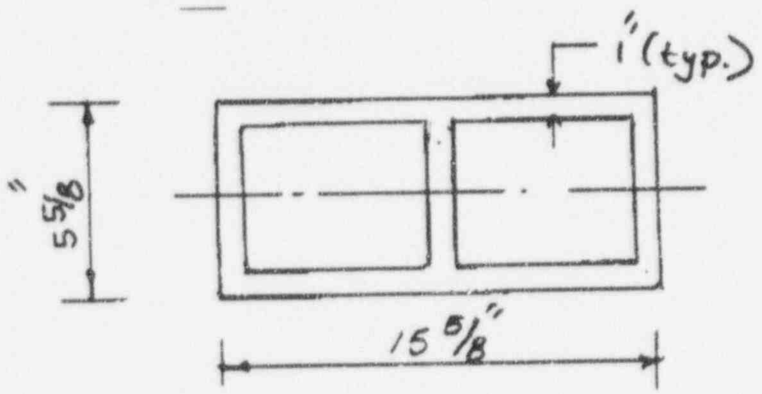
The portion of the Wall No. 9 which is adjacent to the motor control centers was analysed in calculation S775305A using two different analytical models. The portion of the wall between Wall No. 10 and row line Q was analysed as one way slab in the horizontal direction. The portion of the wall between row lines P and Q was analysed using a finite element model. The details of these analyses are summarized below.

3.1 Part of the Wall West of Row Q

The wall west of row line Q was analysed conservatively as a slab with single span and simple support condition.

Maximum horizontal span = 8'-0"=96" (see Figure 2)

Moment of Inertia of the block wall:



$$I = \frac{15.625(5.625)^3}{12} - \frac{12.625(3.625)^3}{12}$$

= 181.6 in^4 per 16" length of wall

$$I = \frac{181.6}{16} x\, 12 = 136.2\ in^4 \text{ per ft. length of wall}$$

Modulus of Elasticity, E = 2.5 x 10^6 psi (based on ACI 531-79, "Building Code Requirements for Concrete Masonry Structures," Table 10.1)

Weight of wall per square foot of the block wall

$$W_w = \frac{[2(15.625 + 3.625) + 3.625]x1"}{144} x\ 125\ pcf\ x\ \frac{12}{15.625} = 28.08\ psf$$

Adding mortar weight, conservatively assume W_w as 30.3 psf.

Additional 5 psf is assumed for wall attachments:

$W_t = 30.3 + 5 = 35.3$ psf

Column added for lateral reinforcement is W 10x60 (see Figures 2 through 4)

I_b	=	344 in^4
W_b	=	60 lb/ft = 5 lb/in
E_b	=	29×10^6 psi
ℓ_b	=	13'-3" = 159" (see Figures 3 and 4)

<u>Frequency of Wall and W-Section</u>

Wall:

$$f_w = 3.55\left[\frac{384EI}{5W\ell^4}\right]^{1/2}$$ (simply supported beam)

$$f_w = 3.55\left[\frac{384 \times 2.5 \times 10^6 \times 136.2}{5 \times \frac{35.3}{12} \times 96^4}\right]^{1/2}$$

$f_w = 36.3\ cps$

W_ Section:

$$f_b = 3.55\left[\frac{384E_bI_b}{5W_b\ell_b^4}\right]^{1/2}$$

$W_b = 35.3\ x\ 8' + 60 = 342.4\ lb/ft = 0.0285\ k/in$

$$f_b = 3.55\left[\frac{384\ x\ 29000\ x\ 344}{5\ x \cdot 0285\,(159)^4}\right]^{1/2}$$

$f_b = 23.02\ cps$

System frequency determination:

Use Dunkerley's equation:

$$\frac{1}{f^2} = \frac{1}{f_w^2} + \frac{1}{f_b^2}$$
$$\frac{1}{f^2} = \frac{1}{23.02^2} + \frac{1}{36.3^2}$$
$$f = 19.4\ cps$$

Wall Stress Computation

The ground response spectra (Fig. F-2 of Appendix F of the UFSAR) gives an acceleration of 0.17g for frequency of 19.4 cps. Per Figure F-29 of the same appendix amplification factor at elevation 1020' of the Auxiliary Building can be calculated as:

$$f_a = \frac{0.257}{0.17} = 1.512$$

Conservatively, a value of 0.2g was used for horizontal ground SSE acceleration. Using that,

$a_{SSE} = 1.512\ x\ 0.2g = 0.3024g$ will be used to qualify the wall as shown below:

$$W_s = 0.3024\ x\ 35.3 = 10.7\ psf\ /\ foot$$

For the portion of the wall under consideration, the moment and bending stress are shown below:

$$M_s = \frac{10.7\ x\ 8^2}{8} = 85.6\ lb - ft\ /\ foot$$
$$f_b = \frac{12\ x\ 85.6\ x\ 5.625/2}{136.2} = 21.2\ psi$$

The compressive strength (fm') of the mortar was determined from onsite test to be 2551 psi, therefore, based on Table 10-1 of ACI 5312-79, an allowable stress of $1.0\sqrt{f_m'} = 1.0\sqrt{2551} = 50\,psi$ is used for normal conditions and 1.5(50) = 75 psi is used for SSE conditions.

The I.C. for the 8' span of the wall is $I.C. = \dfrac{21.2}{75} = 0.28$.

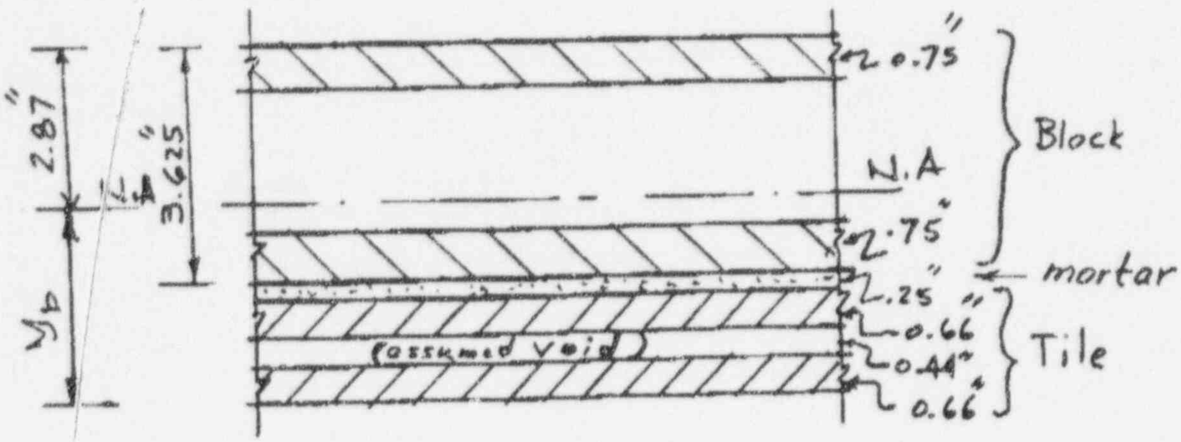
The above calculation summary shows that the portion of wall No. 9 west of Row Q is adequate under SSE conditions and the margin is $1/0.28 = 3.57$.

