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PROGRAM SEISMIC REVIEW.OVERSIZE DRAWINGS ENCL (1)

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FROM: US NRC/TIDC/Distribution Services Branch

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LEON D. WHITE, JR.
VICE PRESIDENT

TELEPHONE
AREA CODE 716 546-2700

May 22, 1979

Director of Nuclear Reactor Regulation
Attention: Mr. Dennis L. Ziemann, Chief
Operating Reactors Branch No. 2
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Systematic Evaluation Program Seismic Review
R.E. Ginna Nuclear Power Plant
Docket No. 50-244

Dear Mr. Ziemann:

During the site visit by the NRC Seismic Review Team, members of the Team requested that we provide additional information on the design and construction of Ginna Station. By letter dated May 7, 1979 we provided some of the requested information. The enclosures to this letter provide additional information.

The items which are addressed and an index to the enclosures is as follows:

		Enclosure	Page
I.	<u>Seismic Analysis and Design Criteria</u> The following information provides responses to functions in item I		
	• Calculations of fundamental frequency of containment internal concrete structure	I	11-11B
	• Dynamic models, design loads (dead load and equipment loads), lumped masses, and member stiffness properties for the following structures	See footnote (1)	
	a. containment shell and containment internal structures	I	12-15
	b. control building	I	16-19
	c. auxiliary building	See footnote (2)	
	d. service building	I	20-26
	e. intermediate building	"	"
	f. turbine building	"	"
	g. diesel generator building	"	"

Dist Per J. Shea

7905300474

None 8/8

		Enclosure	Page
	• Typical calculations showing load path through the structure - Auxiliary building	I	27-42
	• Description of Category I structural foundations	I	82-83
	• Summary of design check on column lines F and 3	I	107-110
II.	<u>Seismic Input to Equipment</u>	I	84-84
III.	<u>Misc. Design and Analysis Topics</u>		
	1. Design of structural masonary walls/ reinforcement details	See footnote (3)	
	2. Design calculations of containment wall and steel liner	I	61-81
	3. Buried service water piping	II	1-3
IV.	<u>Seismic Clarification of Mechanical and Equipment</u>		
	• System component support calculations	I	43-60
	a. pressurizer		
	b. accumulator		
	c. steam generator		
	d. reactor coolant pump		

Footnote (1) Dynamic models are not available for structures from d. through g.
(2) Submitted to NRC 5/7/77.
(3) This information is still being assembled.

As requested by your Staff, eight copies of this letter and the enclosures are being supplied for your use. If there are any questions regarding this material, please contact us.

Very truly yours,

L D White, Jr.

L. D. White, Jr.

LDW:np
Enclosure

[illegible]

ENCLOSURE I

50-244
Ltr 5-22-79
7905300474

"Re Systematic Evaluation
Program Seismic Review.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36·1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE 11

SUBJECT: SEISMIC EVALUATION - CONTAINMENT INTERIOR STRUCTURES

ORIGINATOR: D. BISS DATE 4/16/79

VERIFIER: PCShah DATE 4-30-79

ATTACHED SHEET 1:36·1/11A PRESENT THE CALCULATION PERFORMED TO OBTAIN THE NATURAL PERIOD OF VIBRATION OF THE STRUCTURE. THE DIMESIAN, Z = 28.56' IS THE DISTANCE TO THE STRUCTURE'S CENTER OF GRAVITY WITH RESPECT TO THE BASE (EL 235'-8"). WEIGHT OF MAJOR COMPONENTS (ie STM. GEN, REACTOR VESSEL ECT) WERE CONSIDERED IN ADDITION TO THE STRUCTURE'S MASS FOR DETERMINING THE CENTER OF GRAVITY.

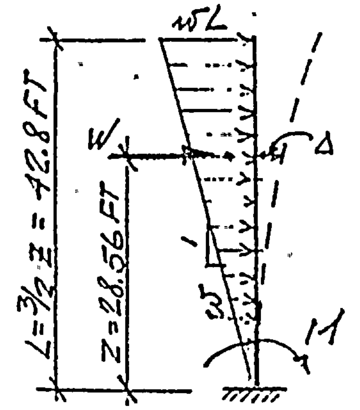
The first fundamental frequency of the internal concrete structure is 30.7 CPS, as results from the 0.0326 seconds period originally calculated.

NEXT SHEET IS 11A

FILING CODE

Natural Period of Simple Harmonic Motion.

Assume a triangular massdistribution as shown



$W = 18{,}940^K$

$\bar{z} = 28.56$ FT

$I_{min} = 376{,}000\ FT^4$

$E = 3.2(10^3)$ KSI

$g = 387$ in/sec²

$W = \frac{1}{2}(wL)L$

$w = \frac{2W}{L^2}$

$$\Delta x = \frac{w}{120EI}\left[L^4(15x - 11L) - x^4(5L - x)\right] \qquad x = \frac{L}{3}$$

$$\Delta x = \frac{2W}{120EIL^2}\left[L^4(-6L) - \left(\frac{L}{3}\right)^4\left(\frac{14L}{3}\right)\right]$$

$$\Delta x = \frac{WL^3}{60EI}\left(-6 - \frac{14}{243}\right) = -\frac{WL^3}{60EI}\left(\frac{1472}{243}\right)$$

$$\Delta x = -\frac{18{,}940(42.8)^3}{60(3.2)(10^3)(376{,}000)}\left(\frac{1472}{243}\right)\left(\frac{1}{12}\right) = \underline{-0.0104 \text{ in}}$$

$$T = 2\pi\sqrt{\frac{\Delta}{g}} = 2\pi\sqrt{\frac{0.0104}{387}} = 0.0326 \text{ sec}$$

For 0.2g groundmotion with 2% damping, the maxim responce is = 20%, or same acceleration of the structure as the ground.

Then $M = W\bar{z}(0.2) = 18{,}940(28.56)(0.2) = 108{,}000$ K-FT

About the x'-axis:

$$f_x' = -\frac{18{,}940}{1431} \pm \frac{108{,}000}{17{,}400} = -13.2 \pm 6.2 \ ; \quad \underline{f_{x_1}' = -19.4\ K/FT^2}, \quad \underline{f_{x_2}' = -7.0\ K/FT^2}$$

About the y'-axis:

$$f_y' = -13.2 \pm \frac{108{,}000}{12{,}750} = -13.2 \pm 8.5 \ ; \quad \underline{f_{y_1}' = -21.7\ K/FT^2}, \quad \underline{f_{y_2}' = -4.7\ K/FT^2}$$

Investigate possible uplift:

$$f_y' = -13.2 \pm \frac{108{,}000}{8{,}850} = -13.2 \pm 12.2 \quad \therefore \ \underline{\text{No uplift!}}$$

$V = 18940(0.2) = 3788^K$

Shear deformation was considered and incorporated in the frequency determination of the containment shell as described in the FSAR on pages 5.1.2-17, -17a, and -17b.

The shear deformation was not considered in the original frequency determination calculation of the internal concrete structure. The subsequent SAP IV dynamic analysis does include shear stiffness, and the first fundamental frequency of the internal concrete was calculated to be 16.5 cps and a corresponding period of 0.0607 sec. From FSAR Figure 5.1.2-8, the spectral acceleration is 0.23g at a period of 0.06 sec. This value of acceleration compares with the value of 0.20g obtained at the 0.0326 sec. period originally calculated.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO. 0414	FILING CODE 1:36.1
	PROJECT NAME: GINNA STATION	W.O. NUMBER 04-4824-000	PAGE 12

SUBJECT: SEISMIC EVALUATION - CONTAINMENT SHELL & INTERIOR STR. MODEL

ORIGINATOR: D. BISS
DATE 4/19/79
VERIFIER: PCAlush
DATE 4-30-79

THE FOLLOWING THREE (3) PAGES PRESENT THE MODELS DEVELOPED TO GENERATE FLOOR RESPONSE SPECTRA FOR THE INTERIOR STRUCTURES AND CONTAINMENT SHELL. THE SPRING ELEMENT BETWEEN THE CRANE SUPPORT STRUCTURE AND CONTAINMENT SHELL IS REPRESENTATIVE OF NEOPRENE BUMPERS WHICH LIMIT DEFLECTIONS OF THE OVER HEAD CRANE SUPPORT STRUCTURE.

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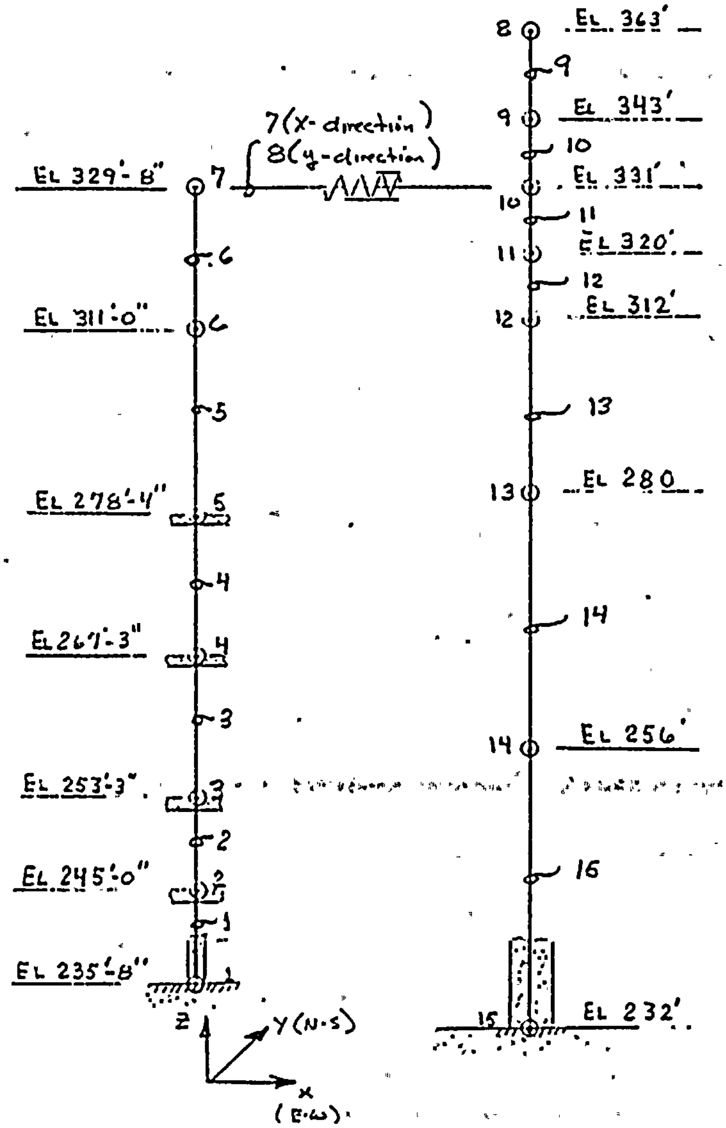
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE: 13

SUBJECT: SEISMIC EVALUATION - CONTAINMENT SHELL & INTERIOR STR. MODEL

ORIGINATOR: D. BISS DATE: 4/19/79
VERIFIER: [illegible] DATE: 4-30-79



INTERIOR STRUCTURE & OVERHEAD CRANE

CONTAINMENT SHELL

O MASS POINT

| STIFFNESS MEMBER

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1.36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04-4824-000	PAGE: 14

SUBJECT: Seismic Evaluation - Containment Shell & Interior Str. Model

ORIGINATOR: D. Biss DATE 4/19/79

VERIFIER: [illegible] DATE 4-30-79

Nodal Point	Description	Coordinates (Ft) X	Y	Z	Mass $k\text{-}sec^2/ft$
1	Base, Int. Str.	0.0	0.0	236.	—
2	Int. Str.	0.0	0.0	245.	128.
3	Int. Str	0.0	0.0	253.	88.5
4	Int. Str	0.0	0.0	267.	122.
5	Int. Str	0.0	0.0	278.	166.8
6	mid height over-head crane	0.0	0.0	311.	3.8
7	over head crane str	0.0	0.0	330.	12.4
8	containment shell	0.0	0.0	363.	81.5
9	containment shell	0.0	0.0	343.	11.6.
10	containment shell	0.0	0.0	330.	64.4
11	containment shell	0.0	0.0	320.	53.1
12	containment shell	0.0	0.0	312.	112.
13	containment shell	0.0	0.0	290.	156.5
14	containment shell	0.0	0.0	256.	134.
15	Base cont. shell	0.0	0.0	232.	—

NODAL MASS & GEOMETRY

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Member	Material	A_x ft^2	A_y ft^2	A_z ft^2	I_x ft^4	I_y ft^4	I_z ft^4
1	Conc.	768.	691.7	1460.7	658,567	438,253	1,000,000
2	Conc	715.8	567.3	1284.1	769,546	450,248	1,000,000
3	Conc	793.7	650.5	1445.2	854,666	608,108	1,000,000
4	Conc	768.1	657.1	1426.2	900,837	560,265	1,000,000
5	Steel	2.83	3.11	5.94	23.27	10.64	1,000
6	Steel	2.83	3.11	5.94	23.27	10.64	1,000
9	Conc	390.	390.	780.	1,011,000	1,011,000	2,022,000
10	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000
11	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000
12	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000
13	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000
14	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000
15	Conc	600.	600.	1200.	1,757,000	1,757,000	3,514,000

Member 7 & 8 are neoprene bumpers
– k = 24,000 k/ft

MEMBER PROPERTIES

NOTE: All properties tabulated are referenced to the global coordinate system.

GILBERT ASSOCIATES, INC.
ENGINEERS AND CONSULTANTS
READING, PA.

SUBJECT: Seismic Evaluation – Containment Shell & Interior Str. Model

DEPARTMENT NAME: Structural Engineering
DEPT. NO. 0414
FILING CODE 1:36.1
PROJECT NAME: Ginna Station
W.O. NUMBER 04 4824 000
PAGE 15
ORIGINATOR D. Biss
DATE 4/19/79
VERIFIER [signature]
DATE 4-30-79

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE: 16

SUBJECT: SEISMIC EVALUATION - CONTROL BUILDING MODEL

ORIGINATOR: D. BISS DATE 4/19/79

VERIFIER: F Paard DATE 4.30.79

THE FOLLOWING PAGES PRESENT THE MODEL DEVELOPED FOR GENERATION OF FLOOR RESPONSE SPECTRA WITHIN THE CONTROL BUILDING COMPLEX. The CONTROL BUILDING COMPLEX WAS ASSUMED TO BE TOTALLY ISOLATED FROM THE TURBINE BUILDING STRUCTURE FOR THIS ANALYSIS.

STIFFNESS PROPERTIES REPRESENTATIVE OF THE STRUCTURAL ELEMENTS WERE OBTAINED BY EVALUATING THE CONCRETE SHEAR WALLS OF THE STRUCTURE. EQUIVALENT SOIL SPRINGS WERE BASED ON THE SHEAR MODULUS OF THE FOUNDATION MATERIAL, POISSON'S RATIO AND DIMENSIONS OF FOUNDATION ELEMENTS.

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	PROJECT NAME: Ginna Station	W.O. NUMBER: 04-4824-000	PAGE: 17

SUBJECT: Seismic Evaluation - Control Building Model

ORIGINATOR: D. Biss
DATE: 4/19/79
VERIFIER: [illegible] Tirard
DATE: 4.30.79

Mass Node	Coordinate (Ft) X (E-W)	Y (N-S)	Z (vert)	Weight k	Mass $\frac{k\text{-sec}^2}{ft}$
20	27.16	16.45	250.0	1532.2	47.584
30	24.12	16.52	269.75	949.5	29.488
40	17.6	16.73	289.25	855.7	26.573
50	23.24	17.60	309.00	972.8	30.21

Note: Mass 50 includes snow load of 40 psf

Lumped Masses

Member	Member Coordinates X (E-W)	Y (N-S)	Z (Vertical) Start	End
1	27.16	16.45	250.0	250.0
2	26.37	0.892	250.0	269.75
3	1.113	0.3586	269.75	289.25
4	1.267	0.3939	289.25	309.0

Structural Geometry

Note: All other nodes are massless and connected to mass nodes with "rigid" beams.

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MEMBER	MATERIAL	A_x ft²	A_y ft²	A_z ft²	I_x ft⁴	I_y ft⁴	I_z ft⁴
2	CONCRETE	85.0	132.7	217.7	36,915	110,031	126,854
3	CONCRETE	87.5	63.6	151.1	20,082	47,373	39,731
4	CONCRETE	87.5	63.6	151.1	20,082	47,373	39,731

Member 1 IS THE SOIL SPRING ELEMENT WITH THE FOLLOWING STIFFNESS:

$k_x = 2342.9 \times 10^3$ k/ft

$k_y = 2342.9 \times 10^3$ k/ft

$k_z = 6355.2 \times 10^3$ k/ft

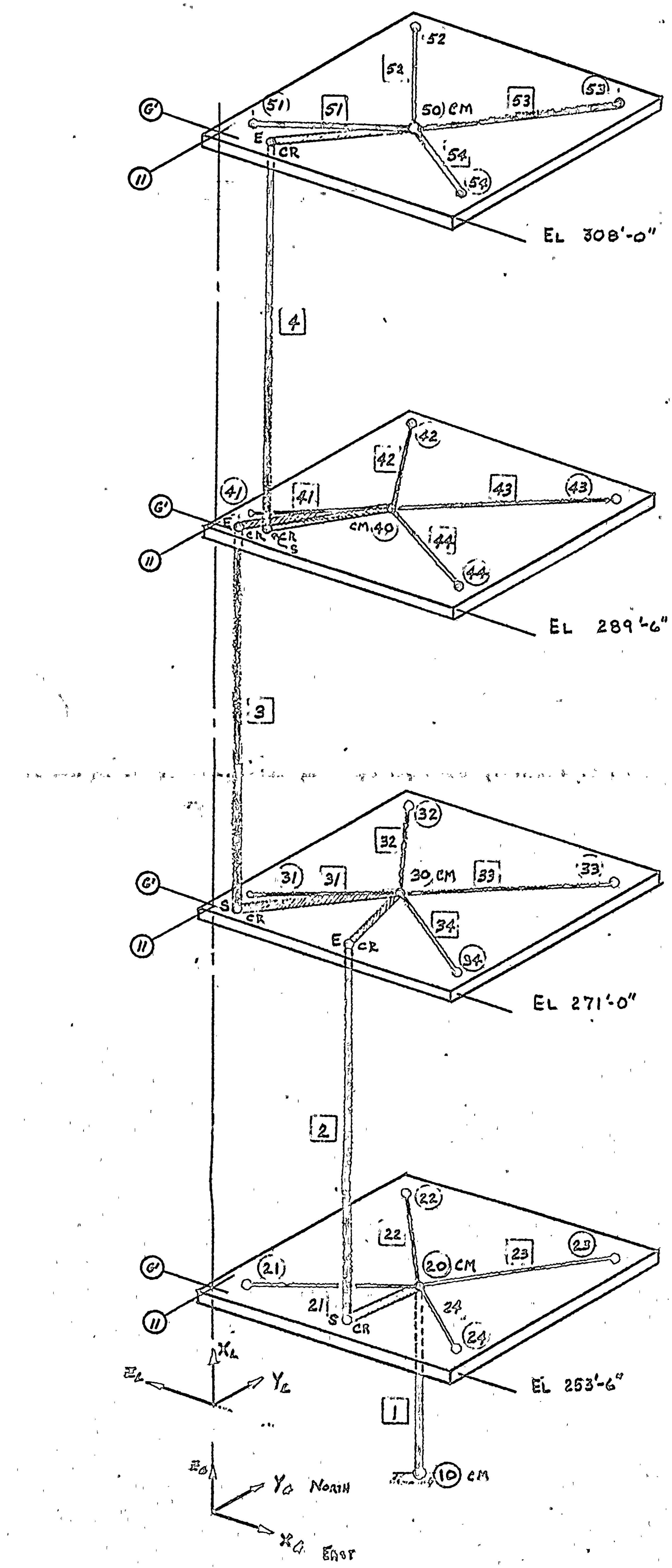
$k\phi_x = 1,361,572. \times 10^3$ k-ft/rad

$k\phi_y = 3,192,889. \times 10^3$ k-ft/rad

$k\psi_z = 2,591,911 \times 10^3$ k-ft/rad

MEMBER PROPERTIES

NOTE: All properties tabulated are referenced to the global coordinate system.



SEISMIC ANALYSIS MODEL

HORIZONTAL EXCITATION

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WORK ORDER

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REV. CH. APP. DATE

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	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04-4824-000	PAGE: 20

SUBJECT: SEISMIC EVALUATION

ORIGINATOR: D. BISS
DATE: 4/20/79
VERIFIER: [illegible]
DATE: 5.1.79

THE FOLLOWING TABLES PRESENT DISTRIBUTED MASS INFORMATION FOR THE SERVICE, INTERMEDIATE TURBINE, DIESEL GENERATOR AND CONTROL BUILDING. TABULATED INFORMATION ACCOUNTS FOR THE STRUCTURAL FLOOR SYSTEM AND EQUIPMENT SUPPORTED ABOVE AND/OR BELOW THE FLOOR. MASSES REPRESENTATIVE OF STEEL COLUMNS, PERIMETER AND PARTITION WALLS WERE NOT INCLUDED IN THE TABULATED VALUES.

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GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1:36·1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04 4824 000	PAGE: 21

SUBJECT: Seismic Evaluation · Service Building

ORIGINATOR: D. Biss
DATE: 4/20/79
VERIFIER: [illegible signature]
DATE: 5.2.79

Level	Structural Dead Weight	Estimated Permanent Equip Weight
EL 253	190 #/ft²	50 psf
EL 271	70 #/ft²	45 psf
EL 297'4" (Roof)	15 #/ft²	15 psf

Distributed Masses

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PROPRIETARY INFORMATION OF GILBERT ASSOCIATES, INC. – FOR INTERNAL USE ONLY

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	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE: 22

SUBJECT: SEISMIC EVALUATION - INTERMEDIATE BUILDING

ORIGINATOR: D. BISS
DATE: 4/20/79
VERIFIER: [signature]
DATE: 5.2.79

LEVEL	STRUCTURAL DEAD WEIGHT	ESTIMATED PERMANENT EQUIP. WEIGHT
EL 253'-9"	175 #/ft²	35 #/ft²
EL 271'	75 #/ft²	15 #/ft²
EL 278'-4"	75 #/ft²	50 #/ft²
EL 293'	75 #/ft²	30 #/ft²
EL 298'-4"	75 #/ft²	20 #/ft²
EL 315'-4"	75 #/ft²	10 #/ft²
EL 318' Low Roof	15 #/ft²	5 #/ft²
EL. 336'-4" High Roof	15 #/ft²	5 #/ft²

DISTRIBUTED MASSES

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1.36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04 4824 000	PAGE: 23

SUBJECT: Seismic Evaluation - Turbine Building

ORIGINATOR: D. Biss
DATE 4/20/79
VERIFIER: [signature]
DATE 5.3.79

Level	Structural Dead Weight	Estimated Permanent Equip. Weight
EL 253	675 #/ft²	25 #/ft²
EL 271	95 #/ft²	190 #/ft²
EL 289'-6"	140 #/ft²	35 #/ft²
EL 330'-0" OVER HEAD CRANE	125T/25T - 120' span	—
EL 354'-11" Roof	20 #/ft²	—

DISTRIBUTED MASSES

NOTE:
THE ABOVE DATA DOES NOT INCLUDE THE TURBINE GENERATOR FOUNDATION NOR THE WEIGHT OF THE TURBINE GENERATOR.

