NRC Questions from 10/22 (assigned to G. Pugh):

Question #36 (Request Number 42)

<u>NRC Question:</u>

Was the original radial tension stresses due to the hoop stresses considered in the original design?

<u>CR3 Response:</u>

Cannot readily determine from the old Gilbert Calculations what the direct answer is to the request. It appears that the tendon design is based on limiting the concrete tensile stress to 212 psi. This limit bounds the tensile stresses in meridional, and hoop directions. See Book 2, Section 1.01.7, pages 1.01.7/6 and 1.01.7/7 for a brief memorandum outlining the critical loading of the cylindrical RB wall. The tendon pre-stress is designed to limit the tensile stresses in the concrete for the load combinations. It does not appear that the calculations considered the tensile stresses in the concrete outside the tendon's influence.

Copies of calculation pages are included at following drive location:

L:\Shared\2009 NRC SPECIAL INSPECTION TEAM Q-A\WILLIAMS Q-A\Request 42, Q36 Response Info- Pugh

0/93

	MADE	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.			
	CHK'D. F KLAT				
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	CF. DFN.				
	ENG.	WORK ORDER	SIZE	DRAWING	REV.
	REV. CH. APP. DATE				

INDEX

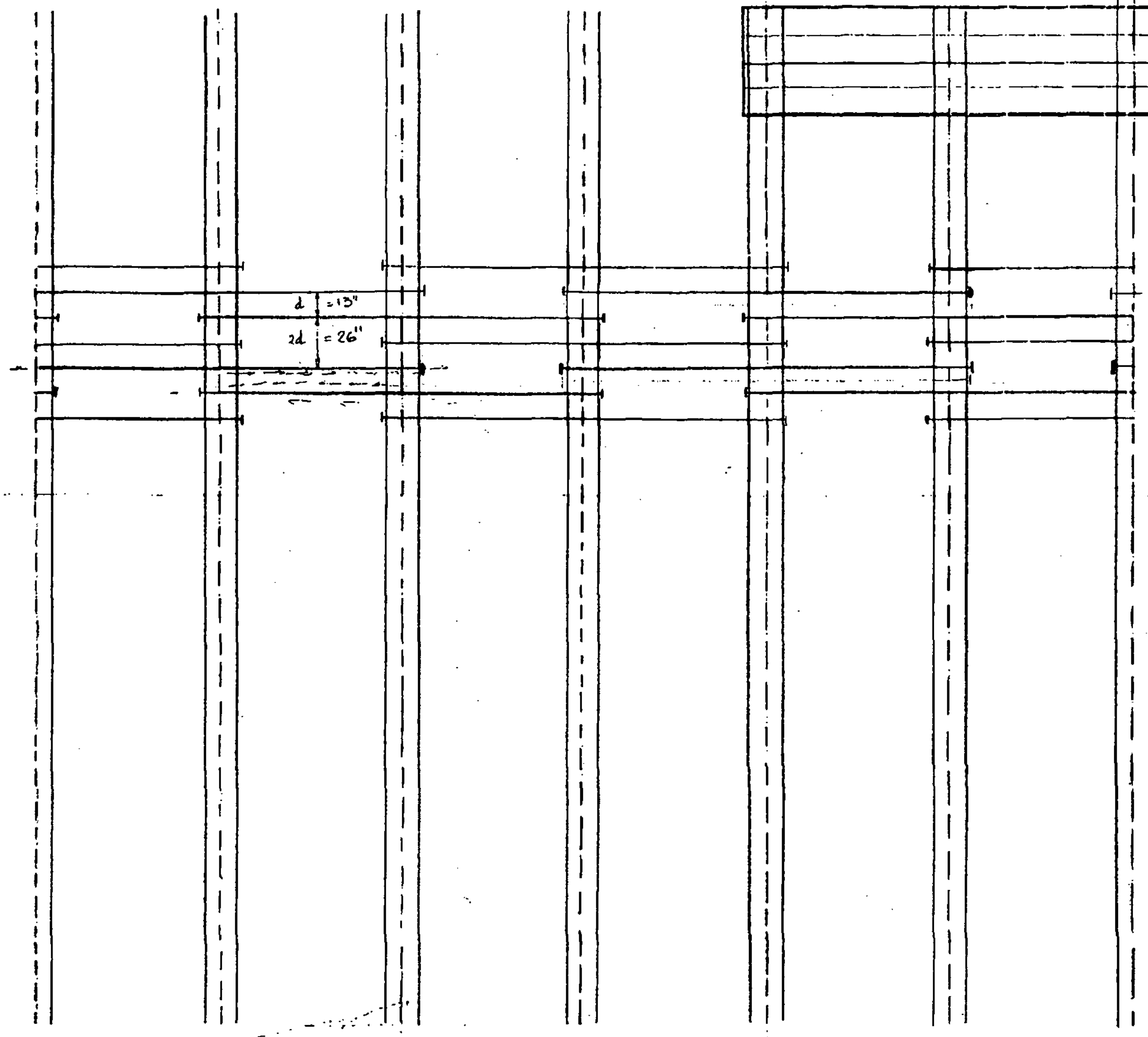
FLORIDA POWER CORPORATION

CRYSTAL RIVER UNIT #3

1.01.7 REACTOR BUILDING DESIGN - PRESTRESSING

1.01.7 | 1

READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS				NEW YORK WASHINGTON
MADE	CHKD	SQ CF	CF ON	APPD	SIZE DRAWING NO. REV. / SHEET
RS	TY.				W.O. 4200-00 DATE 4-24-69
REV.					



CRYSTAL RIVER

TYPICAL HOOP TENDONS

IN THE WALL

FLORIDA POWER CORPORATION	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. F.K.	
CRYSTAL RIVER UNIT #3	BD. CF.	
REACTOR BUILDING	CP. DFN.	4200-00
	ENG.	WORK ORDER / SIZE / DRAWING / REV.
SPACING OF VERTICAL TENDON	DATE 10-15-69	

TOTAL NO OF TENDONS = 144

RADIUS OF ℄ OF TENDONS = 807.375"

SPACING OF TENDON = $\frac{2 \times \pi \times 807.375}{144}$

$= \frac{\pi \times 807.375}{72}$

$= 35.23"$ $= 35\frac{1}{4}"$ SAY

R. Shan

1.01.7|3

MEMORANDUM

GILBERT ASSOCIATES, INC.

March 13, 1969

VOID
See letter of April 10, 69 for revised prestress requirements

TO: DISTRIBUTION LISTED BELOW

FROM: R. Shanmugasundaram

SUBJECT: Crystal River
Reactor Building - Prestress Requirements
GAI-4203

At present prestress for Reactor Building for Met-Ed and Crystal River are designed to take care of the following effects:

1. 1.5 accident pressure.
2. Effects of accident temperature above operating temperature.

During accident condition the liner is heated to in the order of 280° F. whereas concrete is heated to only 110° F. Because of this difference in temperature, the steel liner is trying to expand with reference to concrete. As this movement is restrained the steel liner exerts a pressure on the concrete. Necessary prestress is added to take care of the above effect.

During operating temperature the liner is gradually heated along with the concrete. In order to analyse for this loading condition either one of the following assumptions has to be made.

1. Concrete and liner act as a composite material.
2. Concrete and liner act as two independent materials.

If we consider the first assumption there is only bending stress. Therefore, no prestressing is required.

If we consider the second assumption there are membrane forces (tension) in addition to bending stress. Hence we need additional prestressing.

Additional prestressing required if we consider second assumption.

<u>Vertical Prestressing</u>

Prestressing required without the above additional requirements - 181200 kips

Stress in steel due to operating temperature

$$= \alpha \Delta T E$$
$$= 6.5 \times 10^{-6} \times (110-70) \times 30 \times 10^{6}$$
$$= 6.5 \times 40 \times 30 = 7800 \text{ psi}$$

$N\phi = 7.8 \times \frac{3}{8} \times 12 \times \quad \times 130 \qquad = 14300$ kips

% increase $= \frac{14300}{181200} = 8\%$

<u>Hoop Prestressing</u>

Prestressing required without the above additional requirements = 756^k/ft ht

$N\theta = 7.8 \times \frac{3}{8} \times 12 = 35^k/ft$

% increase $= \frac{35}{756} = 4.6\%$

Therefore a decision is to be made about the assumption to be followed in case of operating temperature.

R. Shanmugasundaram.

R. Shanmugasundaram

RS:flg

cc: H. P. Lorenz
S. N. Dobreff
D. A. Godfrey
D. K. Croneberger
K. E. Nodland
W. J. Leininger
D. A. Skilton

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D. F.K.	
REVISED PRESTRESS REQUIREMENTS	CP. DFN. / ENG.	4203-00 / D / 1/5 (WORK ORDER / SIZE / DRAWING / REV.)
	REV. CH. APP. DATE	4-2-69

PRESTREESS DESIGN REQUIREMENTS AS GIVEN IN THE SPECIFICATIONS

(1) VERTICAL FORCE OF 457 KIPS PER FOOT OF CIRCUMFERENCE AT A RADIUS OF 67'-3"

(2) HOOP FORCE OF 829 KIPS PER FOOT OF VERTICAL HEIGHT AT A RADIUS OF 67'-7". THE HOOP FORCE FOR THE LOWER TEN FEET OF WALL SHALL BE 415 KIPS PER FOOT

(3) DOME FORCE OF 820 KIPS PER FOOT OF CIRCUMFERENCE AT A RADIUS OF 60'-0"

R. Shan

1.01.7/6

MEMORANDUM

GILBERT ASSOCIATES, INC.