3.2 Part of the Wall Between Rows P and Q

The wall plan view and cross section through the portion with the tile facing are shown in Figure 5. Figure 6 shows the wall elevation including the part covered with tile, the door openings, and the perpendicular walls.

In Calculation S775305A, a finite element model was used to model this part of the wall. In this model the walls perpendicular to Wall No. 9 were modeled as lateral truss members. The truss members were limited to a height of 9'-4" which is the actual height of these walls (all walls north of Wall No. 9). The $WF10 \times 60$ columns were not included in this model. The four sides of the model were considered as simply supported. This consideration is justified since the bottom of the wall is embedded in the 2" concrete finish and the top of the wall is anchored to the slab above by 3" x 3" angle anchored to the wall and the slab. In addition, the model included the two door openings and another opening for a tray passing through the wall. Figures 7 and 8 show the truss and plate element details of the finite element model. The frequency analysis of this model resulted in a natural frequency of $20.08\,cps$. This frequency is comparable to the combined frequency ($f_f = 19.4\ cps$) calculated in section 3.1 above.

Moment of Inertia of wall with tile facing:



The ratio of the elastic modulus between the block and the tile was considered to be:

$$\frac{2 \times 10^6}{1.1 \times 10^6} = 1.818.$$

Calculate centroid of the composite section:

Area (A)	Distance to Top (y)	A.y
$0.75 \times 12 = 9$	$\frac{0.75}{2}$	3.375
$0.75 \times 12 = 9$	$3.625 - \frac{.75}{2} = 3.25$	29.250
$\frac{.66}{1.818} \times 12 = 4.356$	$3.625 + .25 + \frac{.66}{2} = 4.205$	18.319
$\frac{.66}{1.818} \times 12 = 4.356$	$3.625 + .25 + \frac{3}{2} \times .66 + .44 = 5.305$	23.111
$.25 \times 12 = 3.0$	$3.625 + \frac{.25}{2} = 3.75$	11.25
Total: 29.712		85.305

$$\bar{y} = \frac{85.305}{29.712} = 2.87''$$

$Y_b = 5.635'' - 2.87'' = 2.765''$

$$I = 2x\frac{12x.75^3}{12} + 9\left(2.87 - \frac{.75}{2}\right)^2 + 9(2.87 - 3.25)^2 + .25x12(3.75 - 2.87)^2$$
$$+ \frac{.66}{1.818}x12(3.75 + .33 - 2.87)^2 + \frac{.66}{1.818}x12(3.75 + .66 + .44 + .33 - 2.87)^2$$
$$= 94.083\, in^4 / foot$$

Block wall section modulus, $S_w = \dfrac{94.083}{2.87} = 32.8\, in^3 / foot$

Tile section modulus,

$$S_t = \frac{94.083}{2.765}x1.818$$
$$= 61.9\, in^3 / foot$$
$$\cong 62\, in^3 / foot$$

The allowable stress in the block perpendicular to the bed joints was considered as $25psi \times 1.3 = 32.5psi$. The $25psi$ is obtained as half the allowable for the parallel to the bed joints based on Table 10.1 of ACI 531-79, and the 1.3 is an SSE increase factor.

The maximum design moments in the 6" block wall obtained from the finite element analysis were:

- M_{yy} = 635 in-lb/foot
- M_{xx} = 913 in-lb/foot
- M_{xy} = 399 in-lb/foot

Conservatively add M_{xy} to M_{xx} and M_{yy} to obtain the design moments in horizontal and vertical directions, respectively:

$M_h = M_{xx} + M_{xy} = 913 + 399 = 1312$ in-lb/foot
$M_v = M_{yy} + M_{xy} = 635 + 399 = 1034$

Then, for the direction parallel to the bed joint:

$$f_{b1} = \frac{1312 \ x\ 5.625}{136.2 \ x\ 2} = 27.1\ psi < 75\ psi\ SSE\ allowable$$

and, in the direction perpendicular to the bed joint:

$$f_{b2} = \frac{1034 \ x\ 5.625}{136.2 \ x\ 2} = 21.4\ psi < 32.5\ psi\ SSE\ allowable$$

The maximum design moments in the composite portion of the block wall computed using the finite element analysis were:

- $M_{yy} = 627$ in-lb/foot
- $M_{xx} = 872$ in-lb/foot
- $M_{xy} = 130$ in-lb/foot

$M_h = (M_{xx} + M_{xy}) = (872 + 130) = 1002$ in-lb/foot
$M_v = (M_{yy} + M_{xy}) = (627 + 130) = 757$ in-lb/foot

These moments were increased by the factor 1.05 to account for variation of elastic modulus in the tile and block: The final design moments used in the evaluation were:

$M_h = 1.05 \times 1002 = 1052$ in-lb/foot
$M_v = 1.05 \times 757 = 795$ in-lb/foot

Stresses in structural tile:

$f_{b1} = \frac{1052}{62.0} = 17.0\ psi \langle 41^{*}\ psi$ (parallel to the bed joint)

$f_{b2} = \frac{795}{62.0} = 12.8\ psi \langle 17.8^{*}\ psi$ (perpendicular to the bed joint)

- * See allowable stresses calculated below. Tile SSE allowables were calculated based upon a mortar strength of 750 psi:

  $f'_{b2} = .5\sqrt{750}\ x\ 1.3 = 17.8\ psi$ for stress perpendicular to bed joint.

  $f'_{b1} = 1.0\sqrt{750}\ (1.5) = 41\ psi$ for stress parallel to bed joint.

Stresses in block wall:

$f_{b1} = \frac{1052}{32.8} = 32.1\ psi \langle 75\ psi$ (parallel to the bed joint)

$f_{b2} = \frac{795}{32.8} = 24.2\ psi \langle 32.5\ psi$ (perpendicular to the bed joint)

Based on the above, the maximum I.C. for the wall is $\frac{24.2}{32.5} = 0.74$, which is the result of flexural stress in the 4" thick part of the wall perpendicular to the bed joints. Therefore, there is a minimum margin of 1/0.74 = 1.35 in this wall.

3.3 Review of Seismic Stresses in Steel Column W10 x 60

Seismic load/unit length of column = contributory dead load per unit length of column x seismic acceleration

Using 0.0285 k/in for the column contributory dead load (see page 3), and 0.3024g for SSE acceleration (see page 4),

Seismic Load/in = 0.0285* x 0.3024 = 0.00862 k/in

$$M = \frac{\omega \ell^2}{8} = \frac{.00862(159)^2}{8} = 27.24\ in\text{-k}$$

$S = 66.7\ in^3$ for W10 x 60 column

$$\text{Stress} = \frac{27.24}{66.7} = 0.408\ ksi \quad \text{(very small)}$$

Steel column has a very high safety margin.