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO. 0414	FILING CODE 1:36·1
	PROJECT NAME: GINNA STATION	W.O. NUMBER 04 4824 000	PAGE 24

SUBJECT: SEISMIC EVALUATION - DIESEL GENERATOR BLDG

ORIGINATOR: D. BISS

DATE 4/23/79

VERIFIER: [signature]

DATE 5.3.79

LEVEL	STRUCTURAL DEAD WEIGHT	ESTIMATED PERMANENT EQUIP WEIGHT
EL 253	310 #/ft²	150 #/ft²
EL 275'-10" (Roof)	20 #/ft²	5 #/ft²

DISTRIBUTED MASSES

NOTE:

STRUCTURAL DEAD WEIGHT AT ELEVATION 253 INCLUDES WEIGHT OF FOUNDATIONS AND CABLE VAULTS BELOW THE STRUCTURAL SLAB. WEIGHT OF THE DIESEL GENERATOR FOUNDATIONS AND THE DIESEL GENERATORS ARE NOT INCLUDED IN THE DEAD WEIGHT FIGURES.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04-4824-000	PAGE: 25

SUBJECT: Seismic Evaluation - Control Building

ORIGINATOR: D. Biss
DATE: 4/27/79
VERIFIER: F. Prard
DATE: 5.1.79

Level	Structural Dead Load	Estimated Permanent Equip. Weight
El 253'-6"	540 #/ft²	30 #/ft²
El 271'	100 #/ft²	25 #/ft²
El 289'-6"	95 #/ft²	55 #/ft²
El 308'	31.5 #/ft²	10 #/ft²

Distributed Masses

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO. 0414	FILING CODE 1:36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER 04-4824-000	PAGE 26

SUBJECT: Seismic Evaluation

ORIGINATOR: D. Biss
DATE: 4/20/79
VERIFIER: [illegible]
DATE: 5.2.79

The following unit weights are estimated weights representative of walls and partitions found through out the plant structure

Description	Weight
12" Block Walls	80 #/ft²
Metal Siding & Insulation	2 #/ft²
Metal Siding, Insul. & Girt System	7 #/ft²

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME STRUCTURAL ENGINEERING	DEPT. NO. 0414	FILING CODE 1.36.1
	PROJECT NAME GINNA STATION	W.O. NUMBER 04 4824 000	PAGE 27

SUBJECT	
SEISMIC EVALUATION	ORIGINATOR D. BISS DATE 5/2/79 VERIFIER [illegible] DATE 5.7.79

THE FOLLOWING PAGES PRESENT TYPICAL CALCULATIONS FOR THE AUXILIARY BUILDING SUPERSTRUCTURE. SUFFICIENT INFORMATION IS PRESENTED FOR THE LOAD PATHS THROUGH THE STRUCTURE TO BE IDENTIFIED.

FILING CODE

0511

BROOKWOOD	READING PENNA	GILBERT ASSOCIATES INC — ENGINEERS AND CONSULTANTS — NEW YORK WASHINGTON
AUX. BUILDING STL DSN	MADE: G	SHEET: 42
	REV.	W.O. 4155 — DATE 7-26-

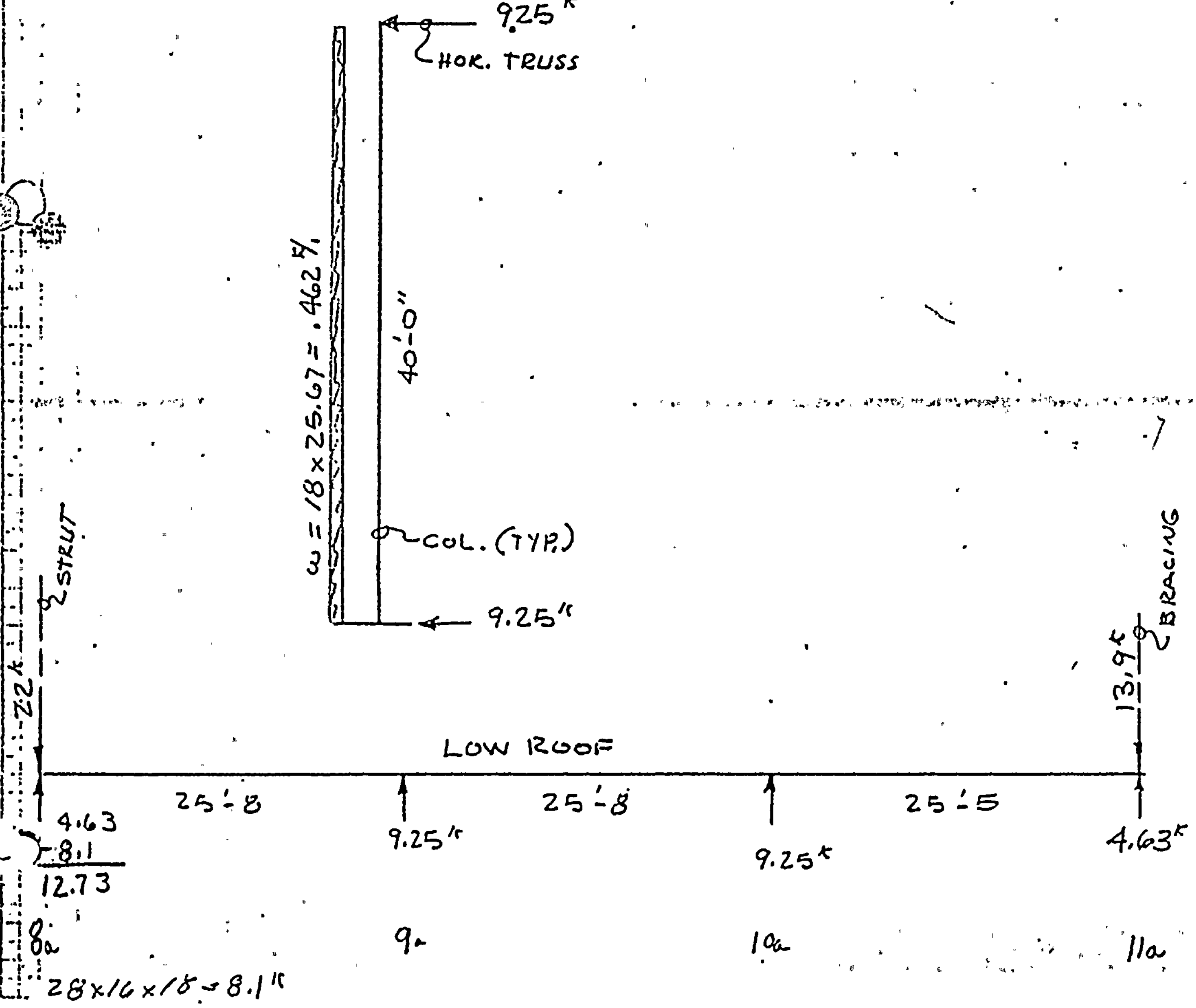
1:36.1/28

LOW ROOF - WIND BRACING -

N.Y. CODE C-304-4

HGT.	PSF
0-15'	12
16-25'	15
26-40'	18
41-60'	21

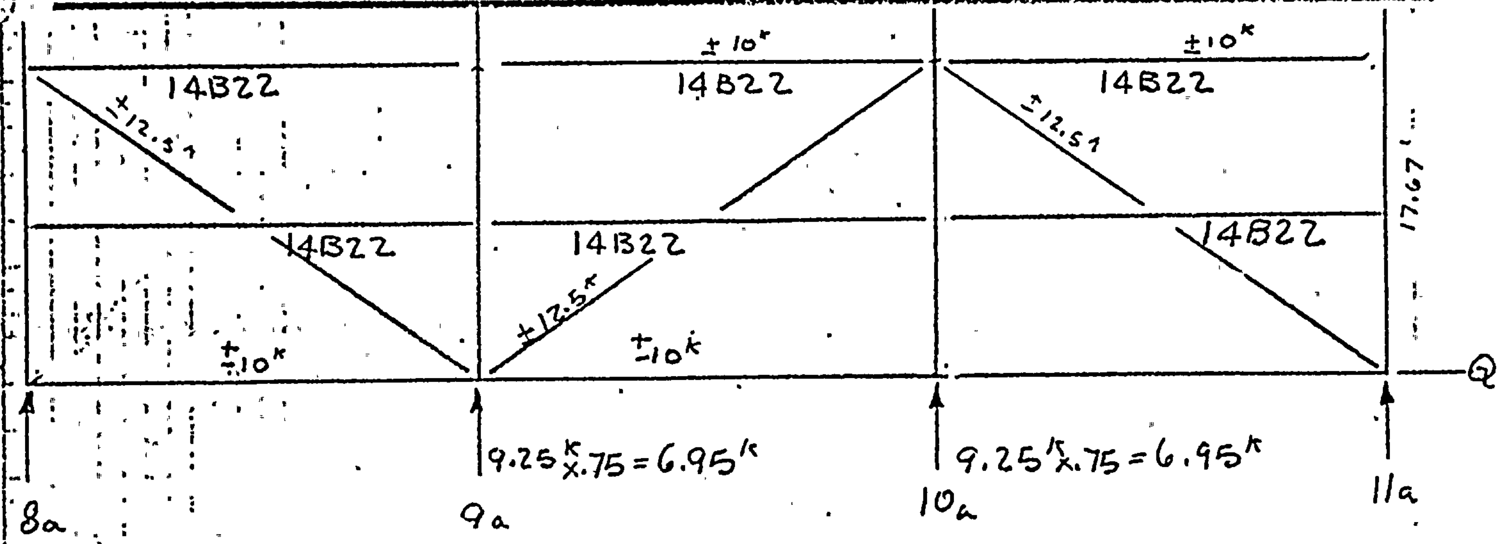
DESIGN LOW ROOF SECTION FOR 18 PSF



1:36-1/29 0512

BROOKWOOD

READING PENNA	GILBERT ASSOCIATES INC. ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
MADE G	W.O. 4155	SHEET 12A DATE 8-2-60



MAX. COMP. = 12.5ᵏ L = 15.5'

TRY 2⊥ 3×3×¼ A = 2.88 r_x = .93 r_y = 1.38

$\frac{15.5\times 12}{.93} = 200$ F_a = 6.22 ksi

6.22 × 2.88 = 17.9ᵏ > 12.5 ok.

$200 \times \frac{.59}{12} = 9.85' < 15.5$ USE ONE RIVET @ ℄ OF ANGLES

CK 14B22 $P/A = \frac{10}{6.47} = 1.55$ ksi

F_a = 16.74 ksi $f_b = \frac{40.7\times 12}{28.8} = 17$ ksi

$f_T = 17 + 1.55 = 18.55$ ok.

$\frac{17}{22} + \frac{1.55}{16.74} = .775 + .093 = .868 < 1.0$ ok

STRUT DESIGN ON 8a LINE FROM "Q TO L" (LOW ROOF)

SEE SH #2

$M = 190 + \frac{.125 \times 55}{1000} \times 46^2 = 190 + 14.6 = 205'^{-k}$

$P = 22^k$ COMP $L_x = 46'$ $L_y = 8.85$ $K = 1.0$

TRY - 21WF 55 $A = 16.18$ $S = 109.7$ $r_x = 8.4$ $r_y = 1.65$

$P/A = 22/16.18 = 1.36$ KSI $K_x = 1.0$ $K_y = 1.0$

$L_c = 8.9$ $L_u = 9.4$

$L_x/r_x = 46 \times 12/8.4 = 66$

$L_y/r_y = 9 \times 12/1.65 = 65.5$ $F_a = 16.89$ $\frac{f_a}{F_a} = \frac{1.36}{16.89} = .081 < .15$

$f_b = 205 \times 12/110 = 22.4$ KSI $F_b = 24$ KSI

USE EQUATION 7a

$.081 + \frac{22.4}{24} = .081 + .935 = 1.016 > 1.0$ ok because .75x mom. will reduce s req.

TRY 21WF 62 $L_c = 8.9$ $L_u = 11.0$

$A = 18.23$ $S = 126.4$ $r_x = 8.53$ $r_y = 1.71$

$f_a = 22/18.23 = 1.21$ KSI

$\frac{L_x}{r_x} = 46 \times 12/8.53 \doteq 65$

$\frac{L_y}{r_y} = 9 \times 12/1.71 = 63$ $F_a = 17.14$ KSI $\frac{1.21}{17.14} = .0705 < .15$

$f_b = 205/126.4 = 16.2$ KSI

$.0705 + \frac{16.2}{24} =$

$.0705 + .685 = .756 < 1.0$ ok.

APPROX. $\Delta = 1.5''$ ok. But use 21WF55

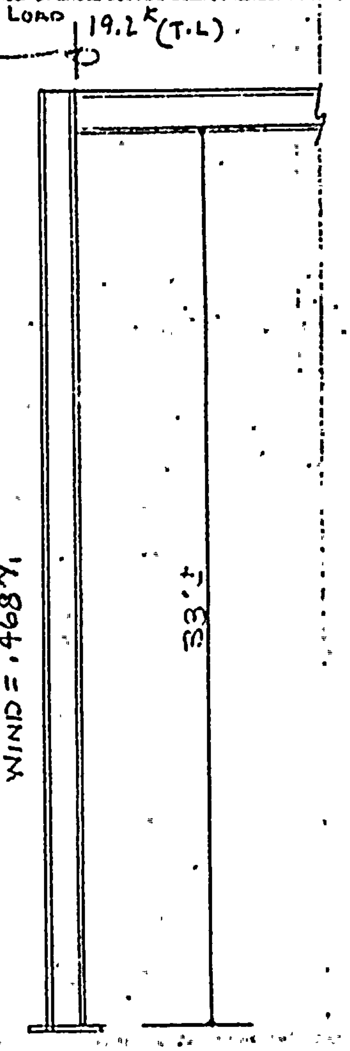
8-2-66

BROOKWOOD	READING PENNA	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON

MADE	CHKD	SU CP	CL DN	APPD	SIZE DRAWING NO. REV	SHEET
G					1:36.1/31	27
					W.O. 4155 DATE 1-28-60	

Note: total load = 25.4k ck & ok

LOW ROOF COL. DESIGN
EXT. COL ("Q" & "L" LINES)



WIND MOM = $.468 \times 38^2/8 = 84.5^{1-k}$

$84.5 \times .75 = 63.4^{1-k}$

D.L. VERT LOAD ≈ 10^k

$Pe = 10 \times .5 = 5^{1-k}$ (NEGLECT)

T.L. VERT LOAD = 19.2^k

$Pe = 19.2 \times .5 = 10^{1-k}$

$84.5 - 10.0 = 74.5^{1-k}$

$M = 74.5 \times .75 = 56^{1-k}$

$P = 19.2 \times .75 = 14.4^k$

$L_x = 39'$ $\quad L_y = 20'$

TRY 10WF45 $\quad A = 13.24 \quad S = 49.1 \quad r_x = 4.33 \quad r_y = 2.0$

$L_c = 8.7 \quad L_u = 17.5$

ASSUME $K = 1.0 \quad d/A_f = 2.04$

$L_x/r_x = 39 \times 12 / 4.33 = 108$

$L_y/r_y = 20 \times 12 / 2 = 120 \rightarrow F_a = 10.28$

$f_a = 14.4 / 13.24 = 1.09$ KSC

$\frac{1.09}{10.23} = .106 < .15$

$C_b = 1.75$
$C_c = 126$

4) $F_b = \left[1.0 - \frac{(l/r)^2}{2 C_c^2 C_b}\right] .6 F_y$

$= \left[1 - \frac{(120)^2}{2 \times (126)^2 \times 1.75}\right] 22 = \left[1 - .26\right] 22 = 16.3$ KSC

5) $F_b = \frac{12 \times 10^6}{20 \times 12 \times 2.08} = 24$ KSC $\quad$ USE 22 KSC

$f_b = \frac{56 \times 12}{49.1} = 13.7$ KSC

0535

BROOKWOOD	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
	MADE: G	DRAWING NO. 1:36-1/32	SHEET 22
	REV.	W.O. 4155	DATE

6) $.106 + \frac{13.7}{22} = .106 + .624 = .730 < 1.0$

TRY - 12 WF 36 $A = 10.59$ $S = 45.9$ $r_x = 5.15$ $r_x = 1.5$

GIRT SPACING @ 7'-0 C.C. CONSIDER AS FLANGE $d/A_f = 3.45$ BRACING

$L_x/r_x = 39\times12/5.15 = 91$ $F_a = 14.09$ KSI $f_a = \frac{14.4}{10.6} = 1.36$ KSI

$L_y/r_y = 7\times12/1.5 = 54$ $\frac{1.36}{14.09} = .0965 < .15$

$f_b = 56\times12/45.9 = 13.6$ KSI. $L_c = 7.1$ $F_b = 22$ KSI

$.0965 + \frac{13.6}{22} = .72 < 1.0$

TRY 12 WF 27 $A = 7.97$ $S = 34.1$ $r_x = 5.06$ $r_x = 1.44$

$L_c = 7.0$ $L_u = 9.9$

$\frac{L_x}{r_x} = \frac{39\times12}{5.06} = 92.5$ $F_a = 13.91$ KSI

$\frac{L_y}{r_y} = \frac{7\times12}{1.44} = 58$ $f_a = \frac{14.4}{7.97} = 1.8$ KSI $\frac{1.8}{13.91} = .13 < .15$

USE $F_b = 22$ KSI $f_b = \frac{56\times12}{34.1} = 19.7$ KSI

$.13 + \frac{19.7}{22} = .13 + .896 = 1.026 > 1.0$

USE 12 W 31

$A = 9.12$ $r_x = 5.11$

$S = 39.4$ $r_y = 1.47$

CONSIDER WIND W/OUT T.L. $P \cong 7.5^k$

$M = 63.4^k$

$F_a = 14.03$ $f_a = .082$

$F_b = \frac{63.4\times12}{39.4} = 19.4$

$\frac{f_a}{F_a} = .059$

$.059 + \frac{19.4}{22} = .059 + .88 = .94 < 1.0$ ok.

USE 12 WF 31

BROOKWOOD	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
MADE: G	CHKD	DRAWING NO. 1:36.1/33	SHEET 26
W.O. 4155			

LOW ROOF COL DESIGN

$M = .468 \frac{(25)^2}{8} = 36.6^{\prime-k}$

$P = 32^k$

DESIGN

$m = 36.6 \times .75 = 27.5$

$P = 24^k$

TRY 12B22 $A = 6.47$ $S = 25.3$

$r_x = 4.91$ $r_y = .84$

$F_a = 12.98$

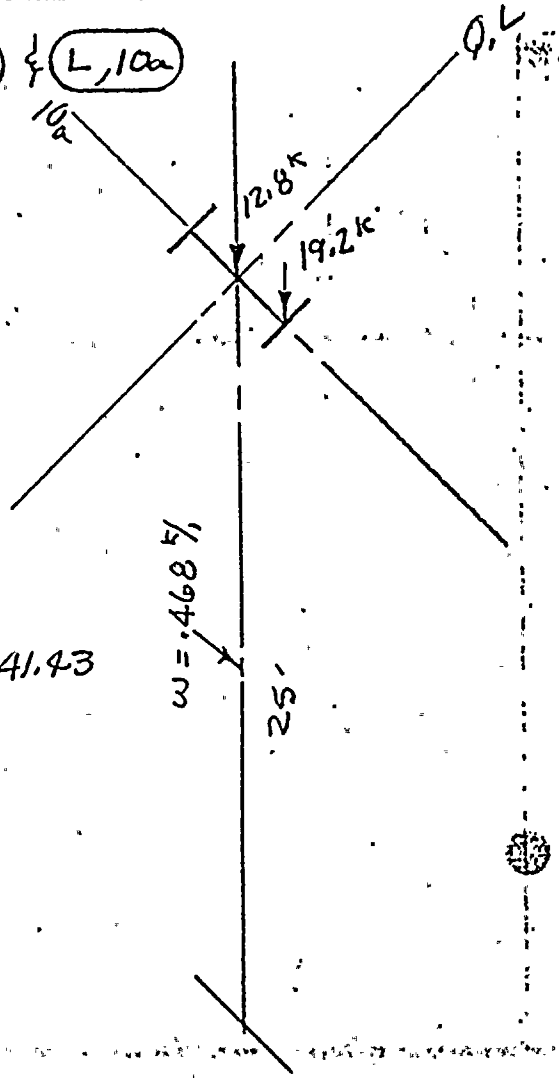
$f_a = 3.7$ $\frac{f_a}{F_a} = .286 > .15$

