GA 1485 (REV. 10/82)

GA Technologies Inc.

ISSUE SUMMARY

TITLE FSV - TENDON REQUIREMENTS BASED ON SAFETY CONSIDERATION

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☒ DESIGN

APPROVAL LEVEL 2

DISCIPLINE	SYSTEM	DOC. TYPE	PROJECT	DOCUMENT NO.	ISSUE NO./LTR.
S	11	CFL	1900	907738	N/C

QUALITY ASSURANCE LEVEL	SAFETY CLASSIFICATION	SEISMIC CATEGORY	ELECTRICAL CLASSIFICATION
I	FSV - 1	FSV - 1	N/A

2 RELEASED

ISSUE	DATE	PREPARED BY	APPROVAL				ISSUE DESCRIPTION/ CWBS NO.
			ENGINEERING	QA	FUNDING PROJECT	APPLICABLE PROJECT	
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CONTINUE ON GA FORM 1485-1

Text 1 - 11 = 11
A1 - A20 = 20
B1 - B80 = 80
C1 - C2 = 2
TOTAL 113

NEXT INDENTURED DOCUMENTS

P.O. N-5159

8502280588 850214
PDR ADOCK 05000267
P PDR

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SH																												
REV																												
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TITLE: FSV - TENDON REQUIREMENTS BASED ON SAFETY CONSIDERATION

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1. SUMMARY

The minimum numbers of tendons required for safely supporting the core cavity pressure of 845 psig (Reference Pressure, RP) and 1268 psig (1.5 RP) without breaching the liner of the Fort St. Vrain PCRV have been determined for: 1) circumferential tendons in the PCRV wall and 2) crosshead and circumferential tendons in the heads of the PCRV. Hand calculations based on the concept of ultimate load analysis are used. No calculation is performed for the vertical tendons in this study. Per Ref. 1, the PCRV can resist up to 1515 psig cavity pressure with only the rebars acting, i.e., without reliance on any vertical prestress.

The results are given in Table 1 and Figs. 1 and 2. These results indicate that the core cavity pressure of 1.0 RP can be safely resisted with considerably less number of tendons than is actually provided. With 1.5 RP, the number of head tendons required is still less than that actually provided. The difference, however, is small if no vertical tendons exist as assumed in the analysis. Existence of vertical tendons will require less number of head tendons to resist 1.0 and 1.5 RP.

The procedure used in the study is described in the following sections. Detailed calculations are given in the appendices.

2. MATERIAL PROPERTIES

Material properties used in this analysis are (Ref. 1):

Concrete:

Compressive strength $f'_c = 6000$ psi

Liner:

Material = SA 537, Gr. B

Yield strength $f_{sy} = 60,000$ psi (at 0.2% offset)

Tensile strength $f'_s = 80,000$ psi (Ref. 2)

Failure strain $\varepsilon' = 18\%$ (Ref. 2)

Modulus of elasticity $E = 29 \times 10^6$ psi

Tendon Wires:

Tensile strength $f'_s = 240{,}000$ psi

Yield strength $f_{sy} = 204{,}000$ psi (at 1% strain)

Failure strain $\varepsilon' = 4\%$

Modulus of elasticity $E = 27 \times 10^6$ psi

Rebars:

Material = A432

Tensile strength $f'_s = 90{,}000$ psi

Yield strength $f_{sy} = 60{,}000$ psi

Failure strain $\varepsilon' = 7\%$

Modulus of elasticity $E = 29 \times 10^6$ psi

3. DEFINITION OF NUMBERS OF TENDONS

The Fort St. Vrain PCRV has, in addition to 90 vertical tendons with 169 1/4-in. diameter wires each, 210 circumferential tendons with 152 1/4-in. diameter wires each, 100 circumferential tendons with 169 1/4-in. diameter wires each, and 48 crosshead tendons with 169 1/4-in. diameter wires each.

All 210 152-wire circumferential tendons are in the barrel section. Of the 100 169-wire circumferential tendons, 34 are in the top head (the top 15'-6" section), 34 in the bottom head (the bottom 15'-6" section), and 16 each in the barrel sections adjacent to the top and bottom heads. Each head has 24 crosshead tendons.

All circumferential tendons are 180° tendons rather than full circle tendons (see Fig. E.15-2, Ref. 1). Because of the arrangement of these tendons, of the 18 circumferential (180°) tendons in a typical five-foot high wall section, a minimum of 12 pass any cross section. Hence 18 actual circumferential tendons provide 12 "effective" circumferential tendons. Similarly in the top or bottom head, 34 actual circumferential tendons provide 22 effective circumferential tendons.

The following definitions are used in this report:

N_b - Number of effective circumferential tendons in a 5-foot high wall section (12 in existence).

N_c - Number of effective circumferential tendons in the top or bottom head (22 in existence).

N_x - Number of crosshead tendons in the top or bottom head (24 in existence).

It is assumed that there is no broken wire in any tendon and that the required tendons in each group are uniformly distributed.

4. CIRCUMFERENTIAL TENDONS IN PCRV WALL

For the determination of the required number of circumferential tendons in the PCRV wall a typical five-foot high wall section was considered. It is assumed that ultimate conditions are reached at 1.0 RP or 1.5 RP for the purpose of this analysis. The core cavity liner is anchored to the concrete by means of studs welded to the liner and embedded in the concrete. The stud spacings are 7-1/2 in. in both circumferential and axial directions. It is assumed that, at ultimate, radial concrete cracks would develop at stud anchor locations and that resistance to the core cavity pressure is provided by the steel elements acting as multiple structural rings. The steel elements include the liner, and circumferential tendons and rebars at various radial locations. With the liner and rebar cross-sectional areas known, the number of tendons required to provide a total pressure resistance capacity for the core cavity pressure of 1.0 RP or 1.5 RP, and meeting the selected limit criteria can be determined from equilibrium and strain compatibility.

The tendon prestress loss at end of life is assumed to be 13.5% (Ref. 1), and the friction loss is assumed to be 11.5% (Ref. 3) in these calculations.

Two limit criteria are used in this case:

1) Liner stress = $0.9\ f_{sy}$,

   Tendon stresses $\leq f'_s$, and

   Rebar stresses $\leq f'_s$

2) Maximum tendon stress = f'_s,

   Liner stress $< f'_s$, and

   Rebar stresses $\leq f'_s$

Obviously the first criterion is the more stringent and results in a larger number of tendons being required. The required numbers of circumferential tendons in the PCRV wall for the above two limit conditions are shown in Table 1.

5. TENDONS IN PCRV TOP AND BOTTOM HEADS

The required number of crosshead and circumferential tendons in the PCRV top and bottom heads to safely support 1.0 RP and 1.5 RP without breaching of the liner boundary are determined by ultimate load analysis of the bottom head.

Four quasi-analytical solutions were originally used in assessing the ultimate capacity of the Fort St. Vrain PCRV heads (Ref. 1). These are: 1) bottom head yield line failure analysis, 2) bottom head punching shear failure analysis, 3) bottom head concrete ligament compressive failure analysis, and 4) top head analysis by grid system simulation. In the case of 34 circumferential and 24 crosshead tendons in each head and 90 vertical tendons, the yield line analysis provided the lowest estimate of the ultimate pressure capacity, while the concrete ligament compressive failure analysis provided the highest, about three times as high as the lowest estimate. The top head grid analysis requires use of a computer program.

Based on the above observations it was decided to use the yield line failure analysis method for the ultimate load analysis in the current study, and to check the results using the punching stress failure analysis.

The assumptions and detailed procedure used in the bottom head yield line analysis follow those used in Ref. 1. Based on an assumed number of crosshead tendons the resultant pressure which must be resisted by the bottom head (cavity pressure reduced by the cavity pressure equivalent of crosshead tendons, Ref. 1) is first calculated. By assuming formation of a plastic hinge at the head-to-wall junction (signified by 0.003 in/in maximum concrete strain and/or yielding of liner and majority of rebars in tension), and a yield line pattern (generally radial along concrete ligaments) the unit yield line moment required to prevent this particular yield line mode of failure under the given cavity pressure (1.0 RP or 1.5 RP) can be determined. The number of circumferential tendons required to provide an ultimate moment capacity along the yield line which is larger than the required unit yield line moment is then established. The ultimate moment capacity of the bottom head is defined by the following stress limits (Ref. 1):

Maximum rebar stress $\leq 0.9\ f_{sy}$,

Maximum tendon stress $\leq 0.9\ f_{sy}$, and

Maximum compressive concrete stress $\leq 0.85\ f'_c$

The tendon prestress losses at end of life are assumed to be 12% for both crosshead and circumferential tendons (Ref. 1) and the friction losses are assumed to be 10% and 11.5%, respectively for these two types of tendons (Ref. 3). The yield line failure analysis in the current study is based on the assumption that no vertical tendons exist.

The required number of tendons based on the results of the yield line failure analysis are shown graphically in Figs. 1 and 2.

For the punching shear failure analysis of the bottom head the failure plane is assumed to be the one formed by the concrete ligaments connecting the steam generator penetrations (Ref. 1). Reference 4 provides an equation to estimate the ultimate shearing strength of PCRV heads as a function of span/depth ratio and radial prestress. Based on this equation and the number of head tendons required as determined by the yield line failure analysis, it is found that the punching shear stress is not critical for either the 1.0 RP or the 1.5 RP cases.

6. CONCLUSIONS

The required numbers of circumferential tendons in the Fort St. Vrain PCRV wall to safely support the cavity pressure of 1.0 RP and 1.5 RP are given in Table 1. The corresponding required numbers of crosshead and circumferential tendons in either top or bottom head of the PCRV, derived under a conservative assumption of no vertical prestress, are given in Figs. 1 and 2.

From Fig. 2, it appears that under 1.5 RP the permissible reduction in the numbers of crosshead and circumferential tendons in the heads is small if no vertical prestressing tendons exist.

7. REFERENCES

1. "Fort St. Vrain Nuclear Generating Station. Updated Final Safety Analysis Report."

2. ASTM, "Specification for Carbon-Manganese-Silicon Steel Plates, Heat Treated for Pressure Vessels. SA-537."

3. Lee, T. T. and Cheung, K. C., "FSV - PCRV Tendon Evaluation," GA Document 907441/A, April 30, 1984.

4. Garas, F. K. and Trowsdale, D. R., "Overload Behavior and Shear Failure Mechanisms of Model No. 2 of the Bottom Head of the Fort St. Vrain Prestressed Concrete Reactor Vessel," Report 14H/69/1411, Taylor Woodrow Construction, Ltd., September 1969.

5. Bresler, B., "Reinforced Concrete Engineering," Vol. 1 Materials Structural Elements, Safety, John Wiley & Sons, New York, 1974.

6. "PCRV Bottom Head, Reinforcing Plan, Sheet 1," Drawing 3614, B-36/J, Sargent & Lundy, March 1969.

7. "PCRV Bottom Head, Reinforcing Schedule and Details, Sheet 1," Drawing 3614, B-37/E, Sargent & Lundy, October 1969.

8. "PCRV Bottom Head, Reinforcing Plan, Sheet 2," Drawing 3614, B-38/K, Sargent & Lundy, April 1969.

9. "PCRV Bottom Head, Reinforcing Schedule & Details, Sheet 2," Drawing 3614, B-39/E, Sargent & Lundy, December 1968.

10. "PCRV Bottom Head, Reinforcing Schedule & Details, Sheet 3," Drawing 3614, B-40/D, Sargent & Lundy.

11. "PCRV Bottom Half Vertical Section," Drawing 3614, B-35/S, Sargent & Lundy, December 1969.

12. "PCRV Bottom Head, Tendon Tubes Details," Drawings 3614, B-21/E and B-22/D, Sargent & Lundy, October 1969.

NOTE: References 5 through 12 are cited in the appendices.

TABLE 1

MINIMUM NUMBERS OF CIRCUMFERENTIAL TENDONS IN WALL SECTION

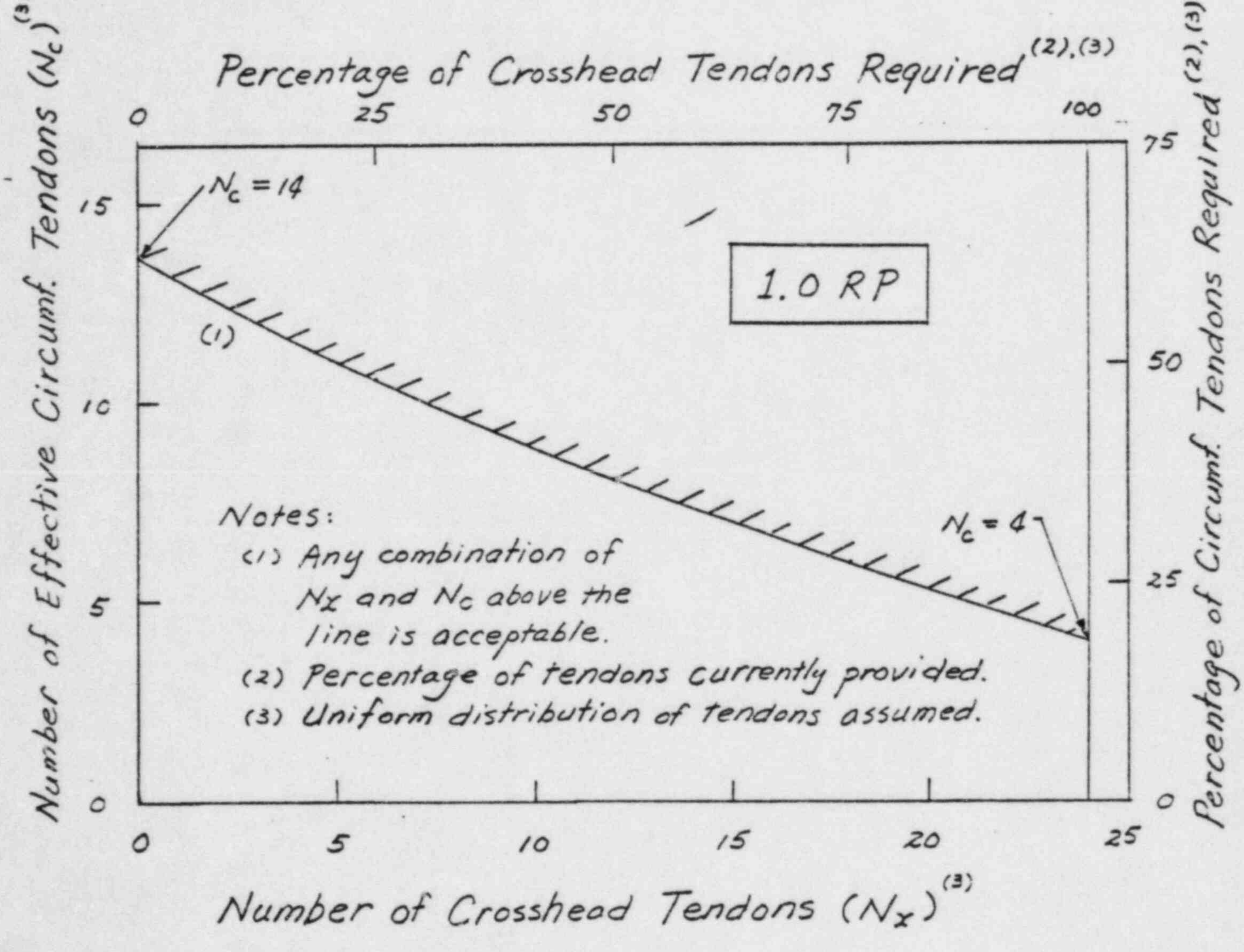
	Number of Tendons Required (N_b)(1)		Percentage of Tendons Required(2)	
Criterion	1.0 RP(3)	1.5 RP	1.0 RP	1.5 RP
Liner Stress = 0.9 f_{sy}	5	9	42%	75%
Max. Tendon Stress = f'_s(4)	3	5	25%	42%

(1) Number of effective tendons required per 5-foot high section. See the text for definition of N_b.

(2) Percentage of tendons currently provided in any region of the PCRV wall. It is assumed that the required tendons are located uniformly in the region under consideration.

(3) 1.0 RP = 845 psig.

(4) The liner strain is 0.046 in./in. when the maximum tendon stress is f'_s.



Notes:
(1) Any combination of N_X and N_C above the line is acceptable.
(2) Percentage of tendons currently provided.
(3) Uniform distribution of tendons assumed.

Figure 1. Number of Head Tendons Required in Each Head to Support 1.0 RP

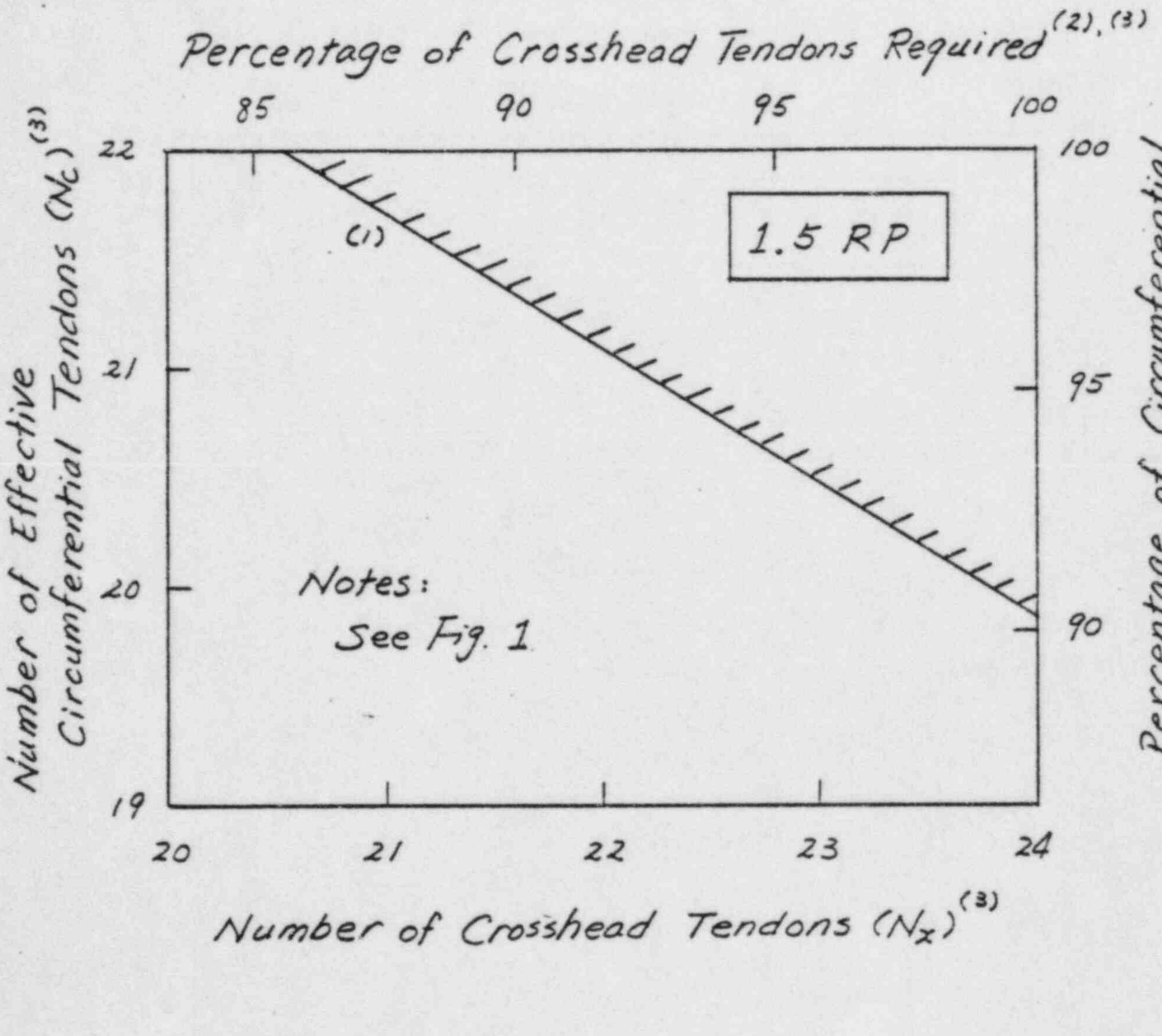


Figure 2. Number of Head Tendons Required in Each Head to Support 1.5 RP

APPENDIX A

NUMBER OF CIRCUMFERENTIAL TENDONS IN THE PCRV WALL

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A.1 Cases Considered

1) Core cavity pressure = Reference Pressure
= 1.0RP = 845 psig.