4.0 CONCLUSION

The calculation shows that the wall is qualified for design basis earthquake using a conservative approach.

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Ft. Calhoun Station
Auxiliary Building
Safety-Related Masonry Wall:

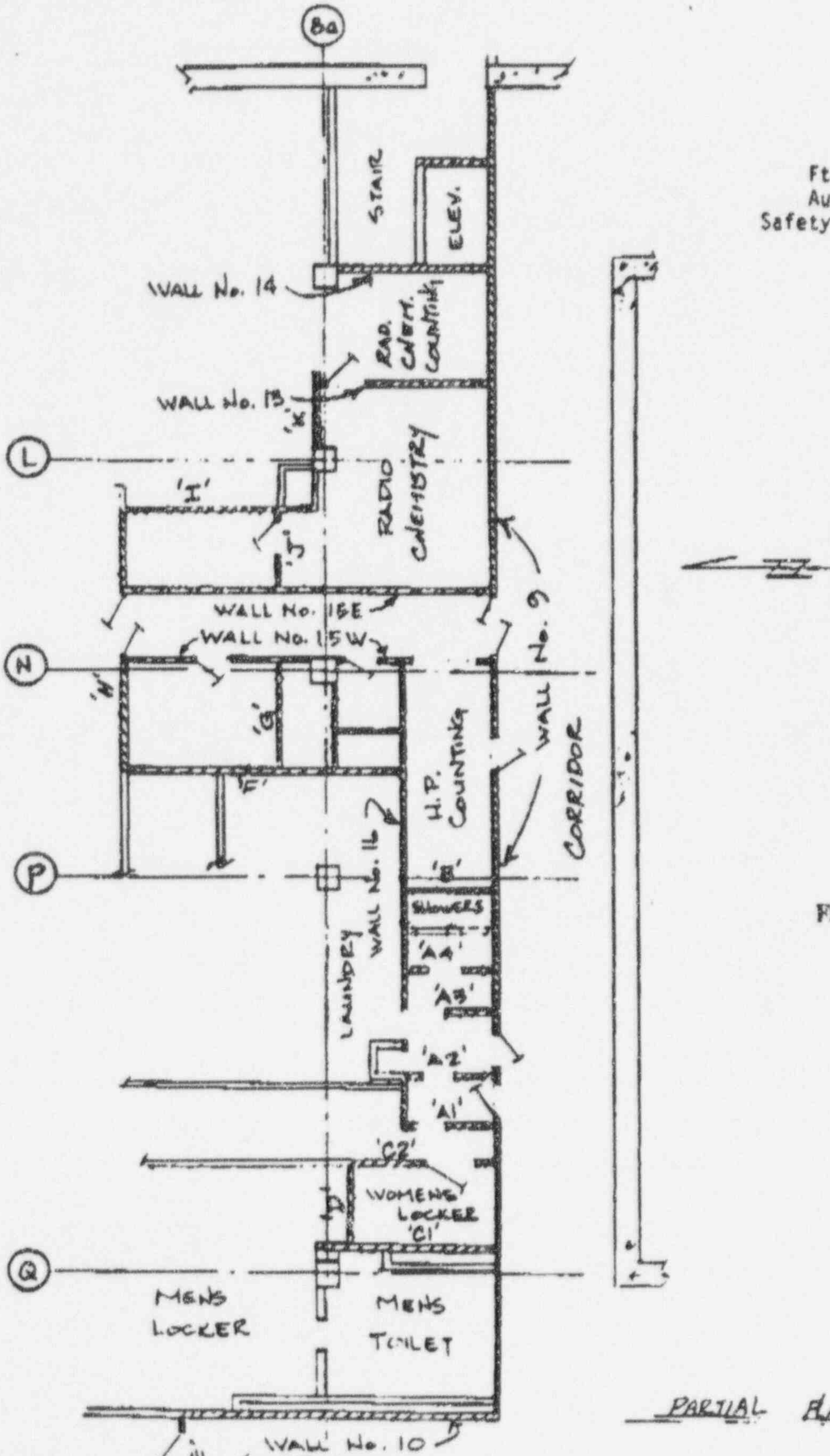


Figure - 1

PARTIAL PLANS AT EL 1007'-0"