April 10, 1969

TO: DISTRIBUTION LISTED BELOW

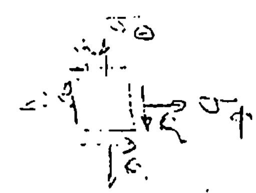
FROM: R. Shanmugasundaram and W. J. Leininger

SUBJECT: Crystal River and Metropolitan Edison Company Reactor Building - Revised Final Prestress Requirements GAI-4203 and GAI-4192

Cylindrical Wall - (R. Shanmugasundaram)

The critical loading condition for prestressing in the cylindrical wall is 0.95 dead load + prestrussing force + 1.5 accident pressure + 1.5 accident temperature. In calculation, the following are considered:

(1) As per PSAR a principal tensile stress of $3\sqrt{f'c}$ = $3\sqrt{5000}$ = 212 psi is allowed. Without exceeding the above value, the following stresses can be allowed.



Tensile stress in meridional direction

$$= \sigma_\phi = 200 \text{ psi}$$

Tensile stress in hoop direction

$$= \sigma_\theta = 212 \text{ psi}$$ 211

Shear stress

$$= Q = 24 \text{ psi}$$ 27.4

(2) Credit is given for 0.95 dead load of the dome only.

(3) Necessary prestressing force is added to limit the tensile stress caused by 1.5 accident pressure.

(4) Creep and shrinkage effects in the concrete due to the prestressing force are taken care of by analysing the vessel with the lower 'E' value of $2.5x10^6$.

(5) During the accident condition the liner is quickly heated to in the order of 280° F. whereas concrete remains at 110° F. Because of this difference in temperature the steel liner is trying to expand with reference to the concrete. As this

1.01.7/7

movement is restrained the steel liner exerts a pressure which causes tension on the concrete. Necessary prestressing force is added to limit the tensile stress in concrete.

(6) With the 1.5 accident pressure and the accident temperature, the vessel expands which stretches the prestressing tendons. The stretching of the tendons gives additional prestressing force. Due credit is given for this effect also.

Detailed calculations for prestress requirements in the cylindrical wall are enclosed.

Dome - (W. J. Leininger)

The criteria for the design of prestress in the dome is the same as the criteria for the cylindrical wall except as noted.

(1) The allowable principle tensile stress in the concrete is 212 psi. But, because one prestressing system provides the required prestressing force in both the hoop and meridional directions, the tensile stress in the meridional direction is dependent upon the tensile stress in the hoop direction which controls the design. The tensile stresses are as follows:

Tensile stress in the meridional direction

$= \sigma_\phi = 56$ psi

Tensile stress in the hoop direction

$= \sigma_\theta = 205$ psi

Shear stress

$= Q = 16$ psi

(2) Credit is given for the dead load stress at the point under consideration.

Calculations to determine the required prestressing force are enclosed.

W. J. Leininger

W. J. Leininger

R. Shanmugasundaram

R. Shanmugasundaram

flg

Distribution
H. F. Lorenz
D. K. Croneberger
S. N. Dobreff
D. A. Godfrey
K. E. Nodland
W. J. Leininger
R. Shanmugasundaram
D. A. Skilton

PRESTRESS CALCULATIONS	MADE RS	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
FOR CYLINDRICAL WALL	CHK'D. F.R.	
OF THE REACTOR BUILDING		WORK ORDER / SIZE / DRAWING 1.01.11 / REV. B

PRESTRESS REQUIREMENT IN MERIDIONAL DIRECTION IN WALL

THE FOLLOWING ARE TABULATED AS STRESSES IN TYPICAL WALL FOR THE LOADING CONDITION OF 0.95 D.L. + PRESTRESSING + 1.5 ACCIDENT PRESSURE + 1.5 ACCIDENT TEMPERATURE WITH THE EXCEPTIONS AS NOTED.

LOADING	STRESS IN STEEL LINER PSI	CONCRETE STRESS PSI
0.95 DEAD LOAD	-704.5	-55.8
OPERATING TEMPERATURE	0.0	0.0
PRESTRESS	-11353.5	-765.6
P 82.5	6653.1	705.3
ACCIDENT TEMPERATURE	-20720.0	85.0

0.95 DEAD LOAD STRESS IS DUE TO THE WEIGHT OF THE DOME ONLY.

OPERATING: DURING SUMMER CONDITION THE TEMPERATURE GRADIENT IS VERY SMALL. THEREFORE IT IS CONSERVATIVE TO NEGLECT THIS EFFECT FOR COMPUTING PRESTRESS REQUIREMENTS.

YIELD STRENGTH OF THE LINER PLATE IS 30,000 PSI. ASSUME 20% INCREASE HENCE ASSUMED YIELD STRENGTH OF THE LINER PLATE IS 30,000 X 1.2 = 36,000 PSI

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	REV. CH. APP. DATE	

THE FOLLOWING TWO CONDITIONS ARE TO BE SATISFIED.

(1) WE HAVE TO CONSIDER THE LINER STRESS UP TO THE YIELD POINT STRESS OF 36,000 PSI ONLY. BEYOND THIS POINT, THE LINER WILL BE IN THE PLASTIC REGION AND DOES NOT GET IN TO THE STRAIN-HARDENING REGION FOR ANY LOADING COMBINATION. THEREFORE THE LINER WILL NOT TRANSMIT ANY FORCE BEYOND THE YIELD POINT.

(2) WE HAVE TO LIMIT THE FINAL TENSILE STRESS IN THE CONCRETE TO 200 PSI.

LET $\sigma_{c/s(P.S.)}$ = REVISED STRESS IN THE CONCRETE/STEEL DUE TO PRESTRESSING.

$\sigma_{c/s(A.T.)}$ = REVISED STRESS IN THE CONCRETE/STEEL DUE TO ACCIDENT TEMPERATURE

$\sigma_{c/s(P82.5)}$ = STRESS IN THE CONCRETE/STEEL DUE TO 1.5 ACCIDENT PRESSURE

$\sigma_{c/s(D.L)}$ = STRESS IN THE CONCRETE/STEEL DUE TO DEAD LOAD.

$$\sigma_{s(A.T.)} = 36000 - \sigma_{s(D.L)} + \sigma_{s\,P82.5} - \frac{11353.5\,\sigma_{c\,P.S.}}{765.6}$$

$$\sigma_{c(A.T.)} = \sigma_{s\,A.T.} \times \frac{185.0}{20720.0}$$

$$= \left[36000 - \sigma_{s\,D.L} + \sigma_{s\,P82.5} - \frac{11353.5 \times \sigma_{c\,PS}}{765.6}\right]\frac{185}{20720}$$

$$= (36000 - 704.5 + 6653.1 - 14.85\,\sigma_{c\,PS})\frac{185}{20720.0}$$

$$\therefore \sigma_{c\,AT} = (41948.6 - 14.85\,\sigma_{c\,PS})\frac{185.0}{20720.0} \quad \leftarrow (1)$$

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$\sigma_{c(A.T.)} + \sigma_{c(PB2.5)} - \sigma_{c(D.L.)} - \sigma_{c(P.S.)} = 200$ ← (2)

SUBSTITUTE FROM EQN (1) FOR σ_{CAT}

$\left(\frac{41948.6 - 14.85\,\sigma_{cPS}}{20720.0}\right)185.0 + 703.3 - 55.8 - \sigma_{cPS} = 200$

$\frac{41948.6 - 14.85\,\sigma_{cPS}}{112} + 647.5 - \sigma_{cPS} = 200$

$41948.6 - 14.85\,\sigma_{cPS} + 72500.0 - 112\,\sigma_{cPS} = 22,400$

$126.85\,\sigma_{cPS} = 92048.6$

$\sigma_{cPS} = \frac{92048.6}{126.85} = 728\ PSI$ ←

$\sigma_{sPS} = \frac{728 \times 11353.5}{765.6} = 10790\ PSI$ ←

$\sigma_{sAT} = 36000 - \sigma_{sDL} + \sigma_{sPB2.5} - \frac{11353.5 \times \sigma_{cPS}}{765.6}$

$= 36000 - 704.5 + 6653.1 - 11353.5 \times \frac{728}{765.6}$

$= 36000 - 704.5 + 6653.1 - 10790.0$

$= 42653.1 - 11494.5$

$= 31,158.6\ PSI$

$\sigma_{CAT} = 200 - \sigma_{cPB2.5} + \sigma_{cDL} + \sigma_{cPS}$

$= 200 - 703.3 + 55.8 + 728.0$

$= 280.5\ PSI$

PRESTRESS FORCE REQD $= 10790 \times \frac{3}{8} + 728 \times 42$

$= 4050 + 30500$

$= 34,550\ \#/IN$

$= 34.55\ KIPS/IN$

$= 414.6\ KIPS/FT$ ←

0.95 DL + Prestress + 1.5 acc. pressure + 1.5 acc. temp.

	Liner Pl.	Conc
0.95 DL	-704	-55.8 comp.
op temp	0	0
prestress	-11,354	-766 comp
pressure	6653	703
ac temp.	-20720	+185

YS liner = 36000 psi

$\sigma_{c/s}(P.S.)$ = Residual stress in the conc/steel due to prestress

$\sigma_{c/s}(\Delta T)$ = " " " " " " due to acc. temp

$\sigma_{c/s}(pres)$ = " " " " " " " " 1.5 acc pres

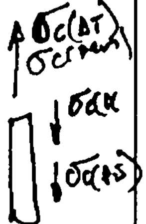
$\sigma_{c/s}(D.L)$ = " " " " " " " " Dead Load

$$\sigma_s(\Delta T) = 36000 - \sigma_s(D.L) + \sigma_s(pres) - \left(\frac{\sigma_s(PS) \cdot \sigma_c(P.S)}{\sigma_c(PS)}\right) \quad \frac{11354}{766}\sigma_c(PS)$$

subst

$$\sigma_c(\Delta T) = \sigma_s(\Delta T) \times \frac{\sigma_c(\Delta T)}{\sigma_s(\Delta T)} = \sigma_s(\Delta T) \times \frac{185}{20720}$$

$$\therefore \sigma_c(\Delta T) = \left(41948.6 - 14.85\,\sigma_c(PS)\right)\frac{185}{20720} \quad \text{— (1)}$$



AND: $\sigma_c(\Delta T) + \sigma_c(pres) - \sigma_c(DL) - \sigma_c(P.S) = 700$ (Tension) (2)

1.01.7/10a

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	REV. CH. APP. DATE	

REVISED STRESSES

LOADING CONDITION	STEEL LINER PSI	CONCRETE STRESS PSI
0.95 DEAD LOAD	-704.5	-55.8
OPERATING TEMPERATURE	0.0	0.0
PRESTRESS	-10790.0	-728.0
P82.5	6653.1	703.3
ACCIDENT TEMPERATURE	-31,158.6	280.5
	-36,000.0	200.0

NUMBER OF TENDONS REQUIRED

THE FOLLOWING 2 TYPES ARE CONSIDERED.