$f_b = 13.0$ ksc $F_b = 19.9$ ksc $F_e' = 41.43$

$$.286 + \frac{13}{\left(1 - \frac{3.7}{41.43}\right)19.9} = .286 + .72$$

$$= 1.006 > 1.0$$

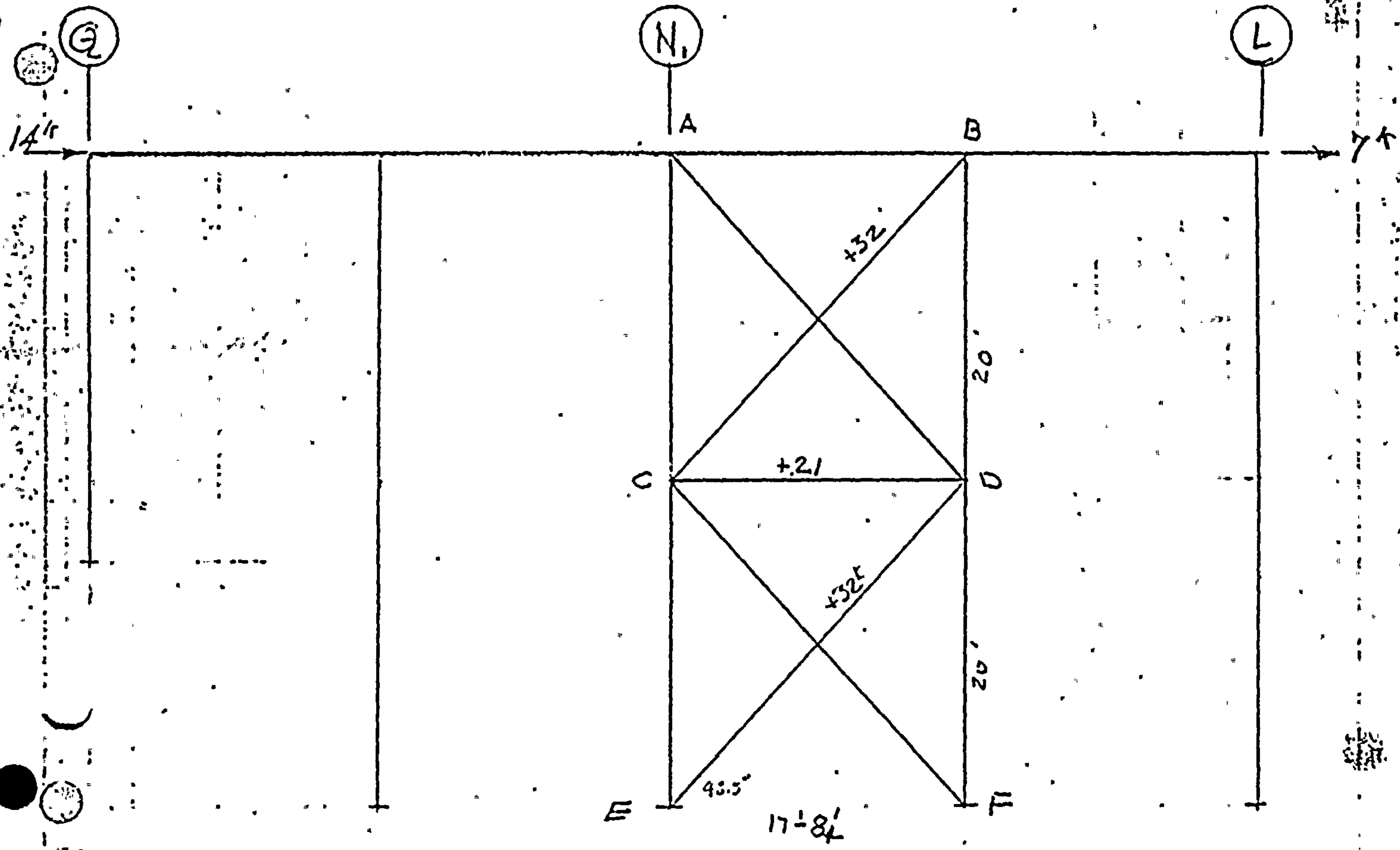
USE 12WF31



READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON

MADE	CHKD	SQ CF	CF DN	APPD	SIZE DRAWING NO. REV	SHEET
G.					1:36.1/34	28
					W. O. 4155 DATE 8-8-66	

REV. 8-19-66



(B-C) $P = 32 \times .75 = 24^k$ $L = 26.8'$

$A_{Req} = 24.0^k/20 = 1.2$ □" USE 4 5x3x5/16 $A_s = 2.4$

$A_{eff} = 2.0$ ok

$\frac{26.8 \times 12}{300} = 1.07$ vs .54 ok

(C-D) $P = 21 \times .75 = 16^k$ $L = 17.67'$

MIN. $r = \frac{17.67 \times 12}{200} = 1.06$

USE 2∠ 3½ x 2½ x 5/16

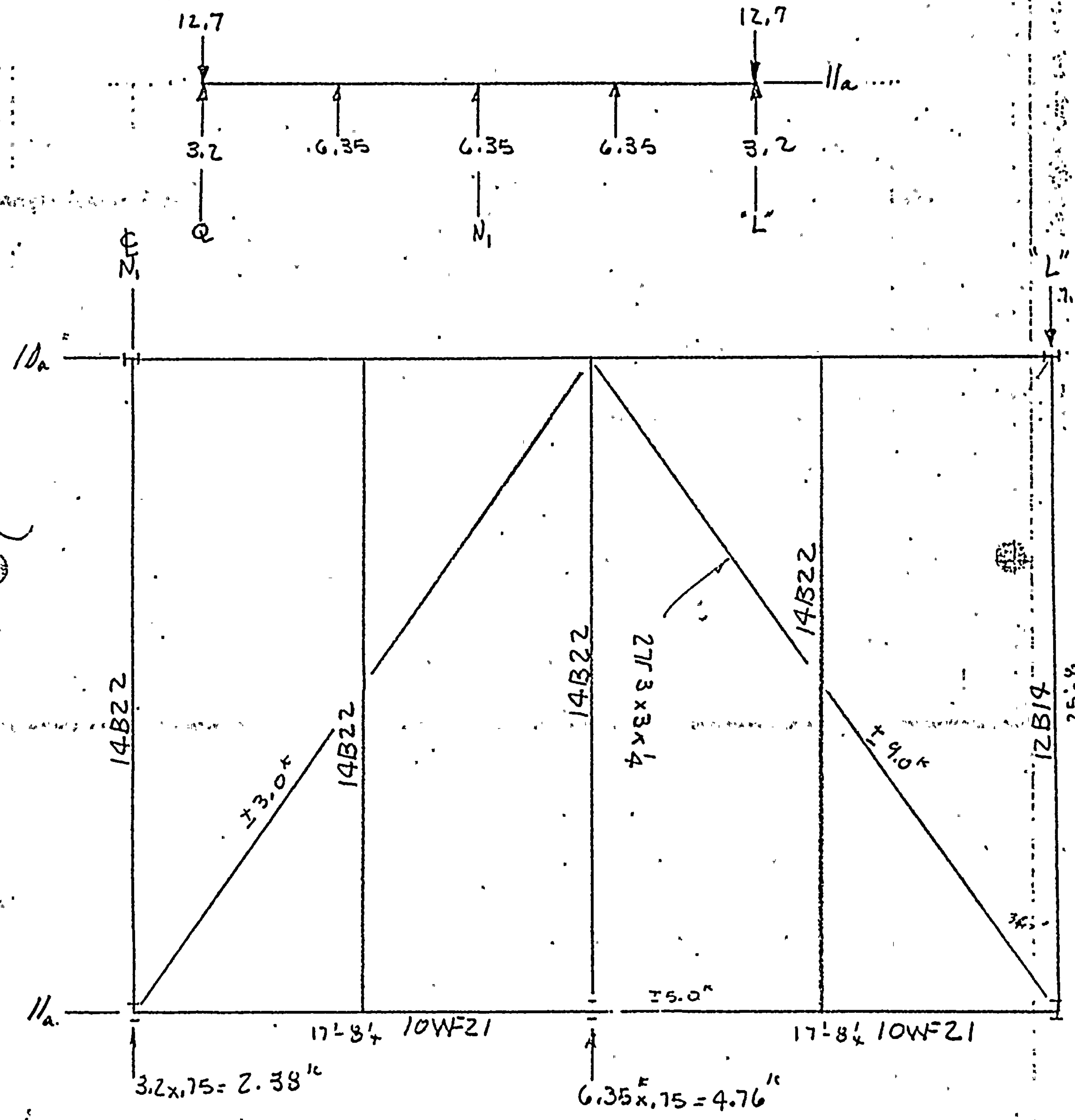
USE ONE RIVET @ ℄

BROOKWOOD

READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON

MADE	CHKD	SQ CF	CF DN	APPD	SIZE DRAWING NO. REV	SHEET
G					1:361/35	39
REV.					W.O. 4155 DATE 8-2-60	

HOR. TRUSS DESIGN LINE "11a"



USE ONE RIVET @ ℄ ANGLE BRACING

BROOKWOOD

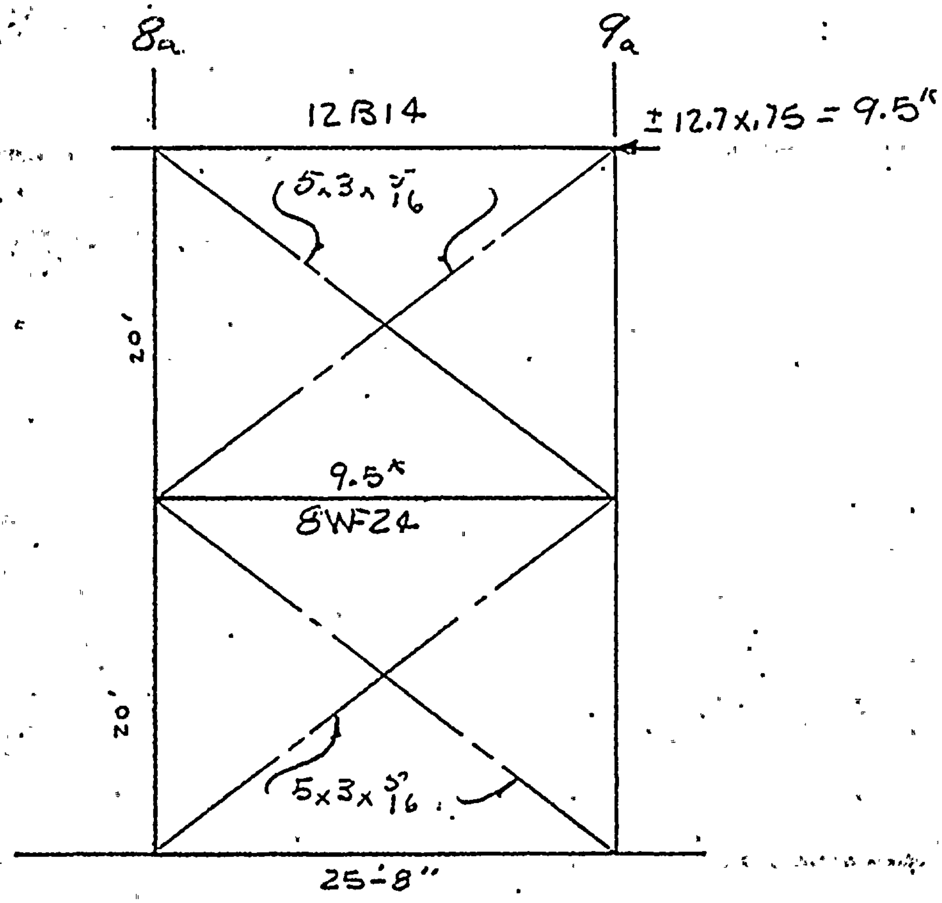
READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON

MADE	CHKD	SQ CF	CF DN	APPD	SIZE DRAWING NO.	REV.	SHEET
G					1:36-1/36		30
					W.O. 4165	DATE 8-2-66	

REV. 8-19-66

VERT BRACING ON COL. LINE Q & L

PLACE BRACING BETWEEN 8a & 9a ON Q & L LINE



$P = 9.5^K \quad L = 25.67'$ FOR COMP.

$\min r = \frac{25.67 \times 12}{200} = 1.54$ TRY 8WF24 $A = 7.06 \quad r_y = 1.61$

$S = 20.8$

$m = \frac{24(25.67)^2}{8} = 1.98^{1-K} \qquad f_b = \frac{1.98 \times 12}{20.8} = 1.14 KSI$

DIAG. FOR TENSION $L = 32.5'$

$\min. r = \frac{32.5 \times 12}{300} = 1.3$ & .65 USE ∡ 5x3x5/16

5" LEG OUT.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36·1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04-4824-000	PAGE: 37

SUBJECT: SEISMIC EVALUATION -

ORIGINATOR: D. BISS
DATE 4/30/79
VERIFIED: L. Praul
DATE 5.7.79

Objective: EVALUATE EQUIVALENT SEISMIC FORCES FOR THE LOW ROOF STRUCTURES OF THE AUXILIARY BUILDING. STIFFNESS AND MASS CALCULATIONS DEVELOPED DURING THE ORIGINAL DESIGN WILL BE THE BASIS FOR THIS EVALUATION.

Reference: Stiffness Calculations for Auxiliary Bldg Box 1044 35 of 42

FLEXIBILITY

$$F_{N-S} = \frac{7580}{E}$$

$$F_{E-W} = \frac{2615}{E}$$

$$Mass = 1.63 \ \frac{k\text{-}sec^2}{ft} \qquad E = 4.176 \times 10^6 \ k/ft^2$$

stiffness is inverse of flexibility ∴

$$k_{N-S} = \frac{4.176 \times 10^6}{7580} \qquad k_{E-W} = \frac{4.176 \times 10^6}{2615}$$

$$k_{N-S} = 550.9 \ k/ft \qquad k_{E-W} = 1596.9 \ k/ft$$

$$f_{N-S} = \frac{1}{2\pi}\sqrt{\frac{k_{N-S}}{m}} \qquad f_{E-W} = \frac{1}{2\pi}\sqrt{\frac{k_{E-W}}{m}}$$

$$= \frac{1}{2\pi}\sqrt{\frac{550.9}{1.63}} \qquad = \frac{1}{2\pi}\sqrt{\frac{1596.9}{1.63}}$$

$$f_{N-S} = 2.93 \ cps \qquad f_{E-W} = 4.98 \ cps$$

$$T_{NS} = 0.34 \ sec \qquad T_{E-W} = 0.20 \ sec$$

From FSAR RESPONSE SPECTRUM

	N-S	E-W
OBE	0.17g	0.20g
SSE	0.44g	0.40g

2.5% damping

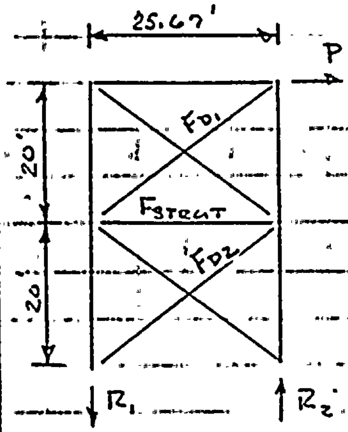
GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04-4824-000	PAGE: 38

SUBJECT: Seismic Evaluation

ORIGINATOR: D. Biss
DATE: 4/30/79
VERIFIER: [illegible]
DATE: 5.7.79

Bracing on Q & L line

Ref Aux Bldg Calc's pg 30 (0545)



	WIND	OBE	SSE
P	12.7^k	10.5^k	21^k
F_{DIAG} 1 (Allowable)	16.1^k (59.6)	13.3^k (59.6)	26.6^k (71.6)
F_{STRUT} (Allowable)	-12.7^k (59.3)	-10.5^k (59.3)	-21^k (71.3)
F_{DIAG} 2 (Allowable)	16.1^k (59.6)	13.3^k (59.6)	26.6^k (71.6)
R_1 & R_2 (Allowable)	$\pm 19.9^k$ ($\pm 42.6^k$)	$\pm 16.4^k$ ($\pm 42.6^k$)	$\pm 32.7^k$ ($\pm 57.7^k$)

Allowable stresses are increased above AISC values by 33% for OBE & WIND, 60% for SSE (equivalent to yield)

For L 5x3x5/16 $A = 2.4\ in^2$

$P_{ALL} = (.85)(2.4)(22) = 44.9^k \times 1.33 \Rightarrow 59.6$

$\times 1.6 = 71.6^k$

For W8x24 Strut

$A = 7.06 \quad r_y = 1.61 \quad \frac{k\ell}{r} = \frac{(25.67)(12)}{1.61} = 191.3$ $\qquad F_b = 6.33\ ksi$

$P_{ALL} = (7.06)(6.33) = 44.6^k \times 1.33 = 59.3$

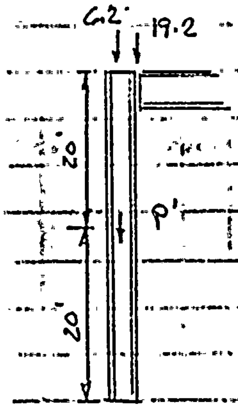
$\times 1.6 = 71.3$

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 044824 000	PAGE: 39

SUBJECT: SEISMIC EVALUATION

ORIGINATOR: D BISS
DATE: 4/30/79
VERIFIED: F Pisarek
DATE: 5.7.79

Column Capacity



Moment @ mid point due to roof load $= \frac{(19.2^k)(6 in)}{2} = 58$ in-k

For W12x31 with $L_x = 40$ ft
$L_y = 20$ ft

$r_x = 5.12 \quad r_y = 1.54 \quad \therefore \frac{kl}{r_y}$ controls

$\frac{kl}{r_y} = \frac{20 \times 12}{1.54} = 156, \quad F_a = 6.14$ ksi

for unbraced length of 20 ft

$F_b = \frac{12 \times 10^3}{ld/A_f} = \frac{12000}{(240)(3.98)} = 12.56$ ksi

@ mid point, $f_b = \frac{58 \text{ in-k}}{39.5 \text{ in}^3} = 1.47$ ksi

using $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$ solve for f_a

$f_a = F_a\left(1 - \frac{f_b}{F_b}\right)$

$f_a = 6.14\left(1 - \frac{1.47}{12.56}\right) \quad f_a = 5.42$ ksi

$A = 9.13$ in²

$P = 49.5^k$

$P' = 49.5^k - 25.4 = 24.1^k$

for 33% increase $\quad f_a = 6.14\left(1.33 - \frac{1.47}{12.56}\right) = 7.44$ ksi

$P = 70^k$

$P' = 70^k - 25.4 = 42.6^k$

for 60% increas $\quad f_a = 6.14\left(1.6 - \frac{1.47}{12.56}\right) = 9.1$ ksi

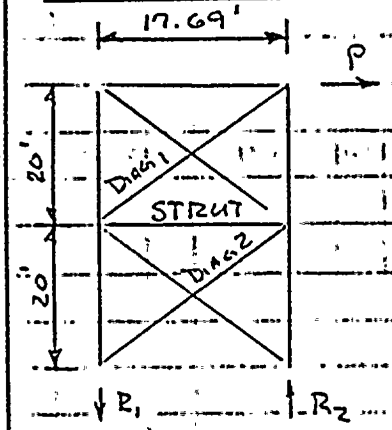
$P' = 57.7^k$ $\qquad P = 83.1^k$

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04 4824 000	PAGE: 40

SUBJECT: Seismic Evaluation

ORIGINATOR: D. Biss
DATE: 4/30/79
VERIFIER: [illegible signature]
DATE: 5.7.79

Bracing on 11a Line



	WIND	OBE	SSE
P	21k	8.9k	23.1k
$F_{DIAG\,1}$ (Allowable)	31.7^k (59.6^k)	13.4^k (59.6^k)	34.9 (71.6^k)
F_{STRUT} (Allowable)	21^k (29.8)	8.9^k (29.8)	23.1^k (35.8)
$F_{DIAG\,2}$ (Allowable)	31.7^k (59.6^k)	13.4^k (59.6^k)	34.9^k (71.6^k)
R_1 & R_2 (Allowable)	$\pm 47.5^k$ (47.9^k)	$\pm 20.1^k$ (47.9^k)	$\pm 52.2^k$ (60.7^k)

For Diagonal L $5 \times 3 \times 5/16$ $A = 2.4\,in^2$

$F_{ALLOW} = 0.85 \times 24 \times 22 \times 1.33 = 59.6^k$

$\times 1.6 = 71.6$

For Strut $2 \sqcap\!\sqcap\ 3\tfrac{1}{2} \times 2\tfrac{1}{2} \times 5/16$ $r_{min} = 1.10$

$\frac{kl}{r} = \frac{(17.69)(12)}{1.1} = 193$, $F_a = 6.31$ ksi

$P = (6.31\,ksi)(3.55\,in^2) = 22.4^k$

with 33% ⇒ 29.8^k

with 60% ⇒ 35.8^k

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE: 41

SUBJECT: SEISMIC EVALUATION

ORIGINATOR: D. Biss
DATE: 4/30/79
VERIFIED: F. Pisarcz
DATE: 5.7.79

For W12x27 columns

$P = 12.8^k$ (6.4^k eccentric)

$M_{mid\ point} = \frac{(6.4^k)(6in)}{2} = 19.2$ in-k

$S_x = 34.2\ in^3$

$d/A_f = 4.60$ $\quad \frac{kl}{r_y} = \frac{20 \times 12}{1.52} = 158$

$A = 7.95\ in^2$

$r_x = 5.07$ $\quad F_a = 5.98$ ksi

$r_y = 1.52$ $\quad F_b = \frac{12000}{l\,d/A_f} = \frac{12000}{(240)(4.6)} = 10.87$ ksi

$f_b = \frac{19.2\ in\text{-}k}{34.2\ in^3} = 0.56$ ksi

for 33% increase

$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1.33$, $\quad f_a = F_a\left(1.33 - \frac{f_b}{F_b}\right)$

$f_a = 5.98\left(1.33 - \frac{.56}{10.87}\right)$

$f_a = 7.64$ ksi

$P = 60.78^k$

$P' = 60.78 - 12.8 = 47.9^k$

for 60% increase

$f_a = 5.98\left(1.6 - \frac{.56}{10.87}\right)$

$= 9.25$ ksi

$P = 73.5^k$

$P' = 73.5 - 12.8 = 60.7$ ksi

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural Engineering	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: Ginna Station	W.O. NUMBER: 04 4824 000	PAGE: 42

SUBJECT: Seismic Evaluation

ORIGINATOR: D. Biss DATE: 4/30/79

VERIFIED: F. Pisarel DATE: 5.7.79

Capacity of 4-¾"φ A307 Anchor Bolts in tension (shear carried by grouted column base pocket)

$A_T = 0.3345 \text{ in}^2/\text{bolt}$

$T_{ALL} = 6.69^k/\text{bolt}$

with 33% increase, $T = 4(6.69)(1.33) = 35.6^k$

with 60% increase, $T = 4(6.69)(1.6) = 42.8^k$

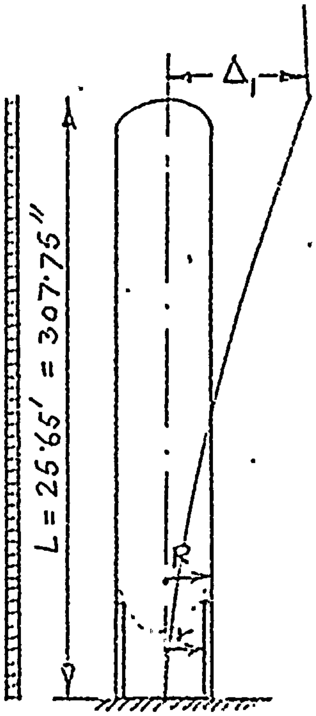
Anchor Bolts satisfactory for applied loads when nominal dead load of structure is considered.

1478

GINNA PROJECT UNIT 1.	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
Pressurisor Support		SIZE DRAWING NO. 1:36.1/43	SHEET

2

TOTAL OPER. WT. 121,330 LBS.
WT. OF INSUL. AND PIPE $0.97^k/'$
(PERIPHERAL)



$R = 3'\text{-}10\frac{3}{16}'' = 46.2''$

$r = 3'\text{-}8\frac{11}{16}'' = 44.7''$

$E = 29 \times 10^6$ psi

Moment of inertia. $I = \frac{\pi}{4}(R^4 - r^4) = \frac{\pi}{4}(46.2^4 - 44.7^4)$

$= \frac{\pi}{4}(3.85^4 - 3.72^4)\,12^4 = \frac{\pi}{4}(220 - 192)12^4$

$= 7\pi\, 12^4 = 22 \times 12^4 \text{ in}^4$

Max. Deflection at the top,

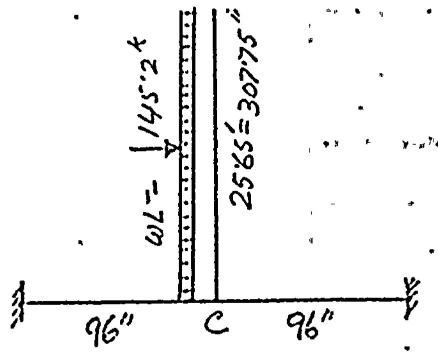
$$\Delta_1 = \frac{wL^4}{8EI} = \frac{wL \cdot L^3}{8EI} = \frac{145.2 \times 25.65^3 \times 12^3 \times 10^3}{8 \times 29 \times 10^6 \times 22 \times 12^4}$$

$$= \frac{145.2 \times 25.65^3}{8 \times 29 \times 22 \times 12 \times 10^3} = \frac{40}{10^3} = .04''$$

Natural frequency, $f = \frac{3.89}{\sqrt{\Delta}} = \frac{3.89}{\sqrt{.04}} = \frac{3.89}{.2} = 19.45$ cycles/sec

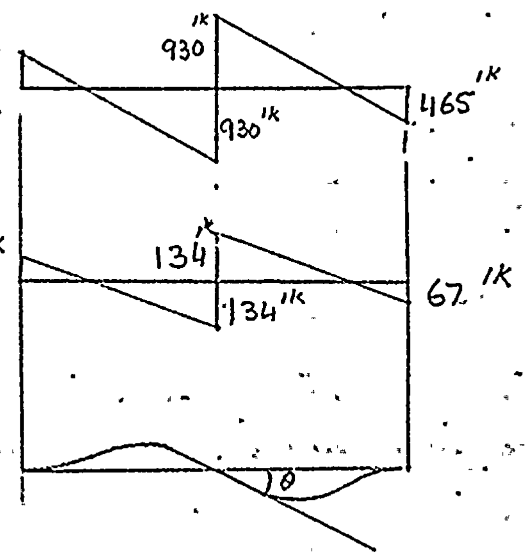
Natural Period $T = \frac{1}{f} = \frac{1}{19.45} = 0.0515$

Rotation of base.



$$M_c = wL\frac{L}{2} = \frac{145.2 \times 25.65}{2}$$
$$= 1860^{'k}$$

1. This moment resisted by 7' wide slab →



Moment per ft. width →

Total depth $d = 36"$

$$I = \frac{bd^3}{12} = \frac{12}{12} \times 36^3 = 46500 \; "^4$$

$$\theta = \frac{67^{'k}}{2EK} = \frac{67^{'k}L}{2EI}$$

$$= \frac{67 \times 12000 \times 96}{2 \times 3 \times 10^6 \times 46500} = \frac{277}{10^6} = .000277$$

Max. defl. due to rotation of base, $\Delta_2 = \theta L = .000277 \times 308$
$= 0.085$

$\therefore$ Total lateral defl. $\Delta_L = \Delta_1 + \Delta_2 = .04 + .085 = .125$

with $d = 30"$ $\Delta_2 = 0.146$ and $\Delta_L = .146 + .04 = .186$

Natural Period $T = 2\pi\sqrt{\frac{\Delta_L}{g}} = 2\pi\sqrt{\frac{.125}{32.2 \times 12}} = 2\pi\sqrt{.000324}$

$= 2\pi \times 0.01794$

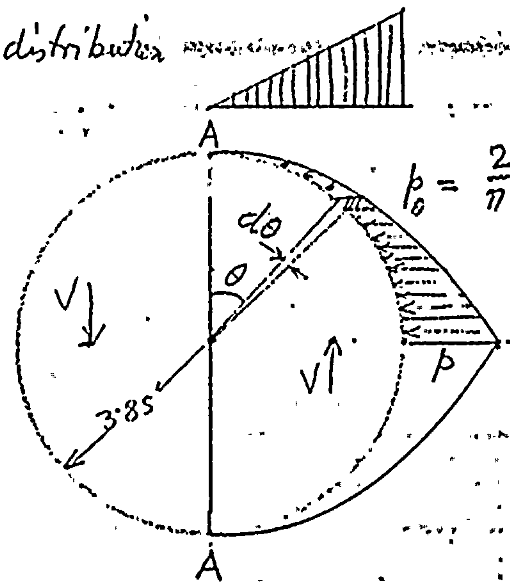
$= 0.113$

From .2g Response Spectra the lateral load coeff with ½% damping the Lateral Load Coefficient is .57 g.