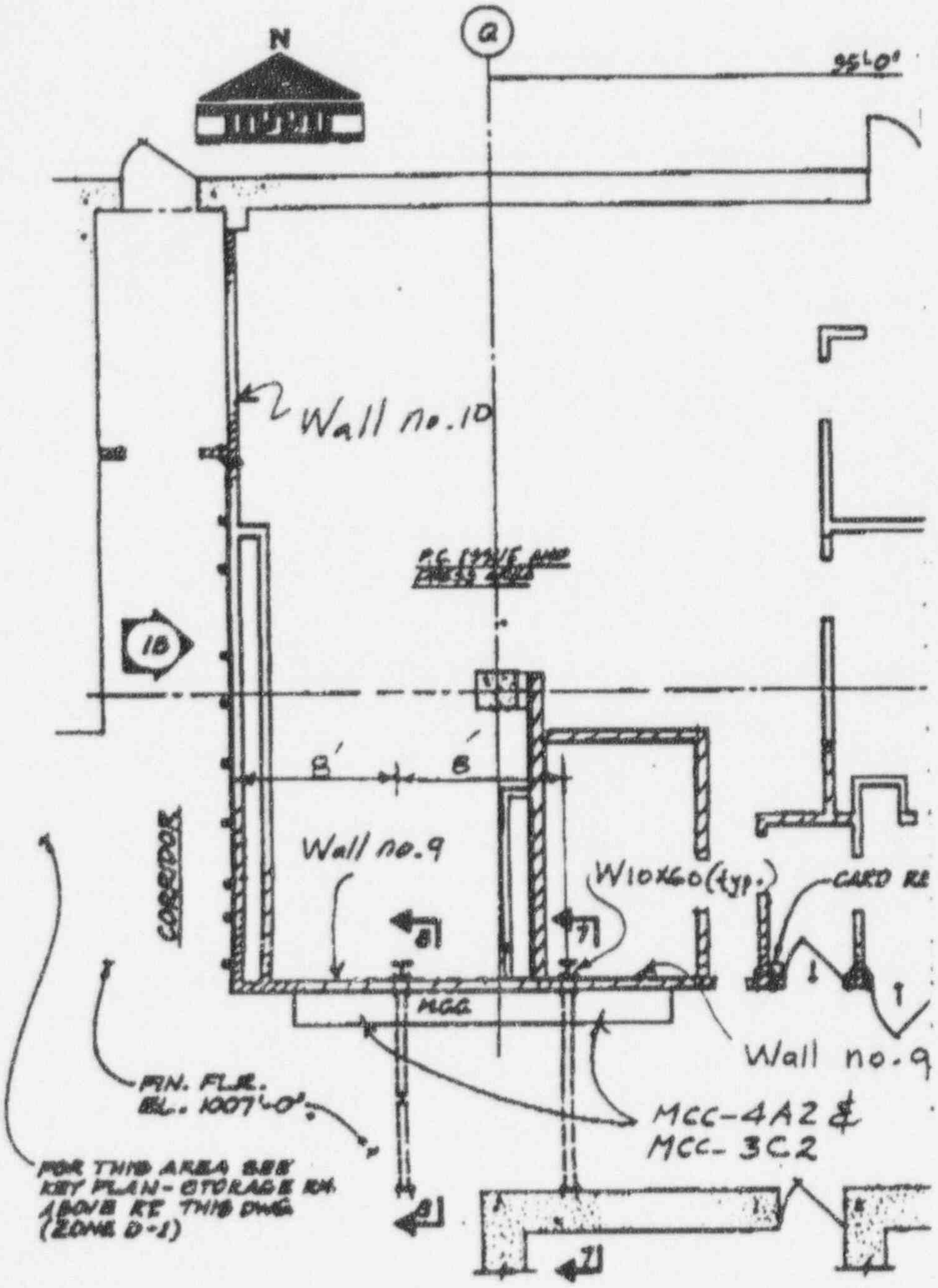

N
Q
Wall no.10
CORRIDOR
8'
8'
Wall no.9
W10x60(typ.)
CARD RE
MCC
Wall no.9
FIN. FLR.
EL. 1007'-0"
MCC-4A2 &
MCC-3C2
FOR THIS AREA SEE
KEY PLAN-STORAGE RM.
ABOVE RE THIS DWG.
(ZONE D-1)

Figure - 2

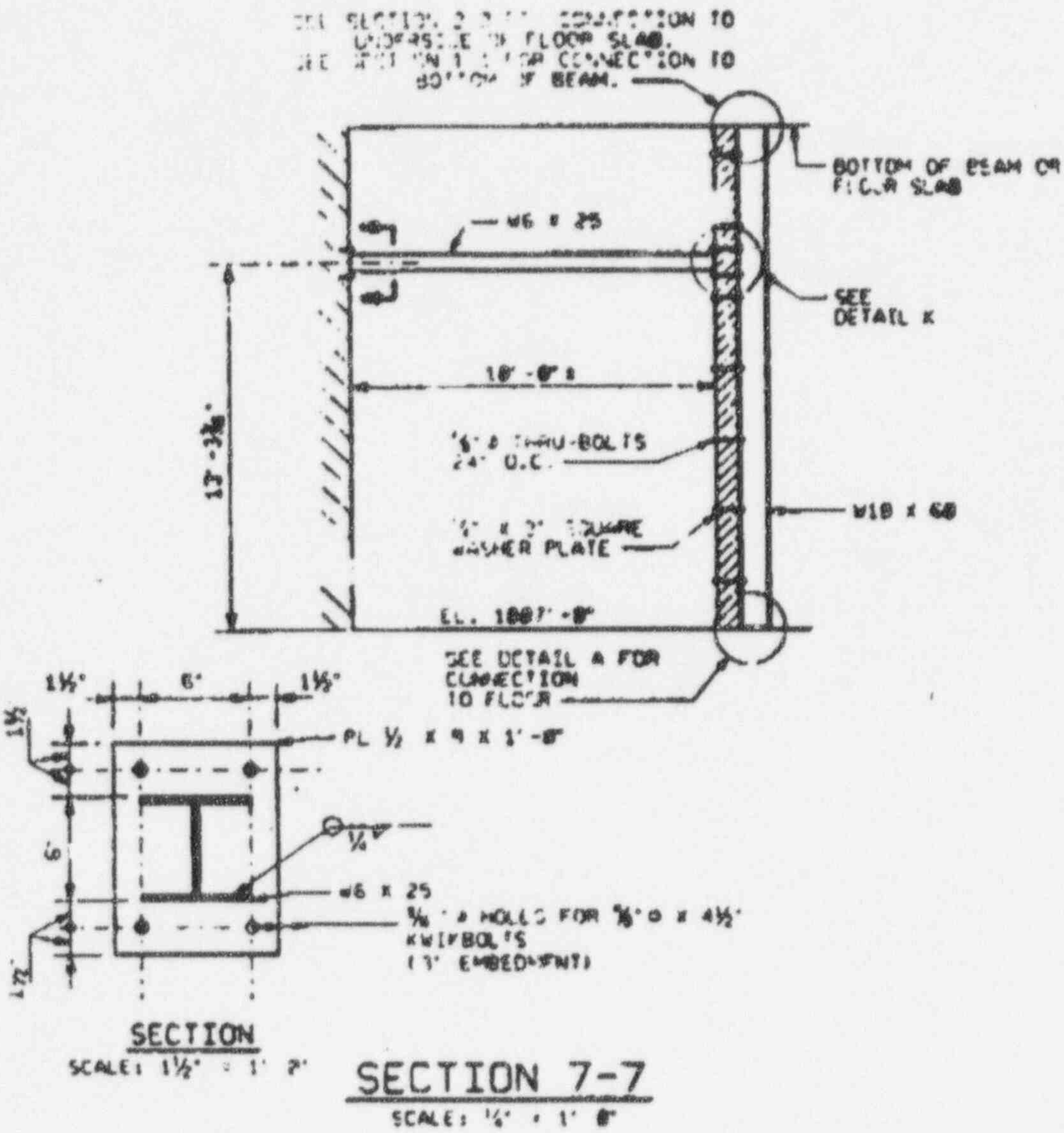

BOTTOM OF BEAM OR FLOOR SLAB
W6 x 25
SEE DETAIL K
18'-0"
W18 x 60
EL. 1007'-0"
SEE DETAIL A FOR CONNECTION TO FLOOR
PL ½ x 9 x 1'-0"
W6 x 25
SECTION
SECTION 7-7