2) Core cavity pressure = 1.5RP = 1268 psig.

A.2 Location of Steel Components

For a 5'-0" barrel section.

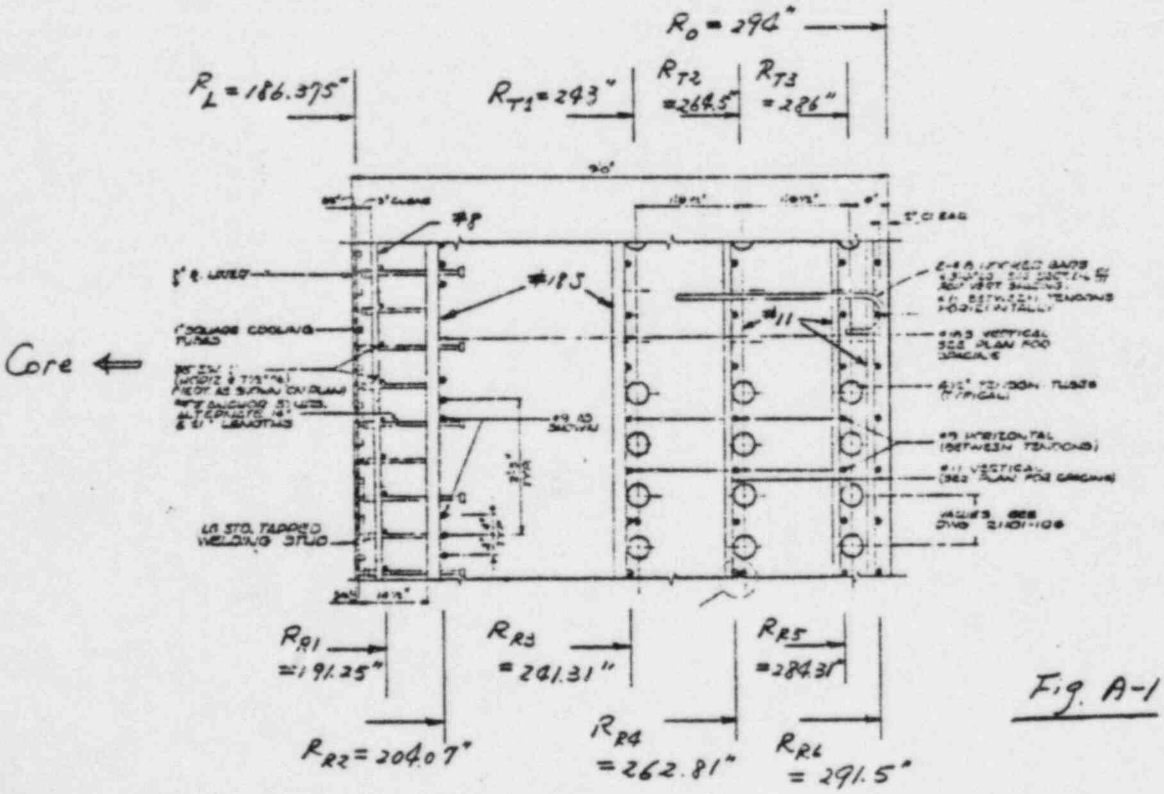

Fig. A-1

(Ref 1. FSAR Fig. E.15-6)

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Rebar Dimensions:

Table A-1.

Bar Size	Diam. (in.)	Area (in²)
#8	1.000	0.79
#9	1.128	1.00
#11	1.410	—
#18S	2.257	—

Rebar Groups: Table A-2

Group	R_R (in.)	Rebars	Number per 5 ft.	Total Area (in²)
R1	191.25	#8 @ 7.5"	8	6.32
R2	204.07	3-#9 @ 27"	6	6.00
R3	241.31	#9 @ 10"	6	6.00
R4	262.81	#9 @ 10"	6	6.00
R5	284.31	#9 @ 10"	6	6.00
R6	291.50	#8 @ 10"	6	4.74

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Liner: 3/4 in. thick at $R_L = 186.375$ in.

Tendons: 152 – 1/4 in. diam. wires → Area = 7.463 in^2

169 – " " " → Area = 8.298

- 3 groups of 6 tendons each (total of 18) for each 5 ft. section, located at R_{T1}, R_{T2} & R_{T3}.
- Minimum of 12 tendons pass any 5 ft section.

A.3 Material Properties

Rebars: A432 (A305-56T. Deformation)

f_{sy} = Minimum guaranteed yield strength (at 0.5 % total extension)

= 60 ksi. (FSAR. App. E Section E.1)

f_s' = Minimum guaranteed tensile strength

= 90 ksi (FSAR. Section E.1)

ϵ' = Minimum guanteed strain at failure

= 7% in 8 in (FSAR. Section E.1)

E = Modulus of elasticity = 29.0×10^3 ksi.

(FSAR. Sec. E.10.3)

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Liner: SA 537, Grade B (Quenched & tempered) (Ref. 1)

f_{sy} = 60 ksi (at 0.2% offset) (FSAR, E.1) (FSAR.E.10.2

f_s' = 80 ksi

ϵ' = 18%

} (ASTM Spec. SA 537. Ref. 2)

E = 29.0×10^3 ksi (FSAR Sec. E.10.3)

Tendons:

Tendon wires: $f_s' = 240$ ksi

$f_{sy} = 204$ ksi at 1% strain

$\epsilon' = 4\%$

See Fig. 4 (From FSAR. Fig. 5.6-1)

$E = 27 \times 10^3$ ksi (Table 5.6-5 FSAR)

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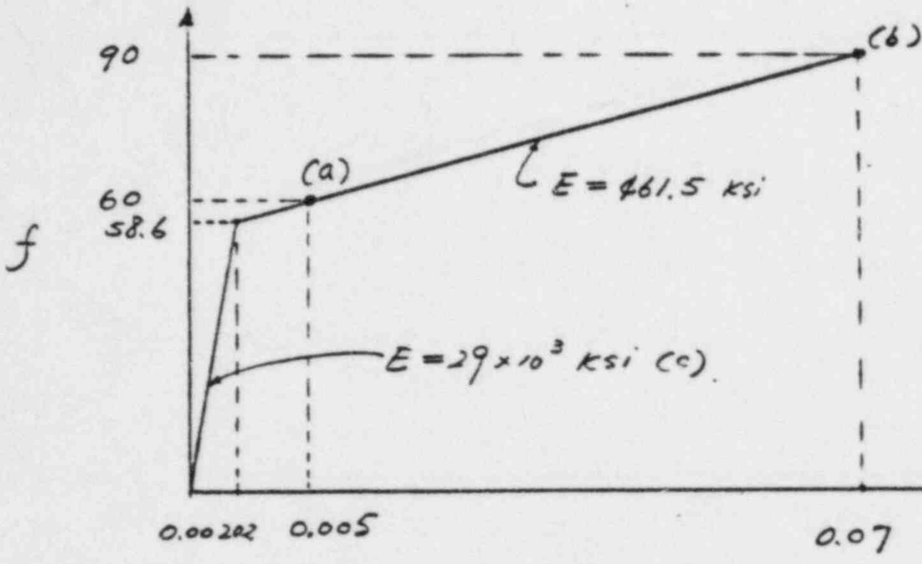
Stress - strain Curves:

Rebars:



Based on (a), (b) and (c)

Fig. A-2

Use $\begin{cases} f = 90 - (0.07 - \epsilon) \times 461.5, & \epsilon \geq 0.00202 \\ f = 29.0 \times 10^3 \epsilon, & \epsilon \leq 0.00202 \end{cases}$

Liner:

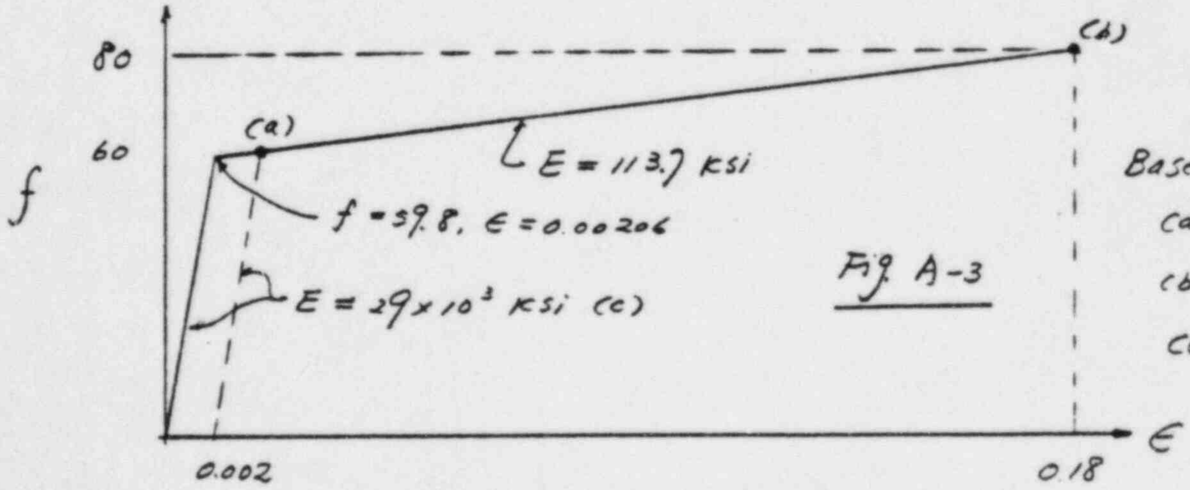


Fig A-3

Based on (a), (b), & (c)

Use $\begin{cases} f = 80 - (0.18 - \epsilon) \times 113.7, & \epsilon \geq 0.00206 \\ f = 29.0 \times 10^3 \epsilon, & \epsilon \leq 0.00206 \end{cases}$

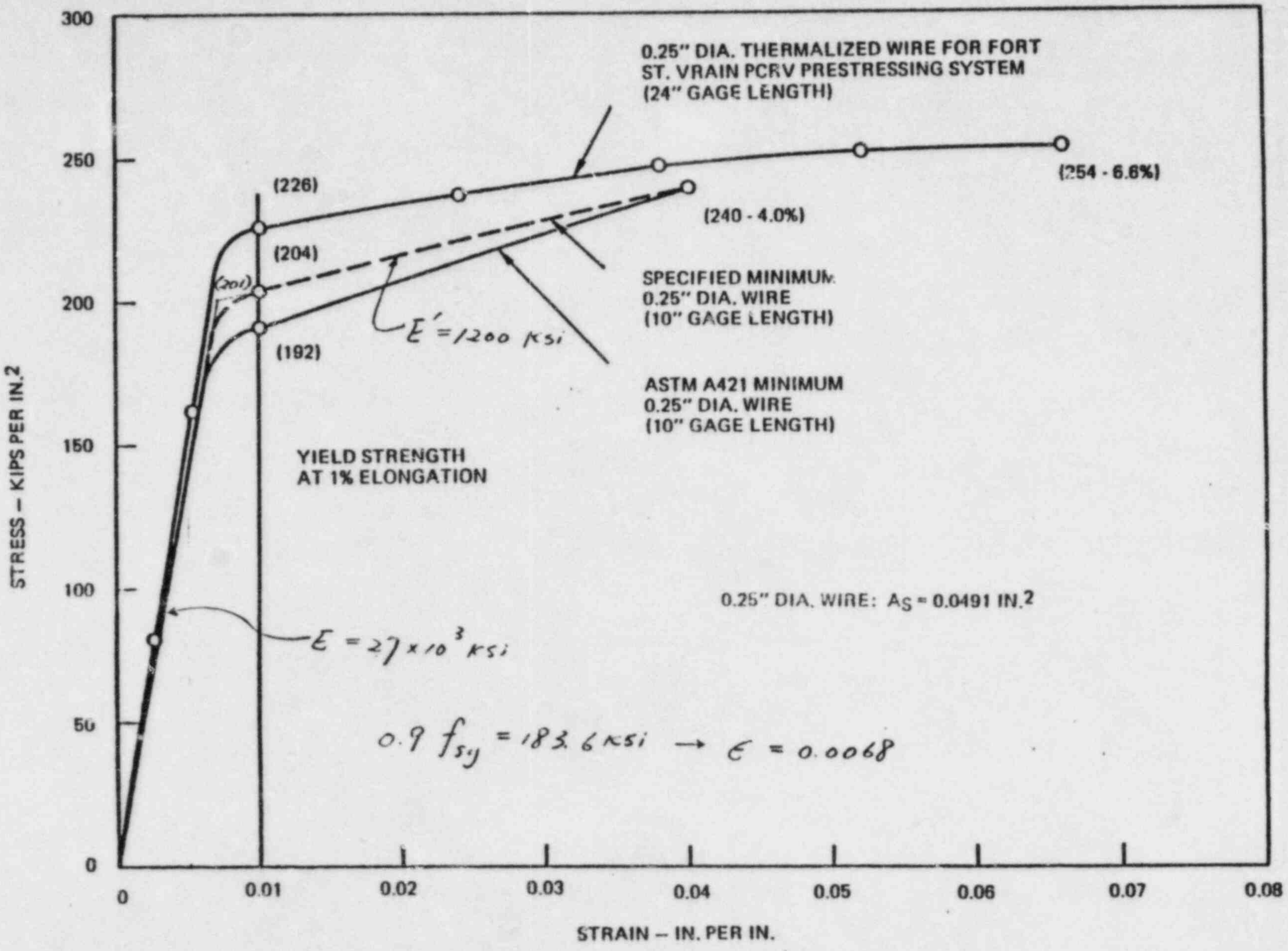


Figure A-4 Typical Stress-Strain Curve of Wire

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A.4 <u>Assumptions</u>

(1) Based on the discussion on the crack propagation given in FSAR, Section E. 8.2.3.1 it is assumed that radial cracks develop at the locations of stud anchors. Hence concrete does not participate in resisting the core cavity pressure. It does transmit the pressure radially.

(2) Each steel component (liner, rebar and tendon) acts as a ring and contributes in resisting the core cavity pressure.

(3) On account of the extensiveness of the concrete crack the bond between rebars and concrete is essentially non-existing and the strain in a given rebar is essentially

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constant (Ref. 5, p 175).

(4) The prestress loss at end of life is 13.5 % (FSAR. Table 5.6-4). The friction loss is 11.5 % (Ref. 3)

(5) Radial shortening of concrete is negligible. Hence the radial displacements of all steel components are the same.

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A.5 Compatibility

Assuming that each steel component displaces radially the same amount (Assumptions) the strains in various components are related as follows:

$$\epsilon_L R_L = \epsilon_{Ri} R_{Ri} = (\Delta\epsilon_{Ti})\, R_{Ti}$$

where ϵ = strain

R = radial distance from the center of core cavity

$\Delta\epsilon_{Ti}$ = additional tendon strain for Group i tendon
= total tendon strain − strain due to prestress.

The subscripts are:

L = liner

R = a rebar group

T = a tendon group

i = group number.

The strain due to prestress ≅ 0.00476 in./in. (Fig A-4)

based on $f = 0.7 f_s' \times (1 - \overset{Loss}{0.135}) \times \overset{Friction}{0.885} = 128.6$ ksi

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A.6 Contribution to Pressure Capacity

Let f = stress (psi)

A_s = total steel area (in²)

p' = pressure capacity if the pressure is applied to the steel component (psi)

p = contribution to the core cavity pressure capacity (psi)

Liner:

$$\frac{p'_L R_L \cdot 60}{A_{s,L}} = f_L \rightarrow p'_L = \frac{3/4 \times 60}{186.375 \times 60} f_L = 4.024 \times 10^{-3} f_L$$

$$p_L = p'_L = 4.024 \times 10^{-3} f_L.$$

Rebar Groups:

$$\frac{p'_{Ri} R_{Ri} \cdot 60}{A_{s,Ri}} = f_{Ri} \rightarrow p'_{Ri} = \frac{A_{s,Ri}}{60 \cdot R_{Ri}} f_{Ri}$$

$$p_{Ri} = p'_{Ri} \frac{R_{Ri}}{R_L} = \frac{A_{s,Ri}}{60 \; R_L} f_{Ri}$$

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For R1: $p_{R1} = \frac{6.32}{60 \cdot 186.375} f_{R1} = 0.565 \times 10^{-3} f_{R1}$

For R2 ~ R5:

$$p_{Ri} = \frac{6.00}{6.32} \cdot 0.565 \times 10^{-3} f_{Ri} = 0.536 \times 10^{-3} f_{Ri}$$

For R6: $p_{R6} = \frac{474}{6.32} \cdot 0.565 \times 10^{-3} f_{R6} = 0.424 \times 10^{-3} f_{R6}$

Tendon Groups:

$$\frac{p'_{Ti} R_{Ti} 60}{A_{s,Ti}} = f_{Ti} \longrightarrow p'_{Ti} = \frac{A_{s,Ti}}{60 R_{Ti}} f_{Ti}$$

$$p_{Ti} = p'_{Ti} \frac{R_{Ti}}{R_L} = \frac{A_{s,Ti}}{60 R_L} f_{Ti}$$

$$= \frac{N' \cdot 7.463}{60 \cdot 186.375} f_{Ti} = 0.667 \times 10^{-3} N'_i f_{Ti} \quad \text{(152-wire)}$$

$$= \frac{N' \cdot 8.298}{60 \cdot 186.375} f_{Ti} = 0.742 \times 10^{-3} N'_i f_{Ti} \quad \text{(169-wire)}$$

where N'_i = Minimum number of tendons passing any 5 ft Section

(for each group, maximum is 4)

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A.7 Calculation of Minimum Tendons Required

(1) Select a limit criterion in terms of ϵ.