TYPE I 163 - 0.276" ϕ WIRE

f'_s = ULTIMATE STRENGTH OF PRESTRESSING STEEL = 240 KSI

RELAXATION OF STEEL STRESS = 12% OF $0.6f'_s$

(DOMESTIC STRESS RELIEVED WIRE.)

TYPE II 163 - 0.276" ϕ WIRE

f'_s = 235 KSI

RELAXATION OF STEEL STRESS = 4% OF $0.6f'_s$

(FOREIGN STABILIZED WIRE)

TYPE I

PRESTRESS FORCE REQD AT 67'-3" RADIUS = 414.6 K/FT

TOTAL PRESTRESS FORCE REQD = 414.6 x 2 x π x 67.25

= 175,200 KIPS.

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	CP. DPM.		/	-5
	ENG.	WORK ORDER	SIZE DRAWING	REV.
	REV. CH. APP. DATE			

LOSSES:

CREEP $= 200 \times 10^{-6} \times 29 \times 10^{6}$

$= 5.8$ KSI

SHRINKAGE $= 1.20 \times 10^{-6} \times 29 \times 10^{6}$

$= 3.48$ KSI

12% RELAXATION $= 0.12 \times 0.6 \times 240$

$= 17.28$ KSI

ELASTIC SHORTENING

$= \frac{1}{2} \times \frac{414.6 \times 29 \times 10^{6}}{42 \times 12 \times 4.08 \times 10^{6}}$

$= \frac{207.3 \times 29}{504 \times 4.08}$

$= 2.92$ KSI

TOTAL LOSSES $= 5.8 + 3.48 + 17.28 + 2.92$

$= 29.48$ KSI ←

(INITIAL) EFFECTIVE PRESTRESS $= 168 - 29.48 = 138.52$ KSI $< 0.6 f_s'$ $= 0.6 \times 240 = 144$ KSI

CREDIT FOR EXPANSION OF THE WIRES

$= 983.8 \times \frac{29 \times 10^{6}}{4.08 \times 10^{6}}$ PSI

$= 6.98$ KSI

ANCHOR STRESS $= 0.7 \times 240$

$= 168$ KSI

FINAL EFFECTIVE PRESTRESS $= 138.52 + 6.98$

$= 145.5$ KSI $< 0.7 f_s = 168$ KSI

AREA OF 163 - 0.276"φ WIRE $= 163 \times 0.0597$

$= 9.73$ □"

	MADE	GILBERT ASSOCIATES, INC.
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	ENG.	
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FORCE / TENDON WITH ϕ FACTOR OF 0.95 = 145.5 × 9.73 × 0.95

= 1348 KIPS.

NO OF TENDONS REQD = $\frac{175,200}{1348}$ = 130 ←

TYPE II

Losses

CREEP = 5.8 KSI

SHRINKAGE = 3.48 KSI

ELASTIC SHORTENING = 2.92 KSI

4% RELAXATION = 0.04 × 0.6 × 235 (141 KSI)

= 5.64 KSI

TOTAL LOSSES = 17.84 KSI

ANCHOR STRESS = $0.7 f'_s$ = 0.7 × 235

= 164.5 KSI

INITIAL EFFECTIVE PRE STRESS = 164.5 − 17.84

= 146.66 > $0.6 f'_s$ = 141 ksi

= 141.0 KSI

CREDIT FOR EXPANSION OF THE WIRES

= 6.98 KSI

FINAL EFFECTIVE PRE STRESS = 141.0 + 6.98

= 147.98 KSI

AREA OF 163 − 0.276" ϕ WIRE = 9.73 □"

FORCE/TENDON WITH ϕ FACTOR OF 0.95 = 147.98 × 9.73 × 0.95

= 1368 KIPS

NO OF TENDONS REQD = $\frac{175,200}{1368}$ = 128.0 ←

NOTE: - TENDON REQUIREMENTS DO NOT CHANGE UNTIL 8% RELAXATION LOSSES.

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D. F.P.	4203.00 — WORK ORDER / SIZE / DRAWING — REV. 7
REVISED PRESTRESS REQUIREMENTS	REV. CH. APP. DATE	3-28-69

PRESTRESS REQUIREMENTS IN HOOP DIRECTION

THE FOLLOWING ARE TABULATED AS STRESSES IN TYPICAL WALL.

LOADING CONDITION 0.95 D.L. + 1.5P + 1.0T

	STEEL LINER	CONCRETE
0.95 D.L.	0	0
OPERATING TEMP.	0	0
PRESTRESS	-17955.0	-1365.1
PRESSURE	11623.0	1424.3
ACC. TEMP.	-21800.0	194.5

REVISED STRESS IN CONCRETE DUE TO ACC. TEMPERATURE

$$= \sigma_{CAT} = \frac{\left[36000 + \sigma_{S(P82.5)} - \frac{17955\,\sigma_{C(P.S)}}{1365.1}\right] 194.5}{21800}$$

WHERE $\sigma_{C(P.S)}$ = STRESS IN CONCRETE DUE TO PRESTRESSING

$$\sigma_{CAT} = \frac{36000 + 11623.0 - 13.15\,\sigma_{C(PS)}}{112}$$

$$= \frac{47623.0 - 13.15\,\sigma_{CPS}}{112}$$

$$= 425 - 0.1175\,\sigma_{CPS}$$

$$\sigma_{CAT} + 0.1175\,\sigma_{CPS} = 425 \quad \text{①}$$

$$\sigma_{CAT} + \sigma_{C(P82.5)} - \sigma_{CPS} = 3\sqrt{f'_c} = 3\sqrt{5000} = 212 \;\; 211$$

$$\sigma_{CAT} - \sigma_{CPS} = 212 \,(211) - 1424.3 = -1212.3 \;\; -1213.3$$

$$\sigma_{CAT} - \sigma_{CPS} = -1212.3 \;\; (-1213.3) \quad \text{②}$$

CRYSTAL RIVER	MADE RS	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D.	4203.00 (WORK ORDER) -8- (REV.)
REVISED PRESTRESS REQUIREMENTS		3-28-69

SUBTRACT ② FROM ①

$1.1175\,\sigma_{cps} = 1637.3$ 1638.3

$\sigma_{cps} = \frac{1637.3}{1.1175}$ $\frac{1638.3}{1.1175}$

$= 1465$ PSI

$\therefore \sigma_{cat} = 1465 - 1212.3 = 252.7$ PSI

$\sigma_{sps} = \frac{1.7955}{1365.1} \times 1465.0 = 19300$ PSI

REVISED STRESSES

	STEEL LINER		CONCRETE
0.95 D.L.	0		0
OPERATING TEMP	0		0
PRESTRESS	−19300		−1465
PRESSURE	11623		424.3
ACC. TEMP	−28323	251.7	252.7
	−36,000	211.0	212.0

N° REQD FOR PRESTRESSING $= 19300 \times \frac{3}{8} + 1465 \times 42$

$= 7240 + 61500$

$= 68740$ #/IN

$= 68.74$ KIPS/IN

$= 824.88$ KIPS/FT

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	ENG.	
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NO OF TENDONS REQUIRED IN HOOP DIRECTION.

REQD PRESTRESSING FORCE FOR TOP 147'-0"

= 824.88 × 147

= 121,000 KIPS

REQD PRESTRESSING FORCE IN THE HAUNCH $= \frac{824.88}{2} \times 10$

= 4124.4 KIPS

TYPE I — TOTAL = 125124.4 KIPS.