$\therefore$ Lateral Load $= .57 \times 145.2 = 82.8^k$

Moment due to Lateral load. $= 82.8 \times \frac{2}{3} 25.65 = 1420^{\prime k}$

Assume a linear Load distribution as shown:



$p_\theta = \frac{2p}{\pi r} r d\theta$

Internal Moment

$M_i = 4 \cdot \int_0^{\pi/2} \left(\frac{2p}{\pi r} \cdot r\theta\right) r\,d\theta\, r \sin\theta$

$= \frac{8pr^2}{\pi} \int_0^{\pi/2} \theta \sin\theta\, d\theta$

$= \frac{8pr^2}{\pi} \left[-\theta\cos\theta + \sin\theta\right]_0^{\pi/2}$

$= \frac{8pr^2}{\pi} [1] = \frac{8}{\pi} pr^2$

External Moment = Internal Moment.

$$\therefore \quad \frac{8}{\pi} p r^2 = 1420'^k$$

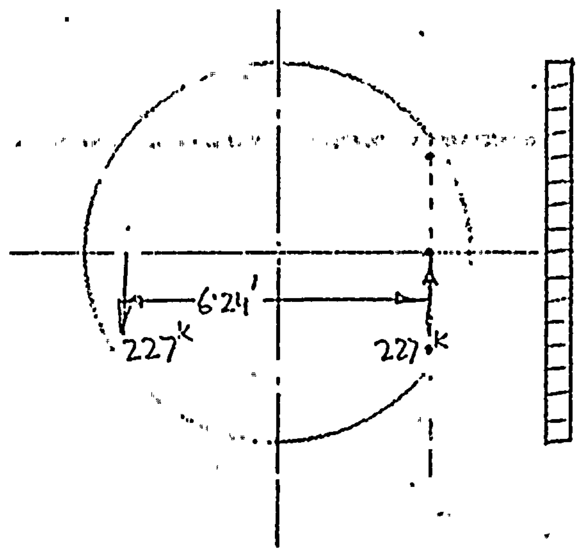
$$\text{or.} \quad p = \frac{1420 \times \pi}{8 \times 3{\cdot}85^2} = 37{\cdot}6^k$$

$$\text{or,} \quad \frac{37{\cdot}6}{\pi \cdot r} = 3{\cdot}14$$

Hence the total force of the internal couple

$$P = 2 \cdot \frac{p}{2} \cdot \frac{\pi r}{2} = \frac{37{\cdot}6 \times 3{\cdot}14 \times 3{\cdot}85}{2} = 227^k$$

$$\text{Lever Arm,} \quad e = \frac{M}{P} = \frac{1420}{227} = 6{\cdot}24'$$



Assuming the load to be distributed uniformly over an area of 3' × 8' [2·66×8)

$$\text{Load} = \frac{227}{3 \times 8} = 9{\cdot}5^k/\square' \approx$$

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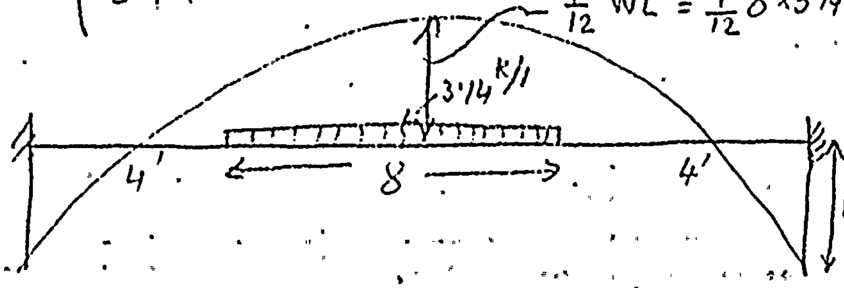
$\frac{1}{2}$ 4 | 2 1.

$\frac{1}{12} WL = \frac{1}{12} 8 \times 3.14 \times 16 = 34^{'K}$ 6

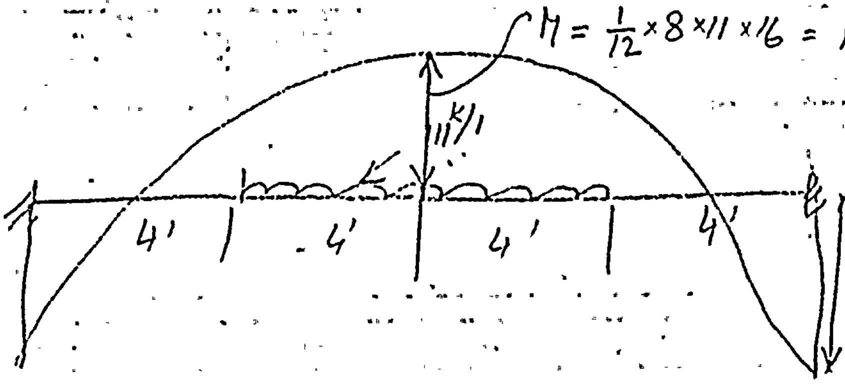
D.L.



$M = \frac{1.375}{12} \times 8 \times 3.14 \times 16$
$= 47^{'K}$

$M = \frac{1}{12} \times 8 \times 11 \times 16 = 118^{'K}$

S.L



$M = 118 \times 1.375$
$= 162^{'K}$

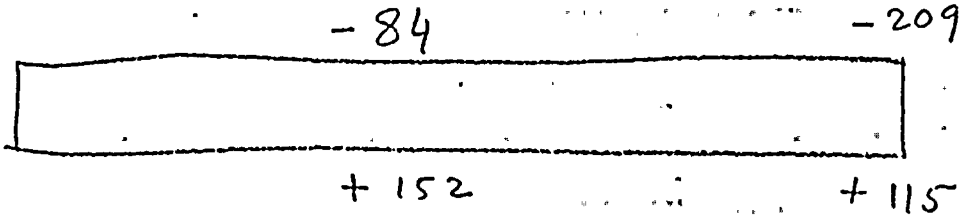
$K = \frac{W}{\Delta}$

$T = 2\pi \sqrt{\frac{W}{Kg}}$

$= 2\pi \sqrt{\frac{\Delta}{g}}$

+34	−47
+118	−162
+152'K	−209

+34	−47
−118	+162
−84	+115



RGE PROJECT UNIT 1	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
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	REV.	W.O. DATE	

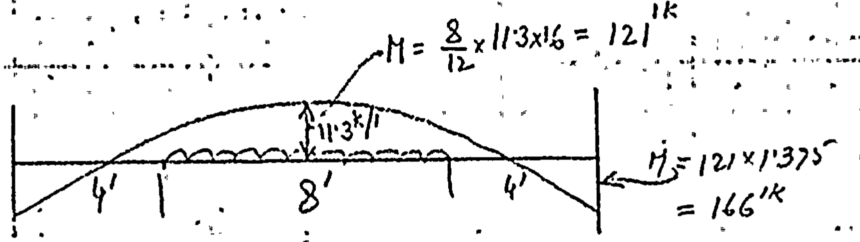
7

Internal opening: 6'-0" dia.

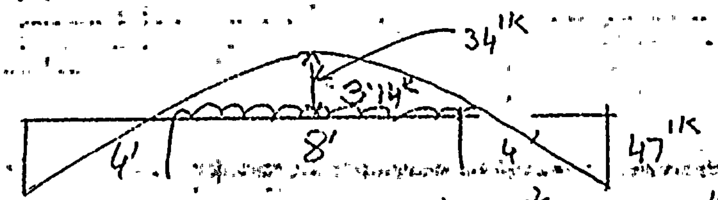
Assuming the load to be distributed over an area $\{(2'10") - 4"\} \times 8' = 20$ sft

Load $= \frac{227}{20} = 11.3^{KSF}$

Seismic Load. S.L.



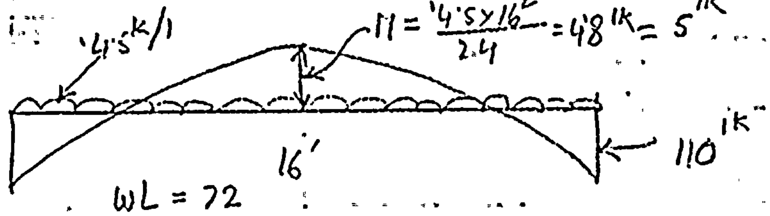
D.L. of Pressurizor D.L.



Wt. of slab D.L.

36" slab.

.45



$$\begin{array}{r} +121 \\ +34 \\ +5 \\ \hline +160^{'K} \end{array} \qquad \begin{array}{r} -166 \\ -47 \\ -10 \\ \hline -223 \end{array}$$

$$\begin{array}{r} -121 \\ \left.\begin{array}{r} +34 \\ +5 \end{array}\right\} 39 \\ \hline -82 \end{array} \qquad \begin{array}{r} +166 \\ \left.\begin{array}{r} -47 \\ -10 \end{array}\right\} 57 \\ \hline +109 \end{array}$$

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$$A_{CL} = 0{\cdot}0099 + \left(\frac{{\cdot}45 \times 16^4}{384 EI} \times 12^3 = \frac{{\cdot}45 \times 16^4 \times 12^3}{384 \times 3 \times 10^6 \times 46600} = \frac{{\cdot}45 \times 1{\cdot}6^3 \times 1{\cdot}2^3}{384 \times 3 \times 46600 \times 10^6} = \frac{{\cdot}6}{10^5}\right.$$

$$= 0{\cdot}0099 + {\cdot}0000606$$

$$= 0{\cdot}0099006$$

$$T = 2\pi\sqrt{\frac{\Delta}{g}} = 0{\cdot}0314 \text{ Sec.}$$

From ·2g. Response curve, the vertical load coeff with 2% damping for concrete is ±·2 i.e. the vertical load varies from 1·2W to ·8W.

1485

R.G.E. PROJECT UNIT 1.	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
Pressurizor Support	MADE / CHKD / ... / APPD	SIZE DRAWING NO REV: 1:36.1/50	SHEET
	REV.	W.O.	DATE

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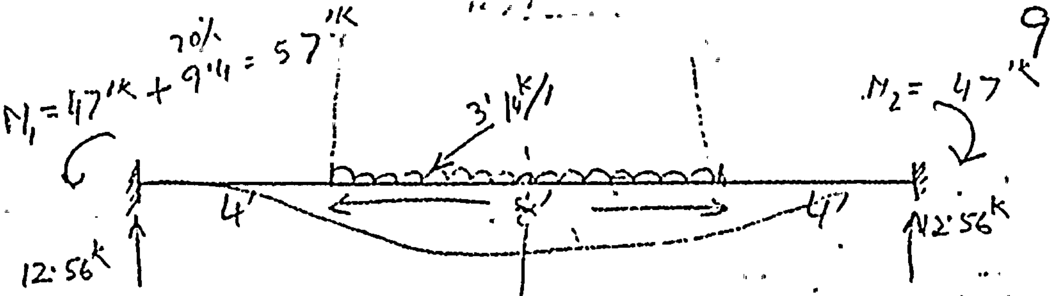

$$\Delta_{CL} = \frac{1}{6EI}\left\{12.56 \times 8^3 - 3 \times 47 \times 8^2 - \frac{1}{4} \, 25.12 \left(\frac{8-4}{8}\right)^4\right\} 12^3 \times 10^3$$

$$= \frac{1}{6EI}\left\{6430 - 9010 - 200.96\right\} 12^3 \times 10^3$$

$$= \frac{2781}{6EI} \times 12^3 \times 10^3 = \frac{2781 \times 12^3 \times 10^3}{6 \times 3 \times 10^6 \times 27000} = \frac{309 \times 32}{10^6}$$

$$= .0099$$

$\left\{ \begin{matrix} 735 \\ 750 \end{matrix} \right.$

$$T = 2\pi \sqrt{\frac{\Delta}{g}} = 2\pi \sqrt{\frac{.0099}{32.2 \times 12}} = 2\pi \, .005 = .0314 \text{ Sec.}$$

From .2g Response curve, the Vertical load Coeff. - with 2% damping for Concrete is ± .2 i.e. the vertical load varies from 1.2 W to .8 W.

1486

R.G.E. PROJECT UNIT 1.

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MADE	CHKD	SQ CP	CP DN	APPD	SIZE DRAWING NO. REV.	SHEET
					1:36.1/51	

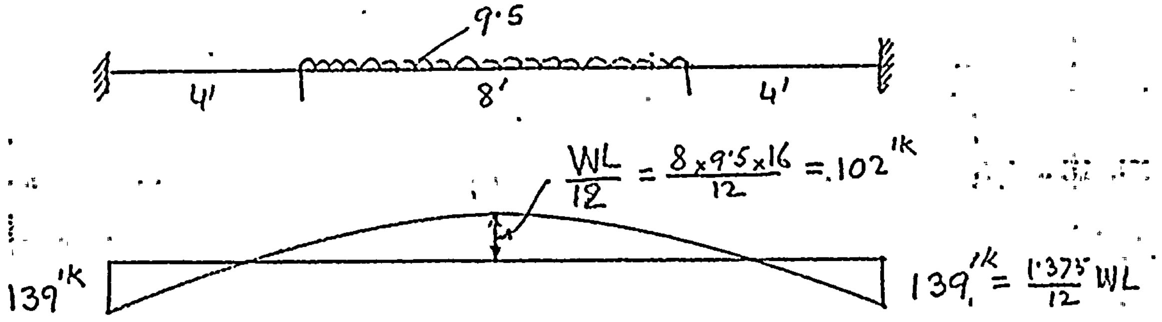
W.O. DATE

PRESSURIZOR SUPPORT

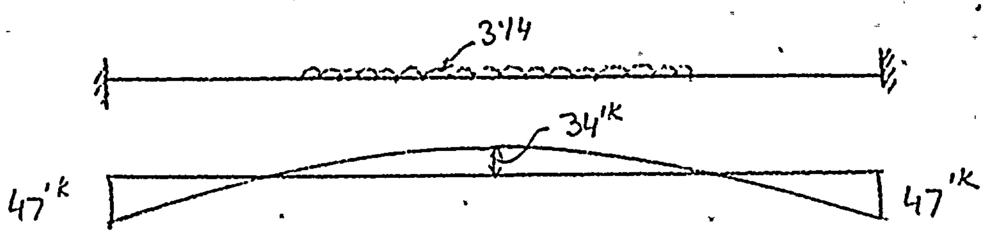
REV.

10

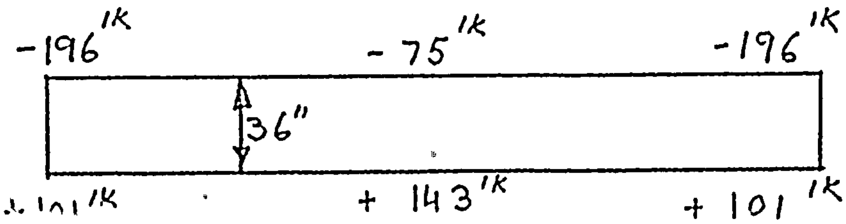
Moment due to Earthquake Load.



Moment due to D.L.



120% D.L.	− 57	+ 41	− 57
80% DL	− 38	+ 27	− 38
E. Load.	∓ 139	± 102	∓ 139
$+M_{max.}$	+ 101	+ 143	+ 101
$-M_{max.}$	− 196	− 75	− 196



1487

RGE PROJECT UNIT 1	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
PRESSURIZOR SUPPORT		SIZE DRAWING NO. 1:36.1/52	

Trial $D = 30''$ 11

T1

Consider - $d = 30''$

$$I = \frac{12}{12} \times 30^3 = 27000 \text{ in}^4$$

$$\theta = \frac{67 \times \cancel{2000}^{2} \times 96}{\cancel{2} \times \cancel{3} \times 10^6 \times 27\cancel{000}} = .000476$$

$$\Delta_2 = .000476 \times 308'' = .1465$$

$$\Delta_L = \Delta_1 + \Delta_2 = .04 + .1465 = .1865$$

$$T = 2\pi \sqrt{\frac{.1865}{32.2 \times 12}} = 2\pi \times \sqrt{.000484} = 2\pi \times .022 = .1382$$

Hor. Load Coefft. .65

Total lateral load $= .65 \times 145.2 = 94.4^k$

Moment $= 94.4 \times \frac{2}{3} \times 25.65 = 1615^{1k}$

$$p = \frac{161.5 \times \pi}{8 \times 3.85^2} = 42^k$$

$$P = 2\frac{p}{2} \times \frac{\pi r}{2} = \frac{42 \times 3.14 \times 3.85}{2} = 232^k$$

$$e = \frac{M}{P} = \frac{161.5}{232} = 6.96$$

U.D.L $\quad w = \frac{232}{378} = 9.66^k$

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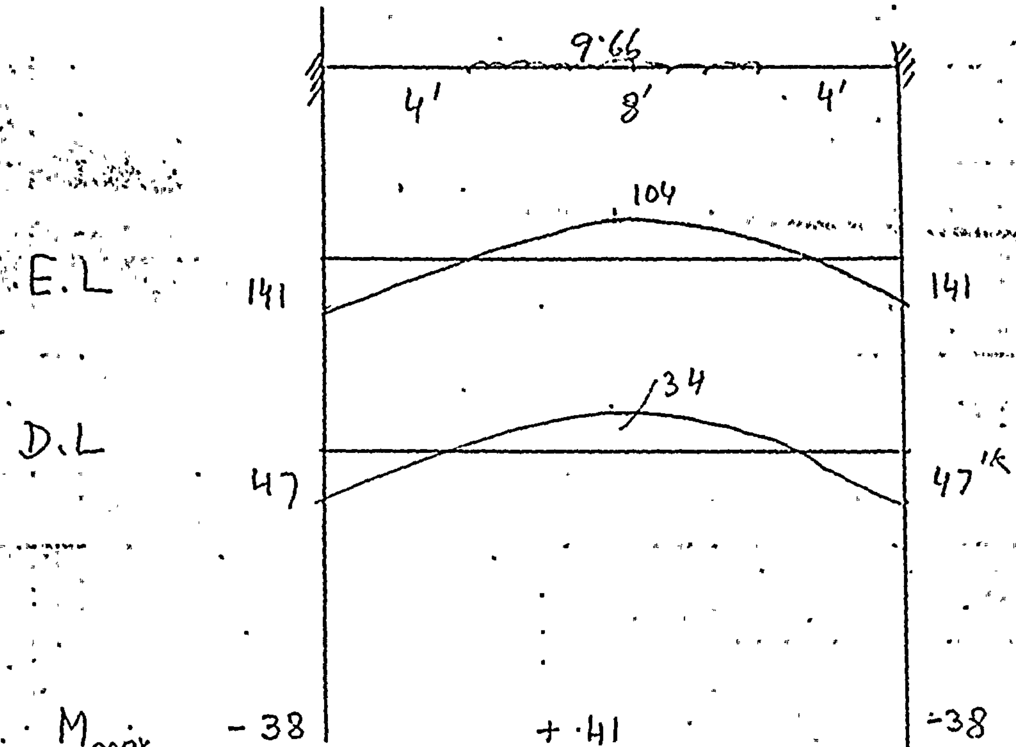
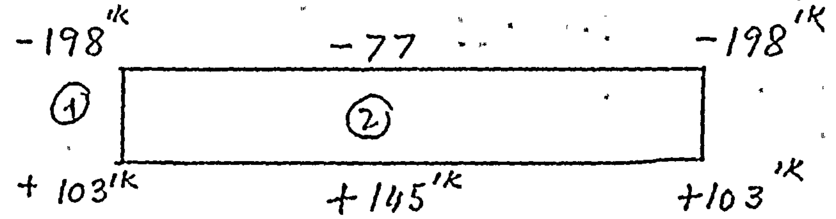
CORRIDOR SUPPORT	MADE / CHKD / SQ CK / CP DN / APPD	DRAWING NO. 1:36.1/53	REV. / SHEET

1400

T2. 12



D.L.	M_{max}	−38	+41	−38
D.L	M_{min}	−57	+27	−57
E.L.	M	∓141	±104	∓141
	$M_{max.}$	+103'k	+145'k	+103'k
	M_{min}	−198'k	−77'k	−198'k



$f_y = 40$ ksi $f'_c = 3$ ksi

$n = 9.2$

$f_s = 18$ ksi $f_c = 1350$

$v = \frac{V}{bd} = \frac{39000}{27 \times 12} = 120$

$b = 12''$ $d = 27''$ $d' = 3''$

$M = 145'^k$ $V = \frac{232}{3 \times 2} = 38.7^k \simeq 39^k$

For Earthquake the allowable stress is increased by $33\frac{1}{3}\%$

i.e. $f_s = 24$ ksi and $f_c = 1.8$ ksi.

From Table 1. for 24000/9.2/1800 : $K = 317$

From Tab. 4. for $b \times d = 12 \times 27$, $F = .729$

Then.

$M = 145$

$KF = 317 \times .729 = 228.'$

$\therefore M - KF = 145 - 228 = -83$

Since M-KF is −ve, no comp. steel reqd..