(2) Calculate strains for all steel components using the compatibility relations. Check $\epsilon \le \epsilon'$.

(3) Calculate f_L, f_{Ri} and f_{Ti} (Figs. A-2 ~ A-4)

(4) Calculate P_L, P_{Ri} and P_{Ti}

(5) Calculate the total pressure capacity to be contributed by tendons:

$$\left(\sum_{i=1}^{3} P_{T,i}\right)_{req'd} = \underset{\text{(or 1268)}}{845} - \left(P_L + \sum_{i=1}^{6} P_{R,i}\right)$$

(6) Calculate the total number of tendons required, using the middle row as an average:

$$N_6 = \sum_{i=1}^{3} N_i' = \frac{\left(\sum P_{Ti}\right)_{req'd}}{\underset{\text{(or 0.742)}}{0.667}\, f_{T2}}, \quad f_{T2} \text{ in ksi.}$$

(152-wire) → 0.667 ; (or 0.742) ~ (169-wire tendons)

Or $N = \frac{18}{12} \sum_{i=1}^{3} N_i'$

where N is the actual total number of tendons required to provide $\sum_{i=1}^{3} N_i'$ effective number of tendons in any 5 ft section.

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1 R P Case

Table A-3.

Criterion: Liner Stress at 0.9 f_{sy} = 54 ksi

Component	R (in.)	$\frac{R_L}{R}$	$\Delta\epsilon_{Ti}$	ϵ	f (ksi)	p (psi)
Liner	186.38	1		0.00186	54.0	217.3
R1	191.25	0.975		0.00181	52.5	29.7
R2	204.07	0.913		0.00170	49.3	26.4
R3	241.31	0.772		0.00144	41.8	23.6
R4	262.81	0.709		0.00132	38.3	20.5
R5	284.21	0.656		0.00122	35.4	19.0
R6	291.50	0.639		0.00119	34.5	14.6
T1	243.00	0.767	0.00143	0.00619	Average 164	109 each tendon
T2	264.50	0.705	0.00131	0.00607		
T3	286.00	0.652	0.00121	0.00597		

$$845 - (p_L + \Sigma p_{Ri}) = 845 - (351.1) = 493.9 \text{ psi}$$

$$\sum_{i=1}^{3} N_i = \frac{493.9}{109} = 4.53 \qquad \rightarrow N = 6.80$$

For $N = 7$ $p = 351 + 109 \times 7 \times \frac{2}{3} = 860$ psi

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1RP case   Table A-4

Criteria 2:

Tendons at R = 243 in have $\epsilon = \epsilon'$

Component	R (in.)	$\frac{R_{T1}}{R}$	$\Delta\epsilon_{Ti}$	ϵ	f (ksi)	p (psi)
Liner	186.38	1.30		0.0458	64.7	260.5
R1	191.25	1.27		0.0447	78.3	44.2
R2	204.07	1.19		0.0419	77.0	41.3
R3	241.31	1.01		0.0356	74.1	39.7
R4	262.81	0.925		0.0326	72.7	39.0
R5	284.31	0.855		0.0301	71.6	38.4
R6	291.50	0.834		0.0294	71.3	30.2
T1	243.00	1	0.0352	0.04	240	160.0 each
T2	264.50	0.919	0.0324	0.0372	236.6	157.8 each tendon
T3	286.00	0.850	0.0200	0.0347	233.6	155.8 each

$845 - (p_L + \Sigma p_{Ri}) = 845 - (493) = 352$

$\sum_{i=1}^{i} N_i' = 352/157.8 = 2.23$ $\rightarrow N = 3.35$

For N = 4. $p = 493 + 4 \times 157.8 \times \frac{2}{3} = 914$ psi

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<u>IRP Case</u> Table A-5

Criterion 3:

<u>Rebars at $R = 191.25$ in. have $\epsilon = \epsilon'$</u>

Component	R (in.)	$\frac{R_{R1}}{R}$	$\Delta\epsilon_{Ti}$	ϵ	f (ksi)	p (psi)
Liner	186.38	1.03		0.072	67.7	272.5
R1	191.25	1		0.07	90.0	50.9
R2	204.07	0.937		0.0656	87.8	47.2
R3	241.31	0.793		0.0555	83.3	44.6
R4	262.81	0.728		0.0509	81.2	43.5
R5	284.31	0.673		0.0471	79.4	42.6
R6	291.50	0.656		0.0459	78.9	33.4
T1	243.00	0.787	0.0551	0.0599		0
T2	264.50	0.723	0.0506	0.0554	> 4 %. $= \epsilon'$	0 each tendon
T3	286.00	0.669	0.0468	0.0516	Failed.	0

$$845 - (p_L + \Sigma p_{Ri}) = 845 - (535) = 310$$

$\sum_{i=1}^{3} N_i' =$ N.A.

→ $N =$ N.A

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Tables A-3 through A-5 are for tendons with 152 wires. For tendons with 169 wires, still for 1 RP, using the data in Tables A-3 and A-4:

Criterion 1. $f_T = 164$ ksi $\rightarrow \frac{\tau_{Ti}}{N_i'} = 121.7$ psi

$$\sum N_i' = \frac{493.9}{121.7} = 4.06 \quad \text{vs} \quad 4.53 \text{ for 152-wire tendons}$$

Criterion 2 $f_T = 236.6 \longrightarrow \frac{\tau_{Ti}}{N_i} = 175.6$

$$\sum N_i' = \frac{352}{175.6} = 2.00 \quad \text{vs} \quad 2.23 \text{ for 152-wire tendons}$$

→ Use the same number of tendons as in the 152-wire tendon case.

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1.5 RP Case

Data in Tables A-3 and A-5 still apply.

The required pressure capacity is 1268 psig instead of 845 psig.

Based on Table A-3:

$P_L + \Sigma P_{Ri} = 351.1$ psi.

Tendons must supply $1268 - 351.1 = 916.9$

$$\sum_{i=1}^{3} N_i' = \frac{916.9}{.109} = 8.41 \longrightarrow N = 12.6$$

Use $N = 13 \longrightarrow P = 351.1 + 109 \times 13 \times \frac{2}{3} = 1293$ psi.

Criterion 2: Tendons at R=243 in. have $\epsilon = \epsilon'$

$$\sum_{i=1}^{3} N_i = \frac{1268 - 493}{1578} = 4.91 \longrightarrow N = 7.37$$

Use $N = 8 \longrightarrow P = 493 + 8 \times 157.8 \times \frac{2}{3} = 1335$ psi.

Criterion 3: Rebars at R= 191.25 in have $\epsilon = \epsilon'$

Tendon strain > 4 %. $P = 535$ psig

N.G.

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A.8 Summary of Calculated Results

Table A-6

Minimum Number of Circumferential Tendons Required per 5 ft. High Wall Section

Limit Criteria	Required 1.0 RP		Required 1.5 RP		Number currently Provided	
	Actual	Effective	Actual	Effective	Actual	Effecti
Liner Stress $= 0.9 f_{sy}$	7	5	13	9	18	12
Max. Tendon strain $= \epsilon'$	4	3	8	5	18	12

APPENDIX B

NUMBER OF PCRV HEAD TENDONS

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B. 1. Yield Line Mode of Failure

The analysis is performed for the bottom head subjected to 1 RP or 1.5 RP cavity pressure. Calculational steps parallel those used in the FSAR Update, Section E. 11. 2. 2 (Ref. 1).

B. 1.1 Basic Assumptions

See Ref. 1, Section. E. 11. 2. 2. 2

Additional assumptions:

1) No vertical tendons.

2) Number of crosshead and circumferential tendons in the heads are to be reduce (Original design has 24 crosshead tendons and 34 circumferential tendons.)

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B.1.2 Calculational Procedure

1) Calculate the net head pressure load.
(Cavity pressure - cavity pressure equivalent of crosshead tendons)
This is done for an assumed N_x (§B.1.3)

2) Calculate the net tensile force (or boundary force) transmitted through the intersection of the PCRV wall and the bottom head (§B.1.4

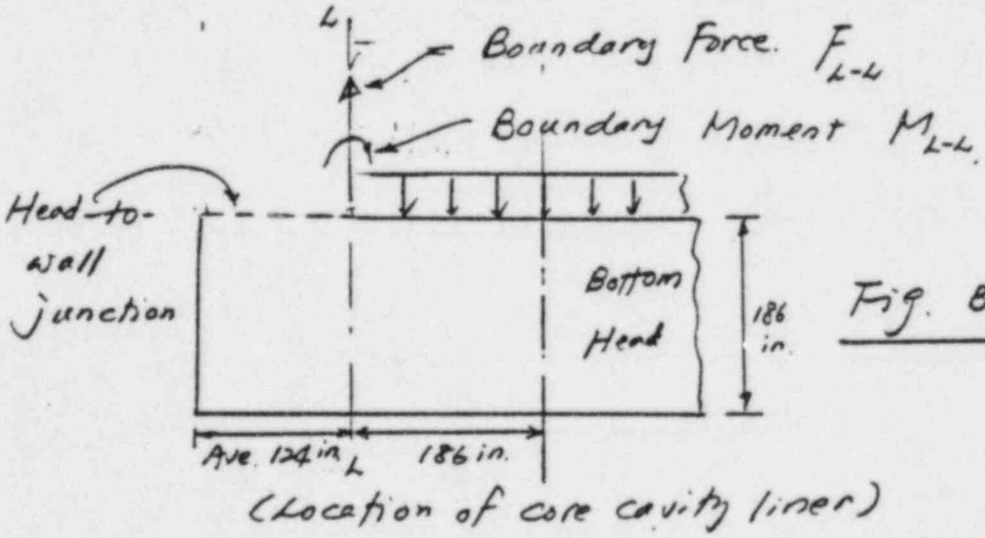

Fig. B-1

3) Assuming that a plastic hinge forms at the head-to-wall junction, calculate the boundary moment corresponding to $F_{℄-℄}$. (§B.1.5

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4) Assuming a yield pattern for the head, calculate the unit yield line moment required to balance the net pressure. F_{L-L} and M_{L-L}. (§ B 1.6)

5) Calculate the number of circumferential (head) tendons required to provide a unit moment capacity along the yield equals to or larger than the required unit yield line moment. (§B.1.7. B.1.8)

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B.1.3 Net Head Pressure Load

The pressure load on the heads is resisted in part by the crosshead tendons. The vertical component of the crosshead tendons is:

$$F_v = (2 N_x) \cdot (A_t)(f_t) \sin \alpha$$

where: N_x = Number of crosshead tendons

A_t = X-sectional area of a tendon = 8.35 in^2.

f_t = tendon stress

α = average inclination of the tendons

= 36° 45'

Allowing $f_t = 0.7 f_s' \underbrace{(1-0.12)}_{\text{loss}}\underbrace{(0.9)}_{\text{friction}} = 0.554 f_s' = 13$ ks.

(Ref. 1, Table 5.6-A), (Ref. 3)

$$F_v = 2 N_x (8.35)(133,000) \sin 36° 45'$$

$$= 1,329,000\ N_x \text{ lbs.}$$

Cavity pressure equivalent of crosshead tendons is assumed constant over the cavity.

$$p_x = \frac{F_v}{3.14 \times 186^2} = 12.23\ N_x \text{ psig}$$ (Cavity rad. = 186 in,

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The net pressure load is:

p = Actual cavity pressure − 12.23 N_x.

Table B-1 Net Pressure and Boundary Force

N_x	1.0 RP		1.5 RP	
	Net pressure (psi)	F_{L-L} (k/in. of liner)	Net pressure (psi)	F_{L-L} (k/in. of liner)
0	845	78.6	1268	117.9
6	772	71.8	1194	111.1
12	698	64.9	1120	104.2
18	625	58.1	1046	97.3
24	551	51.3	972	90.4

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B.1.4 Boundary Force

Since there is no vertical tendons. the net pressure load must be resisted by the vertical rebars. liners and concrete. The net tensile force per circumferential inch at the head-to-wall junction is:

$$F_{L-L} = \frac{\pi (186)^2 (\text{Net pressure in psi})}{2\pi \underbrace{(186)}_{\text{core cavity radius}} (1000)} \quad \text{k/in. of liner.}$$

See Table B-1.

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B.1.5 Boundary Moment

Boundary moment at the head-to-wall junction is determined by establishing a strain diagram which is consistent with the plastic hinge condition and which will result in a net cross-section resistance force equals to F_{L-L}.

A plastic hinge is considered formed when,

Max. concrete strain = 0.003

or

Max. steel strain (rebar) = 0.07 = ϵ'

Liner contribution is neglected in this analysis.

Fig. B-2 shows the location of rebars strain diagram, etc. for a case in which the two limits given above occur simultaneously.

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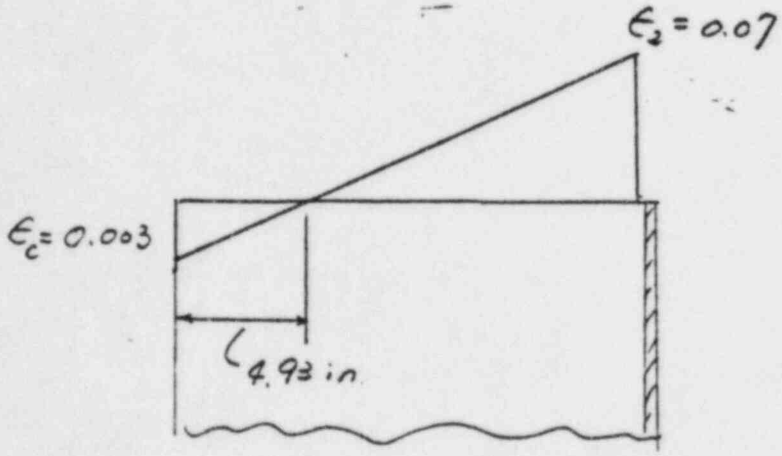
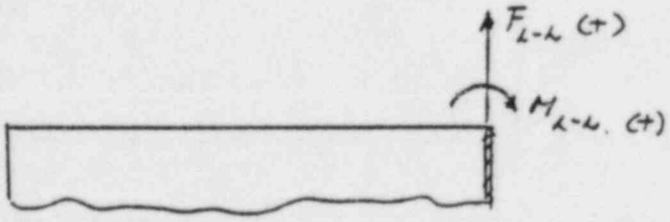
A balanced case:



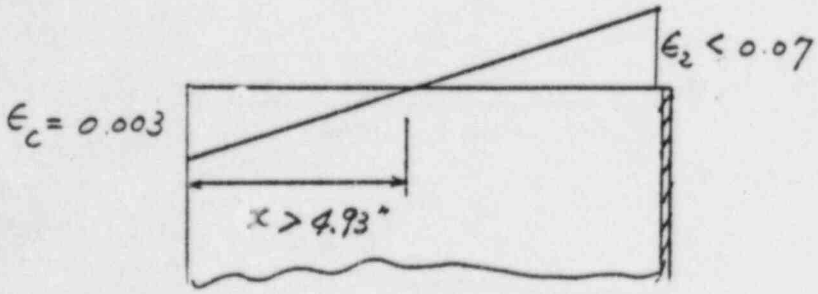
Referring to Fig. B-2 and Table B-2:

$F_{L-L} = 76.6$ k/in. of liner

$M_{L-L} = 1616.06$ k-in./in. of liner.