LOSSES

CREEP $= 300 \times 10^{-6} \times 29 \times 10^{6} = 8.7$ KSI

SHRINKAGE $= 120 \times 10^{-6} \times 29 \times 10^{6} = 3.48$ KSI

12% RELAXATION = 0.12 × 0.6 × 240 = 17.28 KSI

ELASTIC SHORTENING $= \frac{1}{2} \times \frac{824.88 \times 29.0 \times 10^{6}}{42 \times 12 \times 4.08 \times 10^{6}}$

$= \frac{412.49 \times 29.0 \times 10^{6}}{504 \times 4.08 \times 10^{6}}$

= 5.8 KSI

TOTAL LOSSES = 8.7 + 3.48 + 17.28 + 5.8

= 35.26 KSI

FRICTION LOSSES AS PER ACI 318-63-2607

$T_o = T_x e^{(KL + \mu\alpha)}$

$\mu = 0.16$

$K = 0.0003$

$L = 70.75'$

$\alpha = 54.9°$ OR 0.96 RADIANS

$0.7 f_s' = 0.7 \times 240 = 168.0$ KSI

$0.8 f_s' = 0.8 \times 240 = 192.0$ KSI

	MADE	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. F.P.	
	SQ. CP.	
	CP. DPN.	WORK ORDER / SIZE / DRAWING / REV. -10-
	ENG.	
	REV. CH. APP. DATE	

$$192.0 = T_x e^{(0.0003 \times 70.75 + 0.16 \times 0.96)}$$

$$= T_x e^{(0.021 + 0.153)}$$

$$= T_x e^{(0.174)}$$

$$= T_x \; 1.19$$

$$T_x = 192/1.19 = 161.0 \text{ KSI}$$

THE TENDON ABOVE AND BELOW IS CONSIDERED TO BE AT 168.0 KSI

∴ USE AV. VALUE $= \frac{168.0 + 161.0}{2}$

$= 164.5$ KSI

INITIAL EFFECTIVE PRESTRESS $= 164.5 - 35.26$

$= 129.24$ KSI

CREDIT FOR EXPANSION OF THE WIRES $= \frac{1.677 \times 29 \times 10^6}{4.08 \times 10^6}$

$= 11.9$ KSI

FINAL EFFECTIVE PRESTRESS $= 129.24 + 11.9$

$= 141.14$ KSI

AREA OF 163 – 0.276" φ WIRE $= 163 \times 0.0597$

$= 9.730 \square''$

FORCE/TENDON WITH φ FACTOR OF 0.95 $= 141.14 \times 9.73 \times 0.95$

$= 1305$ KIPS

NO OF TENDONS REQD $= \frac{12512.4}{1305}$

$= 9.6$

TOTAL NO OF TENDONS REQD. $= 3 \times 9.6$

$= 28.8$ ←

	MADE	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D.	
	GO. CP.	
	CP. DPN.	11
	ENG.	WORK ORDER / SIZE / DRAWING / REV.
	REV. CH. APP. DATE	

TYPE II

LOSSES

CREEP = 8.7 KSI

SHRINKAGE = 3.48 KSI

4% RELAXATION = 0.04 × 0.6 × 235

= 5.64 KSI

ELASTIC SHORTENING = 5.8 KSI

TOTAL LOSSES = 23.62 KSI

$0.7 f'_s = 0.7 \times 235 = 164.5$ KSI

$0.8 f'_s = 0.8 \times 235 = 188.0$ KSI

$T_x = \frac{188.0}{1.19} = 158$ KSI

THE TENDON ABOVE AND BELOW IS CONSIDERED TO BE AT 164.5 KSI

∴ USE AV. VALUE $= \frac{158 + 164.5}{2}$

= 161.25 KSI

INITIAL EFFECTIVE PRESTRESS = 161.25 − 23.62

= 137.63 KSI

CREDIT FOR EXPANSION OF THE WIRES

= 11.9 KSI

FINAL EFFECTIVE PRESTRESS = 137.63 + 11.9

= 149.53 KSI

FORCE/TENDON WITH φ FACTOR OF 0.95 = 149.53 × 9.73 × 0.95

= 1381 KIPS

NO. OF TENDONS REQD $= \frac{125124.4}{1381} = 91$

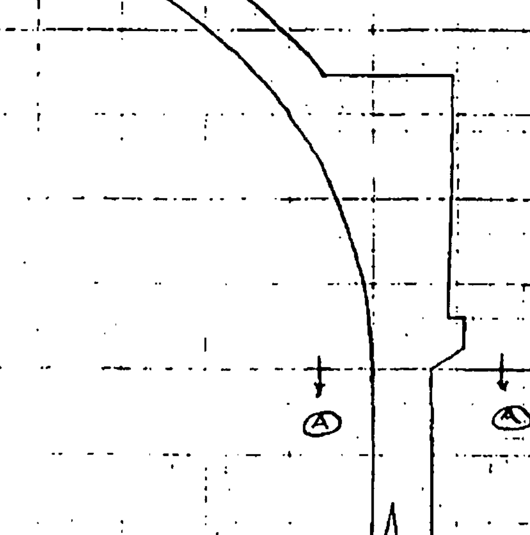
TOTAL NO. OF TENDONS REQD = 91 × 3 = **273**

CRYSTAL RIVER	MADE R.S. / CHK'D. F.R. / SQ. CP.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CP. DFN. / ENG.	4203-00 (WORK ORDER) / SIZE / DRAWING / -17- (REV.)
	REV. CH. APP. DATE	4-21-69

TO FIND ALLOWABLE TENSILE STRESSES IN THE HOOP AND MERIDIONAL DIRECTION

THE CRITICAL LOADING CONDITION FOR PRESTRESSING IN THE CYLINDRICAL WALL IS 0.95 D.L. + PRESTRESSING FORCE + 1.5 ACCIDENT PRESSURE + TEMPERATURE ASSOCIATED WITH 1.5 ACCIDENT PRESSURE.

THE CRITICAL LOCATION FOR DESIGN CONSIDERATION IS AT TOP OF THE WALL.



SHEAR AT TOP OF THE WALL

0.95 DEAD LOAD	=	+210.3#	(CRYSTAL RIVER OUTPUT)
OPERATING TEMP.	=	+518.1#	(")
PRESTRESSING	=	+4464.7#	(MET-ED)
1.5 ACC. PRESSURE	=	-5710.7#	(")
ACC. TEMPERATURE ASSOCIATED WITH 1.5 ACC. PRESSURE	$\frac{-74.3 \times 280.5}{185}$ =	-112.0#	(")

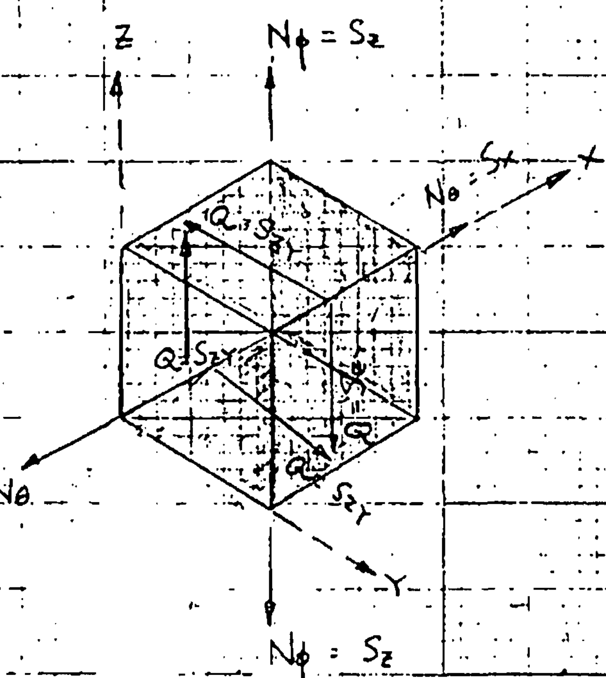
CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D.	
	SQ. CF.	
	CF. DPN.	4203-00 — WORK ORDER / SIZE / DRAWING / REV.
	ENG.	
	REV. CH. APP. DATE	4-21-69

IT IS CONSERVATIVE TO NEGLECT THE SHEAR DUE TO OPERATING TEMPERATURE

$\therefore$ NET SHEAR = 210.3 + 4464.7 − 5710.7 − 112.0

= 1147.7 #/IN

SHEAR STRESS = $\frac{1147.7}{42}$ = 27.4 PSI



THE VALUES OF THE THREE PRINCIPAL STRESSES ARE THE THREE ROOTS OF THE FOLLOWING CUBIC EQUATION

$$S^3 - (S_x + S_Y + S_Z)S^2 + (S_x S_Y + S_Y S_Z + S_Z S_x - S_{xY}^2 - S_{YZ}^2 - S_{Zx}^2)S - (S_x S_Y S_Z + 2 S_{xY} S_{YZ} S_{Zx} - S_x S_{YZ}^2 - S_Y S_{Zx}^2 - S_Z S_{xY}^2) = 0$$

GIVEN:

S = 212 PSI

S_x = 211 PSI

S_z = ?

Q = 27.4 PSI

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D. F.H.	WORK ORDER 4203-00
	REV. CH. APP. DATE	4-21-69
		REV. -19-

$$S^3 - (S_x + S_z) S^2 + (S_z S_x - S_{yz}^2) S + S_x S_{yz}^2 = 0$$

$$212^3 - (211 + S_z) 212^2 + (211 S_z - 27.4^2) 212 + 211 \times 27.4^2 = 0$$

$$212^3 - 211 \times 212^2 - 212^2 \times S_z + 211 \times 212 S_z - 27.4^2 \times 212 + 211 \times 27.4^2 = 0$$