Figure - 3

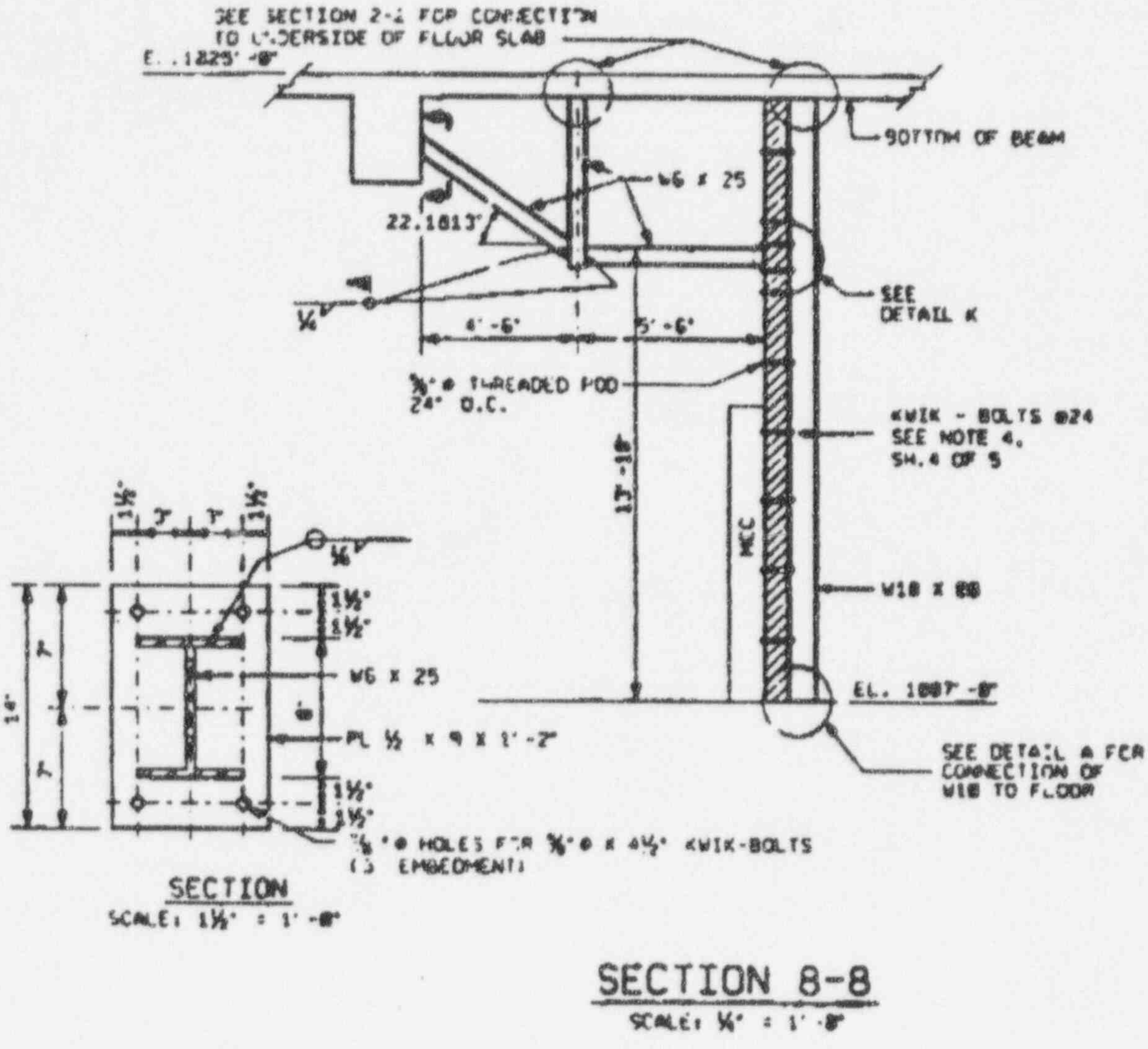
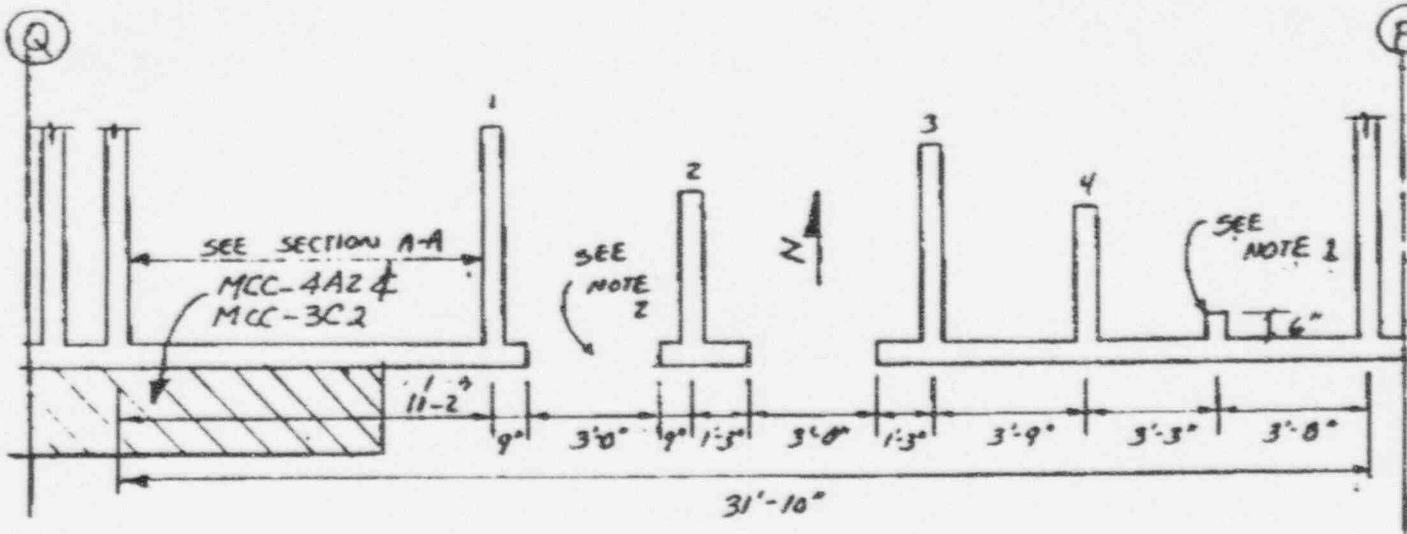
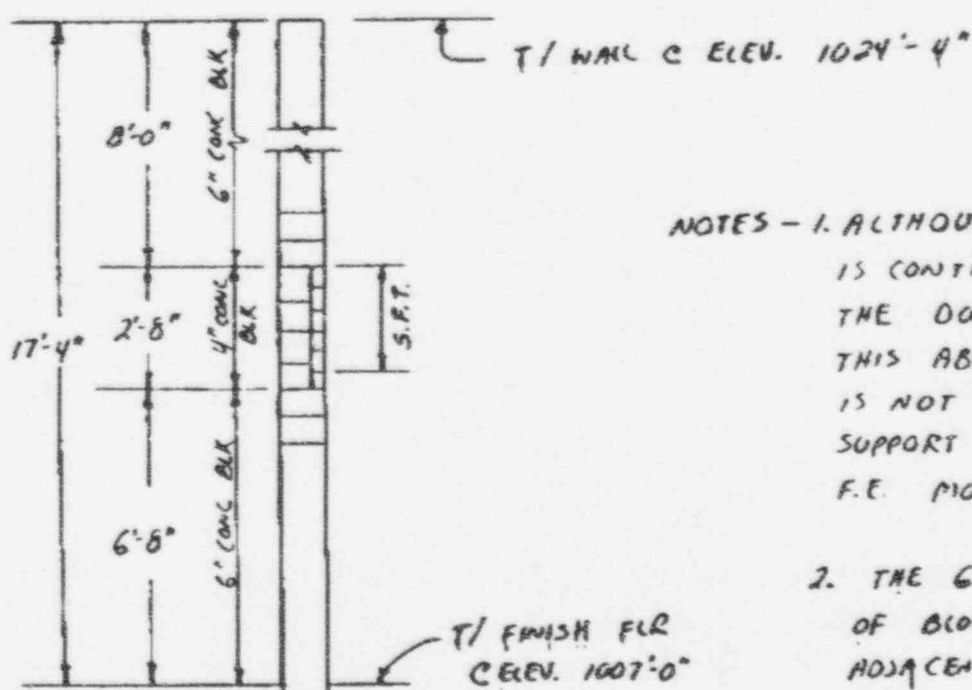