For $F_{L-L} < 76.6$ k/in:



Case 1

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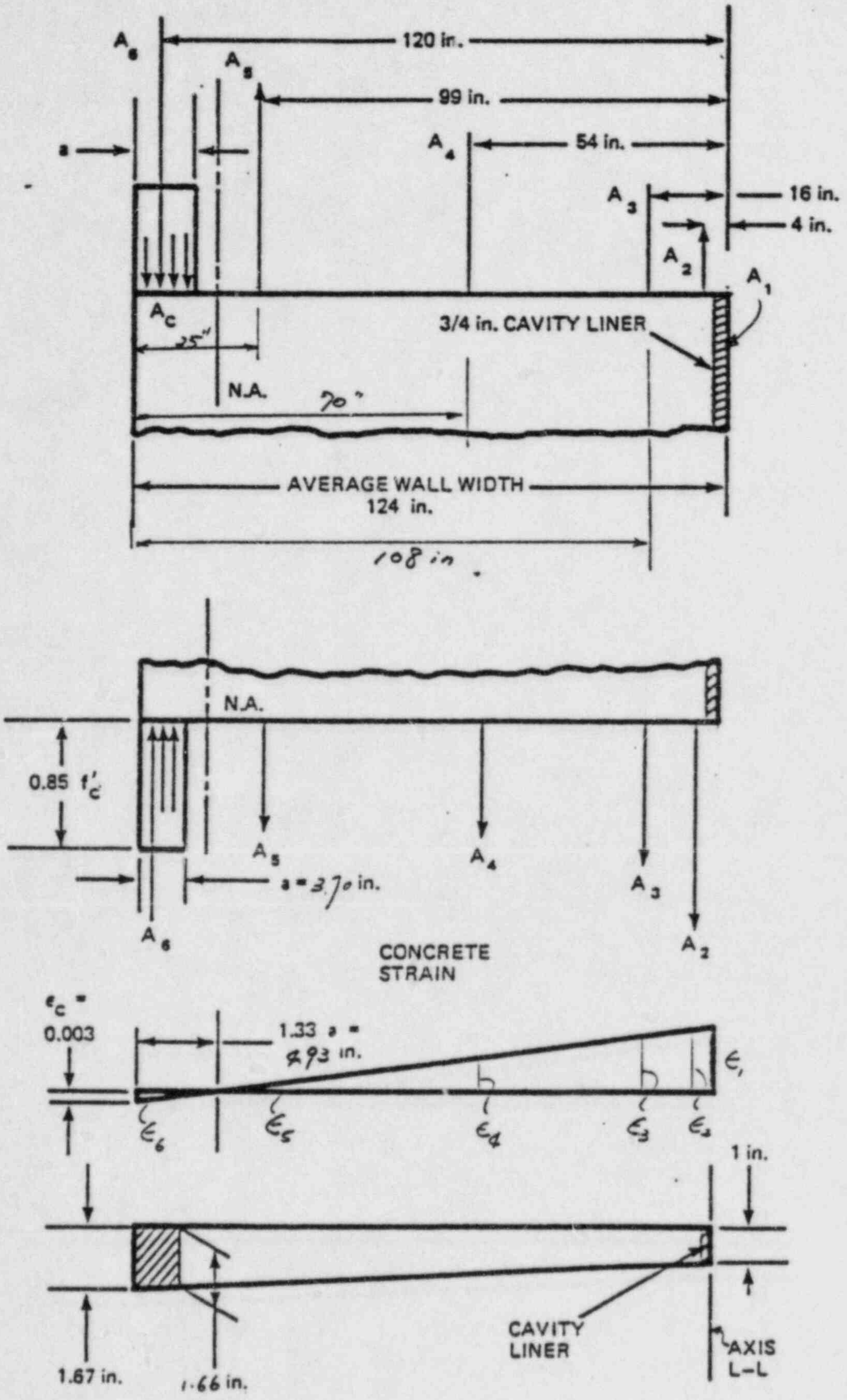

Figure B-2

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Table B-2

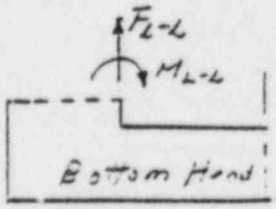
Wall Moment about Liner Axis L-L

Element	Area[(a)] (in²/in.)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in/in.)
A_1	0.75		~	~	0	~
A_2	0.27	0.07	90.	24.300	4.0	97.20
A_3	0.29	0.063	86.640	25.126	16.0	402.02
A_4	0.21	0.040	75.969	15.953	54.0	861.46
A_5	0.77	0.012	63.331	48.765	99.0	4827.74
A_6	0.38	-0.001	-16.412	-6.236	120.0	-748.32
A_c		0.003		-31.306	122.15	-3824.03

(a) Those for A_2 through A_6 are from Ref. 1, Table E.1-1

(b) Based on $x = 4.93$ in.
$\epsilon_c = 0.003$

$F_{L-L} = 76.602$ (vs. . .) $M_{L-L} = +1616.06$



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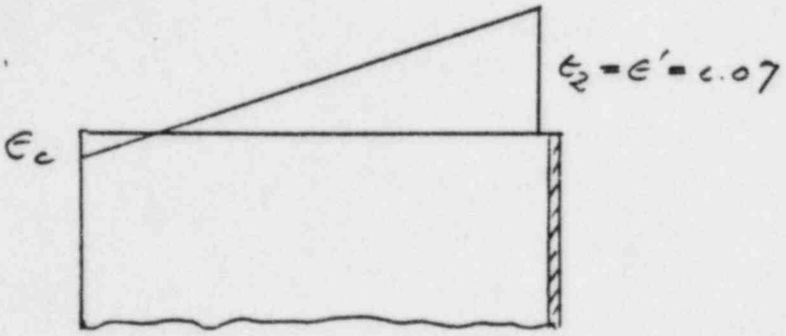
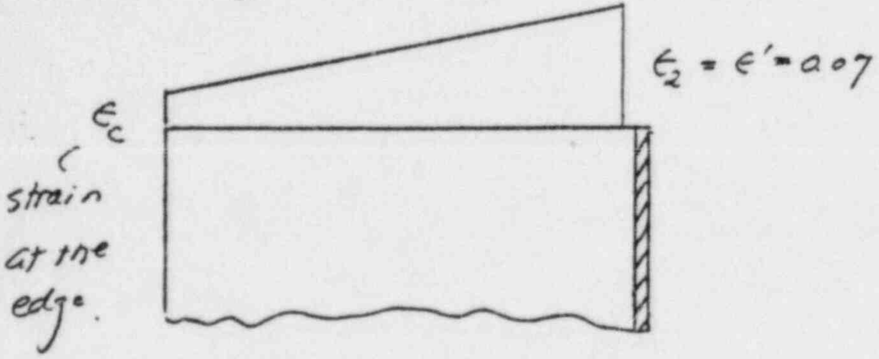
For $F_{L-L} > 76.6$ k/in :

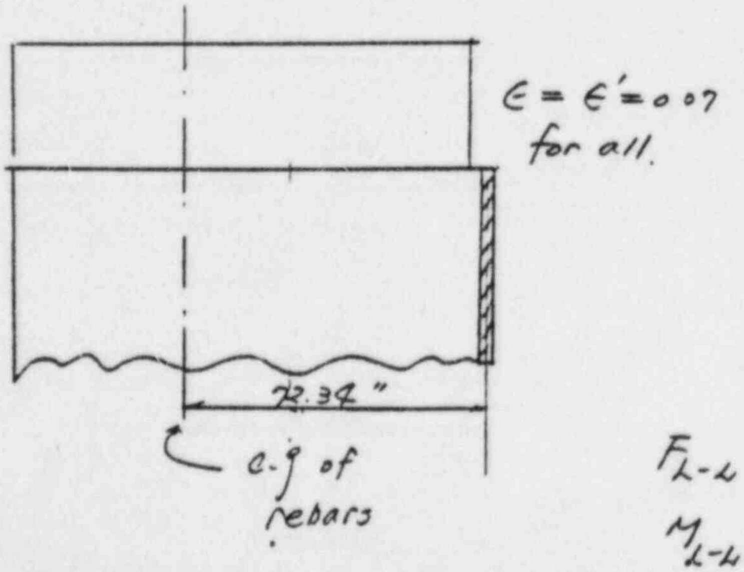


Case 2

Larger F_{L-L} ↓



Larger F_{L-L} ↓



$F_{c.g} = f_s' \cdot \Sigma A_s$

$= 90 \times 1.92$

$= 172.80$ k/in

$M_{c.g} = 0.$

$F_{L-L} = 172.80$

$M_{L-L} = +12500$ k-in/in.

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For Case 1, $F_{L-L} < 766$ k/in:

From the force equilibrium:

$$\sum_{i=2}^{5} F_i - F_6 - F_c = F_{L-L} = \text{Boundary Force}$$

Note: <u>Liner force not included.</u>

Strains:

$$\epsilon_1 = \frac{0.003(124-x)}{x} = \frac{0.372}{x} - 0.003$$

$$\epsilon_2 = \frac{0.003(120-x)}{x} = \frac{0.36}{x} - 0.003$$

$$\epsilon_3 = \frac{0.003(124-16-x)}{x} = \frac{0.003(108-x)}{x} = \frac{0.324}{x} - 0.00$$

$$\epsilon_4 = \frac{0.003(124-54-x)}{x} = \frac{0.003(70-x)}{x} = \frac{0.21}{x} - 0.00$$

$$\epsilon_5 = \frac{0.003(124-99-x)}{x} = \frac{0.003(25-x)}{x} = \frac{0.075}{x} - 0.0$$

$$\epsilon_6 = \frac{0.003(x-4)}{x} = 0.003 - \frac{0.012}{x} \quad \text{(comp.)}$$

Stress:

Let f_i be the stress in element i.

$$f_i = 90 - (0.07 - \epsilon_i) \times 461.5 \quad \text{if } \epsilon_i \geq 0.00202$$

$i = 2 \sim 6$ (Fig. A-2)

$$f_6 = 29000\,\epsilon_6 \quad \text{if } \epsilon_6 \leq 0.00202 \quad \text{(Fig. A-3)}$$

$$f_c = 0.85 f_c' = 5.1 \text{ ksi}$$

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Forces:

$$F_i = f_i \times A_i \qquad i = 2, \cdots, 6$$

$$F_c = f_c \times A_c$$

where $A_c = (0.75\,x)$(Ave width of conc. compression zone)

$$= (0.75\,x)(\bar{w}_c)$$

$$\bar{w}_c = \frac{1}{2}\left(1.67 + 1.67 - 0.67\,\frac{0.75\,x}{124}\right)$$

$$= 1.67 - 0.00203\,x$$

use $\bar{w}_c = 1.66 \qquad x \le 7.39"$

$\bar{w}_c = 1.65 \qquad x \le 12.31"$

$\bar{w}_c = 1.64 \qquad x \le 17.24"$

$$F_c = 6.35\,x \qquad \text{if} \quad x \le 7.39"$$

$$= 6.31\,x \qquad \text{if} \quad x \le 12.31"$$

$$= 6.27\,x \qquad \text{if} \quad x \le 17.24"$$

Distance from L.L. for A_c:

$$124 - \frac{0.75\,x}{2} = 124 - 0.375\,x$$

Calculation of M_{L-L} using the above formulas and other needed relations are done in tabular form.

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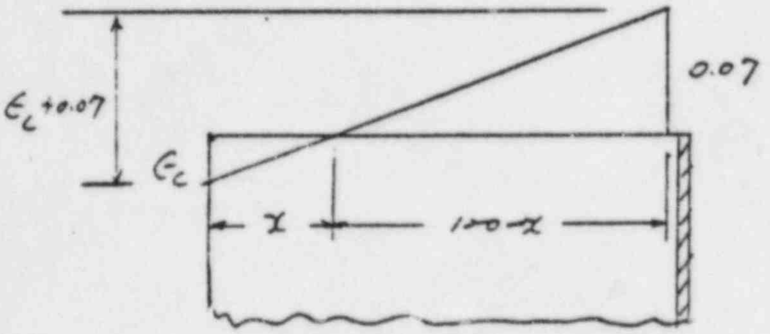
Case 2: For $F_{x-L} > 76.6$ k/in



$$\epsilon_3 = 0.07 - (\epsilon_c + 0.07)\frac{12}{120} = 0.0630 - 0.1\,\epsilon_c$$

$$\epsilon_4 = 0.07 - (\epsilon_c + 0.07)\frac{50}{120} = 0.0408 - 0.4167\,\epsilon_c$$

$$\epsilon_5 = 0.07 - (\epsilon_c + 0.07)\frac{95}{120} = 0.0146 - 0.7917\,\epsilon_c$$

$$\epsilon_6 = 0.07 - (\epsilon_c + 0.07)\frac{116}{120} = 0.0023 - 0.9667\,\epsilon_c$$

$$x = \frac{120\,\epsilon_c}{0.07 + \epsilon_c}$$

$$\left.\begin{array}{l}\text{Use } F_c = 6.35x \\ \text{\& arm} = 124 - 0.375x\end{array}\right\} \text{For } \epsilon_c \geq 0.001$$

(i.e, assume $0.85 f_c'$ rect. block)

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To avoid a tedious iteration process, calculation of M_{L-L} for various F_{L-L} given in Table B-1 is done as follows:

1) Compute a series of F_{L-L}, M_{L-L} pairs for selected values of x or ϵ.
Use formulas for Case 1 or Case 2, as neede
(Tables B-2 through B-13)
2) Interpolate to obtain M_{L-L} for various F_{L-L} of interest
(Table B-14)

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Table B-3 Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in/in.
A_1	0.75	0.051	—	—	0	—
A_2	0.27	0.049	80.389	21.705	4.0	86.820
A_3	0.29	0.044	77.981	22.614	16.0	361.852
A_4	0.21	0.027	70.356	14.775	54.0	797.859
A_5	0.77	0.008	61.327	47.222	99.0	4674.972
A_6	0.38	0.001	-36.565	-13.895	120.0	-1667.374
A_c		0.003		-43.815	121.413	-5319.689

$F_{L-L} = 48.61$ $M_{L-L} = -1065.63$

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

(b) Based on $x = 6.9$ in.

$\epsilon_c =$

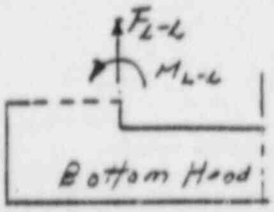

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Table B-4 Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in/in.)
A_1	0.75	0.055	—	—	0	—
A_2	0.27	0.053	82.270	22.213	4.0	88.851
A_3	0.29	0.048	79.674	23.105	16.0	369.680
A_4	0.21	0.030	71.453	15.005	54.0	810.282
A_5	0.77	0.009	61.719	47.523	99.0	4704.817
A_6	0.38	0.001	-32.625	-12.398	120.0	-1487.700
A_c		0.003		- 40.640	121.6	-4981.824

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 54.85$ $M_{L-L} = -455.894$

(b) Based on $x = 6.4$ in.

$\epsilon_c =$

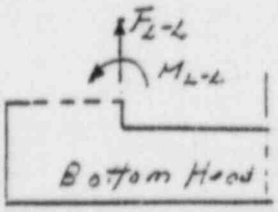

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Table B-5 Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in/in)
A_1	0.75	0.060	—	—	0	—
A_2	0.27	0.058	84.375	22.781	4.0	91.125
A_3	0.29	0.052	81.568	23.655	16.0	378.477
A_4	0.21	0.032	72.681	15.263	54.0	824.206
A_5	0.77	0.010	62.157	47.861	99.0	4738.244
A_6	0.38	0.001	-28.216	-10.722	120.0	-1286.659
A_c		0.003		-37.592	121.78	-4577.954

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 61.25$ $M_{L-L} = 167.439$

(b) Based on $x = 5.92$ in.

$\epsilon_c =$

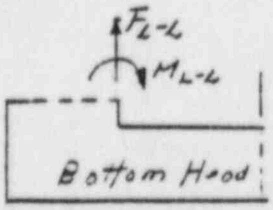

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Table B-6 Boundary Force and
Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in/in.
A_1	0.75	0.065	—	—	0	—
A_2	0.27	0.062	86.52	23.36	4.0	93.4
A_3	0.29	0.056	83.50	24.21	16.0	387.4
A_4	0.21	0.035	73.93	15.53	54.0	838.4
A_5	0.77	0.011	62.60	48.20	99.0	4772.3
A_6	0.38	-0.001	-23.73	-9.02	120.0	-1082.0
A_c		0.003		-34.92	121.94	-4258.7

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 67.36$ $M_{L-L} = +750.8$

(b) Based on $x = 5.5$ in.

$\epsilon_c = 0.003$

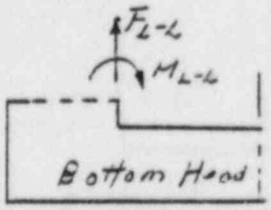

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Table B-7. Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in.)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in./in.
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0627	86.64	25.12	16.0	402.0
A_4	0.21	0.0396	75.97	15.95	54.0	861.5
A_5	0.77	0.0123	63.37	48.80	99.0	4830.8
A_6	0.38	−0.0005	−14.60	−5.55	120.0	−665.7
A_c		0.0029		−30.29	122.21	−3701.7

(a) Those for A_2 through A_6 are from Ref. 1, Table E.1-1

$F_{L-L} = 78.3$ $M_{L-L} = 1824.1$

(b) Based on $x = 4.77$ in.

$\epsilon_c = 0.0029$

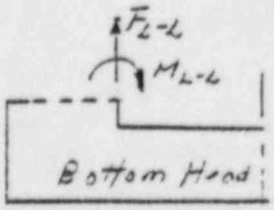

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Table B-8 Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in.)/in.
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0628	86.66	25.13	16.0	402.1
A_4	0.21	0.0398	76.08	15.98	54.0	862.8
A_5	0.77	0.0128	63.59	48.97	99.0	4847.7
A_6	0.38	0.0001	2.22	0.84	120.0	101.3
A_c		0.0023		-24.24	122.57	-2971.0

$F_{L-L} = 90.98$ $M_{L-L} = 3340.0$

(a) Those for A_2 through A_6 are from Ref. 1, Table E.1-1

(b) Based on $x = 3.82$ in.

$\epsilon_c = 0.0023$

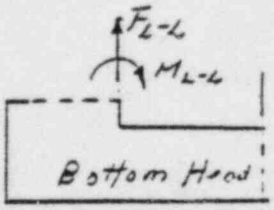

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Table B-9 Boundary Force and Moment about Liner Axis L-L

Element	Area[a] (in²/in)	Strain[b] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[a] (in.)	Moment abt. L-L (k-in./in.
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0628	86.68	25.14	16.0	402.2
A_4	0.21	0.0400	76.14	15.99	54.0	863.4
A_5	0.77	0.0130	63.70	49.05	99.0	4856.0
A_6	0.38	0.0004	11.60	441	120.0	529.0
A_c		0.002		-21.17	122.75	-2598.6

$F_{L-L} = 97.72$ $M_{L-L} = 4149.2$

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

(b) Based on $x = 3.33$ in.

$\epsilon_c = 0.002$

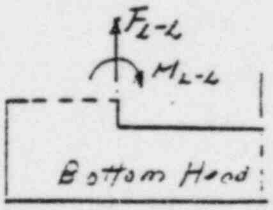

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Table B-10 Boundary Force and Moment about Liner Axis L-L

Element	Area[a] (in²/in)	Strain[b] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[a] (in.)	Moment abt. L-L (k-in/in.)
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0628	86.69	25.14	16.0	402.2
A_4	0.21	0.0401	76.20	16.00	54.0	864.1
A_5	0.77	0.0133	63.81	49.14	99.0	4864.4
A_6	0.38	0.0007	19.04	7.24	120.0	1047.6
A_c		0.0017		-18.07	122.9	-2221.4

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 103.75$ $M_{L-L} = 5055.1$

(b) Based on $x = 2.84$ in.

$\epsilon_c = 0.0017$

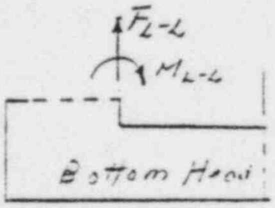

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Table B-11 Boundary Force and Moment about Liner Axis L-L

Element	Area[a] (in²/in)	Strain[b] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[a] (in.)	Moment abt. L-L (k-in/in.)
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0629	86.71	25.14	16.0	402.3
A_4	0.21	0.0403	76.27	16.02	54.0	869.9
A_5	0.77	0.0136	63.96	49.25	99.0	4875.5
A_6	0.38	0.0010	30.26	11.50	120.0	1379.6
A_c		0.0013		-13.89	123.18	-1710.9

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 112.32$ $M_{L-L} = +5913.6$

(b) Based on $x = 2.19$ in.