DIVIDE BY 212

$$212^2 - 211 \times 212 - 212 \times S_z + 211 S_z - 27.4^2 + 750 = 0$$

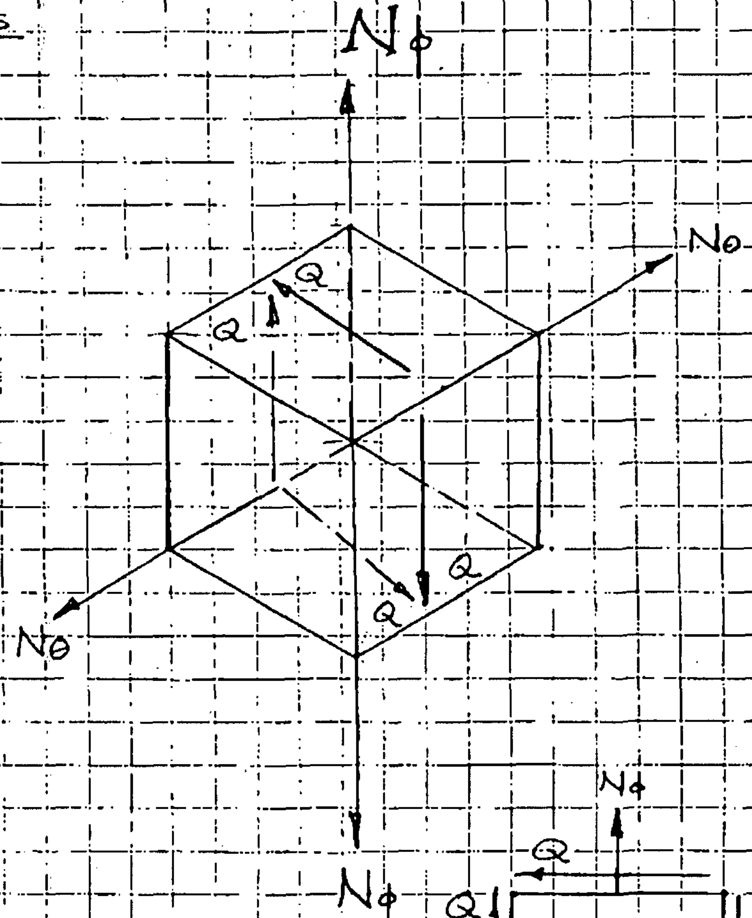
$$44{,}944 - 44{,}732 - S_z - 751 + 750 = 0$$

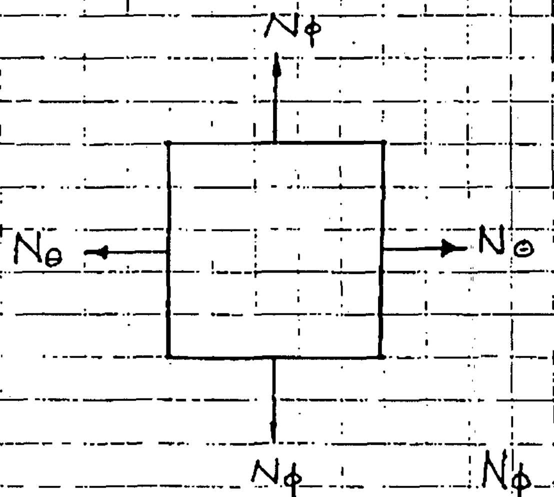
$$S_z = 211 \text{ PSI}$$

CRYSTAL RIVER

REACTOR BUILDING

STRESS ANALYSIS

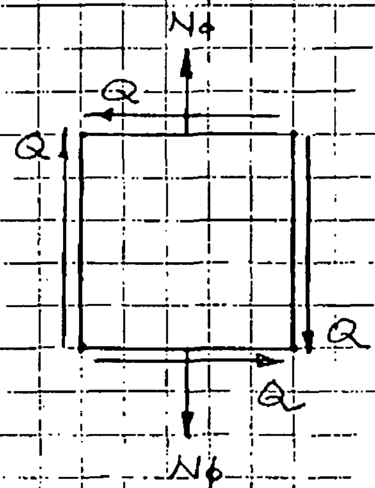




N_ϕ = 211 PSI

N_θ = 211 PSI

Q = 27.4 PSI



(a) (b)

(a) PRINCIPAL STRESS = 211 PSI

(b) PRINCIPAL STRESS

$$= \sigma_2 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2}$$

$$\sigma_2 = \frac{211}{2} + \sqrt{\left(-\frac{211}{2}\right)^2 + 27.4^2}$$

$$= 105.5 + \sqrt{105.5^2 + 27.4^2}$$

$$= 105.5 + \sqrt{11{,}100 + 750}$$

$$= 105.5 + 108.5$$

$$= 214.0 \text{ PSI} \longleftarrow$$

2nd TRIAL

$N_\phi = 200$ PSI

$N_\theta = 211$ PSI

$Q = 27.4$ PSI

(a)

$$\sigma_1 = \frac{211+200}{2} + \sqrt{\left(\frac{211-200}{2}\right)^2 + 0}$$

$$= 205.5 + 5.5$$

$$= \underline{\underline{211 \text{ PSI}}} \leftarrow$$

(b)

$$\sigma_1 = \frac{200}{2} + \sqrt{\left(\frac{200}{2}\right)^2 + 27.4^2}$$

$$= 100 + \sqrt{10000 + 750}$$

$$= 100 + \sqrt{10750}$$

$$= 100 + 103.7$$

$$= 203.7 \text{ PSI} \leftarrow$$

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SCHUPACK & ASSOCIATES
CONSULTING ENGINEERS

APPENDIX 5J

REPORT ON

RECOMMENDED CONCRETE CREEP AND SHRINKAGE VALUES

FOR COMPUTING PRESTRESSING LOSSES

UP TO DATE

12/15/75

Prepared for: Jersey Central Power & Light Co.
Parsippany, New Jersey

Prepared by: Schupack and Associates
Consulting Engineers
Stamford, Connecticut

Dated: June 1968

Project No. (67-535)

TABLE OF CONTENTS

1. SUMMARY

Grouted tendons have been selected for the containment structure at Oyster Creek Nuclear Station - Unit 2. The purpose of this report is to estimate prestress losses for these tendons due to concrete shrinkage and creep over a 50-year period at an average temperature of 85°F ± 10%.

Deformational behavior of controlled concrete similar to this structure is shown to be predictable on the basis of test data on basic creep and shrinkage, and correlation with observations on actual full size beams. For the volume/surface ratio of the concrete in this structure, basic creep is the dominating deformational component and depends primarily on the concrete's specific strength and age at loading. Shrinkage strain is shown to be approximately equal to the lower limit of possible accuracy in estimating total long-term losses.

A uni-axial creep strain of 0.40×10^{-6} inch/inch per psi, and a final shrinkage of 0.0001 inch/inch are recommended for the concrete specified for this structure.

SCHUPACK & ASSOCIATES
CONSULTING ENGINEERS

2. INTRODUCTION

The force initially imparted to prestressing tendons decreases — asymptotically with time due to:

A. Shrinkage of concrete

B. Creep of concrete

C. Relaxation of prestressing steel

Grouted tendons have been selected for the containment structure in the Jersey Central Power and Light Company project. This requires a conservative evaluation of the total losses in prestressing force over a 50-year period in order to ensure the structure's satisfactory function during its entire service life.

3. PURPOSE OF REPORT

It is the object of this report to recommend a conservative estimate of probable shrinkage and creep of concrete in this structure. The evaluation applies specifically to:

A. Steel inside-lined dome and walls with concrete 36" - 42" thick.

B. Concrete with approximately 5,000 psi strength and of normal workability (i.e., six to seven sacks cement per cubic yard, approximately 1" to 4" slump and such normal aggregate that the concrete's modulus of elasticity is approximately equal to the ACI formula).

C. Average ambient temperature 85°F ± 10%

D. Bi-axial compression less than 50% of f'_c

4. NATURE OF SHRINKAGE AND CREEP

4.1 Shrinkage

Shrinkage is the deformation of concrete due to other physical or chemical causes than applied load and temperature dependent volumetric changes. (i.e., variation in moisture content and hydration of cement). Shrinkage strain is time dependent and influenced by the following factors:

A. Internal properties of the concrete, as affected by:

1. Cement quality
2. Water cement ratio
3. Proportion cement-aggregate
4. Aggregate properties (expansive or permeable aggregate)
5. Admixtures
6. Degree of compaction
7. Reinforcement percentage
8. Curing methods

B. Ambient temperature

C. Ambient humidity as affected by:

1. Size and shape (volume to surface ratio) of element
2. Environment

4.2 Creep

Creep is the time dependent deformation of concrete due to load. Creep strain at any particular time is defined as the difference between total strain and the sum of initial elastic strain, shrinkage strain and strains due to temperature dependent volumetric changes at that time. Creep strain is influenced by the following factors:

A. Internal properties of the concrete (same as for shrinkage, except A-7 above, Reinforcement percentage, that arbitrarily can be considered as an internal property or an external load factor).

B. Ambient temperature (same as for shrinkage)

C. Ambient humidity (same as for shrinkage)

D. External load

   1. Time of loading

   2. Duration of loading

   3. Uni-or bi-axial loading

4.2.1 Evaluation of Creep

For practical evaluation and prediction, creep can be separated into two components:

A. Drying creep results from stress-induced moisture diffusion and consequential shrinkage or swelling during the period of sustained loading. For given stress conditions, drying creep is primarily a function of the humidity factors (C) mentioned above.

B. Basic creep is the creep of concrete not exposed to moisture diffusion. For any given concrete mix, it is a characteristic, predictable function of load and temperature alone [2].

The complexity of variables influencing shrinkage and creep must be viewed realistically. The property-factors, for example, are substantially the same as determine the concrete's strength and workability. For the structure in question, therefore, this set of parameters can be substituted with values typical for the specified concrete.

Similarly, the humidity factors can be limited to a single parameter for size and shape by assuming typical average environmental conditions.

A comprehensive investigation in the range 0 - 95°C (32 - 203°F) of temperature effects on creep and shrinkage by D. J. Hannant [1A], indicates that creep increases approximately linearly with temperature [2]. Applied to this structure, with an average temperature of 85°F ± 10% (110°F inside, 0 to 100°F outside), the influence of the temperature range on creep is negligible. Temperature variations within this range are without measurable influence on the final shrinkage, and can also be neglected.[1B]

The influence of mild reinforcement is to increase the elastic resistance of the composite section and reduce creep and shrinkage effects proportionately. These recommendations will concern the concrete section alone, and give conservative values for the final shrinkage and creep.