SEE SECTION 2-2 FOR CONNECTION
TO UNDERSIDE OF FLOOR SLAB
EL. 1825'-0"
BOTTOM OF BEAM
W6 x 25
22.1813°
1/4"
SEE
DETAIL K
4'-6"
5'-6"
5/8"⌀ THREADED ROD
24" O.C.
KWIK - BOLTS @24
SEE NOTE 4,
SH.4 OF 5
MCC
W18 X 88
EL. 1807'-0"
SEE DETAIL A FOR
CONNECTION OF
W18 TO FLOOR
1 1/2"
5"
5/16"
W6 x 25
PL 1/2 x 9 x 1'-2"
5/8"⌀ HOLES FOR 5/8"⌀ x 4 1/2" KWIK-BOLTS
(3" EMBEDMENT)
SECTION
SCALE: 1 1/2" = 1'-0"
SECTION 8-8
SCALE: 1/4" = 1'-0"

Figure - 4

THE FOLLOWING IS THE LAYOUT OF WALL ? BETWEEN P & Q:





SECTION A-A

NOTES – 1. ALTHOUGH THE WALL IS CONTINUOUS ABOVE THE DOOR OPENING THIS ABUTTING SECTION IS NOT CONSIDERED A SUPPORT POINT IN THE F.E. MODEL.

2. THE 6" WIDE SECTIONS OF BLOCKS DIRECTLY ADJACENT TO THE DOOR OPENING WILL NOT BE MODELED.

Figure - 5

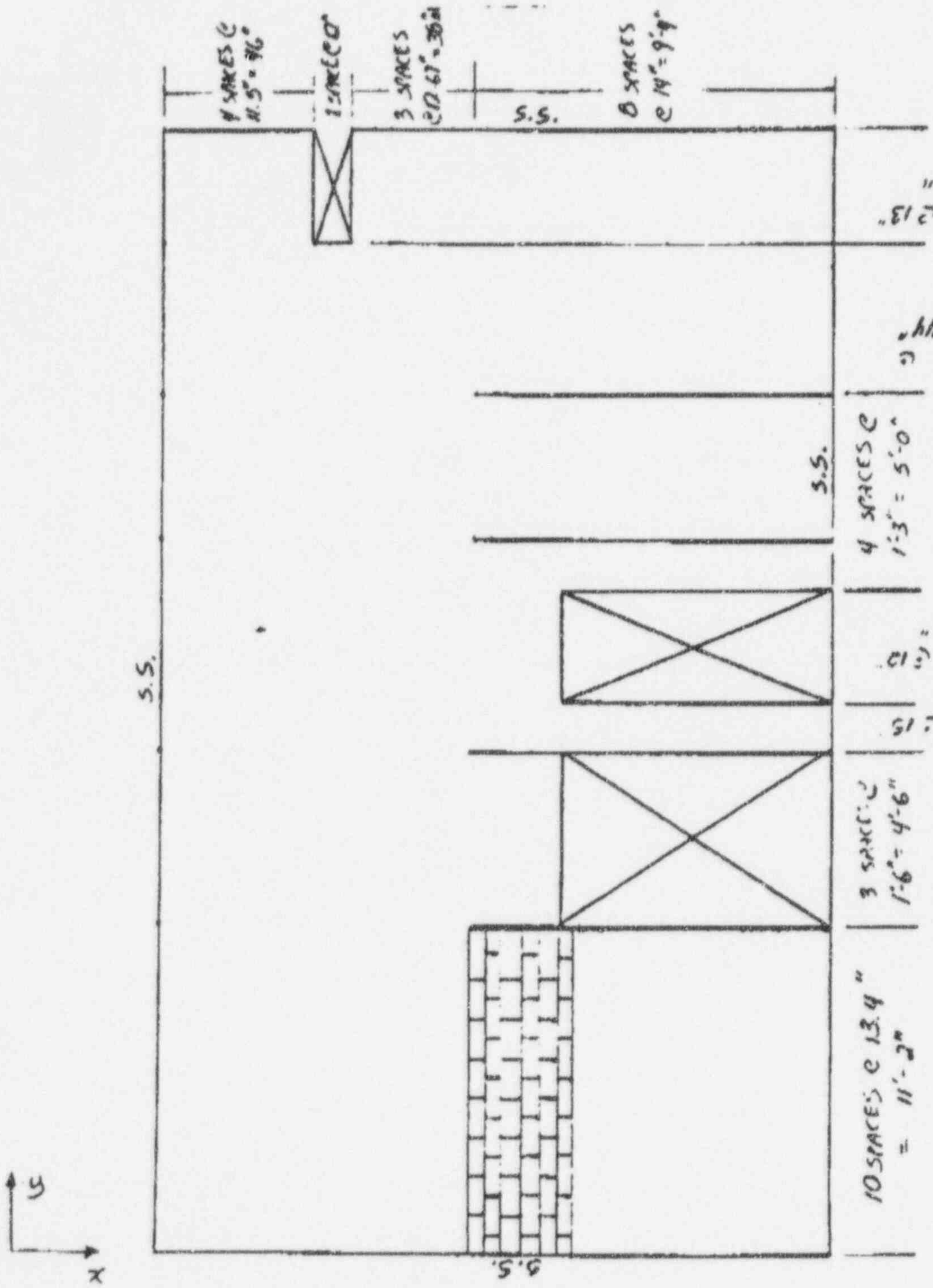

3 SPACES @ 13" = 3'-3"
4 spaces @ 11" = 44"
4 SPACES @ 1'-3" = 5'-0"
1 SPACE @ 15
3 SPACES @ 1'-6" = 4'-6"
10 SPACES @ 13.4" = 11'-2"
8 SPACES @ 14" = 9'-4"
S.S.
y
x