From Table 1. for $f_s = 20000$, $a = 1.76$

$A_{s2} = \frac{M}{ad} = \frac{145}{1.76 \times 27} = 3.06^{\square''}$

$A'_{s2} = \frac{77}{1.76 \times 27} = 1.62^{\square''}$

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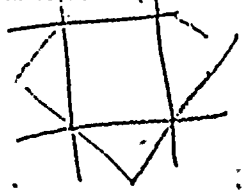
PRESSURIZOR SUPPORT		MADE	CHKD			APPD	SIZE DRAWING NO. 1:36·1/55 REV. SHEET
		REV.					W.O. DATE

14



$$A_{s1} = \frac{198}{1{\cdot}76 \times 27} = 4{\cdot}17\ \square''$$

4″

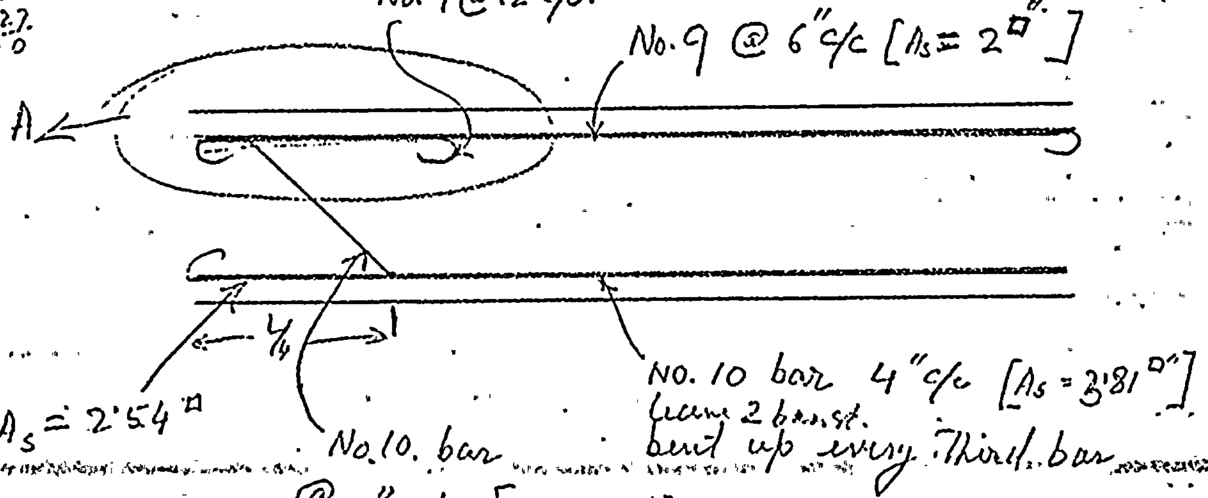
4″

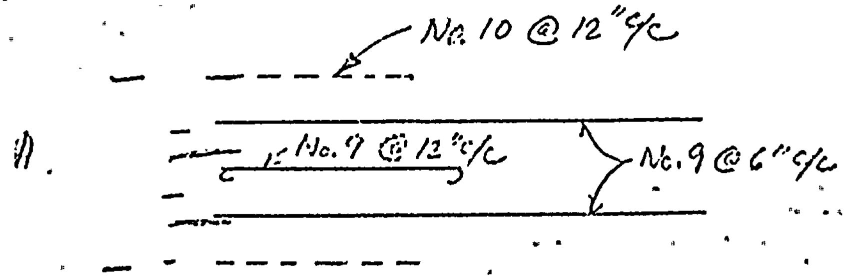
4″

$$A'_{s1} = \frac{103}{1{\cdot}76 \times 27} = 2{\cdot}17\ \square''$$

4·17
3·27
·90

1·27
2·00
1·27
0·54



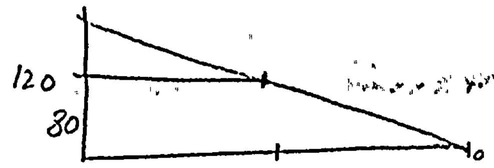


RGE PROJECT UNIT 1	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
PRESSURIZOR SUPPORT	MADE / CHK'D / NO CF / CK'D / APPD / REV	SIZE DRAWING NO. 1:36·1/56 REV. / W. O. / DATE	SHEET

Shear:

$\frac{7\times 12}{120}\ 40 = 28''$ to

$$v_{c} = \frac{V}{bd} = \frac{39000}{12\times 27} = 120\text{ psi}$$



$v_c = 60$ psi

133% $v_c = 80$ psi

Steel shear — $120-80 = 40$ psi

$V' = 40\times 12\times 27 = 13000$ #

$$A_v = \frac{V'}{f_v'\cdot \sin\alpha} = \frac{13000}{24000\times \sin 45^{\circ}} = \frac{13}{24\times .707}$$

$= 0.766$

Av provided $= 1.27\ \square''$ O.K.

Bond: $\frac{V}{jd} = \frac{39000}{7/8\cdot 27} = 1650$

		ΣO
3 No. 9 bars	3 $\square''$	14.2
1 " 10 "	1.27	4.0
	4.27	18.2

$u = \frac{1650}{18.2} = 90.5$ psi

1492

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PRESSURIZOR SUPPORT	MADE / CHKD / SCALE / CK DN / APPD	SIZE DRAWING NO 1:36.1/57 W.O. DATE	REV SHEET

16

Allowable bond stress

$$U = \frac{3.4\sqrt{f'_c}}{D} = \frac{3.4\sqrt{3000}}{1.25} = \frac{3.4 \times 54.6}{1.25}$$

$$= 149 \text{ psi} < 350 \quad O.K.$$

FILE CODE 1:36·1/58

BROOKWOOD UNIT No. 1 CONTAINMENT VESSEL ACCUMULATOR SUPPORT	READING PENNA	GILBERT ASSOCIATES INC — ENGINEERS AND CONSULTANTS — NEW YORK WASHINGTON
	MADE: KEN	CHKD / SUPV / APPD / SIZE / DRAWING NO. / REV. / SHEET: IV-1-2
	REV.	W.O. 4755 DATE 12/5/6

1750 FT^3 ACCUMULATOR

Volume = $\pi/4\,(9.5^2)\,18.5 + 4/3\,(\pi)\,4.75^3 = 1310 + 448 = 1758\ FT^3$

Total weight = $1758 \times 62.5 \times 1.2 = \underline{131{,}000}^{\#}$

Determine natural frequency.

Assume all mass lumped at C.G. (See p. III-3-2a)

Assume $t = 1\tfrac{1}{2}''$ for the cylinder

$I = 0.049087\,(9.75^4 - 9.5^4) = 43.7766\ FT^4$

$A = \frac{\pi}{4}\,(9.75^2 - 9.5^2) = 3.7778\ FT^2$

Bending deflection:

$\Delta_B = \frac{PL^3}{3EI} = \frac{131\,(17.67)^3}{3\,(29)(10^3)(43.78)(12)} = 0.015{,}813''$

Shear deflection:

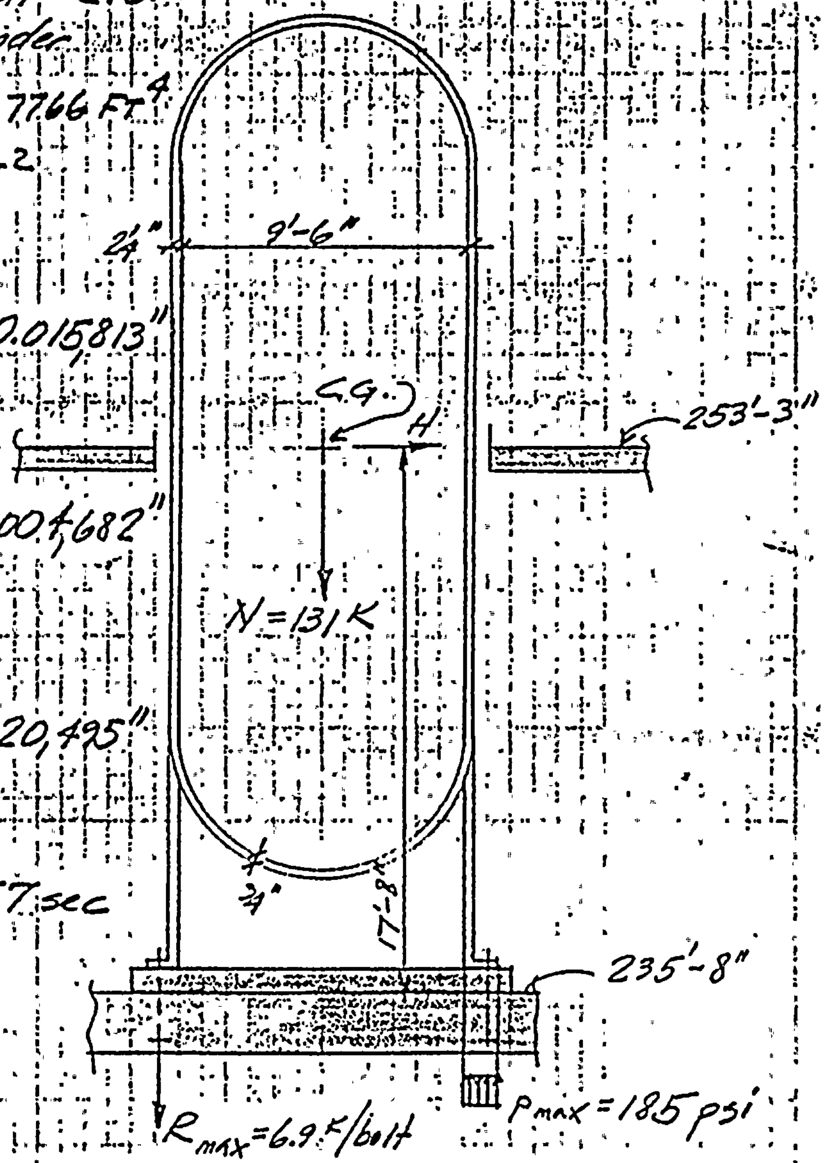
$\Delta_S = \frac{VL}{AG} = \frac{131\,(17.67)}{3.78\,(10.9)(10^3)(12)} = 0.004{,}682''$

$G = \frac{29(10^3)}{2.66} = 10.9\,(10^3)$

$\Delta_T = 0.015{,}813 + 0.004{,}682 = 0.020{,}495''$

Natural period

$T = 2\pi\sqrt{\frac{0.020{,}495}{387}} = 0.0457\ sec$



Max. response under 0.2g ground motion and 1% damping → $\underline{24.5\%}$

$H = 0.245\,(131) = 32.1^K$

$M = 32.1\,(17.67) = 567\ K\text{-}FT$

$N_{min} \approx 131\,(\underbrace{1 - 0.245}_{0.755}) = 99^K$ $\qquad N_{max} = 131\,(1 + 0.245) = 163^K$

$f = -\frac{99}{3.78} \pm \frac{567}{43.78}\left(\frac{9.75}{2}\right) = -26.2 \pm \frac{567}{9} = -26.2 \pm 63.0 \left\{ \begin{array}{l} f_1 = +36.8\ K/FT^2 \\ f_2 = -89.2\ K/FT^2 \end{array} \right.$

or

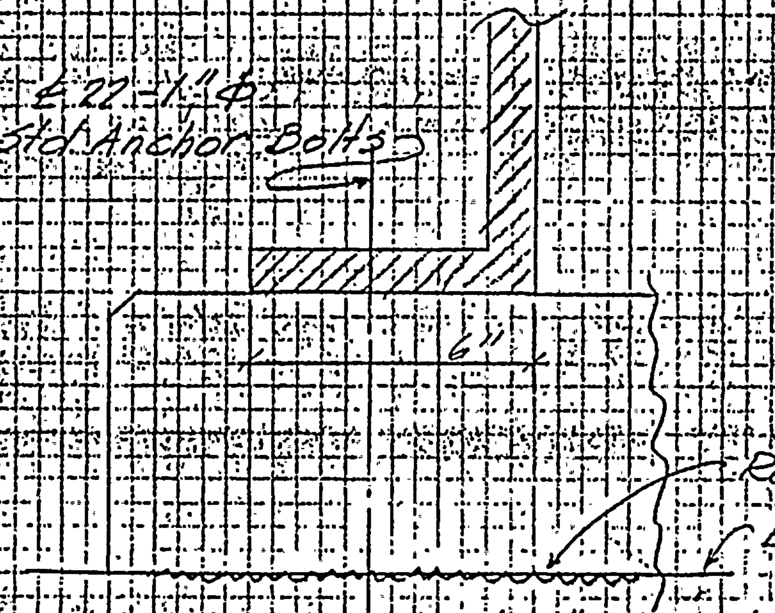
$f = -\frac{163}{3.78} \pm 63.0 = -43.2 \pm 63.0 \left\{ \begin{array}{l} f_3 = +19.8\ K/FT^2 \\ f_4 = -106.2\ K/FT^2 \end{array} \right.$

FILE CODE 1:36.1/59

BROOKWOOD UNIT No. 1 CONTAINMENT VESSEL	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
	MADE: KEN	CHKD / SQ CF / CF DN / APPD	SIZE DRAWING NO. REV. SHEET: IV-1-3
ACCUMULATOR SUPPORT	REV.		W.O. 4155 DATE 12/5/66

$R_{max} = 36.8(1)\left(\frac{1.5}{12}\right) = 4.6\ ^{K}/_{FT}$ (circumferential uplift)

$P_{max} = 106.2(1)\left(\frac{1.5}{12}\right)\left[\frac{1}{(6)(12)}\right] = \frac{13.3}{72} = \underline{0.185\ ksi}$ (bearing stress)



Max uplift per bolt $= 4.6(1.5) = \underline{6.9^{K}/bolt}$

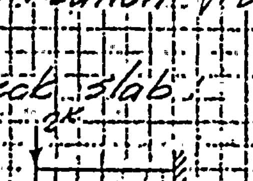
Uplift resistance:

D.L. Slab $\quad 2.5(16.5)(2)(\tfrac{1}{2})(0.15) = \quad 6.2^{K}$

Contribution from embedded struct. T's ~ $\quad \underline{2.0}$

$8.2^{K} > 6.9^{K}$ Say O.K.

Check slab



$W \cong 2.5(6)(2)(0.15) = 4.5^{K}$

$M = 4.5(3) + 2(6) = 13.5 + 12 = 25.5\ ^{K-FT}/1.5\ FT$

or $M = 17\ ^{K-FT}/FT$

$K = 226 \qquad F = \frac{17}{226} = 0.075 \qquad d_{req'd} = 9\ in$

Top steel req'd: $A_s = \frac{17}{1.44(16.5)} = 0.716\ in^2/FT$ circumferential

$d = 16.5\ in$

Use: #8 @ 12" c/c, radial @ D = 10'-5

top steel

Steam Generator and Reactor Coolant Pump Supports

Support structures of the above major components were designed for loads resulting from ruptures of the primary coolant piping and main steam piping. Equivalent static seismic forces equal to the components weight, accelerated by the peak response of the applicable seismic response spectra, applied through the component's center of gravity, were evaluated against the corresponding pipe rupture loads. For both the steam generators and reactor coolant pumps, the resulting seismic forces were smaller than the pipe rupture loads, therefore, supports were designed for pipe rupture loads.

Consideration of the Liner in Structural Analysis and Design

The liner is included in the structural analysis of the containment shell. Its inclusion would appear in the calculation of k, I_z, and β on FSAR page 5.1.2-46b.

The liner is taken to participate with the hoop reinforcement steel in arriving at the hoop rebar stresses in the cylinder and dome tabulated on FSAR page 5.1.2-51h. The meridional rebar stresses in the dome also reflect inclusion of the liner. The concrete and meridional reinforcement stresses tabulated for the wall on FSAR page 5.1.2-51h were calculated without including the liner because the liner was located on the compression face of the section.

Buckling Evaluation

Justification for including the liner in the analysis and in the cracked section stress evaluation, relies on whether or not the liner is buckled. This condition is evaluated below for the liner stresses and buckling strengths appearing in the FSAR.

- Cylinder

  Operation Condition

  The maximum compressive liner stress appearing on FSAR page 5.1.2-46e is 14.3 ksi compression in the meridional direction and is accompanied by a 2.6 ksi compressive hoop stress. The theoretical buckling capacity in the meridional direction is 16.6 ksi for no accompanying hoop stress,

as shown on FSAR page 5.1.2-51f. Since the actual tested buckling capacity may be less than its theoretical value, the absence of meridional buckling cannot be assured in the cylindrical wall. The small compressive hoop stress precludes buckling in the hoop direction.

Accident Condition

The insulation of the liner for accident temperature prevents an increase of the 14.3 ksi compressive stress. As shown on FSAR page 5.1.2-46e, the meridional compressive stress is actually reduced to -2.9 ksi, due to the accident pressure while accompanied by 27.0 ksi hoop tension. This tensile stress would allow for a meridional buckling strength even greater than the 38.1 ksi shown in the FSAR page 5.1.2-51f table.

Dome

Operation Condition

Since the dome is not prestressed, only a small compressive liner stress of -2.4 ksi exists, due mainly to the operation temperature. Consequently, the liner is not stressed high enough for buckling to be a concern.

Accident Condition

Even though the liner is not insulated, practically all of the thermal stress is relieved as the dome expands under the accident pressure. As a result, the net liner stresses are actually tensile and are less than 10 ksi (equal biaxial).

Incidentally, the equal biaxial buckling strength is 26.4 ksi compression as noted on FSAR page 5.1.2-51d.

Results

Liner buckling in the cylindrical wall due to meridional operation stresses cannot be ruled out. Buckling of the liner in the hoop direction in the wall and in both directions in the dome is not predicted to occur.

As discussed above, the liner was not considered in the cracked section investigation of the concrete and rebar stresses in the meridional direction of the cylindrical wall. However, the structural analysis did include the liner.

Effect of Liner in Structural Analysis

In the structural analysis, the effect of the liner in the meridional direction is thru I_z and β, which are defined on FSAR page 5.1.2-46b (factor of 4 is missing in the denominator under the radical). A comparison of these parameters is given below for the original design, in which the liner is included, and for the condition of no liner.

Original Design

$$I_z = \frac{1}{12}(1)\left[42 + n(.375)\right]^3$$

$$I_z = 7600 \text{ in}^4 \text{ (used)}$$

$$\beta = \sqrt[4]{\frac{k}{4EI_z}} \quad \text{(Page 5.1.2-46c)}$$

$$\beta = \sqrt[4]{\frac{116.5}{(4)(4.1 \times 10^6)(7600)}} = 0.00553/\text{in. (used)}$$

Without Liner

$$I_z = \frac{1}{12}(42)^3 = 6174 \text{ in}^4$$

$$\beta = \sqrt[4]{\frac{116.5}{4(4.1 \times 10^6)(6174)}} = 0.00582/\text{in.}$$

This small difference in β would have an insignificant effect on the moment and shear given by the equations on FSAR page 5.1.2-46b.

Conclusion

The discussion above provides support for inclusion of the liner in the original design structural analysis and section investigations.

Tangential shears are discussed in the FSAR as "Longitudinal" and "Horizontal" shears on pages 5.1.2-44 and-45. Related information appears on FSAR pages 5.1.2-18 and -51b.

These shears are induced due to the restraint on the containment shell at its base subjected to the horizontal earthquake inertial forces of the shell. Physically, this restraint would be available (in varying degrees) from (1) the radial tension rods, (2) the 2' structural mat plus the 2' floor from El. 231'-8" to El. 235'-8", and (3) frictional shear resistance that the Neoprene Pad provides on the wall where net compression exists at this interface.

On FSAR page 5.1.2-51b, a maximum tangential shear value of 67.2^k/ft. is given, which is due to 0.2g ground acceleration. This shear value is produced by the conservative 0.46g response acceleration discussed on FSAR page 5.1.2-17. A more accurate seismic analysis is discussed on FSAR pages 5.1.2-17a and -17b, and the resulting shear force is shown in FSAR Figure 5.1.2-8F. From FSAR Figure 5.1.2-8F, the maximum value of tangential shear, due to 0.2g ground acceleration, is

$$V_{max} = 2V_{ave} = \frac{2 \times 6700^k}{\pi \times 108.5 \text{ ft.}} = 39.3^k/\text{ft.}$$

(Containment Diameter = 108.5 feet)

The corresponding tangential shear stress on the gross concrete section, v'_u , is

$$v'_u = \frac{Vmax}{12" \times 42"} = \frac{39.3^k}{504 \text{ in}^2} = 78 \text{ psi}$$

The 78 psi (39.3^k/ft.) tangential shear is evaluated (below) for the horizontal and vertical shearing planes of the wall.

Horizontal Plane

Due to the presence of the vertical prestress, the horizontal plane at $\theta = 90^\circ$ and 270° ($0^\circ - 180^\circ$ axis being the direction of earthquake) is under 134^k/ft membrane compression for load combination #41. The 78 psi (39.3^k/ft) tangential shear corresponds to 2E in this load combination and it occurs at $\theta = 90^\circ$ and 270°. The 134^k/ft membrane compression provides more than enough shear friction clamping force to resist the 39.3^k/ft shear.

Vertical Plane

Since the containment is not prestressed in the hoop direction, its ability to resist the tangential shear is evaluated using the provisions of CC-3421.5.1 of the ASME Section III, Division 2 Containment Code. These provisions apply to tangential shear on reinforced concrete sections which do not have the benefit of prestress compression.

The 3 #18 @ 9" hoop bars in the shell wall provide a reinforcement ratio , ρ , of 0.0317. The corresponding allowable shear stress carried by the concrete, v_c, is its maximum value of 160 psi. Since v_c of 160 psi exceeds v_u of 78 psi, the concrete can carry this tangential shear provided the three requirements of item (b) in CC-3421.5.1 are met. Item (1) is met because the minimum specified concrete compressive strength is 5000 psi, which is greater than the 3000 psi requirement. Item (2) is met bacause ASTM C131 tests (conducted in 1975) of aggregate from the same geographic area as that used in the containment shell indicate a maximum weight loss of 29% which is less than the 40% maximum requirement. Item (3) is met because the cracked surface is produced by tensile fracturing of the concrete due to 1.0 P and, consequently, would be of sufficient roughness.

NRC Question III.2:

GAI calculations Filing Code 1:36.1, pages 69 thru 81

GINNA STA. — UNIT No. 1 Containment Vessel Shearintecaction between liner concrete and dowel action	MADE KEN	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
		4155 WORK ORDER
		11-13-68 II-A-1

SHEAR INTERACTION — LINER, CONCRETE, DOWEL ACT.

The purpose with this study is to determine the stress in the liner during an earthquake. The earthquake shear stress will be combined with liner stresses due to Dead Load, Vertical Prestress incl. the effect of creep and shrinkage, Internal Pressure, and Temperature.

Longitudinal shear due to "0.2g" groundmotion:

Dead Load at base of C.V. $= 71^{K}/_{FT}$

Max Earthquake responce $= 0.474$

Max. Shear: * $V_{max} = \alpha \frac{V_E}{A} = 2.0(0.474)(71) = \underline{67.2\ ^{K}/_{FT}}$

Determine the shear distribution between the liner and the concrete Containment Wall. Analyse the wall for composite action in shear, considering the presence of vertical cracks with dowels.