Based on the above, a realistic estimate of creep and shrinkage for this structure can be determined by:

A. An evaluation of shrinkage and drying creep for the one significant humidity parameter - the actual volume/surface ratio.

B. A prediction of basic creep for the specified concrete, the given loads, and the concrete age at loading.

5. REVIEW OF LITERATURE AND TEST DATA

5.1 Shrinkage and Creep Research in General

The American Concrete Institute's publication SP-9 [3] presents a summary of the research done on creep up to 1964. A corresponding review of research on shrinkage has been prepared by the California Producers Committee [7] in 1966. Papers presented at the Conference on Prestressed Concrete Pressure Vessels in London in 1967 [1] cover research on the special creep and shrinkage problems arising in prestressed concrete pressure vessels resembling the one in question.

Most investigations concern effects of a single, or a few, of the factors influencing shrinkage and creep and supply excellent information on the nature of these phenomena, especially for basic creep. The small scale models and laboratory specimens tested, and the comparatively short period of observation (maximum about 4 years), is, however, not representative of 50 years performance of the large concrete structural dimensions in question.

Reported observations on actual structures show the significant variation of combined shrinkage and creep effects with size and shape of member, but suffer from the impossibility of separating shrinkage and creep strain without having unloaded specimens for comparison.

The data reported on a few long-term full scale tests including unloaded companion specimens for shrinkage measurements (*), is insufficient in quantity and homogeneity to support conclusions on shrinkage and creep for this specific structure and extrapolation over 50 years.

All test data can, however, be utilized for this estimate by separating the deformational behavior of the concrete into a primary component - basic creep, and a modifying component - drying creep and shrinkage. The investigations of size and shape influence quoted below show that the volume/surface ratio of this structure completely governs shrinkage in the possible range of prediction, and that drying creep can be neglected. Similarly, the research listed on basic creep shows that it dominates the concrete's deformational behavior on this structure. General test data applies directly to basic creep, and available information in correlation with reported observations on full scale tests is sufficient to warrant regression and extrapolation of this structure's estimated basic creep behavior during its entire service life.

(*) Notably, the AASHO Road Test, Report 4, "Bridge Research" and Technical Report R212 from U. S. Naval Laboratories, Port Hueneme, California

5.2 Research and Test Data on Influence of Size and Shape

A. H. Mattock has investigated shrinkage and creep of precast concrete bridges (4), and Mattock and Torben C. Hansen together (5) have reported data from the first four years of an investigation in progress regarding the influence of size and shape of members on the shrinkage and creep of concrete. Based on the accumulated data and G. Pickett's theory of moisture diffusion (6), Hansen and Mattock have established a relationship between shrinkage and creep, and the volume to surface ratio (V/S inch) of a concrete member.

5.2.1 Shrinkage

The following graph shows the variation of final shrinkage with volume-surface ratio for Hansen and Mattock's main tests (Elgin gravel) and an auxiliary test on concrete made from an unusually porous crushed sandstone aggregate:

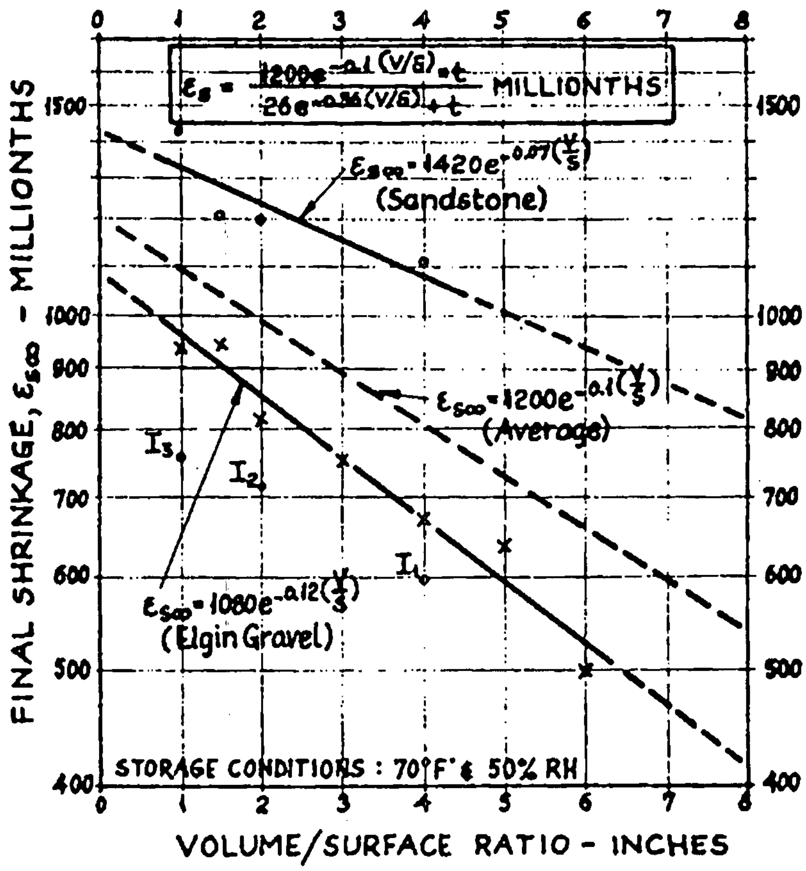

FIG. 1. VARIATION OF FINAL SHRINKAGE WITH VOLUME/SURFACE RATIO

The plotted average represents a conservative prediction on the concrete specified for the structure in question, and the shrinkage at time "t" may be predicted from the given formula. Based on the underlying general theory of moisture diffusion, extrapolation of this curve is justified up to volume/surface ratios of approximately 24". Further extrapolation results in values outside the scope of accuracy possible in estimating total losses. For a volume/surface ratio of 24", the final shrinkage will be

$$\varepsilon_{s\infty} = 1200 \times e^{-0.10 \times 24} \times 10^{-6} = 0.0001 \text{ inch/inch}$$

All concrete in the present structure has a volume/surface ratio greater than 24", and the above value (0.0001) can be used as a conservative estimate of the final shrinkage strain.

5.2.2 Creep

Hansen and Mattock have established the following relationship between the coefficient for total (= basic + drying) creep ϕ, the coefficient of creep for sealed specimens (= basic creep) ϕ_s, and the volume/surface ratio V/S, valid for concrete similar to the structure in question:

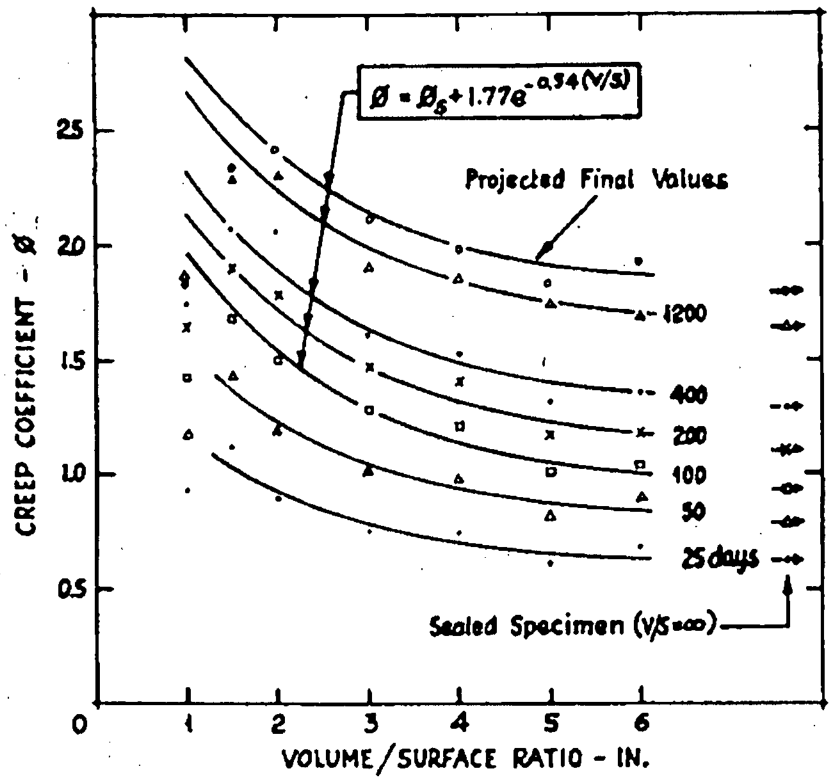

FIG. 2. VARIATION OF CREEP COEFFICIENT WITH VOLUME/SURFACE RATIO AT DIFFERENT AGES. (ELGIN GRAVEL AGGREGATE CONCRETE CYLINDERS)

Based on the above, and assuming a coefficient of basic creep $\phi_s = 1.5$ (*), the drying creep component for a volume/surface ratio ≥ 6 is found to be:

$$\frac{\Delta \phi}{\phi_s} \leq \frac{1.77}{1.5} \times e^{-.54 \times 6} = 0.046$$

i.e., drying creep is less than 5% of basic creep for volume/surface ratios larger than 6, and can be neglected.

As all volume/surface ratios in the present structure are greater than 6, the size and shape of this structure makes basic creep the only significant creep component.

5.2.3 Research on Basic Creep

A summary of research on basic creep has been published by Torben C. Hansen [2] tabulating all work done up to 1960. Based on Powers and Brownyard's [8], and A. Hummel's [9] investigations of the properties of hardened Portland cement, Hansen has fit the accumulated data to Reiner's general rheological creep equation [10] and established a practical relationship between specific basic creep (i.e., basic creep per unit stress) and time. For given concrete properties the only parameter is concrete age at loading.