Figure - 6

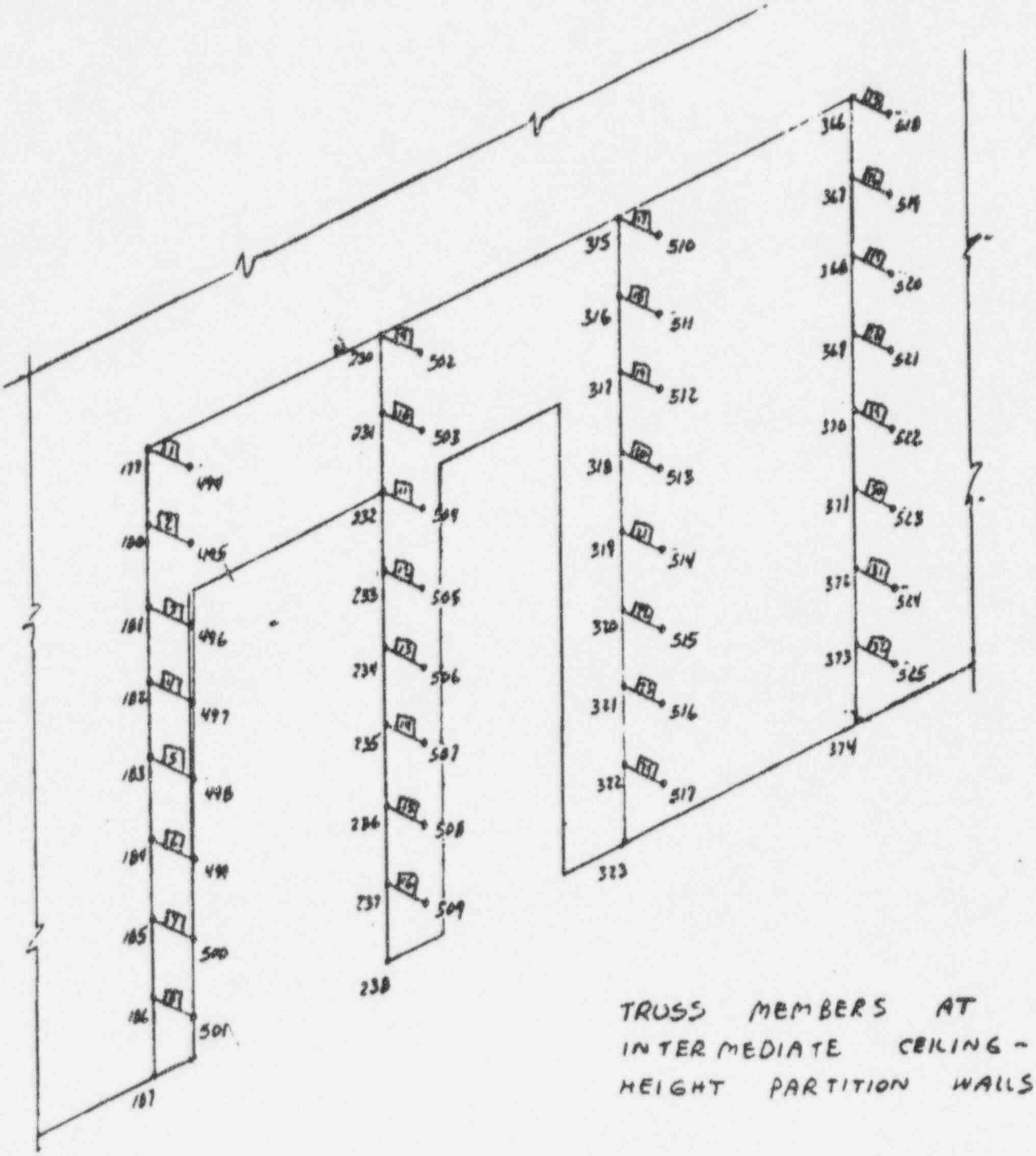

TRUSS MEMBERS AT INTERMEDIATE CEILING-HEIGHT PARTITION WALLS

Figure - 7

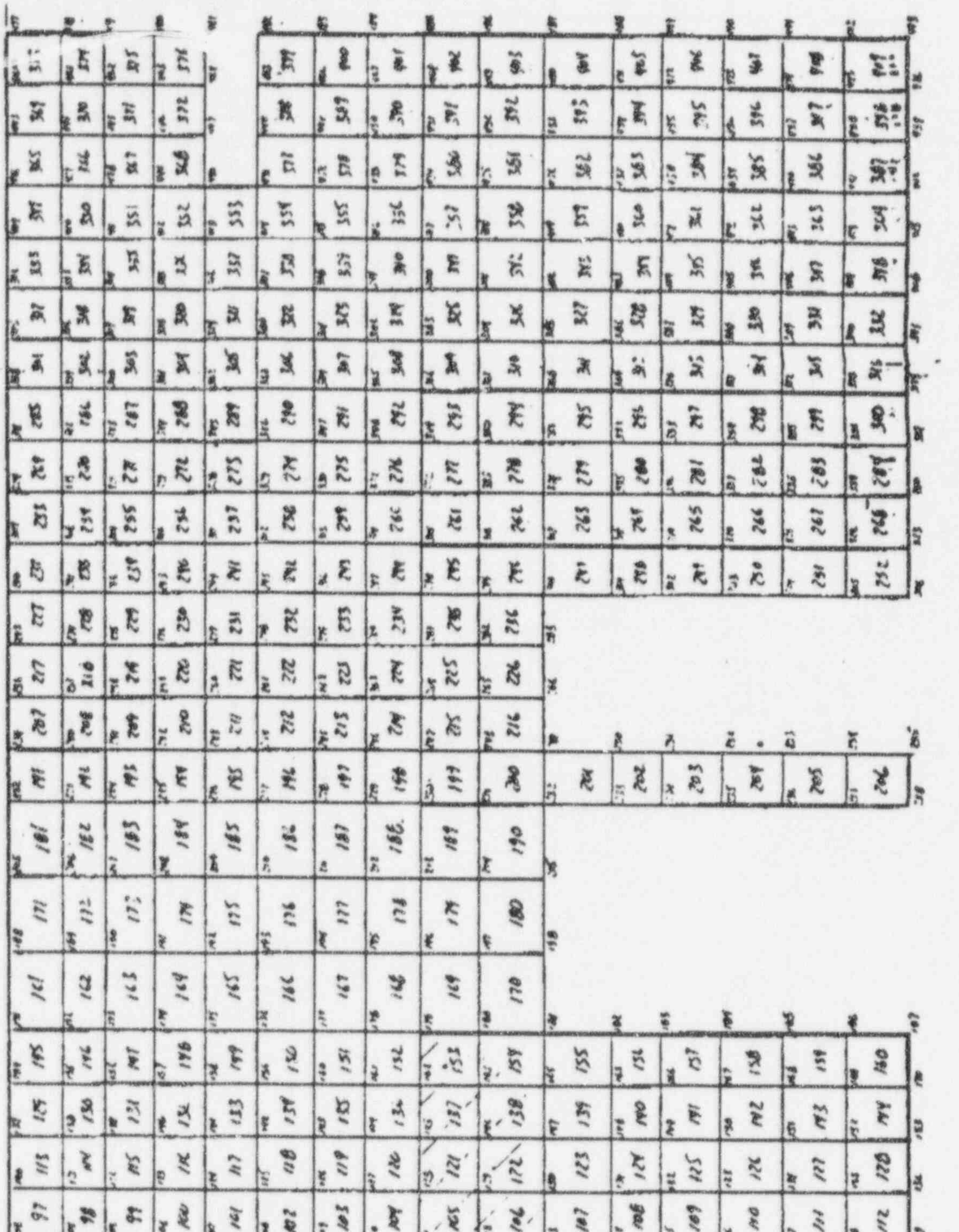


Figure - 8

LIC-96-0121

ATTACHMENT 6

Question 16

Summary of FCS Review of USI A-46
Safe Shutdown Path and SSEL R/5

#16

From: J.K. Mathew x6652 (FC 2-4 Admn.) JKM 9/12/94

To: G. E. Guliani x6025 (FC 3-1 Trg.)

J. F. Friedrichsen x6827 (FC 1-2 Plant)

J. D. Kecy x6794 (FC 1-1 Plant)

W. O. Weber x7280
Per (K. C Holthaus x7275) (FC 2-4 Admn.)

R.F. Mehaffey x6505 (FC 2-4 Admn.)

Subject: Review of USI A-46 Safe Shutdown Path and SSEL R/5

The purpose of this transmittal is to request the documentation of the acceptance of the Seismic Safe Shutdown Path, Safe shutdown Equipment List and Associated Relay List Report, Revision- 5, generated by Vectra (Impell). Please forward this report to the next review team member after you have placed your signature.

The Generic Implementation Procedure (GIP) was generated by Seismic Qualification Utilities Group (SQUG), and has been endorsed by the NRC in their SER as an acceptable method of resolving the Unresolved Safety Issue (USI) A-46. OPPD in their response to the Generic Letter (GL) 87-02, Supplement 1, committed to use the GIP methodology to resolve USI A-46 at Fort Calhoun Station. The GIP requires that the plant operations department review the SSEL to confirm its compatibility with the plant normal and emergency operating procedures. Section 3.7 and 3.8 of the GIP which delineate this requirement is attached for your information.

Over the last two years, during the development of the SSEL, as part of the SQUG technical review team you have participated in the reviews which meets the intent of the GIP plant operations department reviews. All the comments provided have been incorporated in the various revisions to the SSEL. The various review cycles including the last one with your initials are documented in Attachment G.

Please document your review and concurrence to the incorporation of all your comments on the SSEL by placing your signature and date next your name. This review page will be added to the Attachment-G

G. E. Guliani (OPERATIONS TRG.) [signature] / 9/15/94

J. F. Friedrichsen (SYSTEMS ENGG.) [signature] / 10-27-94

J. D. Kecy (PLANT OPERATIONS) [signature] / 1-3-95

W. O. Weber ~~K. C Holthaus~~ (NUCLEAR ENGG.) [signature] / 11/12/94

R.F. Mehaffey (E/I&C ENGG.) R F Mehaffey / 9/22/94

J. K. Mathew (MECHANICAL ENGG.) J.K. Mathew / 9/12/94

- Print out the Screening and Verification Data Sheets (SVDSs). (The SVDSs are described in Section 4.)

Use of a computer data base management program is optional.

3.7 OPERATIONS DEPARTMENT REVIEW OF SSEL

The Safe Shutdown Equipment List (SSEL) generated for resolution of USI A-46 should be reviewed for compatibility with the plant procedures for shutting down the plant. The purpose of this section is to provide suggested methods for performing this review by the plant's Operations Department. Note that the individuals performing this review should be familiar with the General Criteria and Governing Assumptions contained in Section 3.2 and the Scope of Equipment for the USI A-46 program contained in Section 3.3.