$\epsilon_c = 0.0013$

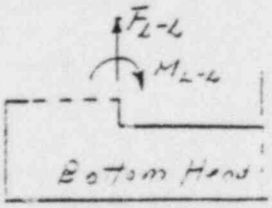

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Table B-12 Boundary Force and Moment about Liner Axis L-L

Element	Area[(a)] (in²/in)	Strain[(b)] (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L[(a)] (in.)	Moment abt. L-L (k-in)/in.
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90.0	24.3	4.0	97.2
A_3	0.29	0.0629	86.723	25.15	16.0	402.4
A_4	0.21	0.0404	76.34	16.03	54.0	865.7
A_5	0.77	0.0138	64.07	49.33	99.0	4883.9
A_6	0.38	0.0013	37.7	14.33	120.0	1719.1
A_c		0.001		−10.73	123.37	−1323.7

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 117.65$ $M_{L-L} = 6644.6$

(b) Based on $x = 1.69$ in. $\varepsilon_c = 0.001$

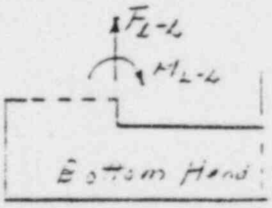

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Table B-13 Boundary Force and Moment about Liner Axis L-L

Element	Area(a) (in²/in)	Strain(b) (in./in.)	Stress (ksi)	Force (k/in. liner)	Dist. from L-L(a) (in.)	Moment abt L-L (k-in/in
A_1	0.75		—	—	0	—
A_2	0.27	0.07	90	24.3	4.0	97.2
A_3	0.29	0.0630	86.77	25.16	16.0	402.6
A_4	0.21	0.0408	76.52	16.07	54.0	867.8
A_5	0.77	0.0146	64.43	49.61	99.0	4911.7
A_6	0.38	0.0023	58.76	22.33	120.0	2679.3
A_c						

(a) Those for A_2 through A_6 are from Ref. 1, Table E.11-1

$F_{L-L} = 137.47$ $M_{L-L} = 8958.6$

(b) Based on $x =$ in

$\epsilon_c = 0$

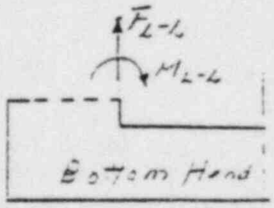

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Table B-14 M_{L-L} Values by Interpolation

Calculated (See Tables)
F_{L-L} vs M_{L-L}

F_{L-L}	M_{L-L}
48.61	-1065.6
54.85	-455.9
61.25	167.4
67.36	750.8
76.60	1616.1
78.30	1824.1
90.98	3340.0
97.72	4149.2
103.75	5054.1
112.32	5908.6
117.65	6644.6
137.47	8958.6

Interpolated
F_{L-L} vs M_{L-L}

F_{L-L}	M_{L-L}
51.3	-802.8
58.1	-139.4
64.9	515.9
71.8	1166.6
78.6	1860.0
90.4	3270.7
97.3	4098.8
104.2	5099.0
111.1	5787.0
117.9	6673.8

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B.1.6 Unit Yield Line Moment

Consider two failure modes as shown in Fig. E-3 For each mode, calculate the unit yield line moment required to maintain equilibrium.

Forces acting on each segment of slab bounded by the yield lines and the boundaries are:

1) Net cavity pressure
2) Boundary forces and moments
3) yield line moment

Equilibrium of these forces are established by computing their moments about Line C-C' (or E-E') shown in Fig B-4.

The following calculation is performed for the case of 1.0 RP with $N_x = 0$. Unit yield moments for other cases are done by proportion from this basic case (Tables B-15 ~ B-18).

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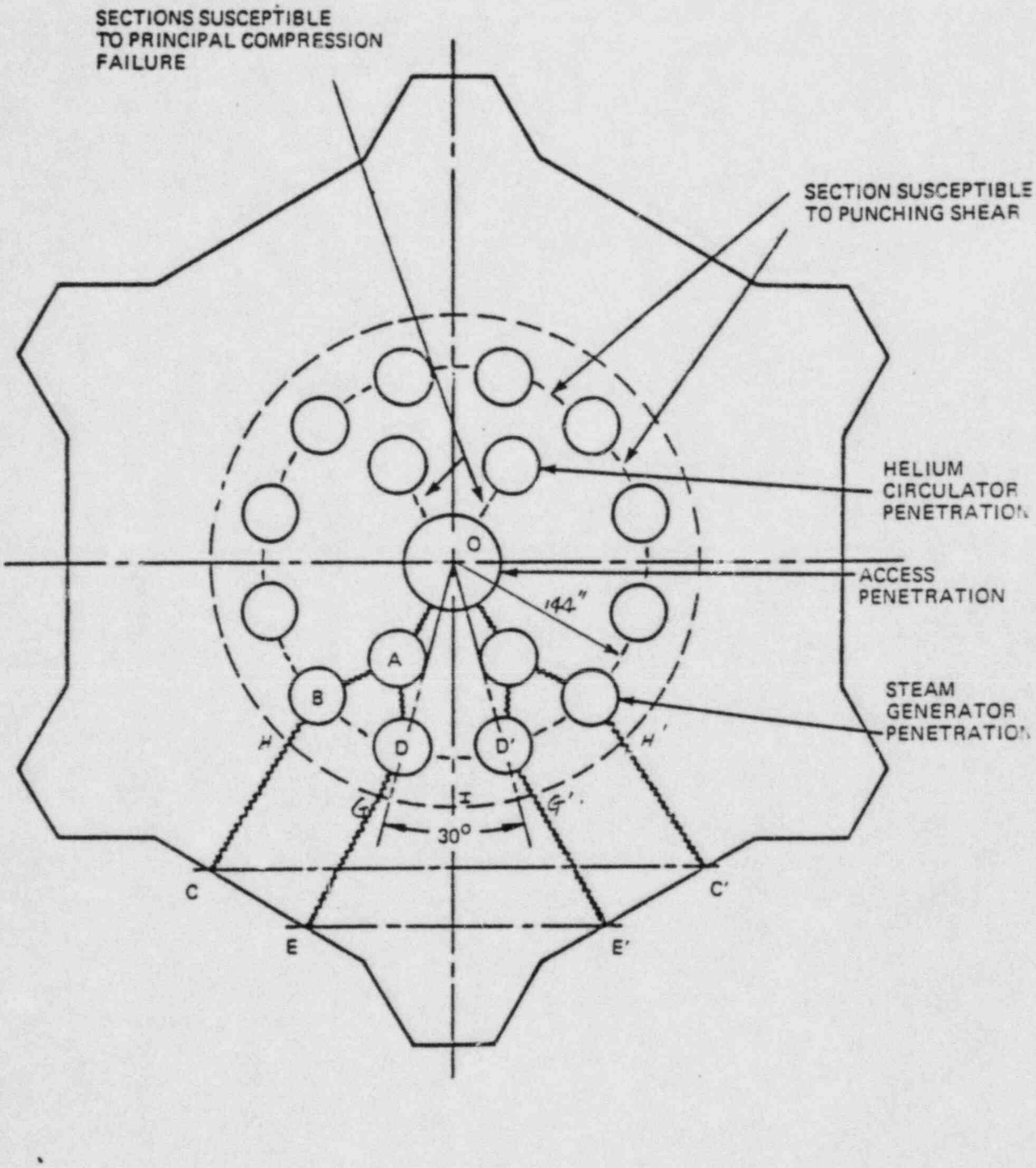

Figure B-3 Arrangement of Lower Head Penetrations and Potential Failure Modes

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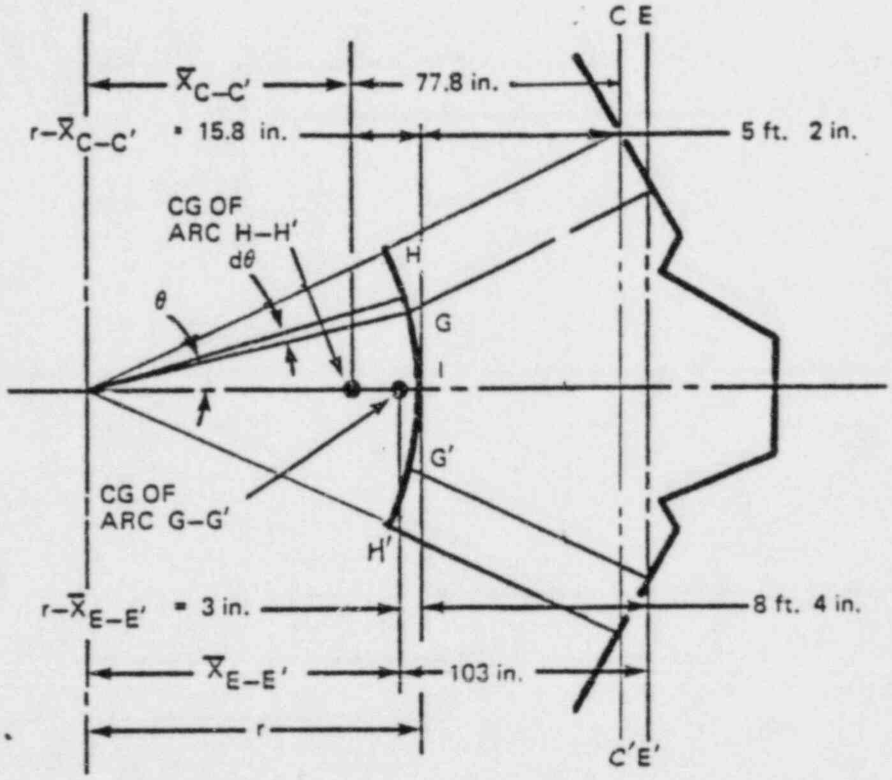

Fig. B-4

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Yield pattern O-A-B-C (See Fig. B-3):

Referring to Fig. B-5, the moment about Line A-A' due to the uniform distributed moment m is:

$$M_{A-A'} = 2\int_0^{\theta_1} mr\cos\theta\, d\theta = 2mr\left[\sin\theta\right]_0^{\theta_1}$$

$$= 2mr\sin\theta_1$$

For this case, $\theta_1 = 41°30'$ (Fig. B-6)

Hence, due to M_{L-L} (See Table B-14):

$$M_{C-C'} = M_{H-H'} = 2(1860\times1000)(186)\sin 41°30'$$

$$= 0.458\times10^9 \text{ lbs-in}$$

Due to F_{L-L} over H-H'.

$$M_{C-C'} = 78.6\times10^3\times\frac{2\times41.5\times\pi}{180}\times186\times77.8$$

$$= 1.648\times10^9 \text{ lbs-in}$$

where 77.8 in. is the distance between the c.g. of F_{L-L} over H-H' and Line C-C' (See Fig. B-4).

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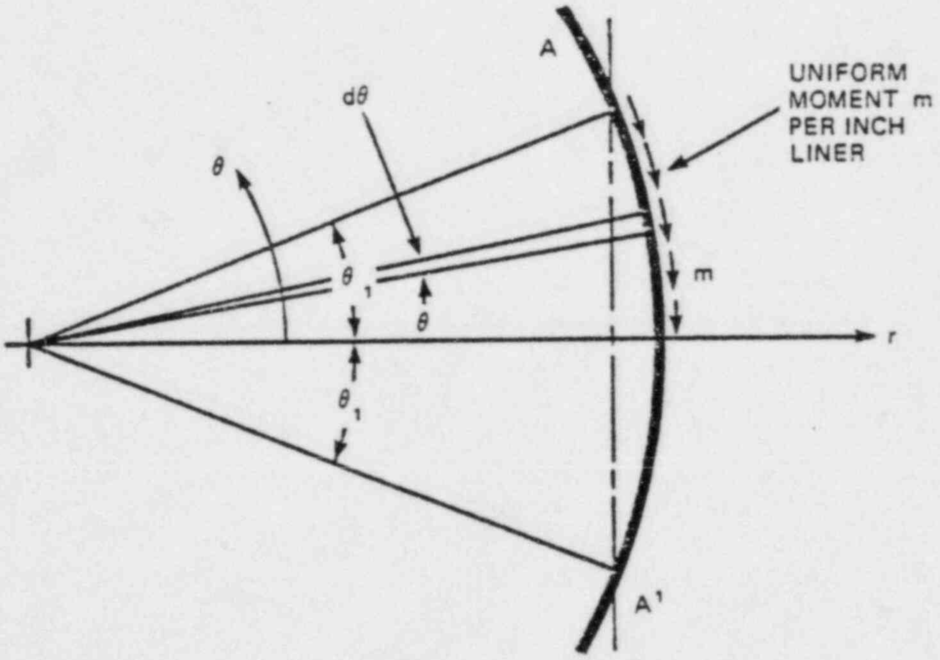

Fig. B-5

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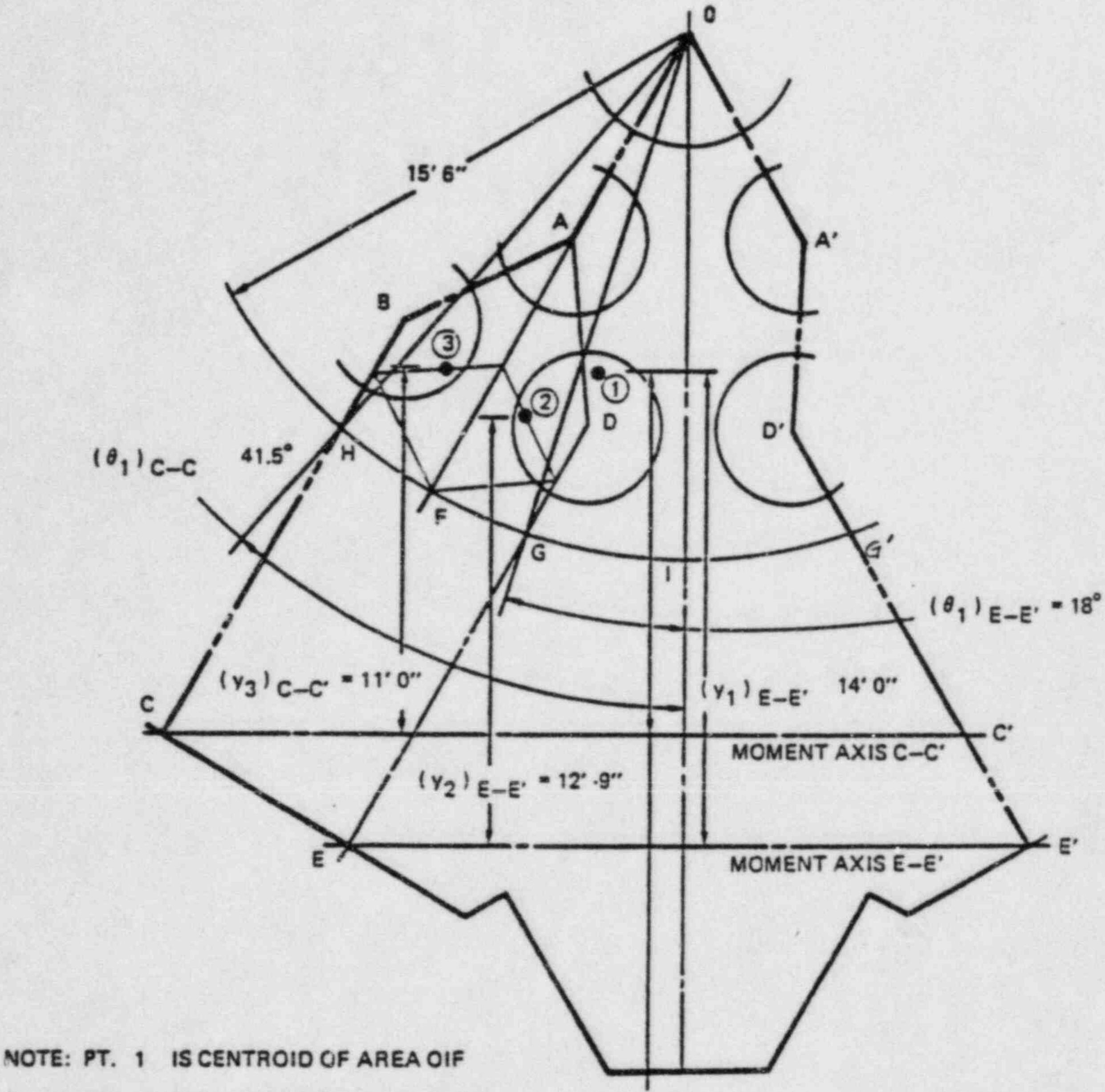

NOTE: PT. 1 IS CENTROID OF AREA OIF

PT. 2 IS CENTROID OF AREA ADGF

PT. 3 IS CENTROID OF AREA ABHF

Fig. B-6

Potential Failure Mode

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Referring to Fig. 10, and Ref. a p. E.11-7.

Area OAFI $= \frac{1}{12}\pi(186)^2 = 9050 \text{ in}^2$

Centroidal dist. from c-c' = 10'10" = 130 in

Area ABHF $= \frac{1}{2}(102+42)\times 38 = 2736 \text{ in}^2$

Centroidal dist. from c-c' = 11' = 132 in.