This fundamental rheological creep equation has been adapted to the concrete specified for the present structure, and curves for the estimated basic creep have been plotted on a graph, Appendix C, for loading ages 8 and 22 days, 1, 2, 3, 4-1/2 and 6 months. The curves are conservatively designed to over-estimate the probable basic creep by approximately 10%. For the specific conditions of this structure, the percentage deviation on the estimated creep values is expected to be approximately 14%.

Derivation of the basic creep curves and an evaluation of the accuracy of the estimate are presented in Appendix A.

(*) See Basic Creep graph (Appendix C)

5.2.4 Effects of Bi-axial Stressing

Based on Arthanari and Yu's investigations [11] and the data reported to the London Conference on Prestressed Concrete Pressure Vessels by Browne [1B], Barrett and Murray [1C] and D. J. Hannant [1A], Poisson's ratio for creep under sustained multi-axial load at temperatures 85°F ± 10% can be considered constant and equal to 1/6. The estimated uni-axial creep strains must be evaluated separately for each member of the present structure as the creep effect due to multi-axial loading varies with the different ratios between specific uni-axial loads, and with the degree of restraint due to reinforcement.

6. CORRELATION WITH FULL SCALE TESTS

The estimated shrinkage and creep for the present structure has been checked against data from full scale tests on 22 prestressed I-section beams conducted by the U. S. Naval Engineering Laboratory, Port Hueneme, California. These 42-foot beams had a volume/surface ratio of 2.13" and were constructed with three proprietary systems of prestressing and designed to support a uniform load of 760 lbs. per linear foot. They were tested under different loading conditions and observations made of concrete strains over periods of 6 to 7 years. Eleven additional 21-foot non-loaded specimens were made for shrinkage measurements.

Evaluation of observations relative to the recommended shrinkage and creep strains is presented in Appendix B, and for comparison, creep curves for loading ages of 8 and 22 days derived from these observations are plotted on the graph of estimated basic creep (Appendix C).

For creep, these modified observations (apart from a minor early age discrepancy on the 8-day curve) clearly fall within the recommended values, especially for extrapolated later age conditions.

For shrinkage, the final average value observed is half of the value predicted for a volume/surface ratio of 2.13" by the recommended conservative formula (observed 0.0005, calculated 0.001 inch/inch).

7. CONCLUSIONS

Shrinkage and creep are complex phenomena affected by a variety of factors reflecting the internal properties of the concrete, ambient temperature and humidity conditions, and the load pattern involved. Attempts to estimate long-term deformation of structures with small volume/surface ratios (say less than 2") result in a broad band of predictions. This is due to the difficulty of evaluating creep behavior when the volume/surface ratios are so small that the drying creep component predominates.

For the temperature range ($85^{\circ}F \pm 10\%$) and volume/surface ratio (larger than 24") of this structure, and the concrete properties and load conditions specified, it is, however, possible to obtain a conservative estimate of long-term deformations. This is done by separating basic creep strains from those due to drying creep and shrinkage.

Basic creep is by far the dominating deformational component and can be predicted with an accuracy of 10 - 15% based on the specified concrete strength and age at load application.

Drying creep is concluded to be negligible for the volume/surface ratio of the concrete in this structure.

Shrinkage is governed by the volume/surface ratio and is conservatively estimated to be at the lower limit of possible accuracy in predictions of total long-term losses.

For bi-axial stress conditions at the considered temperature, Poisson's ratio for creep can be assumed constant throughout the entire evaluation period. Multi-axial creep effects must, however, be evaluated separately for varying ratios of bi-axial stresses.

8. RECOMMENDATIONS

The following losses due to shrinkage and creep are a reasonably conservative estimate of the values to be expected after 50 years sustained tendon loads at a temperature level of 85°F ± 10% on the Jersey Central Power and Light Company's Prestressed Concrete Containment Structure at Oyster Creek Nuclear Station - Unit 2:

Shrinkage Strain After 50 Years:.................. 0.0001 inch/inch

Uni-axial Creep Strain After 50 Years:

Concrete Age at Prestressing Months	Specific Creep Strain inch/inch per psi	Creep Coefficient ϕ_c (E_c 4 x 10^6 psi)
1	$.45 \times 10^{-6}$	1.8
2	$.40 \times 10^{-6}$	1.6
3	$.37 \times 10^{-6}$	1.5
6	$.33 \times 10^{-6}$	1.3

Modification of Uni-axial Creep Strains for Bi-axial Stresses:

Poisson's ratio for creep can be considered constant over 50 years sustained tendon load at temperatures averaging 85°F ± 10%. The recommended value is 1/6.

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SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX A Basic creep	SHEET NO. A1 OF BY CS DATE 2.5.68 CHK'D. DATE

Torben C. Hansen's formulation of Reiner's general rheological creep equation is:

$$\frac{\varepsilon_{creep}}{\sigma} = \beta \frac{\left(0.31\cdot g(t_0) + \frac{w_0}{C}\right)\cdot V_1}{(N\cdot k_1 + 0.31)\, g(t_0)} \left(1 - e^{-m(t_1 - t_0)}\right) + \alpha_1 \frac{w_0}{C}\cdot V_1 \cdot \ln\left(\frac{t_1}{t_0}\right)$$

spec. creep — delayed elasticity — viscosity

- t_1 = age of concrete at time t_0, in days
- t_0 = age of concrete when loaded, in days
- $\frac{w_0}{C}$ = water-cement ratio by weight, corrected for bleeding
- V_1 = volume concentration of cement paste in concrete
- $g(t_0)$ = degree of hydration of cement at time t_0
- k_1 = weight ratio of nonevaporable water to cement when all cement is hydrated. k_1 is a function of the proportions of cement components:
  $k_1 = 0.187(C_3S) + 0.158(C_2S) + 0.655(C_3A) + 0.213(C_4AF)$
- N = $0.75(1+4k)$, where
  $k = 0.230(C_3S) - 0.320(C_2S) + 0.317(C_3A) + 0.368(C_4AF)$

α_1, β and m are coefficients to be determined by experiment.

Hansen has fit the results of 42 test series to the equation above to determine α_1, β and m. Powers & Brownyard's values $k = .25$ and $k_1 = .23$ have been used where the cement composition was unknown, as have Hummel's values for degree of hydration with time for standard and rapid hardening Portland cement. Hansen found:

$\beta = 1.76\times10^{-6}$, $m = 0.0333$, $\alpha_1 = 5.7\times10^{-6}$ (units: specific creep/kg/cm^2)

The general equation for basic creep becomes: (units: sp. creep pr. psi)

$$\frac{\varepsilon_{creep}}{\sigma}\times10^6 = 0.189\cdot\frac{\left(0.31\cdot g(t_0) + \frac{w_0}{C}\right)\cdot V_1}{g(t_0)}\cdot\left(1 - e^{-\frac{t_1 - t_0}{30}}\right) + 0.401\cdot\frac{w_0}{C}\cdot V_1\cdot\ln\left(\frac{t_1}{t_0}\right) \quad [R]$$

The average percentage deviation between measured and calculated creep values was 14%, largest (16%) for 50-100 days sustained load, smallest (12%) for 900 days loading. For long term estimates, where the effect of poorly defined early age conditions is minimalized, the accuracy of the equation is conservatively set at ±10%.

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX A Basic creep	SHEET NO. A2 OF BY CS DATE 7.5.68 CHK'D. DATE

Values of $r = \frac{W_0}{C}$ and V_1 to be used for the specified concrete in this estimate are as follows:

$r = 0.45,\ \left(\frac{\Delta r}{r}\right) = 7.5\%$; $V_1 = 0.29,\ \frac{\Delta V_1}{V_1} = 7.5\%$; $\frac{\Delta g}{g} \sim 10\%$;

$$\frac{\Delta R}{R}(r) = \frac{\frac{dR}{R}}{\frac{dr}{r}}\left(\frac{\Delta r}{r}\right) = \frac{\left(\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g(t_0)} + .4\ln\frac{t_1}{t_0}\right) \times dr}{\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g(t_0)}(.31g(t_0)+r) + .4r\ln\frac{t_1}{t_0}} \times \frac{r}{dr}\left(\frac{\Delta r}{r}\right) = .95 \times \left(\frac{\Delta r}{r}\right) = .95 \times 7.5\%$$

$$\frac{\Delta R}{R}(V_1) = \frac{\frac{dR}{R}}{\frac{dV_1}{V_1}}\left(\frac{\Delta V_1}{V_1}\right) = \frac{dV_1}{V_1} \times \frac{V_1}{dV_1} \times \left(\frac{\Delta V_1}{V_1}\right) \text{ - - - - - - - - - } = 1.0 \times \left(\frac{\Delta V_1}{V_1}\right) = 1.0 \times 7.5\%$$

$$\frac{\Delta R}{R}(g) = \frac{\frac{dR}{R}}{\frac{dg}{g}}\left(\frac{\Delta g}{g}\right) = \frac{\left(\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g^2} \times r\right) \times dg}{\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g}(.31g+r) + .4r\ln\frac{t_1}{t_0}} \times \frac{g}{dg} \times \frac{\Delta g}{g}\Big) \cong .09 \times \left(\frac{\Delta g}{g}\right) \cong .10 \times 10\%$$

[computed for: $t_0 \cong 60$ days, $t_1 \cong 18000$ days, $g(t_0) \cong 0.85$, $r \cong 0.45$]

To be on the conservative side, the basic creep curves have been plotted for $r = 0.50$ and $V_1 = 0.30$. According to the above, estimated creep values will be found about $.95 \times 10 + 1.0 \times 3 \cong 13\%$ to the safe side of the probable values, with an average deviation of approximately: $\sqrt{10^2 + 7.5^2 + 7.0^2} \cong 14\%$ ($\frac{\Delta g}{g}$ is already included in the 10%)

As 13% ≈ 14%, the curves will represent the upper limit of the confidence interval for equation [R]. The probable overestimation of creep values will be ~ $2/3 \times 13 \approx 10\%$.