Then: $\eta_V = \frac{G_{LINER}}{G_{WALL}}$

A. Determine G_{LINER}

$$G_L = \frac{E}{2(1+\mu)} = \frac{29(10^3)}{2(1.3)} = \underline{\underline{11.2(10^3)\ \frac{ksi}{in/in}}}$$

* Shear in containment Vessel based on a dynamic modal analysis is: (FSAR, Figure 5.1.2-8F)

$V_{base} = 6700^{K}$, $v_{avg} = \frac{6700}{\pi(108.5)} = \frac{6700}{341} = 19.7\ ^{K}/_{FT}$, $v_{max} = 2(19.7) = \underline{39.4\ ^{K}/_{FT}}$

$V_{20'} = 6500^{K}$, $v_{max} = \underline{38.2\ ^{K}/_{FT}}$

$V_{40'} = 6000^{K}$, $v_{max} = \underline{35.3\ ^{K}/_{FT}}$

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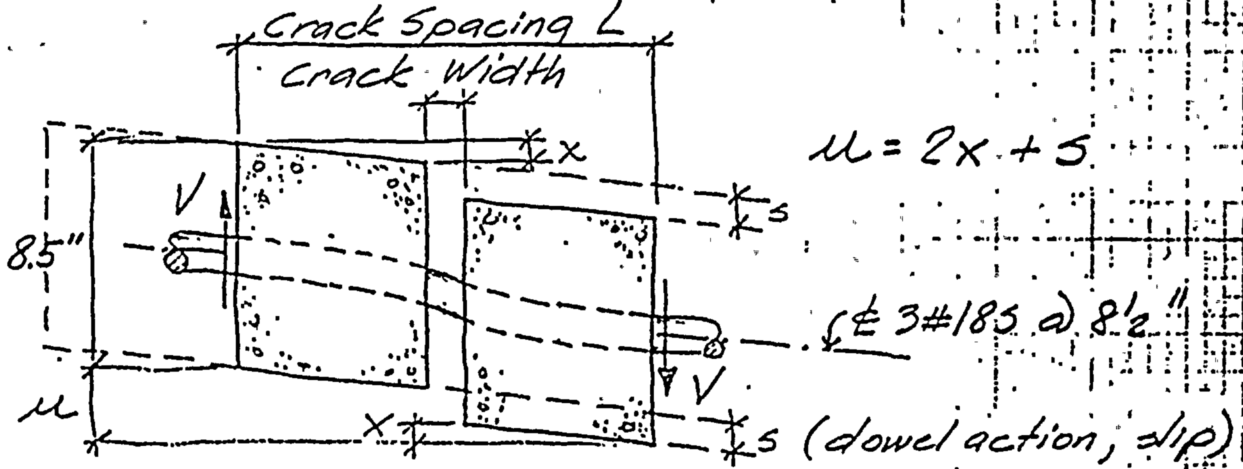
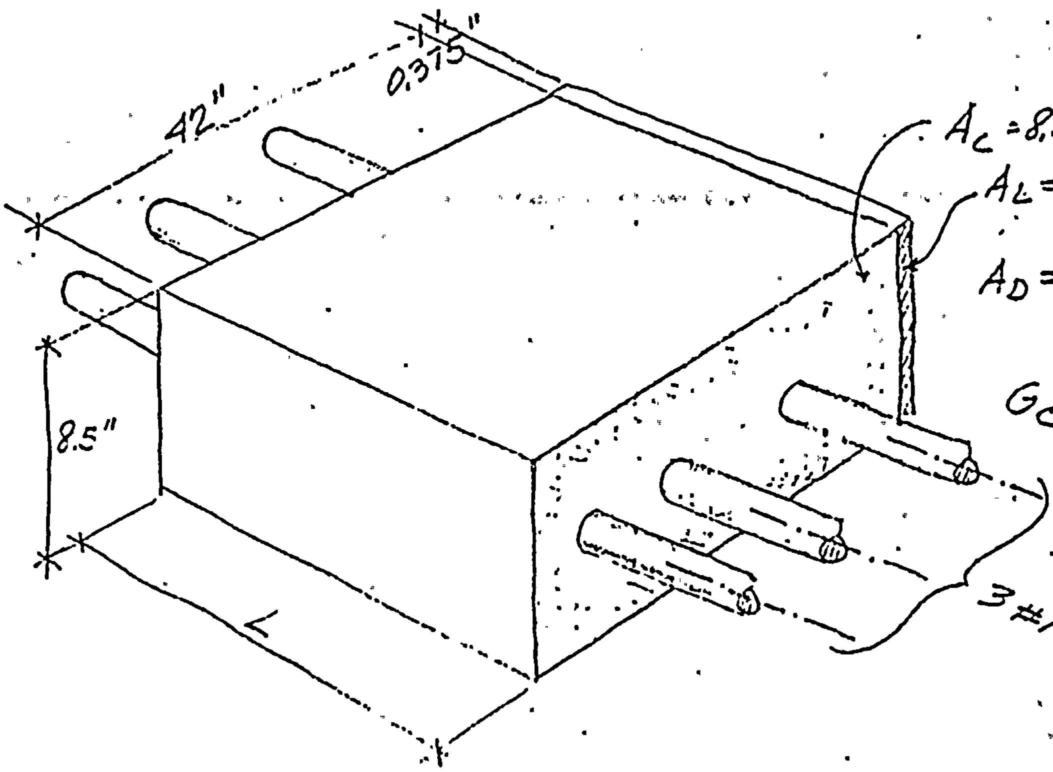
B. Determine G_{WALL}

The effective shear modulus of the concrete wall will be based on pure shear on the uncracked concrete plus the dowel action of the horizontal reinforcement across a vertical crack.

Model





$A_C = 8.5(42) = 357.\ in^2$

$A_L = 8.5(.375) = 3.19\ in^2$

$A_D = 3(4) = 12.\ in^2$

$G_C = \dfrac{4.1(10^3)}{2(1.15)} = 1.8(10^3)\ k/in.$

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$$u = 2x + s$$

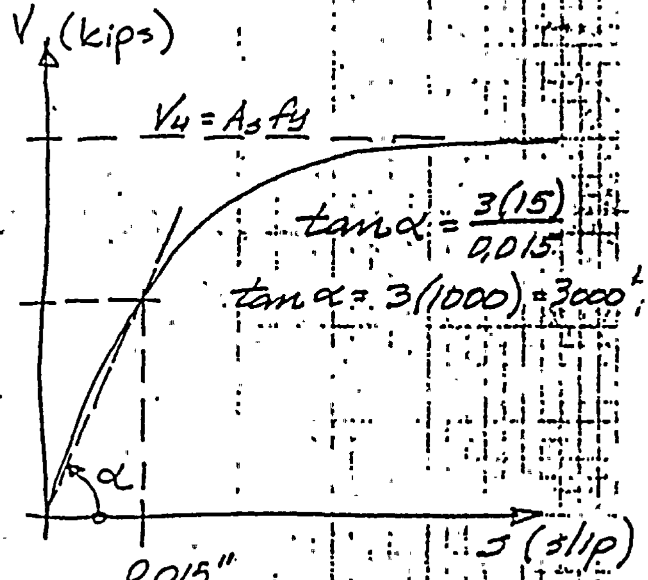
$$x = \frac{V}{A_c G_c} \times \frac{L}{2}$$

$$V = 3000(s)$$

$$s = \frac{V}{3000}$$

$$u = \frac{VL}{A_c G_c} + \frac{V}{3000}$$

$$\frac{u}{L} = \frac{V}{A_c G_c} + \frac{V}{3000L} = \gamma$$



DOWEL ACTION
Load - Slip Curve

Then:

$$\frac{\cancel{V}}{A_c G_W} = \frac{\cancel{V}}{A_c G_c} + \frac{\cancel{V}}{3000L}$$

$$\frac{1}{G_W} = \frac{1}{G_c} + \frac{A_c}{3000L}, \quad \frac{1}{G_W} = \frac{3000L + A_c G_c}{3000 G_c L}$$

$$G_W = \frac{3000 G_c L}{3000L + A_c G_c}, \quad \boxed{G_W = \frac{G_c}{1 + \frac{A_c G_c}{3000L}}}$$

For $L = 25''$: $G_{W_{25}} = \dfrac{1.8(10^3)}{1 + \dfrac{357(1.8)(10^3)}{3(10^3)(25)}} = \dfrac{1.8(10^3)}{1 + 8.6} = \underline{0.188(10^3)\text{ ksi}}$

For $L = 50''$: $G_{W_{50}} = \dfrac{1.8(10^3)}{1 + 4.3} = \underline{0.340(10^3)\text{ ksi}}$

For $L = 100''$: $G_{W_{100}} = \dfrac{1.8(10^3)}{1 + 2.15} = \underline{0.572(10^3)\text{ ksi}}$

0.095 (10^3) ksi

For $L = 12''$: $G_{W_{12}} = \dfrac{1.8(10^3)}{1 + 17.9} = \underline{0.1(10^3)\text{ ksi}}$

SHEAR DISTRIBUTION

Crack Spacing L	G_W	$\mu_V = \frac{G_L}{G_W}$	$\mu_V A_L$	$A_C + \mu_V A_L$	Liner Shear $V_L = V \times \frac{\mu_V A_L}{A_C + \mu_V A_L}$	Liner Shear stress $f_{VL} = \frac{V_L}{A_L}$	Conc. Shear $V_C = V \times \frac{A_C}{A_C + \mu A_L}$	Conc. Shear stress $f_{VC} = \frac{V_C}{A_C}$
25	$0.188(10^3)$	59.5	190.0	547.	16.7	5230 psi	31.0 *	87.0 psi
100	$0.572(10^3)$	19.6	63	420	7.2	2260 psi	40.5 **	113.5 psi
12	$0.095(10^3)$	118	376	733	24.5	7,680 psi	23.2 (1)	65.0 psi
8	$0.065(10^3)$	172	550	907.	28.9	9050 psi	18.8 (2)	52.6 psi

$G_L = 11.2(10^3)$ ksi

$A_L = 3.19$ in^2

$A_C = 357.0$ in^2

$V = 8.5/12\,(67.2) = 47.7^K$

* Check:

$$u = \frac{\tau}{G_W} L = \frac{0.087}{0.188(10^3)}(25) = 0.0116 \text{ in}$$

$$-2x = \frac{VL}{A_C G_C} = \frac{0.087}{1.8(10^3)}(25) = \underline{0.0012}$$

$$s = \underline{\underline{0.0104 \text{ in}}}$$

$$V = \tan\alpha(s) = 3000(0.0104) = \underline{\underline{31.2^K \cong 31.0^K}} \quad \text{O.K.}$$

** Check:

$$u = \frac{0.1135}{0.572(10^3)}(100) = 0.0198 \text{ in}$$

$$-2x = \frac{0.1135}{1.8(10^3)}(100) = \underline{0.0063}$$

$$s = \underline{0.0135 \text{ in}}$$

$$V = 3000(0.0135) = \underline{40.5^K} \quad \text{O.K.}$$

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CHECK :

(1) $\mu = \frac{.065}{.095(10^3)}(12) = .00821$

$-2x = -\frac{.065}{1.8(10^3)}(12) = -.00043$

$.00778$

$U = \tan\alpha\,(s)$

$= 3000(0.00778)$

$= 23.3^k \approx 23.2^k$ ok

(2) $\mu = \frac{.0526}{.065(10^3)}(8) = .00650$

$-2x = \frac{.0526}{1.8(10^3)}(8) = -.00023$

$.00627$

$U = 3000(.00627)$

$= 18.8^k$ ok

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		11-13-68 · II-A-6

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Maximum allowable liner shear:

Determine τ_{max}.

Existing stresses,

Stress reversal in liner during accident pressu...

$\sigma_y = -14100 + \overbrace{600(10)} = -8100$ psi

$\tau_{max} = ?$

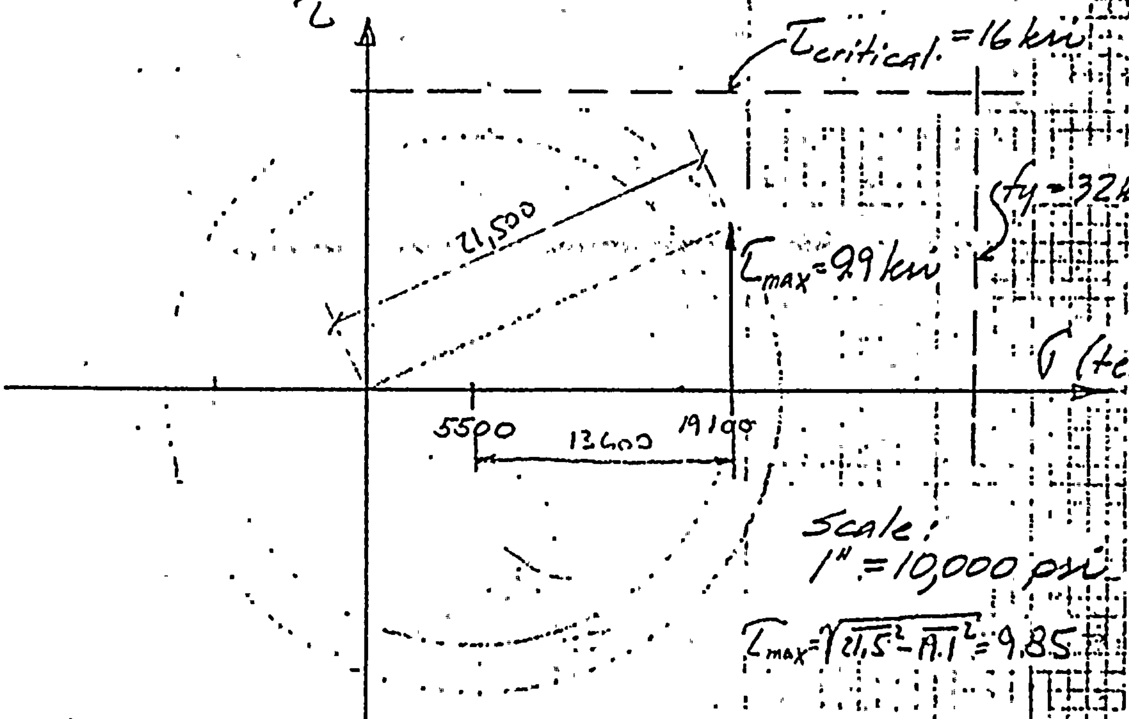
Liner

$\sigma_x = 28700\left(\frac{60}{90}\right) = +19100$ psi

By Mohr's circle,

$$\frac{\sigma_x + \sigma_y}{2} = \frac{19100 - 8100}{2} = \underline{5500\ psi}$$

$$\frac{\sigma_x - \sigma_y}{2} = \frac{19100 + 8100}{2} = \underline{13600\ psi}$$



Scale:
1" = 10,000 psi

$\tau_{max} = \sqrt{21.5^2 - 19.1^2} = 9.85$

$\tau_{max} = \underline{\underline{9.9\ ksi > 5.23\ ksi}}$ p.4 O.K.

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GINNA	KEN	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
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Longitudinal shear – Earthquake		II-A-7

LINER – DOWEL ACTION

A. Investigate liner stresses at the base of C.V. for maximum principal stress due to 0.2g groundmotion plus the other loads associated with the earthquake load.

Load Comb. #46-42: (3' above base)

$N_\phi = \frac{1}{2}(-476-370.8) = -184.2$ K/FT $N_\theta = +222.4$

$M_\phi = +98.8$ K-FT/FT $M_\theta = +99.5$ (Winter)

Earthquake in plane vertical shear p. II-A-1. $V_{max} = 39.4$ K/FT

$\tau = \frac{39.4}{12(.375)} = \underline{8.8 \text{ ksi}}$ (ignoring dowel action)

Meridional liner stress: (Ref.: p. I-H-3)

$f_\phi = 0.0318(-184.2) - 0.0183(98.8) = -10.2 - 1.8 =$ -5.9 ksi

p. I-E-12 Accident temper. $f_\phi = -8.0/4.5 =$ -1.8

p. I-E-19c Operating temper. $f_\phi = (-.125)(7.1) =$ -0.9

$f_\phi =$ -8.6 ksi

Hoop liner stress: (Ref.: p. I-H-4)

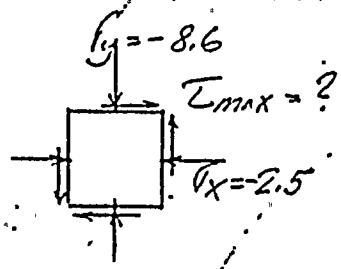
$f_\theta = 0.0465(+222.4) - 0.0613(+99.5) = +10.4 - 6.1 =$ $+4.3$ ksi

p. I-E-12 Accident temper. $f_\theta = -6.7/4.5$ -1.5

p. I-E-18 Operating temper. $f_\theta =$ -5.3

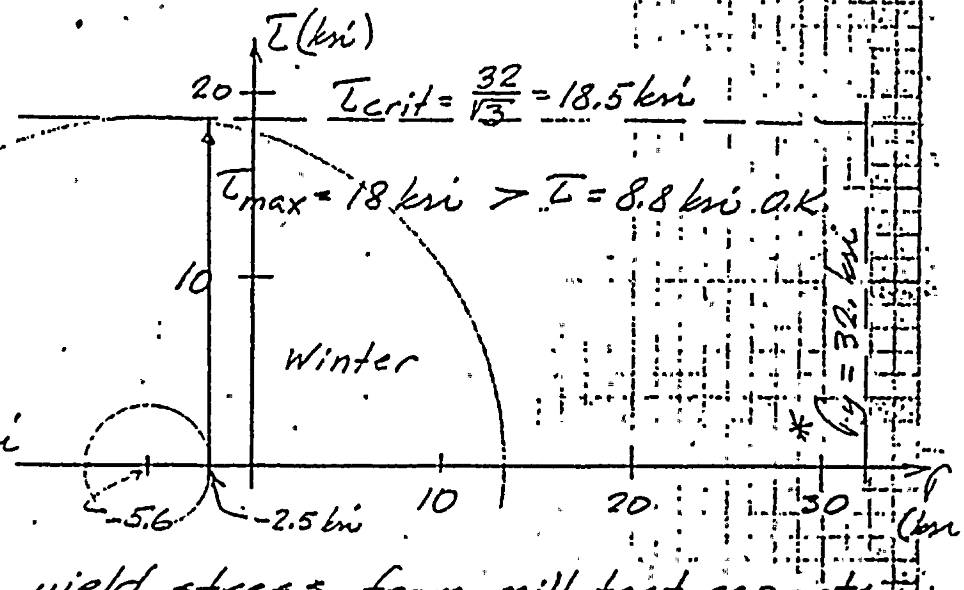
$f_\theta =$ -2.5 ksi

Principal stress:



$\frac{\sigma_x + \sigma_y}{2} = \frac{-2.5-8.6}{2} = -5.6$ ksi

$\frac{\sigma_x - \sigma_y}{2} = \frac{-2.5+8.6}{2} = 3.1$ ksi



* Note:

Minimum liner plate yield stress from mill test reports $\sigma_y = 45.0$ ksi

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C.V.

Longitudinal shear - Earthquake

$\tau_{max} = 18.0$ ksi > 8.8 ksi. Therefore, the liner should be able to resist the in-plane earthquake shear stresses without the contribution of hoop rebar dowel action. Furthermore, two rows of anchor studs have been provided between the channel anchors. These studs will eliminate the possibility of a premature liner buckling before it could develop its ultimate capacity.

The anchor studs are used up to 17'-0" above the base.

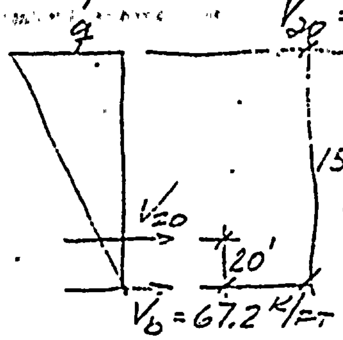
B. Investigate liner stresses 20' above base of C.V. for maximum principal stress due to 0.2g ground motion plus the other loads associated with the earthquake.

Load Comb. # 46-42: (20' above base)

$N_\phi = \frac{1}{2}(-289.1 - 61.5) = -175.3$ K/FT $\quad N_\theta = +410.2$ K/FT

$M_\phi = +310.5$ K-FT/FT $\quad M_\theta = +99.5$ K-FT/FT (Winter)

Eartquake in plane vertical shear: (based on max responce)



$V_{20} = \underline{\underline{38.2 \text{ K/FT}}}$ based on dynamic model

$\frac{1}{2} q\, 150 = 67.2, \quad q = 0.9$

$q_{20} = 0.9\left(\frac{20}{150}\right) = 0.12$

$V_{20} = 67.2 - \frac{1}{2}(0.12)(20) = 66.0$ K/FT

$\tau_{liner} = \frac{38.2}{12(0.375)} = \underline{\underline{8.5 \text{ ksi}}}$ (Ignoring dowel action)

Meridional liner stress: (Ref.: p. I-4-3)

$f_\phi = 0.0318(-175.3) - 0.0183(310.5) = -5.6 - 5.7 =$ -11.3 ksi

Accident temp. -1.8

Operating temp. -0.9

$f_\phi =$ $\underline{-14.0 \text{ ksi}}$

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C.V.
Longitudinal shear - Earthquake

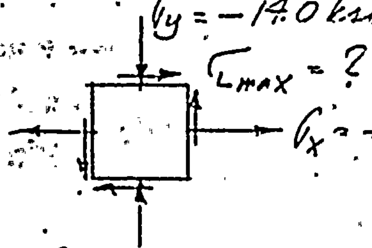
Hoop liner stress: (Ref.: p. I-H-4)

$f_\theta = 0.0465(+410.2) - 0.0613(+99.5) = +19.1 - 6.1 =$ $+13.0$ ksi

Accident temp. -1.5

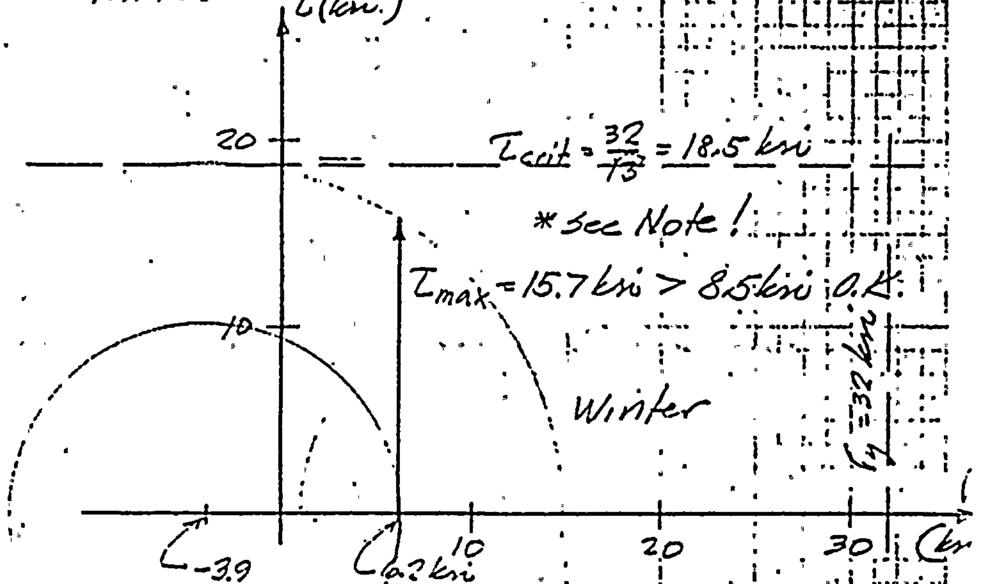
Operating temp. -5.3

f_θ $= +6.2$ ksi



$$\frac{\sigma_x + \sigma_y}{2} = \frac{6.2 - 14.0}{2} = -3.9 \text{ ksi}$$

$$\frac{\sigma_x - \sigma_y}{2} = \frac{6.2 + 14.0}{2} = +10.1 \text{ ksi}$$



* Note:

$\tau_{max} = 15.7$ ksi > 8.5 ksi O.K.

Min. liner plate yield stress from mill test reports $\underline{\sigma_y = 45 \text{ ksi}}$

Investigate liner stability:

stability of the liner plate under biaxial stresses and shear, ref.: Roark, "Formulas for Stress and Strain," Table XVI, Case E-13, p. 350.

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II-A-10

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Longitudinal shear - Earthquake

$S_x = +14.0$ ksi (compr.)
$S_y = -6.2$ ksi (tens.)

$$C = \frac{0.823}{1-0.3^2}\left(\frac{0.375}{49.5}\right)^2 = 520(10^{-6})$$

Critical unit shear stress:

$$S_s' = \sqrt{(520(10^{-6}))^2 \Bigg(\underbrace{\underbrace{2.31\sqrt{4-\frac{-6200}{520(10^{-6})}}}_{3450 \;\; 7970} \underbrace{+\frac{4}{3}-\frac{14000}{520(10^{-6})}}_{-26,900,000}}_{-26,892,030}\Bigg)\Bigg(\underbrace{\underbrace{2.31\sqrt{4-\frac{-6200}{520(10^{-6})}}}_{+7,970} \underbrace{+8-\frac{176000}{520(10^{-6})}}_{-26,900,000}}_{-26,892,000}\Bigg)}$$

$+722.(10^{12})$

$$S_s' = \sqrt{(270,000)\cancel{(10^{-12})}(722)\cancel{(10^{12})}} = \sqrt{195,000,000} = \underline{14,000\ psi}$$

$S_s' = 14.0$ ksi > 8.5 ksi, therefore the liner is stable.