Hence moment about c-c'. due to the cavity pressure on (OABHI) × 2 is:

$$M_{c-c'} = 2p\left[(9050\times 130) + (2736\times 132)\right]$$

$$= 3.076\, p \times 10^6 \text{ lbs-in}$$

$$= 3.076 \times 845 \times 10^6 = 2.599 \times 10^9 \text{ lbs-in}$$

Total moment about c-c' to be resisted by the yield lines O-A-B-C is:

$$M_{c-c'} = (2.599 - 1.648 + 0.458) \times 10^9$$

$$= 1.410 \times 10^9 \text{ lbs-in}$$

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Let m_{B-c} be the unit yield line moment for all segments along 0-A-B-C

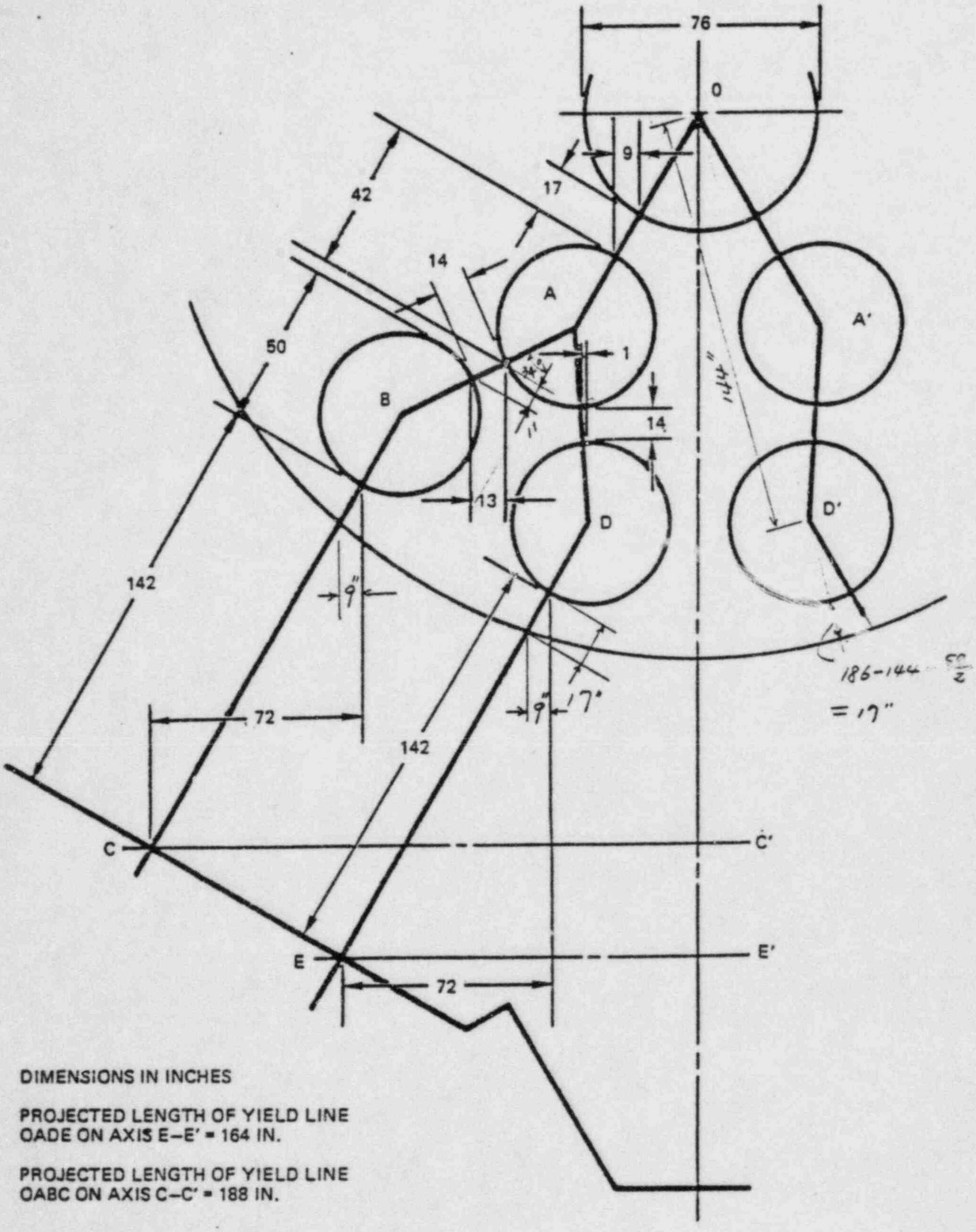
The components of total yield line moments for individual segments in the C-C' direction is (See Fig. B-7):

$$2\left(17\, m_{B-c} \cdot \frac{9}{17} + 14\, m_{B-c} \cdot \frac{13}{14} + 142\, m_{B-c} \cdot \frac{72}{142}\right)$$

$$= 188\, m_{B-c}$$

This balances the $M_{c-c'}$ total, hence

$$m_{B-c} = \frac{M_{c-c'} \text{ total}}{188} = \frac{1.410 \times 10^{9}}{188} = 7.50 \times 10^{6} \text{ lb-in/.}$$



DIMENSIONS IN INCHES

PROJECTED LENGTH OF YIELD LINE OADE ON AXIS E–E' = 164 IN.

PROJECTED LENGTH OF YIELD LINE OABC ON AXIS C–C' = 188 IN.

Figure B-7 Projected Length of Yield Line

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Yield pattern O-A-D-E:

Repeating the similar calculation done for yield pattern O-A-B-C.

Due to M_{L-L} over G-G',

$$M_{E-E'} = 2\,(1860 \times 1000)\,(186)\,\sin 18^\circ = 0.214 \times 10^9 \text{ lbs-in}$$

Due to F_{L-L} over G-G',

$$M_{E-E'} = (78.6 \times 1000) \times \frac{2 \times 18 \times \pi}{180} \times 186 \times 103$$

$$= 0.946 \times 10^9 \text{ lbs-in}$$

Due to cavity pressure over (OADGI)×2:

$$M_{E-E'} = 2\,p\,\left[(9050 \times 168) - (2736 \times 153)\right]$$

$$= 2.204\,p \times 10^6 \text{ lbs-in}$$

$$= 2.204 \times 845 \times 10^6 = 1.862 \times 10^9 \text{ lbs-in}$$

Total to be resisted by the yield lines O-A-D-E is:

$$M_{E-E'} = (1.862 - 0.946 + 0.214) \times 10^9$$

$$= 1.130 \times 10^9 \text{ lbs-in.}$$

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Let m_{D-E} be the unit yield line moment for all segments along O-A-D-E.

The components of total yield line moments for individual segments along the E-E' axis is (Fig. B-7):

$$2(9\,m_{D-E} + m_{D-E} + 72\,m_{D-E})$$

$$= 164\,m_{D-E}$$

This balances the $M_{E-E'}$ total, hence

$$m_{D-E} = \frac{M_{E-E',\,total}}{164} = \frac{1.130 \times 10^9}{164} = 6.89 \times 10^6$$

lbs-in/in

The m_{B-C} of 7.50×10^6 lbs-in/in and the m_{D-E} of 6.89×10^6 " computed for the case of 1 RP and $H_x = 0$ are used as the basis for proportioning in the following tables.

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Table B-15

Unit Yield Line Moments for Yield Pattern O-A-B-C, 1 RP

N_x		0	6	12	18	24
M_{L-L}		18600	1166.6	515.9	-139.4	-802.8
F_{L-L}		78.6	71.8	64.9	58.1	51.3
Resid. Cavity Pressure		845	772	698	625	551
$M_{c-c'}$ Due to (10^9 lb-in.)	M_{L-L} *	0.458	0.287	0.127	-0.034	-0.198
	F_{L-L} *	-1.648	-1.505	-1.361	-1.218	-1.076
	Press. *	2.599	2.374	2.147	1.922	1.695
Total $M_{c-c'}$		1.410	1.156	0.913	0.670	0.421
m_{B-C} * (10^6 lb-in/in.)		7.50	6.15	4.86	3.56	2.24

↑ Base case

Note: For Tables B-15 and B-17

* Numbers in these rows are computed by proportion from the base case in Table B-1.

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Table B-16

Unit Yield Line Moments

for Yield Pattern O-A-D-E, 1 RP

N_x		0	6	12	18	24
M_{L-L}		1860.0	1166.6	515.9	-139.4	-802.[illegible]
F_{L-L}		78.6	71.8	64.9	58.1	51.3
Resul. Cavity Pressure		845	772	698	625	551
$M_{E-E'}$ Due to (10^9 lb-in)	M_{L-L}^{**}	-0.214	0.134	0.059	-0.016	-0.092
	F_{L-L}^{**}	-0.946	-0.864	-0.781	-0.699	-0.617
	Press.**	1.862	1.701	1.538	1.377	1.214
Total $M_{E-E'}$		1.130	0.971	0.816	0.662	0.505
m_{D-E}^{**} (10^6 lb-in/in.)		6.89	5.92	4.98	4.04	3.08

Base Case

Note:
(For Tables B-16, B-18)

** Numbers in these rows are computed by proportion from the base case in Table B-16.

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Table B-17

Unit Yield Line Moments for Yield Pattern o-A-B-C, 1.5 RP

N_x		0	6	12	18	24
M_{L-L}		6673.8	5787.0	5099.0	4098.8	3270.7
F_{L-L}		117.9	111.1	104.2	97.3	90.4
Resul. Cavity Pressure		1268	1194	1120	1046	972
$M_{c-c'}$ Due to (10^9 lb-in)	M^*_{L-L}	1.643	1.425	1.256	1.009	0.805
	F^*_{L-L}	-2.472	-2.329	-2.185	-2.040	-1.895
	Press.*	3.900	3.672	3.445	3.217	2.990
Total $M_{c-c'}$		3.071	2.768	2.516	2.186	1.900
m^*_{B-C} (10^6 lb-in/in)		16.34	14.72	13.38	11.63	10.11

m_{B-C} = 11.27

if. N_x = 19.42 by interpolation

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Table B-18

Unit Yield Line Moments

for Yield Pattern O-A-D-E, 1.5 RP

N_x		0	6	12	18	24
M_{L-L}		6673.8	5787.0	5099.0	4098.8	3270.7
F_{L-L}		117.9	111.1	104.2	97.3	90.4
Resul. Cavity Pressure		1268	1194	1120	1046	972
$M_{E-E'}$ Due to (10^9 lb-in)	M_{L-L} **	0.768	0.666	0.587	0.472	0.376
	F_{L-L} **	-1.419	-1.337	-1.254	-1.171	-1.088
	Press. **	2.794	2.631	2.468	2.305	2.142
Total $M_{E-E'}$		2.143	1.960	1.801	1.606	1.430
m_{D-E} ** (10^6 lb-in./in.)		13.07	11.95	10.98	9.79	8.72

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B.1.7 Moment Capacity Along Yield Lines

Moment capacity in the hoop direction provided by the circumferential tendons, circumferential rebars and concrete is established for a cross section with unit width along a radial yield line.

Criteria:

1) Maximum tendon stress = $0.9\ f_{sy}$ (Ref. 1)

2) Maximum concrete strain ≤ 0.003

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Circumferential Rebars:

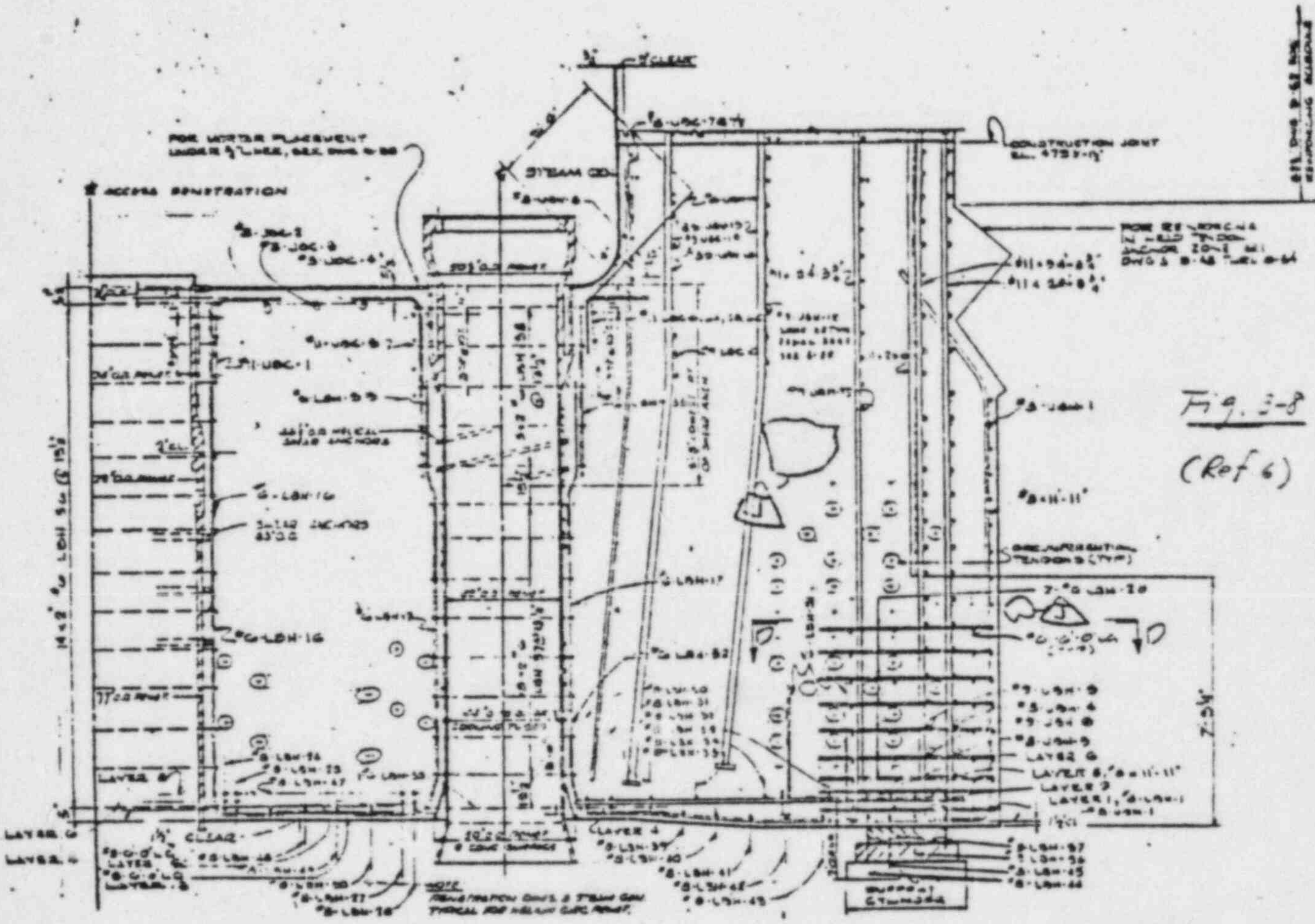

Fig. 3-8

(Ref 6)

For 17" long concrete ligament between O. and A (Fig. 3-7):

$$6 - \#8 \text{ bars} \rightarrow \frac{6 \times 0.79}{17} = 0.28 \text{ in}^2/\text{in}$$

For 14" long concrete ligament between A and D:

$$6 - \#8 \text{ bars} \longrightarrow 0.28 \text{ in}^2/\text{in}$$

For 142" length between D + E:

$$22 - \#8 \text{ bars} \rightarrow \frac{22 \times 0.79}{142} = 0.12 \text{ in}^2/\text{in}$$

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Rebars: LBH-30 (2), -31(2), -32(2)
(Fig. B-8) LBH-33 (1), -34(1), -35(1)
LBH-40 (1), -41(1), -42(1)
LBH-43 (1), -44(2), -45(2)
LBH-56 (2), -57 (2)

Each contains three sections, forming a complete circle.

Use $A_r = 0.12$ in^2/in in the following calculation

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Effect of Internal Pressure within Penetration:

For concrete compression zone:

$$p_c \times x \times \frac{\text{Sum of pen. I.D. over an yield line}}{\text{Total yield line length}}$$

$$= 0.722\ p_c\, x = 0.722 \times 845\ x = 610\ x$$

where p_c = cavity pressure.

x = height of compression zone

0.722 is from Ref. 1. p. E.11-10

For tension zone:

Stress in penetration is

$$\frac{p_c \cdot r}{h} = \frac{845 \times 21}{1} = 17{,}700\ \text{psi.}$$

$$< f_{sy}.$$

r = radius of penetration = 21 in.

h = thickness of penetration = 1 in. } (Ref. 1.

Hence this effect is applied to compression zone only in the calculation of moment capacit

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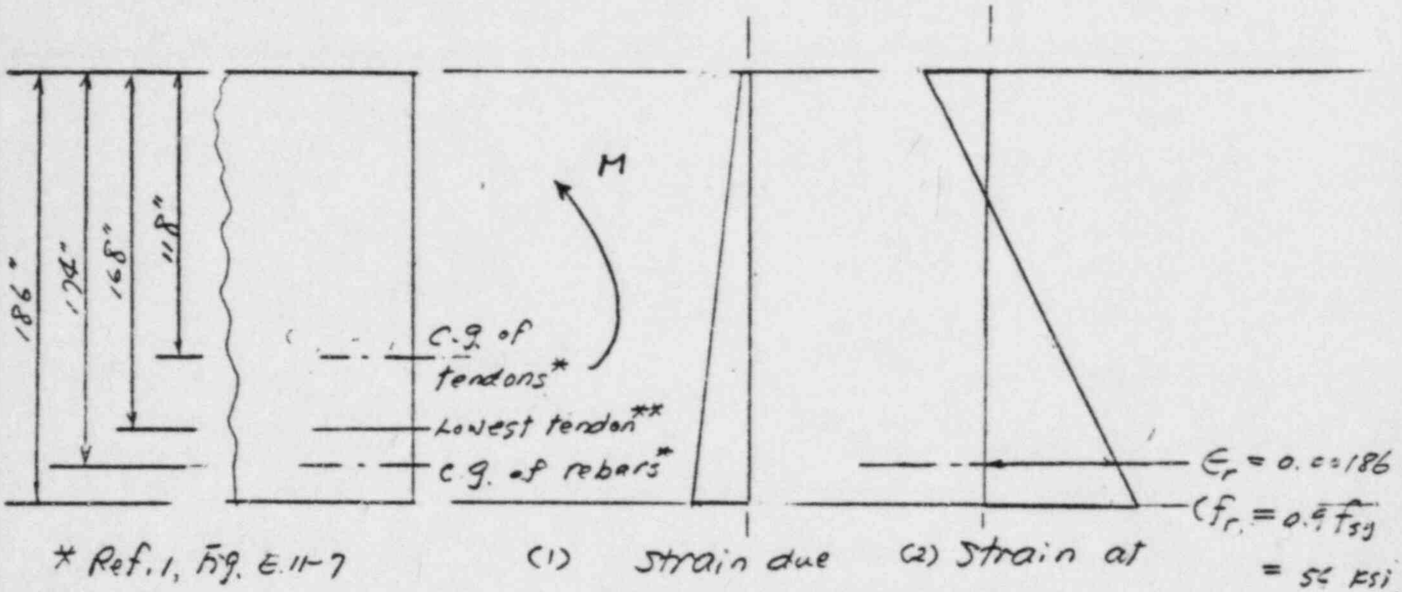
Moment Capacity Per Unit Width Along Yield Line:

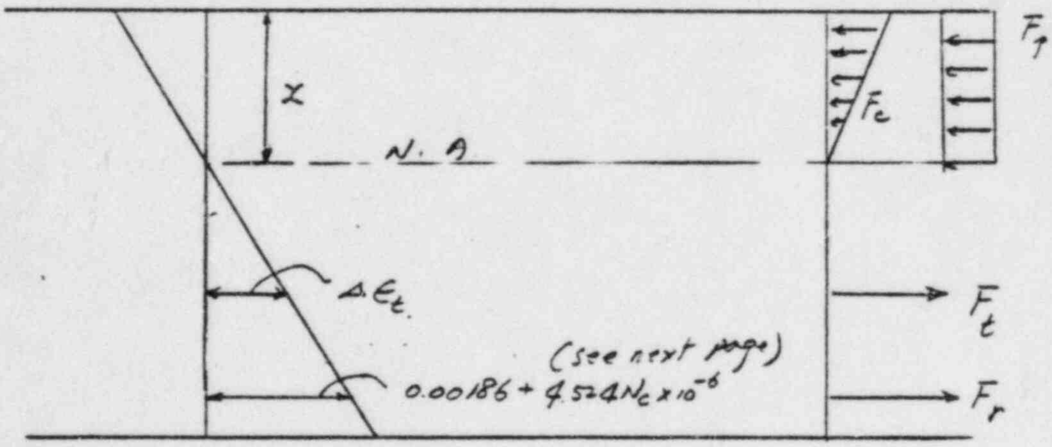


* Ref. 1, Fig. E.11-7

** Ref. 5.

(1) strain due to prestress

(2) Strain at limit



strain due to moment

Forces due to moment & prestress

Fig. B-9

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Due to prestress:

Tendon stress at end of life:

$$0.7 f_s'(1-0.12)(0.885) = 0.545 f_s' = 130.8 \text{ ksi}$$

Loss (Ref. 1, Table 5.4-4), Friction effect (Ref 3)

corresponding strain in tendon is.