A review of the SSEL by a representative of the plant's Operations Department is required to confirm compatibility with the plant normal and emergency operating procedures. The intent of the Operations Department review of the SSEL is to verify that a trained operator, following existing plant procedures, will eventually be directed to the use of the safe shutdown equipment and instruments even though the operator may have first tried to shut down using equipment not included in the USI A-46 SSEL. It is <u>not</u> the intent that the operator be directed to use the USI A-46 shutdown path as his first priority or to change the symptom-based emergency operating procedures. Rather, this review is to ensure that the shutdown path selected for USI A-46 and included in the SSEL is a legitimate safe shutdown path consistent with plant procedures and operator training.

One method of reviewing the SSEL against the plant operating procedures is to do a "desk top" review of the applicable procedures. Using this method, the normal and emergency operating procedures are reviewed by an

experienced Operations Department representative to check whether all equipment called out in the operating procedures for the selected path are included on the SSEL. This review should also verify that there are no paths from which an operator could not recover with the selected set of SSEL equipment. For those steps in the procedure which rely upon operator training (i.e., steps which only give an overview summary of the actions to be taken; detailed steps are omitted), the reviewer should mentally walk through the actions an operator would take and verify that all the equipment needed is on the SSEL.

Another method of reviewing the SSEL against the plant operating procedures is to use a simulator. A loss of offsite power could then be simulated. An operator could go through this simulated transient and be observed and/or interviewed to determine whether any problems are encountered.

Another method of reviewing the SSEL against the plant operating procedures is to perform a limited control room walkdown in which an operator talks and walks through a plant shutdown following a postulated loss of offsite power. This could include not only the actions taken by the operator in the control room, but also operator actions taken in the plant where the equipment is operated from a local control panel or station.

The Operations Department of the plant should decide which of these approaches or combination of approaches would best accomplish the review of the SSEL against the plant's operating procedures.

3.8 DOCUMENTATION

A summary of the systems selected for shutting down the plant following a Safe Shutdown Earthquake (SSE) and the basis for selecting those systems should be documented. This summary can be similar to the generic summaries contained in Appendix A for PWRs or BWRs.

The scope of the equipment included on the Safe Shutdown Equipment List (SSEL) for each of the systems used to shut down the plant should be identified; this can be done using marked-up schematic drawings (P&IDs, electrical one-lines, etc.).

The Safe Shutdown Equipment Lists (SSELs) should be retained along with any special explanations for including or excluding certain items of equipment.

The method used by the plant's Operations Department to verify the compatibility of the SSELs with the plant operating procedures should be documented.

Section 9 summarizes the type of documentation which should be generated and that which should be included in the report submitted to the NRC.