Load Comb. # 48-44: (20' above base)

$N_\phi = \frac{1}{2}(-289.1-61.5) = -175.3$ K/FT $\quad N_\theta = +422.1$ K/FT (Summer)

$M_\phi = +236.0$ K-FT/FT $\quad M_\theta = 0.0$

Meridional liner stresses:

$f_\phi = 0.0318(-175.3) - 0.0183(236.0) = -5.6 - 4.3 =$ -9.9 ksi
Accident temp. -1.8
Operating temp. 0.0

$f_\phi = \underline{-11.7\text{ ksi}}$

Hoop liner stresses:

$f_\theta = 0.0465(422.1) =$ $+19.6$ ksi
Accident temp. -1.5
Operating temp. 0.0

$f_\theta = \underline{+18.1\text{ ksi}}$

$\tau_{liner} = 8.5$ ksi

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KEN

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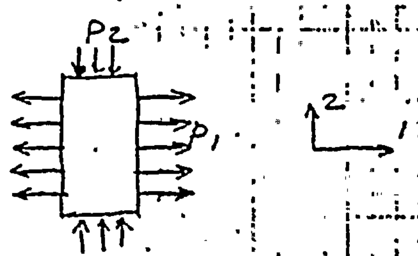
4155			
ORDER	SITE	DRAWING	REV.
		II-A-106	

GINNA

C.V.

Liner Stability:

$$\sigma_{2\,cr} = -11.6 \frac{1-f\nu}{(1-\nu)(1+f)}$$



$$f = \frac{p_1}{p_2}$$

a) $p_1 = +6.2$ ksi, $p_2 = -14.0$ ksi } Winter, 20' above base

$\tau = 8.5$ ksi

$$f = -\frac{6.2}{14} = -0.443$$

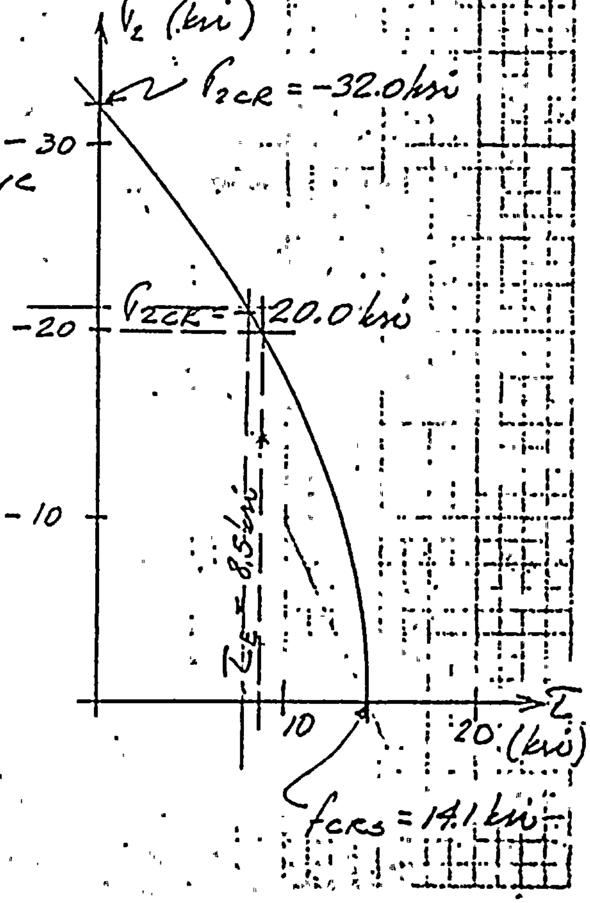
$$\sigma_{2\,cr} = -11.6 \frac{1+0.443(0.3)}{(0.7)(0.557)} = -11.6 \frac{1.133}{0.39} = -33.7 \text{ ksi}$$, Use: $\sigma_{2\,cr} = -32.$

$$\sigma_1 = f\,\sigma_{2\,cr} = (-0.443)(-33.7) = +14.95 \text{ ksi}$$

Critical shear: Ref.: USS, "Steel Design Manual" p. 79

$$f_{crs} = \frac{26.2(10^6)(8.98)}{\left(\frac{49.5}{0.375}\right)^2} = \frac{245(10^6)}{17400} = 14,100 \text{ psi}$$

$\sigma_{2\,cr} = -20.0$ ksi for $\tau = 8.5$ ksi, $p_2 = -14.0$ ksi $<$ -20.0 ksi, therefore the liner is stable.



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II-A-10c

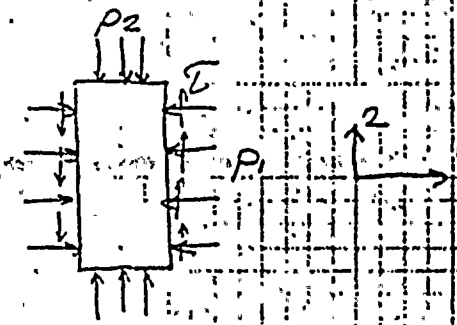
GINNA

C.V.

Longitudinal Shear - Earthquake

LINER STABILITY

Applied stresses



$$* \begin{cases} \sigma_{2cR} = -11.6 \dfrac{1-f\nu}{(1-\nu)(1+f)} \\ f = \dfrac{p_1}{p_2} \\ \sigma_{1cR} = f\,\sigma_{2cR} \end{cases}$$

By large Deflection Theory. Assumed a wave surf. and a cylindr. surface.

Liner stresses 20' above base due to: $DL + VP + OT + IP + E_{0.2g}$

$p_1 = +6.2$ ksi
$p_2 = -14.0$ ksi
$\tau = 8.5$ ksi
} Winter.

$f = \dfrac{6.2}{-14} = -0.443$, $\sigma_{2cR} = -11.6 \dfrac{1+0.443(0.3)}{(1-0.3)(1+(-0.443))} = \underline{-33.7 \text{ ksi}}$

$\sigma_{1cR} = (-0.443)(-33.7) = +\underline{14.95 \text{ ksi}}$

Critical shear:

Ref.: "Theory of Elastic Stability", by Timoshenko & Gere, p. 384. Also USS, "Steel Design Manual", p. 79.

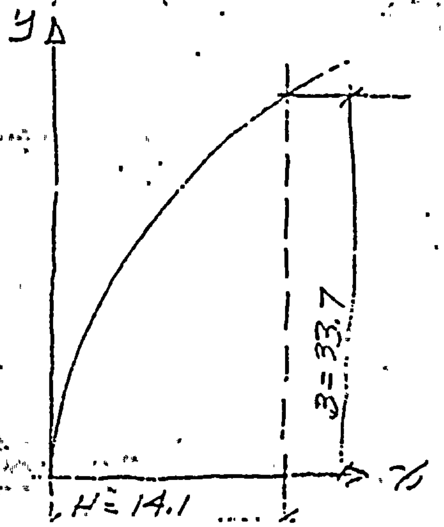
$f_{cRs} = \dfrac{26.2(10^6)(8.98)}{\left(\dfrac{49.5}{0.375}\right)^2} = \underline{14,100 \text{ psi}}$

Interaction diagram:

The interaction curve will be represented by a parabolic curve $y^2 = px$, $p = \dfrac{B^2}{H} = \dfrac{33.7^2}{14.1} = 80.5$

$y^2 = 80.5x$

For $x = 3.5 \longrightarrow y = 16.8$
For $x = 7.0 \longrightarrow y = 23.8$
For $x = 10.5 \longrightarrow y = 29.0$



* Lehigh University - Dep. of Civil Engr.
Post-Buckling Deflection Modes of Liner Plates in Prestressed Concrete Reactor Vessels
by Umut Ulkumen - Dr. A. Ostapenko

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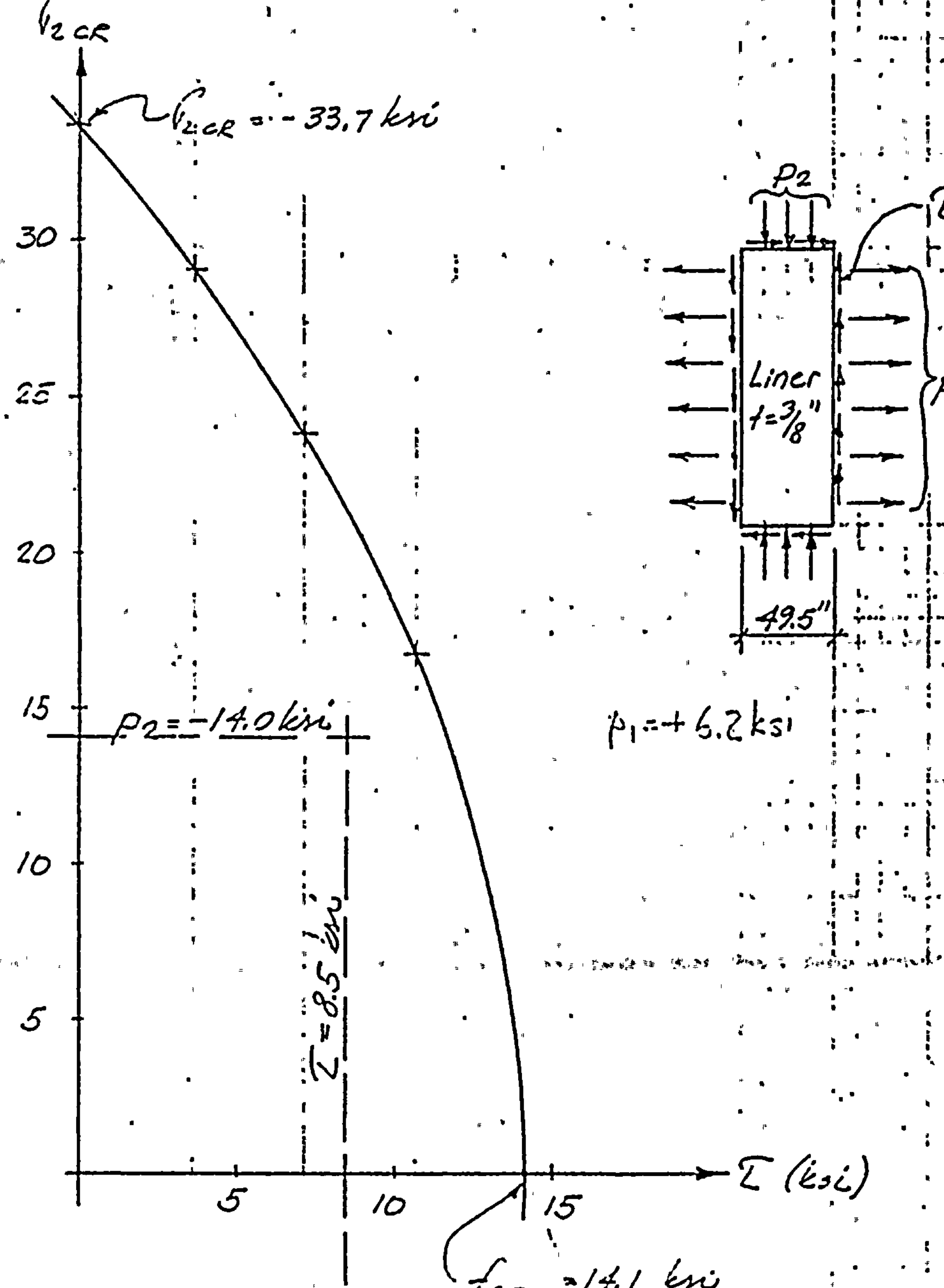
W.O. ORDER	SIZE	DRAWING	REV.
4155		II-A-10d	

FILE CODE 1:36 / Page 81

GINNA

C.V.

Longitudinal Shear - Earthquake



INTERACTION DIAGRAM
FOR
BIAXIAL LINER STRESSES
AND
EARTHQUAKE SHEAR

The plant buildings are located in a relatively level meadow area with finished grade elevation approximately 270'-0". The major plant structures are supported on the Queenston Formation bed rock (red sandstone) or atop natural or compacted granular soils immediately above the bed rock. The Queenston Formation is generally found at a depth of 30 to 40 feet below natural grade. Unsuitable soil removed below foundation level is replaced either with lean concrete or compacted structural fill.

Structural backfill where used, has been placed on layers approximately 8" in thickness and each layer is compacted to a density of at leat 95% of the maximum density obtainable by the modified AASHO method (American Association of State Highway Officials).

The Containment Building is founded on rock. The bottom of the foundation mat elevation is 231'-8" with the deepest foundation around the reactor vessel at elevation 208'-0". The containment cylinder is founded on rock (sandstone) and anchored by means of post-tensioned rock anchors whereby rock acts an an integral part of the containment structure. The containment building is isolated from other buildings, and in areas where it is not surrounded by buildings, a retaining wall spaced 2'-6" clear of the containment wall is provided. This eliminates lateral earth pressure due to backfill on the containment wall. Rock elevation in the area is approximately at 236'-0".

The Auxiliary Building is located south of the Containment Building and founded on rock. The bottom of foundation mat elevation is 233'-8", with the deepest foundation for decay heat removal area at elevation 217'-0" with sump at elevation 214'-0". Rock elevation in this area is approximately at elevation 236'-0". The west end of the superstructure of Auxiliary Building is connected with a portion of the Service Building, and on the northwest with the Intermediate Building. However, the foundation of the Auxiliary Building is independent of these building foundations.

Intermediate Building is located on the north and west of the Containment Building, and is founded on rock. The west end has a retaining wall where the floor at elevation 253'-6" is supported. The bottom of the retaining wall footing is at elevation 233'-6". Rock elevation in this area is approximately at elevation 239'-0". Foundations for interior columns are on individual column footings and embedded a minimum of 2'-0" in solid rock.

The Service Building is located west of the Intermediate Building and is founded on compacted soil. The bottom of the mat is approximately at elevation 252'-8" with localized thickened mat for column footings, and deepest foundation for sump is at elevation 247'-3". Natural compacted granular soil is approximately at elevation 255'-0" to 260'-0". Mat is supported on the east side by a retaining wall on col. line 3 with the Intermediate Building.

The Turbine Building is located North of the Intermediate Building and is supported by a combination of perimeter grade beams and a structural mat. The mat foundation of the turbine generator is independent of the surrounding turbine building foundations. The Turbine Building foundation is supported on the natural compacted granular material which overlie the natural rock. Rock elevation in this area is approximately at elevation 239'-0". Bottom of the trubine generator foundation mat is at elevation 243'-0". Bottom of the perimeter column foundation mat varies from elevation 245'-3" on south side along the Intermediate Building to approximately 246'-9". Circulative water discharge tunnel is supported at elevation 242'-2". Some of the area where Condensate Pumps are located, the entire area is filled with lean concrete, with bottom elevation 229'-8". The area between turbine generator foundation and the permiter column mat foundation is supported on compacted granular material with bottom of mat elevation approximately 251'-6".

The Control Building is located adjacent to the south-east corner of the Turbine Building and is supported by a mat foundation. Foundation of the Control Building is supported on the natural compacted granular material. The rock elevation in this area is approximately at elevation 240'-0". Bottom elevation of the deepest portion of foundation mat is at elevation 245'-4", with a structural slab supported at elevation 250'-6" with thickened slab for column footing.

The Diesel Generator Building is located beyond the north-east corner of the Turbine Building, and is supported on strip and spread footings at elevation 243'-0". The Diesel Generators are supported on individual concrete pedestals, at elevation 243'-0". The rock elevation in this area is at elevation 240'-0". The foundation structures are supported on the natural compacted granular material.

Foundation of all the above mentioned structures, except the Containment Building, are connected into the adjoining building by nominal dowels.

Safety Class I equipment and components were analyzed for seismic accelerations by the response spectra approach. The response spectra specified for the Ginna plant are presented in Figures 5.1.2-7 and 5.1.2-8 of the FSAR. The original design basis considered the seismic response of systems and components to the simultaneous application of an acceleration in the horizontal and vertical planes and combined them to maximize the support design load condition.

Information pertaining to the qualification and analysis of individual systems and components is presented in the Tables of RG&E's letter to the NRC dated April 3, 1979, (L. D. White, Jr. to Dennis L. Ziemann, Subject: Systematic Evaluation Program Seismic Reviews).

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: STRUCTURAL ENGINEERING	DEPT. NO.: 0414	FILING CODE: 1:36.1
	PROJECT NAME: GINNA STATION	W.O. NUMBER: 04 4824 000	PAGE: 107

SUBJECT	ORIGINATOR
SEISMIC EVALUATION - F LINE ANALYSIS	D. BISS
	DATE 5/8/79
	VERIFIER Kroem
	DATE 5-11-79

VERTICAL BRACING SYSTEMS ALONG F LINE WERE EVALUATED FOR WIND, OBE, AND SSE LOADS. BRACING MEMBERS WERE ASSUMED TO BE EFFECTIVE IN BOTH TENSION AND COMPRESSION. WHERE THE MEMBER'S COMPRESSIVE CAPACITY IS EXCEED OR THE MEMBER IS TOO SLENDER TO SUPPORT COMPRESSIVE REACTIONS, THE LOAD WILL REDISTRIBUTE INCREASING THE TABULATED REACTION IN THE ADJOINING TENSION MEMBER BY AN APPROXIMATE FACTOR OF TWO (2).

BELOW THE OPERATING LEVEL (EL. 289'-6") THE VERTICAL BRACING IN THE CRANE BAYS WAS ASSUMED TO BE EFFECTIVE IN RESISTING THE APPLIED HORIZONTAL FORCES. NO CONSIDERATION HAS BEEN GIVEN TO DIAPHRAGM ACTION OF THE TWO FLOOR SYSTEMS OF THE TURBINE BUILDING OR BRACING IN INTERIOR BAYS OF THE TURBINE BUILDING.

ALLOWABLE STRESSES HAVE BEEN INCREASED BY 33% for WIND AND OBE LOADS AND 60% (.9 YIELD) FOR SSE LOADS IN ACCORDANCE WITH THE ORIGINAL FSAR DESIGN CRITERIA.

Rochester Gas & Electric Co Ginna Station Seismic Evaluation F Line Analysis Column Reactions Summary	Made 5/7/79 Chk'd Kvelin Co. Cp. 5-11-79 Cp. Dpn. Eng. D. Biss Rev. Ch. App. Date	Gilbert Associates, Inc. Engineers and Consultants Reading, Penna. 01 4824 000 Work Order / Size / Drawing / Rev. File Code 1:361 / Page 108

Column	Member	Dead Load	Wind Load	OBE Load	SSE Load	Combination #1			Combination #2			Combination #3		
						D+W		Nominal[1]	D+E		Nominal[1]	D+E'		Nominal[2]
						Min	Max	Capacity	Min	Max	Capacity	Min	Max	Capacity
F-3	W24x130	-221.7k	± 12.9k	± 6.1k	± 14.9k	-234.6k	-208.8k	-887.8k	-227.8k	-215.6k	-887.8k	-236.6k	-206.8k	-1068.1k
F-4	W24x160	-405.9k	± 7.7k	± 4.2k	± 10.2k	-413.6k	-398.2k	-1098.1k	-410.1k	-401.7k	-1098.1k	-416.1k	-395.7k	-1321.0k
F-5	W24x160	-393.8k	± 335.4k	± 278.5k	± 683.6k	-729.2k	-58.4k	-1098.1k	-672.3k	-115.3k	-1098.1k	-1077.4k	+289.8k	-1321.0k / +640k
F-6	W24x160	-291.3k	∓ 316.4k	∓ 268.4k	∓ 659.0k	-607.6k	+25.1k	-1098.1k / +532k	-559.7k	-22.9k	-1098.1k	-950.3k	+367.7k	-1321.0k / +640k
F-7	W24x160	-372.7k	∓ 35.1k	∓ 18.3k	∓ 44.8k	-407.8k	-337.6k	-1098.1k	-391.0k	-354.4k	-1098.1k	-417.5k	-327.9k	-1321.0k
F-8	W24x160	-182.2k	∓ 0.7k	∓ 0.1k	∓ 0.3k	-182.9k	-181.5k	-1098.1k	-182.3k	-182.1k	-1098.1k	-182.5k	-181.9k	-1321.0k
F8a	W24x100	-106.8k	∓ 0.5k	∓ 0.1k	∓ 0.3k	-107.3k	-106.3k	-640.7k	-106.9k	-106.7k	-640.7k	-107.1k	-106.5k	-770.7k
F-9	W24x160	-152.3k	± 9.4k	± 5.9k	± 14.5k	-161.7k	-142.9k	-1098.1k	-158.2k	-146.4k	-1098.1k	-166.8k	-137.8k	-1321.0k
F-10	W24x160	-157.4k	± 239.8k	± 196.7k	± 482.7k	-392.1k	+89.5k	-1098.1k / +532k	-349.0k	+44.4k	-1098.1k / +532k	-635.0k	+330.4k	-1321.0k / +640k
F-11	W24x160	-281.3k	∓ 207.8k	∓ 168.1k	∓ 412.6k	-489.1k	-73.5k	-1098.1k	-449.4k	-113.2k	-1098.1k	-693.9k	+131.3k	-1321.0k / +640k
F-12	W24x160	-391.1k	∓ 41.4k	∓ 34.3k	∓ 84.2k	-432.5k	-349.7k	-1098.1k	-425.4k	-356.8k	-1098.1k	-475.3k	-306.9k	-1321.0k
F-13	W24x160	-211.5k	∓ 3.2k	∓ 1.9k	∓ 4.8k	-214.7k	-208.3k	-1098.1k	-213.4k	-209.6k	-1098.1k	-216.3k	-206.7	-1321.0k

F Line Column Reactions

Notes:

1. Allowable stress increased 33% above AISC Allowables as per FSAR. Compression Capacity Based on Gross Area of Column Member, Tension Capacity Based on Anchor Bolts.
2. Allowable stress increased 60% above AISC Allowables. (≈ yield) as per FSAR, Compression Capacity Based on Gross Area of Column Member. Tension capacity Based on Anchor Bolts.

Location	Member	Wind Member Load	Wind Member Capacity	OBE Member Load	OBE Member Capacity	SSE Member Load	SSE Member Capacity
EL 386' to EL 370.5'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 21.5^k$	$+96.1^k$ $-32.^k$	$\pm 5.0^k$	$+96.1^k$ $-32.^k$	$\pm 12.3^k$	$+115.7^k$ -38.5^k
EL 370.5' to EL 356.5'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 22.2^k$	$+96.1^k$ -32.3^k	$\pm 8.9^k$	$+96.1^k$ -32.3^k	$\pm 21.8^k$	$+115.7^k$ -38.8^k
EL 356.5' to EL 342.7'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 18.1^k$	$+96.1^k$ -32.3^k	$\pm 11.4^k$	$+96.1^k$ -32.3^k	$\pm 28.1^k$	$+115.7^k$ -38.8^k
EL 342.7 to EL 328.2'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 35.9^k$	$+96.1^k$ -32.2^k	$\pm 15.7^k$	$+96.1^k$ -32.2^k	$\pm 38.5^k$	$+115.7^k$ -38.7^k
EL 328.2 to EL 308.2'	JL $4\times3\times\frac{7}{16}$	$\pm 62.1^k$	$+142.7^k$ —	$\pm 35.0^k$	$+142.7^k$ —	$\pm 85.9^k$	$+171.7^k$ —
EL 308.2' to EL 288'	JL $4\times3\frac{1}{2}\times\frac{7}{16}$	$\pm 71.1^k$	$+153.6$ -53.4	$\pm 47.5^k$	$+153.6$ -53.4	$\pm 116.6^k$	$+184.8$ -64.3 *
EL 288' to EL 270'	JL $6\times4\times\frac{3}{4}$	$\pm 70.2^k$	$+345.6^k$ -151.2^k	$\pm 70.5^k$	$+345.6^k$ -151.2^k	$\pm 173.2^k$	$+415.8^k$ -181.9^k
EL 270' to EL 254.5'	JL $4\times3\frac{1}{2}\times\frac{3}{8}$	$\pm 31.9^k$	$+132.7^k$ -48.9^k	$\pm 46.2^k$	$+132.7^k$ -48.9^k	$\pm 113.5^k$	$+159.7^k$ -58.9^k *

Vertical Bracing Summary

F-5 → F-6

* Though the stress exceeds yield stress, it is less than minimum tensile strength of the members.