0.00484 in./in. (Fig 4)

Prestress in concrete:

Total tendon force at end of life:

$$F_{t,e} = \frac{2}{3} N_c' \times 8.35 \times 130.8 / 294$$

$$= 2.477 N_c' \text{ kip/in. width}$$

Concrete stresses. N_c' = actual number of circ. head tendons.

Top (of bottom head): $\frac{2.477 N_c}{186} - \frac{(2.477 N_c')(118 - \frac{186}{2}}{\frac{(1) 186^2}{6}}$

$$= 0.0133 N_c' - 0.0107 N_c' = 0.0026 N_c' \text{ k/in}^2.$$

Bottom $= 0.0133 N_c' + 0.0107 N_c' = 0.024 N_c'$ k/in^2.

Concrete strains:

$E_c = 5.0 \times 10^6$ psi (Ref. 1, §5.4.1)

$\epsilon_{top} = 0.52 N_c' \times 10^{-6}$ in./in., $\epsilon_b = 4.8 N_c \times 10^{-6}$ in./i

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Concrete strains:

at rebar location:

$$\left[4.8 - \frac{4.8 - 0.52}{186} \cdot 12\right] N_c \times 10^{-6} = 4.524 N_c \times 10^{-6}$$

Due to Moment:

Limit: Max. rebar stress $= 0.9 f_{sy} = 54$ ksi.

Rebar strain at limit:

$$\frac{54}{29000} = 0.00186 \text{ in./in}$$

Rebar strain due to moment: $\epsilon_r = 0.00186 + 4.524 N_c \times 10$

With $N_c' = 34$ (Max.). additional ϵ_r over 0.0018

is 0.000154 or about 8 % of 0.00186.

For simplicity and conservatism, assume

$$\epsilon_r = 0.00186 \text{ in./in}$$

This gives:

f_r = rebar stress = 54 ksi

$F_r = 0.12 \times 54{,}000 = 6480$ lbs/in

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Tendons:

$$\Delta \epsilon_t = \frac{118 - x}{174 - x} \cdot 0.00186$$

$$f_t = 130.8 + \Delta \epsilon_t \cdot 27{,}000 = 130.8 + \frac{118-x}{174-x} \cdot 50.28$$

(ksi)

$$F_t = \frac{2}{3} N_c' \times 8.35 \times f_t / 294$$

$$= 18.9 \underset{\uparrow \text{(in ksi)}}{f_t} N_c' \quad \text{(lbs/in.)}$$

Concrete:

$$\text{Max. strain: } \frac{x}{174-x} \epsilon_r = \epsilon_c = \frac{x}{174-x} \, 0.00186$$

$$\text{Max. stress. } f_c = E_c \epsilon_c = 5 \times 10^6 \times \frac{x}{174-x} \times 0.00186$$

$$= \frac{9300 x}{174-x} \quad \text{(psi)}$$

Total force in concrete:

$$F_c = \frac{1}{2} x \cdot f_c = \frac{4650 x^2}{174-x} \quad \text{(lbs/in)}$$

Effect of pressure inside penetration.

$$F_p = 610 x \quad \text{(lbs/in.)}$$

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Equation of equilibrium:

$$F_c + F_p = F_t + F_r$$

$$\frac{4650 x^2}{174 - x} + 610 x = 18.9 N_c \left[130.8 + \frac{118 - x}{174 - x} \cdot 50.28 \right]$$

$$+ 6480$$

$$4650 x^2 + 610 x (174 - x) = 18.9 N_c' \cdot 130.8 (174 - x)$$

$$+ 18.9 N_c' (118 - x) 50.28$$

$$+ 6480 (174 - x)$$

$$(4650 - 610) x^2 + (610 \cdot 174 + 18.9 \cdot 130.8 N_c'$$

$$+ 18.9 \cdot 50.28 N_c' + 6480) x$$

$$- (18.9 N_c' \times 130.8 \times 174 + 18.9 N_c' \cdot 118 \times 50.28$$

$$+ 6480 \times 174) = 0$$

$$\underbrace{4040}_{a} x^2 + \underbrace{(112{,}620 + 3422 N_c')}_{b}$$

$$- \underbrace{(1127{,}520 + 542{,}283 N_c')}_{c} = 0$$

$$x = \frac{1}{2a} \left(-b + \sqrt{b^2 + 4ac} \right)$$

Capacity: $M = \frac{2}{3} x F_c + 305 x^2 + F_t (118 - x) + F_r (174 - x)$

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Table B-19 Moment Capacity

N'_c	34	32	30	28	
a					
b					
c					
N. A. x (in.)	46.80	45.52	44.18	42.78	
$f_{c, max}$ (psi)	3422	3295	3165	3031	
F_c (lbs/in.)	80070	74988	69914	64854	
f_t (ksi)	158.94	159.16	159.39	159.62	
F_t (lbs/in.)	102138	96263	90375	84472	
F_r (lbs/in.)	6480	6480	6480	6480	
M (10^6 lbs-in. per in.)	11.27	10.72	10.17	9.61	

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Table B-19 Moment Capacity (Continued)

N_c'	24	22	20	16	
a					
b					
c					
N. A. x (in.)	39.78	38.17	36 46	32.73	
$f_{c,max}$ (psi)	2756	2613	2465	2155	
F_c (lbs/in.)	54823	49877	44942	35261	
f_t (ksi)	160.10	160.35	160.61	161.15	
F_t (lbs/in)	72622	66674	60710	48731	
F_r (lbs/in.)	6480	6480	6480	6480	
M (10^6 lbs-in per in)	8.49	7.92	7.34	6.17	

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Table B-19 Moment Capacity (Continued)

N_c'	14	12	10	8	6
a					
b					
c					
N. A. x (in.)	30.66	28.43	26.00	23.33	20.35
$f_{c,max}$ (psi)	1989	1816	1634	1440	1232
F_c (lbs/in.)	30495	25819	21239	16798	12533
f_t (ksi)	161.44	161.74	162.06	162.39	162.75
F_t (lbs/in.)	42716	36682	30628	24554	18456
F_r (lbs/in.)	6480	6480	6480	6480	6480
M (10^6 lbs-in. per in.)	5.57	4.96	4.35	3.73	3.09

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<u>Effect of pressure inside penetration.</u> - a check.

If $F_p = 610\,x$ is neglected, the equation of equilibrium becomes.

$$F_c = F_l + F_r.$$

$$4650\,x^2 + (6480 + 3422\,N_c') - (1127,520 + 542,283\,N_c')$$

$$= 0$$

$$x = \frac{1}{2a}\left(-b + \sqrt{b^2 + 4ac}\right)$$

$$M = \frac{2}{3}\,x\,F_c + F_t\,(118 - x) + F_r\,(174 - x)$$

Check $N_c' = 34.$ $\rightarrow$ $x = 52.99$

$$M = 11.19 \times 10^6$$

If F_p is included. $x = 46.80$, $M = 11.27 \times 10^6$

$$\frac{11.27 - 11.19}{11.19} = 0.01 \quad \text{off.}$$

Thus F_p effect is small.

Also, if $N_c' = 0.$ $\rightarrow$ $M = 1.095 \times 10^6$ vs 1.10×10^6

(No F_p) (with F_p)

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B. 1.8 Required Number of Head Tendons

Table B-20

N_x	1.0 RP			1.5 RP		
	Unit Yield Line Moment Req'd	N_c' Req'd $^{(1)}$	N_c Req'd $^{(1)}$	Unit Yield Line Moment Required	N_c' Req'd $^{(1)}$	N_c Req'd $^{(1)}$
	lb-in/in.			lb-in/in.		
0	7.50×10^6	20.55	13.70	16.34×10^6	(2)	—
6	6.15×10^6	15.93	10.62	14.72×10^6	(2)	—
12	4.98×10^6	12.07	8.05	13.38×10^6	(2)	—
18	4.04×10^6	9.00	6.00	11.63×10^6	$34^{(3)}$	$22.67^{(3)}$
24	3.08×10^6	6.00	4.00	10.11×10^6	29.79	19.86
Source	Tables B-15, B-16	Table 19 by interpolation	$N_c' \times \frac{2}{3}$	Tables B-17, B-18	Table 19 by interpolation	$N_c' \times \frac{2}{3}$

(1) Required to provide (See Fig. 1) (See Fig. 2)
a moment capacity equal to the unit yield line moment.

(2) Not possible.

(3) Corresponds to $N_x = 19.42$. Max N_c' is 34. $N_x = 18$ req'd $N_c' > 34$.

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The calculated effective number of circumferential head tendons and the corresponding number of crosshead tendons required are shown graphically in Figs. 1 and 2, main text.

As a further conservatism, the curve for 1.5 RP (Fig. 2) is truncated at $N_c = 22$ since in each head, the minimum N_c passing any section is 22, out of 34 actual number of tendons currently provided.

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B.2 Potential Failure Mode with Yield Line at Edge of Core Cavity

N_c, N_x requirements were determined from consideration of yield patterns O-A-B-H-C and O-A-D-G-E. (See Fig. B-3).

Potential yield line failure along the yield pattern O-A-B-H-I and O-A-D-G-I are checked to assure that any of these will not be critical. (See Fig. B-3)

Four N_x, N_c' combinations representing the extreme conditions in Figs. 1 and 2 are checked. They are

$$\left.\begin{array}{l} N_x = 0, \quad N_c' = 20 \quad (N_c = 13) \\ N_x = 24, \quad N_c' = 6 \quad (N_c = 4) \end{array}\right\} \text{Fig. 1}$$

$N_x = 21, \quad N_c' = 33 \quad (N_c = 22)$

$N_x = 24, \quad N_c' = 30 \quad (N_c = 20)$

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Bottom Rebars for Bottom Head (Ref. 7)

Layer #1 : All #8. A432

LBH-1, 2, ..., 9 = 102

LBH-10, 11, 12, 13, 14 = 72

} 184 × (0.79) = 145.36 in².

Layers #3, 5, 7: All #8. A432

LBH-18, 64

LBH-21 32

11'-11" Lg. 24

11'-2" " 36

12'-7" " 24

7'-6" " 12

} 192 × (0.79) = 151.68 in²

Total in radial dir. = 297.04 in².

Or. $\frac{297.04}{2\pi \cdot 186}$ = 0.254 in²/in liner

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Top Rebars for Bottom Head

Diagonal Bars (Refs. 8. & 10) A431. #11

$12 \times 16 \times (1.56) \times \cos 19°50' \times \cos 45°$

#11 (plan) (Elevation)

$= 192 \times 1.56 \times \cos 19°50' \times \cos 45°$

$= 299.52 \text{ in}^2 \times \cos 19°50' \times \cos 45°$

(19.83°)

$= 199.23 \text{ in}^2$ total in radial direction

Radial Bars (Refs. 8 and 9) A431. #14s

(Area)

UBH-5. 5A, 6. 6A: $8 \times 2.25 \times \cos 27.5° \times \cos 9° =$ 15.77

UBH-7, 7A, 8. 8A: $8 \times 2.25 \times \cos 27.5° \times \cos 18° =$ 15.18

UBH-9. 9A, 10. 10A $8 \times 2.25 \times \cos 27.5° \times \cos 25° =$ 14.47

UBH-11. 11A. 12. 12A: $8 \times 2.25 \times \cos 27.5 \times \cos 33° =$ 13.39

UBH-17. 17A. 18. 18A. $16 \times 2.25 \times \cos 28° \times \cos 9° =$ 31.39

UBH-19. 19A. 20. 20A. $16 \times 2.25 \times \cos 28° \times \cos 18° =$ 30.23

UBH-21, 21A. 22, 22A. $16 \times 2.25 \times \cos 28° \times \cos 25° =$ 28.81

UBH-23, 23A. 24, 24A, $16 \times 2.25 \times \cos 28° \times \cos 33° =$ 26.66

UBH-33. 33A. 34. 34A: $96 \times 2.25 \times \cos 27.5° \times \cos 27°15' =$ 170.33

Total in radial dir. = 346.23

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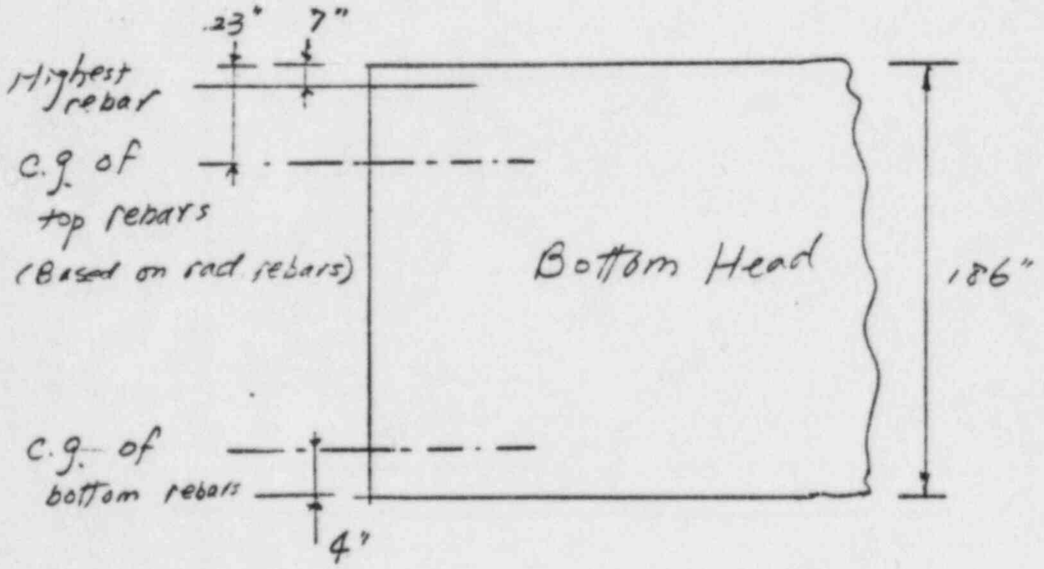
Total in radial dir. = 199.23 + 346.23 = 545.46 in²

$$\text{Or} \quad \frac{545.46}{2\pi \cdot 186} = 0.467 \text{ in}^2/\text{in.}$$

Location of Rebars

(Refs. 8 and 11)



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Radial Prestress due to Crosshead Tendons

Referring to Ref. 10,

Of 24 tendons, 12 are essentially radial at the edge of core cavity, and 12 are inclined about 30° w.r.t. the radial direction. Vertically, the largest inclination is 43°. Hence the prestress force is:

$$2 \cdot \frac{N_x}{2} (1 + \cos 30°) \cos 43° \times 8.35 \times f_x$$

(stress in X-head tendons)

$$= 2 \times 5.698\ f_x N_x \quad \text{kips} \qquad (f_x \text{ in ksi})$$

or

$$\frac{2 \times 5.698\ f_x N_x}{2 \pi \cdot 186} = 9.75 f_x N_x \quad \text{lbs/in.}$$

$(f_x$ in ksi.)

From Ref. 11, the location of crosshead tendons at the edge of core cavity is, about 8 ft from the top of bottom head

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Moment Capacity Along a Radial Yield Lines

Same as before (See § B.1.7) except

use $F_t = 18.9\ (130.8)\ N_c = 2472.\ N_c{}^*$

$F_c + F_p = F_t + F_r$

$$\frac{4650\ x^2}{174 - x} + 610\ x = 2472\ N_c + 6480$$

$$4650\ x^2 + 610x\ (174 - x) = (24,- N_c + 6480)(178 - x)$$

$$4040 x^2 + (610 \times 174 + 2472 N_c + 6480) x$$

$$- 174 (2472 N_c + 6480) = 0$$

$$\underbrace{4040}_{a} x^2 + \underbrace{(1,126\,20 + 2472\,N_c)}_{b} x - \underbrace{(1127,520 + 430,128)}_{c} = 0$$

$$x = \frac{1}{2a}\left(-b + \sqrt{b^2 + 4ac}\right)$$

$$M = \frac{2}{3} x F_c + 305 x^2 + F_t (118 - x) + F_r (174 - x)$$

* The increase in prestress due to further extension of tendons is ignored.