GILBERT ASSOCIATES, INC.
ENGINEERS AND CONSULTANTS
READING, PA.

SUBJECT: Seismic Evaluation - F Line Analysis

DEPARTMENT NAME: Structural Engineering
PROJECT NAME: Ginna Station
DEPT. NO.: C414
W.O. NUMBER: 04 4824 000
FILING CODE: 1256.1
PAGE: 110
ORIGINATOR: D. Bise DATE: 5/8/79
VERIFIER: [signature] DATE: 5/11/79

Location	Member	Wind: Member Load	Wind: Member Capacity	OBE: Member Load	OBE: Member Capacity	SSE: Member Load	SSE: Member Capacity
El 356.5' to El 342.7'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 7.0^k$	$+96.1^k$ -32.3^k	$\pm 4.8^k$	$+96.1^k$ -32.3^k	$\pm 11.8^k$	$+115.7^k$ -38.8^k
El 342.7' to El 328.2'	JL $3\frac{1}{2}\times3\times\frac{5}{16}$	$\pm 32.3^k$	$+96.1^k$ -32.2^k	$\pm 17.7^k$	$+96.1^k$ -32.2^k	$\pm 43.3^k$	$+115.7^k$ -38.7^k
El 328.2' to El 308.2'	JL $3\frac{1}{2}\times3\times\frac{3}{8}$	$\pm 51.7^k$	$+114.1^k$	$\pm 31.5^k$	$+114.1^k$	$\pm 77.4^k$	$+137.3^k$
El 308.2' to El 288.'	JL $3\frac{1}{2}\times3\times\frac{3}{8}$	$\pm 57.4^k$	$+114.1^k$	$\pm 39.7^k$	$+114.1^k$	$\pm 97.5^k$	$+114.1^k$
El 288.' to El 270.'	JL $6\times4\times\frac{1}{2}$	$\pm 62.3^k$	$+236.2^k$ -91.6^k	$\pm 54.5^k$	-236.2^k -91.6^k	$\pm 133.8^k$	$\pm 284.2^k$ -110.2^k
El 270' to El 261.1'	JL $5\times3\times\frac{1}{2}$	$\pm 50.6^k$	$+186.5$ -88.4	$\pm 52.1^k$	$+186.5$ -88.4	$\pm 127.9^k$	$+224.4$ -106.3 *

Vertical Bracing Summary

F-10 → F-11

* Though the stress exceeds yield stress, it is less than minimum tensile strength of the members.

FILING CODE

2

ENCLOSURE II

Response to Item III.(3) of the NRC Docket No. 50-244, SEP Seismic Review.

Buried Service Water Piping

1.	Size:	16" to 20" nominal I.D.
2.	Material:	Prestressed concrete cylinder (steel) pipe, class 150, Rubber and Steel joint, AWWA C301,58
3.	Pipe Bedding:	All buried service water pipe bedding take either of two forms: (a) concrete cradle (b) concrete encasement. Typical details of concrete cradle and encasement are shown in Figures 1 and 2.
4.	Penetration:	Details of pipe penetration through wall is shown in Figure 3. Concrete is poured around pipe monolithically with wall.
5.	Thrust Blocks:	Concrete thrust blocks were provided at various points throughout the entire line.
6.	Subgrade:	All subgrade on which concrete bedding were constructed were compacted to at least 95 percent of the maximum density at optimum moisture content as determined in accordance with the modified AASHO Spec.
7.	Backfill:	All backfill at sides and immediately over pipe were compacted uniformly with compacted material in successive layers of 6" or less to at least 95% of the maximum density at optimum moisture content as determined in accordance with modified AASHO spec.

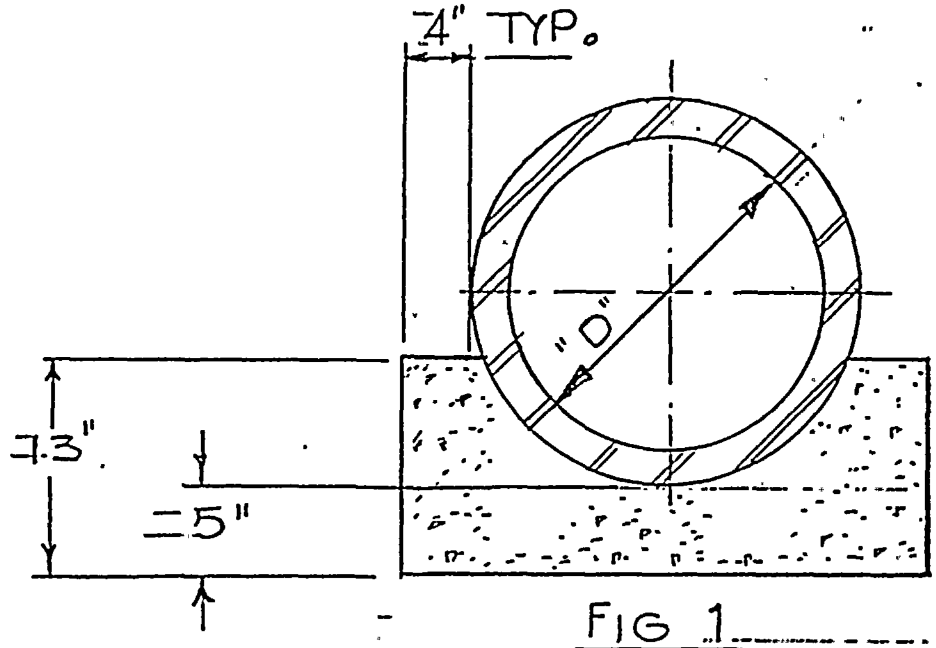


FIG 1

TYPICAL CONCRETE CRADLE

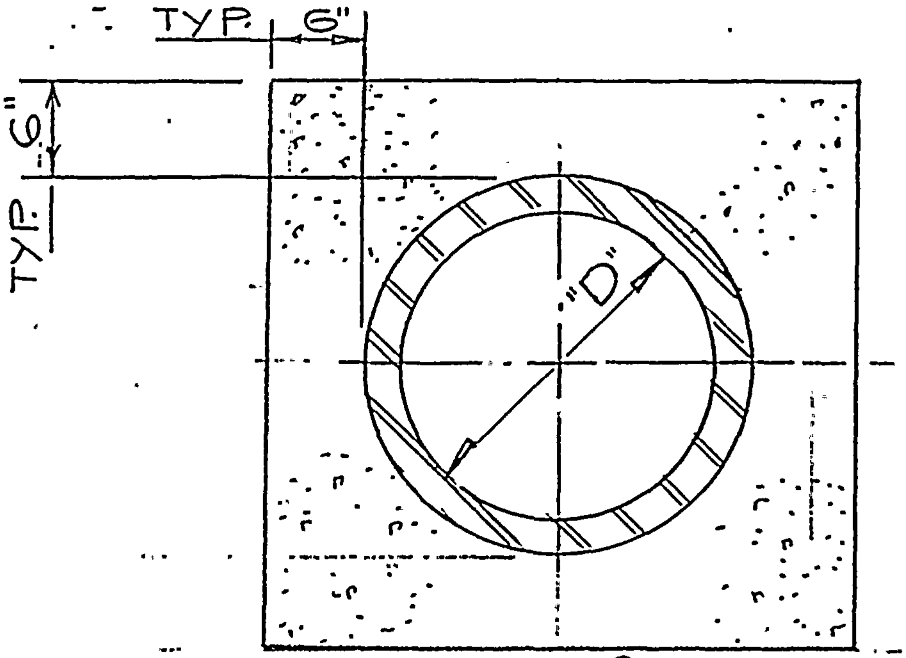


FIG. 2

TYPICAL CONCRETE ENCASEMENT

NOTES:

D = PIPE SIZE; 16" TO 20" NOMINAL I.D.

CONCRETE STRENGTH; 3000 PSI @ 28 DAYS.

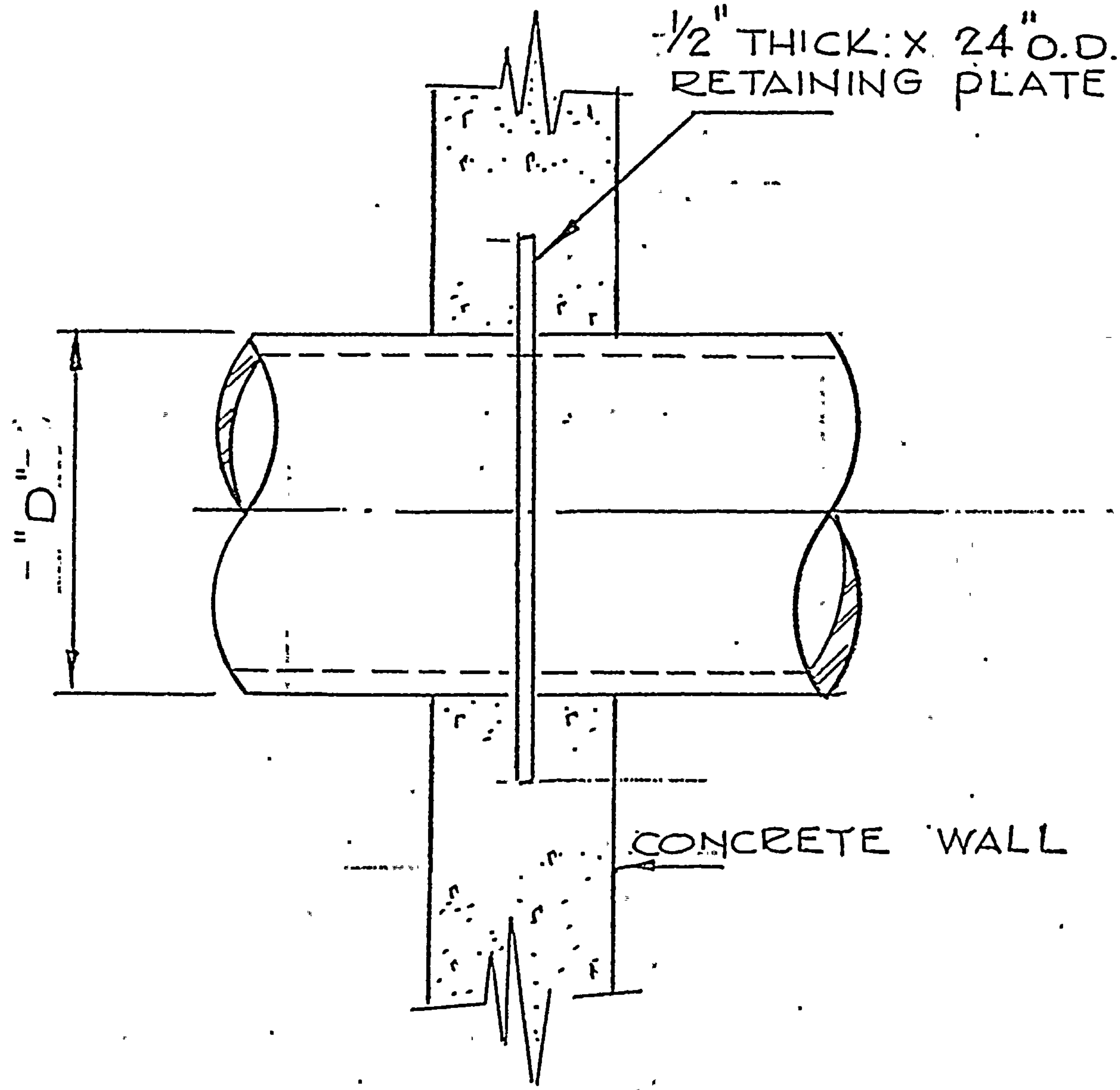
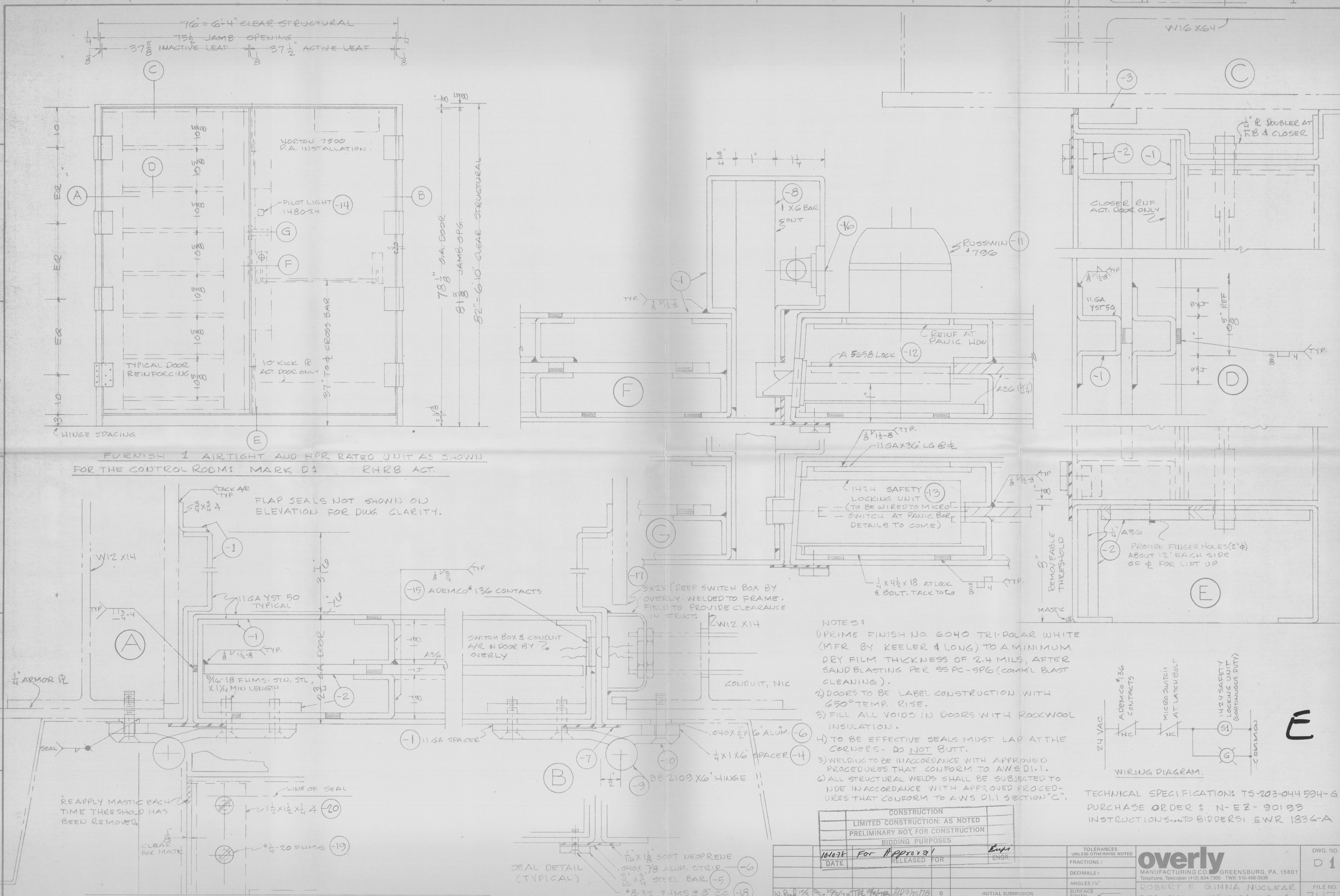


FIG. 3

TYPICAL DETAIL OF PIPE PENETRATION
THRU CONCRETE WALL

"D" = PIPE SIZE - 16" TO 24" NOMINAL I.D.



NOTES:

1) PRIME FINISH NO. 6040 TRI-POLAR WHITE (MFR BY KEELER & LONG) TO A MINIMUM DRY FILM THICKNESS OF 2.4 MILS, AFTER SAND BLASTING PER SSPC-SP6 (COMM'L BLAST CLEANING).
2) DOORS TO BE LABEL CONSTRUCTION WITH 650° TEMP. RISE.
3) FILL ALL VOIDS IN DOORS WITH ROCKWOOL INSULATION.
4) TO BE EFFECTIVE SEALS MUST LAP AT THE CORNERS - DO NOT BUTT.
5) WELDING TO BE IN ACCORDANCE WITH APPROVED PROCEDURES THAT CONFORM TO AWS D1.1.
6) ALL STRUCTURAL WELDS SHALL BE SUBJECTED TO NDE IN ACCORDANCE WITH APPROVED PROCEDURES THAT CONFORM TO AWS D1.1 SECTION "C".

TECHNICAL SPECIFICATION: TS-203-044594-G5
PURCHASE ORDER: N-EZ-90193
INSTRUCTIONS....TO BIDDERS: EWR 1836-A

	CONSTRUCTION	
	LIMITED CONSTRUCTION: AS NOTED	
	PRELIMINARY NOT FOR CONSTRUCTION	
	BIDDING PURPOSES	
10-10-78 DATE	For Approval RELEASED FOR	Eupi ENGR.

QA REVIEW DATE	REVIEW/APP. DATE	CHECKED DATE	PREPARED BY DATE	REV.	DESCRIPTION
W. Read 10/3	DCS 10/2/78	TFL 10/3/78	9/25/78	0	INITIAL SUBMISSION

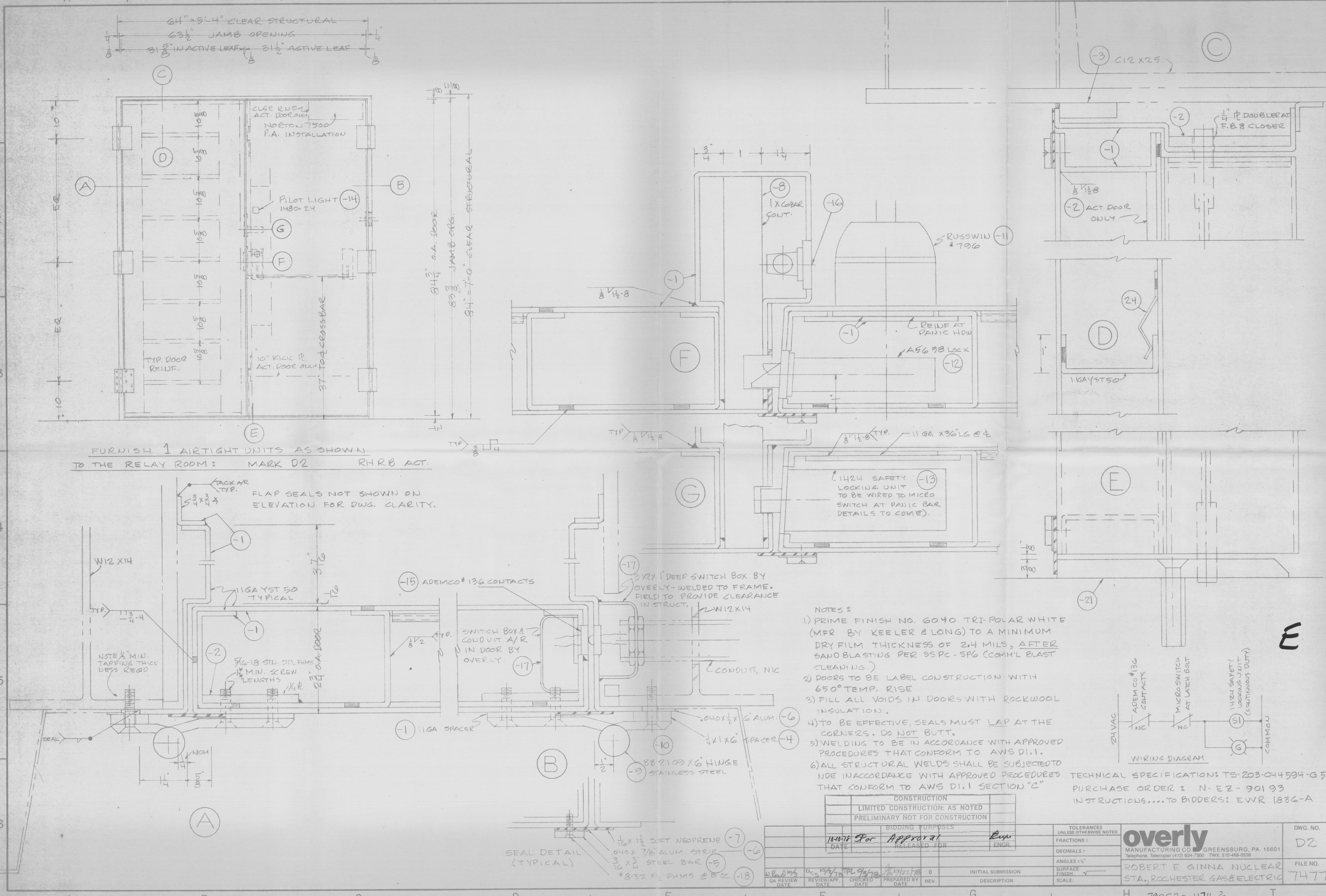
TOLERANCES UNLESS OTHERWISE NOTED
FRACTIONS ±
DECIMALS ±
ANGLES ±½°
SURFACE FINISH √
SCALE:

overly
MANUFACTURING CO. GREENSBURG, PA. 15601
Telephone, Telecopier (412) 834-7300 TWX: 510-468-0539

ROBERT E. GINNA NUCLEAR STA., ROCHESTER GAS & ELECTRIC

DWG. NO. D 1
FILE NO. 7477

7905300474-1



NOTES:

1) PRIME FINISH NO. 6040 TRI-POLAR WHITE (MFR. BY KEELER & LONG) TO A MINIMUM DRY FILM THICKNESS OF 2.4 MILS, AFTER SANDBLASTING PER SSPC-SP6 (COMM'L BLAST CLEANING.)
2) DOORS TO BE LABEL CONSTRUCTION WITH 650° TEMP. RISE
3) FILL ALL VOIDS IN DOORS WITH ROCKWOOL INSULATION.
4) TO BE EFFECTIVE, SEALS MUST LAP AT THE CORNERS. DO NOT BUTT.
5) WELDING TO BE IN ACCORDANCE WITH APPROVED PROCEDURES THAT CONFORM TO AWS D1.1.
6) ALL STRUCTURAL WELDS SHALL BE SUBJECTED TO NDE IN ACCORDANCE WITH APPROVED PROCEDURES THAT CONFORM TO AWS D1.1 SECTION "C"

TECHNICAL SPECIFICATION: TS-203-044594-G5
PURCHASE ORDER: N-EZ-90193
INSTRUCTIONS....TO BIDDERS: EWR 1836-A

CONSTRUCTION
LIMITED CONSTRUCTION: AS NOTED
PRELIMINARY NOT FOR CONSTRUCTION
BIDDING PURPOSES
10-10-78 For Approval RELEASED FOR
DATE ENGR.

QA REVIEW DATE	REVIEW/APP. DATE	CHECKED DATE	PREPARED BY DATE	REV.	DESCRIPTION
10/3	DCS 10/2/78	TFL 10/3/78	9/22/78	0	INITIAL SUBMISSION

TOLERANCES UNLESS OTHERWISE NOTED
FRACTIONS ±
DECIMALS ±
ANGLES ±1/2°
SURFACE FINISH √
SCALE:

overly
MANUFACTURING CO. GREENSBURG, PA. 15601
Telephone, Telecopier (412) 834-7300 TWX: 510-468-0539

ROBERT E GINNA NUCLEAR STA., ROCHESTER GAS & ELECTRIC

DWG. NO. D2
FILE NO. 7477

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