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Table B-21 Moment Capacity
Along a Radial Yield Line

N'_c	20	6	33	30	
a	4040	4040	4040	4040	
b	162060	127182	194196	186,780	
c	9.730,080	3,708,288	15,321.744	14.031,360	
N.A. x (in.)	3296	18.40	42.07	40.19	
$f_{c,max}$ (psi)	2173	1100	2966	2793	
F_c (lbs/in.)	35,817	10118	62386	56128	
f_t (ksi)	130.8	130.8	130.8	130.8	
F_t (lbs/in)	49440	14832	81576	74160	
F_r (lbs/in.)	6480	6480	6480	6480	
M (10^6 lbs-in per in)	6.24	2.71	9.34	8.63	

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Moment Capacity Along a Hoop Yield Line

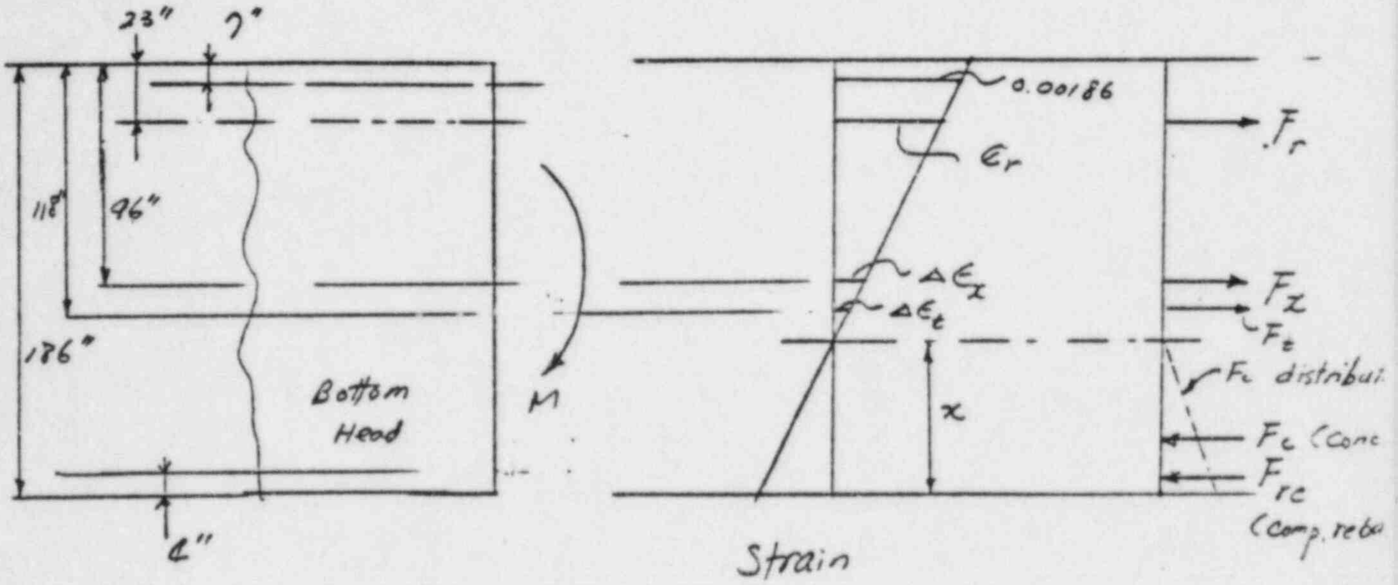

Fig. B-10

$$\text{Max. rebar strain} = \frac{54}{29000} = 0.00186$$

$$\text{Ave. rebar strain} = \frac{186-23-x}{186-7-x} \cdot 0.00186 = \frac{163-x}{179-x} \cdot 0.001$$

Total rebar force (tensile):

$$F_r = \underset{(\text{Area/in})}{0.467} \times \frac{163-x}{179-x} (0.00186)(29{,}000{,}000)$$

$$= 25190 \times \frac{163-x}{179-x} \text{ lbs/in.}$$

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Total rebar force (compressive):

Strain: $0.00186 \dfrac{x-4}{179-x}$

$F_{rc} = 0.00186 \dfrac{x-4}{179-x} \cdot 29{,}000{,}000 \times 0.254$ (Area/in)

$= 13700 \dfrac{x-4}{179-x}$ lbs/in.

Crosshead tendons:

$\Delta \epsilon_x = \dfrac{186-96-x}{179-x} \cdot 0.00186 = 0.00186 \dfrac{90-x}{179-x}$

Stress at end of life:

$0.7 f_s' \underset{\text{loss}}{(1-0.12)} \underset{\text{friction}}{(0.9)} = 0.554 f_s' = 133$ ksi

f_x = Stress in X-head tendons

$= 133 + \Delta \epsilon_x \cdot 27{,}000 = 133 + \dfrac{90-x}{179-x} \times 50.28$ (ksi)

$F_x = 9.75 \underset{\text{(in ksi)}}{f_x} \cdot N_x$ (lbs/in.)

Concrete:

Max. strain - $\epsilon_c = 0.00186 \dfrac{x}{179-x}$

$f_{c(max)} = E_c f_c = 5 \times 10^6 \times 0.00186 \dfrac{x}{179-x} = \dfrac{9300x}{179-x}$ (psi)

$F_c = \frac{1}{2} x f_c = \dfrac{4650x^2}{179-x}$

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(ksi)

$$F_t = \frac{2}{3} N_c' (8.35) \cdot f_t / 22.17 \times 12 = 20.9 \, f_t \, N_c' \text{ (lb/in.}$$

↳ Ave. radius of cir. tendons

Use $f_t = 130.8$ ksi. (Inc. in f_t due to $\Delta\epsilon_t$ ignored,

$$\therefore F_t = 20.9 \times 130.8 \, N_c = 2734 \, N_c$$

Equilibrium:

$$F_r + F_x + F_t = F_c + F_{rc}$$

$$25190 \, \frac{163-x}{179-x} + 9.75 \left(133 + \frac{90-x}{179-x} \, 50.28\right) N_x$$

$$+ 2734 \, N_c = \frac{4650 x^2}{179-x} + 13700 \, \frac{x-4}{179-x}$$

$$25190 (163-x) + 9.75 (133)(179-x) N_x + 9.75 (90-x) 50.28 N$$

$$+ 2734 \, N_c (179-x) = 4650 x^2 + 13700 (x-4)$$

$$4650 x^2 + (13700 + 25190 + 9.75 \times 133 N_x + 9.75 \times 50.28 N_x + 2734 N_c)x$$

$$-(13700 \times 4 + 25190 \times 163 + 9.75 \times 133 \times 179 N_x + 9.75 \times 90 \times 50.28 N_x$$

$$+ 2734 \times 179 N_c) = 0$$

$$\underbrace{4650}_{a} x^2 + (\underbrace{38890 + 1787 N_x + 2734 N_c}_{b}) x$$

$$-(\underbrace{4,160,770 + 276,239 N_x + 489,386 N_c}_{c}) = 0$$

$$x = \frac{1}{2a} \left(-b + \sqrt{b^2 + 4ac}\right)$$

$$M = F_r (163-x) + F_x (90-x) + F_t (68-x) + F_c \left(\tfrac{1}{3} x\right) + F_{rc} (x-$$

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Table B-22 Moment Capacity Along a Hoop Yield Line

(At R = 186 in)

N_x	0	24	21	24	
N_c'	20	6	33	30	
x (in.)	45.62	4479	59.13	58.47	
F_r (lbs/in.)	22168	22187	21878	21846	
f_x (ksi)	—	149.94	145.95	146.15	
F_x (lbs/in.)	0	35085	29883	34200	
$\hat{f}_t$ (ksi)	130.8	130.8	130.8	130.8	
F_t (lbs/in.)	54680	16404	90222	82020	
f_c (psi)	3181	3104	4588	4512	
F_c (lbs/in)	72556	69507	135631	131894	
F_{rc} (lbs/in.)	4275	4164	6301	6191	
M (lbs-in/in)	6.21	6.83	9.68	9.62	

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Yield Line Pattern OADGI

Referring to Fig. B-6. the forces acting on the slab section bounded by OADGIG'D'A consist of the effective pressure on the surface. vertical force (F_{L-L}) along GIG'. shearing forces along OADG and OA'D'G'. moment along the two radial yield lines (OADG and OA'D'G'), and moment along the hoop yield line GIG'. For simplicity ignore the shear along OADG. & OA'D'G'.

From equilibrium. the moment of the effective pressure and F_{L-L} about any horizonta axis must be balanced by the moments along all yield lines. For convenience, use E-E' as the axis to compute the moment due to pressure and F_{L-L}.

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Check 1 RP. $N_x = 0$ $N_c' = 20$ Case

From Table B-16,

(1) $M_{E-E'}$ due to pressure $= 1.862 \times 10^9$ lb-in

(2) " " " F_{L-L} $= -0.946 \times 10^9$ lb-in

Sum $= 0.916 \times 10^9$

$= 916 \times 10^6$ lb-in

Resisting Moment

(3) Along OADG + OA'D'G' $= 2 \times \underbrace{6.24 \times 10^6}_{}\times \underbrace{(9+1+9)}_{}$

(Table B-21)

(See Fig. B-7)

$= 237 \times 10^6$

(4) Along GIG' $= 2 \times \underbrace{6.21 \times 10^6}_{} \times \underline{186 \times \sin 18}$

(Table B-22)

(See Fig. B-6)

$= 714 \times 10^6$

Sum $= 951 \times 10^6$ lb-in

$> 916 \times 10^6$ lb-in.

∴ This yield line failure mode does not control

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Check 1 RP, $N_x = 24$, $N_c' = 6$ Case

$M_{E-E'}$ due to pressure $= 1.214 \times 10^9$

" " . F_{L-L} $= -0.617 \times 10^9$

Sum $= 0.597 \times 10^9$

$= 597 \times 10^6$ lb-in.

Resisting Moment.

Along OADG & OA'D'G' $= 2 \times 2.71 \times 10^6 \times 19 = 103 \times 10^6$

Along GIG' $= 2 \times 6.83 \times 10^6 \times 186 \times \sin 18°$

$= 785 \times 10^6$

Sum $= 888 \times 10^6$ lb-in

$> 597 \times 10^6$ lb-in

O.K.

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Check 1.5 RP, $N_x = 21$. $N_c' = 33$ Case

From Table B-18.

$M_{E-E'}$ due to pressure $= 2.224 \times 10^9$ (by interpolation

" " " F_{L-L} $= -1.130 \times 10^9$ (by interpolatio

Sum $= 1.094 \times 10^9 = 1094 \times 10^6$ lb-in

Resisting moment

Along OADG, OA'D'G' $= 2 \times 9.34 \times 10^6 \times 19 = 355 \times 10^6$

Along GIG' $= 2 \times 9.68 \times 10^6 \times 186 \times \sin 18° = 1113 \times 10^6$

Sum $= 1468 \times 10^6 > 1094 \times 10^6$ lb-in

O.K

Check 1.5 RP, $N_x = 24$. $N_c' = 30$ Case

$M_{E-E'}$ due to pressure $= 2.142 \times 10^9$

" " " F_{L-L} $= -1.088 \times 10^9$

Sum $= 1.054 \times 10^9 = 1054 \times 10^6$ lb-in

Resisting moment

along OADG, OA'D'G' $= 2 \times 8.63 \times 10^6 \times 19 = 328 \times 10^6$

along GIG' $= 2 \times 9.62 \times 10^6 \times 186 \times \sin 18°$

$= 1106 \times 10^6$

Sum $= 1434 \times 10^6 > 1054 \times 10^6$ lb-in

O.K

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Yield Line Pattern OABHI

Refer to Fig. B-6

Check 1.0 RP, $N_x = 0$, $N_c' = 20$ case

From Table B-15.

$M_{c-c'}$ due to pressure = 2.599 × 10⁹

" " " F_{l-l} = −1.648 × 10⁹

Sum = 0.951 × 10⁹

= 951 × 10⁶ lb-in

Resisting moment

along OABH, OA'B'H' = 2 × 6.24 × 10⁶ × (9+13+9) = 387 × 10

along HIH' = 2 × 6.21 × 10⁶ × 186 × sin 41.5° = 1531 × 10⁶

Sum = 1918 × 10⁶ lb-in > 951 × 10⁶

O.K

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Check 1.0 RP, $N_x = 24$, $N_c' = 6$ Case

$M_{c-c'}$ due to pressure = 2.374×10^9

" " " $F_{\lambda-\lambda}$ = -1.505×10^9

Sum = $0.869 \times 10^9 = 869 \times 10^6$ lb-in

Resisting moment

along OABH, OA'B'H' = $2 \times 2.71 \times 10^6 \times (31) = 168 \times 10^6$

along HIH' = $2 \times 6.83 \times 10^6 \times 186 \times \sin 41.5° = 1684 \times 10^6$

Sum = $1852 \times 10^6 > 869 \times 10^6$ lb-in

o.k.

Check 1.5 RP. $N_x = 21$, $N_c' = 33$ Case

$M_{c-c'}$ due to pressure = 3.104×10^9

" " " $F_{\lambda-\lambda}$ = -1.968×10^9

Sum = $1.136 \times 10^9 = 1136 \times 10^6$ lb-

Resisting moment

along OABH, OA'B'H' = $2 \times 9.34 \times 10^6 \times 31 = 579 \times 10^6$ lb-

along HIH' = $2 \times 9.68 \times 10^6 \times 186 \times \sin 41.5° = 2386 \times 10^6$

Sum = $2965 \times 10^6 > 1136 \times 10^6$ lb-in

ok

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Check 1.5 RP. $N_x = 24$. $N_c' = 30$ Case

$M_{c-c'}$ due to pressure $= 2.990 \times 10^9$

" " " $F_{L-L} = -1.895 \times 10^9$

Sum $= 1.095 \times 10^9 = 1095 \times 10^6$ lb-in

Resisting moment:

along OABH, OA'B'H' $= 2 \times 863 \times 10^6 \times 31 = 535 \times 10^6$

along HIH' $= 2 \times 9.62 \times 10^6 \times 186 \times \sin 41.5° = 2371 \times 10^6$

Sum $= 2906 \times 10^6 > 1095 \times 10^6$ lb-in

O.K

Conclusion (§B.1.9)

Yield line failure along OABHI or OADGI will not develop, if N_x, N_c shown in Table B-20 (or Figs 1 and 2) are used. Hence Table B-20, or Figs 1 and 2 remain valid

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B. 3 <u>Punching Shear Mode of Failure</u>

(Bottom Head)

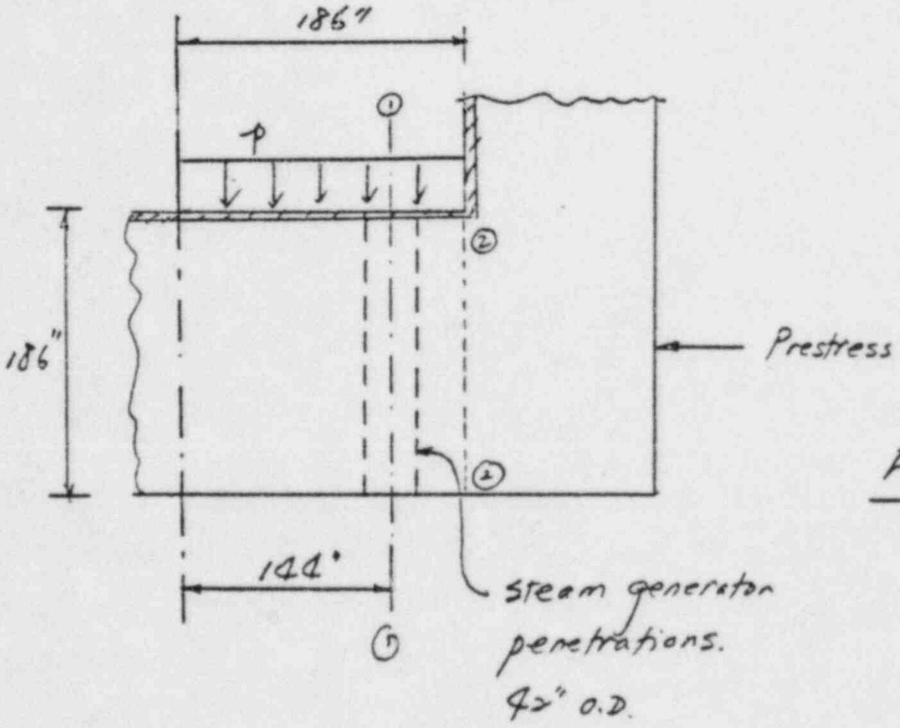

Fig. B-11

Section susceptible to punching shear failure is the one connecting the S.G. penetrations, Fig. B-11, ①-①. See Fig. B-3 also.

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The punching shearing stress is

$$\frac{p\cdot\pi\,(144)^2}{[2\pi(144)-12(42)]\cdot 186} = 0.874\,p. = f_v$$

Ref. 4 gives:

$$p = \frac{d}{D}\left(0.842 f_c' + 0.1676 f_t\right)$$

where p = ultimate pressure (psi)

d = slab thickness = 186 in.

D = pressurized diameter in inches = 288 in

f_c' = concrete strength = 6000 psi

f_l = prestress (psi)

From this. the shear allowable is:

$$\frac{p\,\frac{\cancel{\pi}\cdot D^{\cancel{2}}}{4}}{\cancel{\pi}\cancel{D}\cdot d} = \frac{pD}{4d} = f_{v,\text{ allowable}}$$

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$$f_{v.\,allowable} = \frac{1}{4}(0.842 f_c' + 0.1676 f_t)$$

$$= 0.210 f_c' + 0.0419 f_t$$

$$= 1260 + 0.0419 f_t$$

For $p = 845$ psig (1.0 RP). $f_v = 738$ psi

$p = 1268$ psig, (1.5 RP) $f_v = 1108$ psi

Both are smaller than 1260 psi, the allowable ignoring the constraining effect of the radial prestress.

Check Section ②-② (Fig. B-11):

Allowable $p = \frac{186}{2\times186}(0.842 f_c' + 0.1676 f_t)$

$= 2526 + 0.0838 f_t$

Actual $p = 845$ and 1268 psig. < 2526 psi

Hence punching shear is not a problem

APPENDIX C

CALCULATION REVIEW REPORT

GA1543(REV. 11/80)

907738 N/C P.c2

CALCULATION REVIEW REPORT

TITLE: FSV - Tendon Requirements Based on Safety Consideration

APPROVAL LEVEL 3

QAL LEVEL I

DISCIPLINE	SYSTEM	DOC. TYPE	PROJECT	DOCUMENT NO.	ISSUE NO./LTR.
S	11	CFL	1900	907738	N/C

INDEPENDENT REVIEWER:

NAME Arnold A. Schwartz

ORGANIZATION Structural Design and Analysis.

REVIEWER SELECTION APPROVAL: BR MGR DATE 10.29.84.

REVIEW METHOD:

	YES	NO	ERROR DETECTED
ARITHMETIC CHECK		✓	
LOGIC CHECK	✓		No
ALTERNATE METHOD USED		✓	
SPOT CHECK PERFORMED	✓		No
COMPUTER PROGRAM USED		✓	

REMARKS: (ATTACH LIST OF DOCUMENTS USED IN REVIEW)

Calculation 907738

FSV - Structural Dwgs for PCRV

CALCULATIONS FOUND TO BE VALID AND CONCLUSIONS TO BE CORRECT:

INDEPENDENT REVIEWER Arnold A Schwartz DATE 11/30/84

SIGNATURE