Curves have been plotted for $t_0 = 8, 22, 30, 60, 90, 135$ & 180 days using the corresponding values for $g(t_0)$ taken from Hummel's diagram and $r = 0.50$, $V_1 = 0.30$. The equation is: ?

$$\frac{\varepsilon_{creep}}{\sigma} \times 10^7 = \frac{0.18\,g(t_0) + .28}{g(t_0)}\left(1 - e^{-\frac{t_1-t_0}{30}}\right) + 0.6\ln\left(\frac{t_1}{t_0}\right) \qquad [S]$$

After about the first 90 days of loading, delayed elasticity has lost its effect on rate of creep and the viscous properties govern.

1.017/42

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX B Port Hueneme Test Data	SHEET NO. B1 OF ... BY CS DATE 20.6.68 CHK'D. H.A. DATE 24.6.68

Shrinkage.

The maximal average shrinkage on 11 beams after 2200 days exposure was measured on bottom fibres at 445 μinch/inch ~ 0.0005.

Volume/surface ratio for these specimens was 2.13. Computed by the recommended formula, shrinkage after 2200 days being practically the same as final shrinkage, it is found:

$$\varepsilon_{s,2200} \approx \varepsilon_{s,\infty} = 1200 \cdot e^{-0.213} = 970\ \mu inch/inch \sim 0.0010$$

Comparison of these values shows that the computed estimate is conservative, and that large percentual deviations may be expected on the prediction of small shrinkage values.

Creep.

The recorded creep observations include data on conditions governed by creep recovery, cracked section and tensile stresses, and stress patterns at so low a level as not to be comparable to the creep conditions of the structure in question. The following two cases are considered representative and have been investigated for this comparison:

A. Creep strains at center of gravity of steel (cgs) on 6 beams
Load: DL + 0% LL, t_o = 8 days, E_c(8 days) = 3.36×10^6 psi, σ_c = 1330 psi;

B Differential creep at top fibre, 2×6 beams, Load: (DL + 100% LL) − DL,
t_o = 22 days, E_c(22 days) = 3.36×10^6 psi, $\Delta\sigma$ = 1880 − 50 = 1830 psi;

The reported observations have been averaged for 6 (or 2×6) beams and tabulated below. The following conservative approximations have been made in evaluating the basic creep component from the test data:

1. Mean values of average shrinkage have been used for shrinkage reduction.
2. No reductions have been made for plastic set.
3. The observed values (at cgs) of non linearly distibuted overall creep effect (partial load removal at cgs (DL + DL + LL)) have been referred to cgs by being takin into account with 2/3 of the actual values.
4. Volume/surface ratio is 1.55" for top flanges. 1.85" has been used.

Hansen & Mattock's formula determines basic creep from specific creep:

$$\phi_{(v/s)} = \phi_{basic} + 1.77 e^{-0.54 (v/s)}; \quad \phi_{basic} = \phi_{2.13} - 0.56 = \phi_{1.85} - 0.65;$$

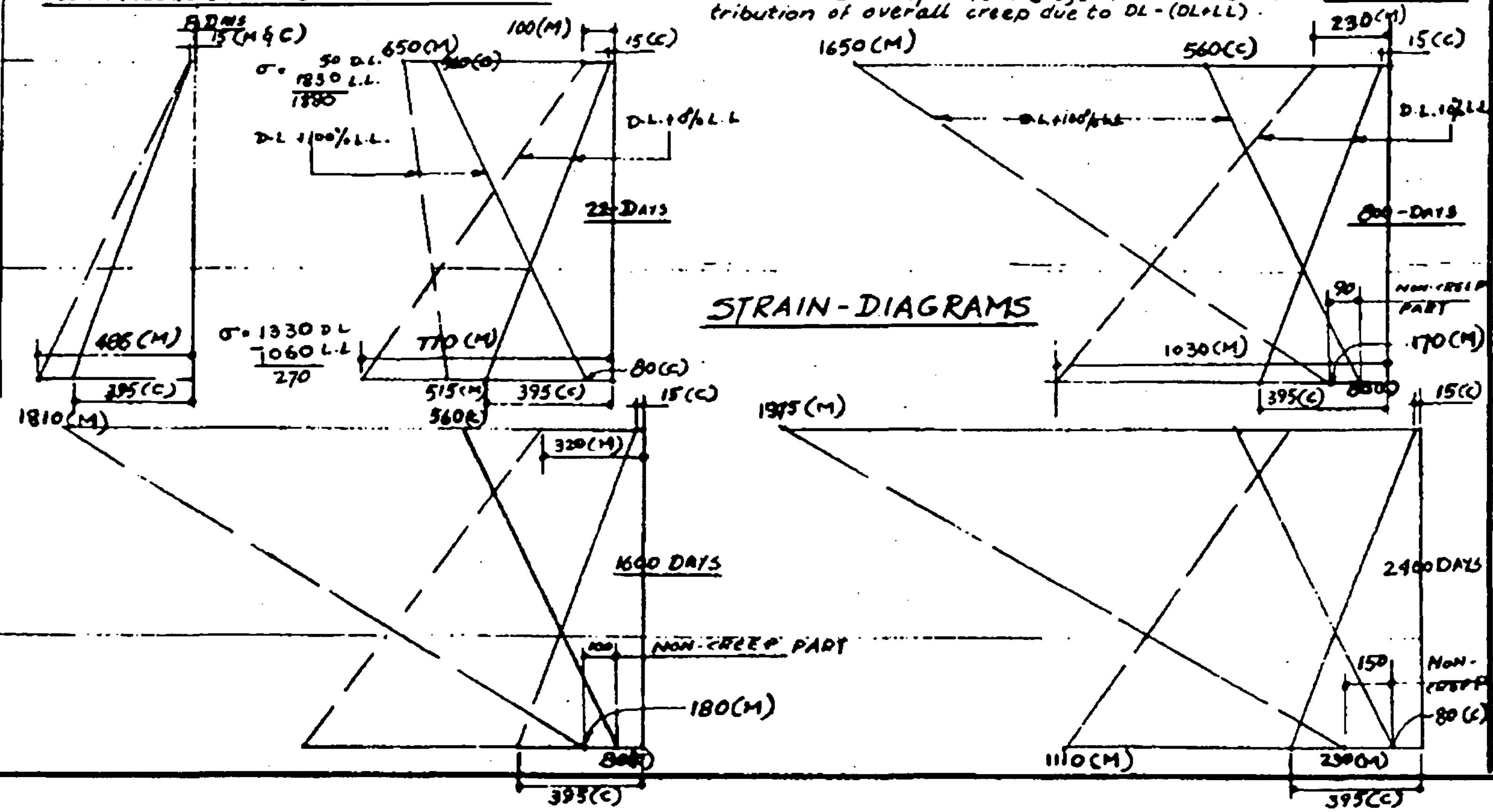
Specific basic creep curves as determined are plotted, see graph, appendix C.

TABULATION OF TEST DATA @ CGS. (DL+100%LL)

	8 d.	(8-22)	22 days		(22-800)	800 days	(800-1600)	1600 days	(1600-2400)	2400 days	NOTES
	1. DL-M	2. DL-M	3. 1.+2.	4. LL-C	5. DL&LL-M	6. 3.+4.+5.	7. DL&LL-M	8. 6.+7.	9. DL&LL-M	10. 8.+9.	M=measured
PRELOAD (GROUTED)	480	250	730	-270	-280	180	-250	210	-280	180	C=calculated strains in μinch/inch
PRELOAD (NON-GROUTED)	495	295	790	-240	-280	270	-310	240	-150	400	
ROEBLING (NON-GROUTED)	475	310	795	-250	-470	65	-450	95	-430	115	shrinkage values are deducted from M.
AVERAGE VALUES	485	285	770	-255 *)	-345	170 **)		180 **)		230 **)	

*) Instantaneous elastic strain = 770-255 = 515.

**) Retardanded creep effect @cgs of non-linear distribution of overall creep due to DL-(DL+LL).



SCHUPACK AND ASSOCIATES
CONSULTING ENGINEERS
STAMFORD, CONN.

APPENDIX B
Port Hueneme Test Data

SHEET NO. B2 OF
BY H.A. DATE 24.6.68
CHK'D CS DATE 25.6.68

1.01.7/43

1.017144

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX B Port Hueneme Test Data	SHEET NO. 83 OF BY H.A. DATE 24.6.68 CHK'D [illegible] DATE 25.6.68

Total @ C.G.S., average of 6 beams, V/S = 2.13.

Conc. Age →	8 DAYS	22 DAYS	800 DAYS	1600 DAYS	2400 DAYS
$\varepsilon_{TOT} - \varepsilon_{SHRINK}$ $\times 10^6$	485	770	1030	1065	1110
$\varepsilon_{elast.\ instant.}$ (1300 p.s.i.) $\times 10^6$	395	395	395	395	395
ε_{creep} $\times 10^6$	0 90	375	635	670	715
$\phi_{2.13} = \varepsilon_c / \varepsilon_{elast.}$	0.229	0.948	1.605	1.695	1.810
$\phi_{2.13} - \phi_{BASIC}$	0.56	0.56	0.56	0.56	0.56
EQUIVALENT ϕ_{BASIC}	—	0.338	1.045	1.135	1.250
$\varepsilon_{cr}/_{p.s.i.} = \phi_{BAS}/E_c$ $\times 10^6$	0	0.115	0.311	0.338	0.372

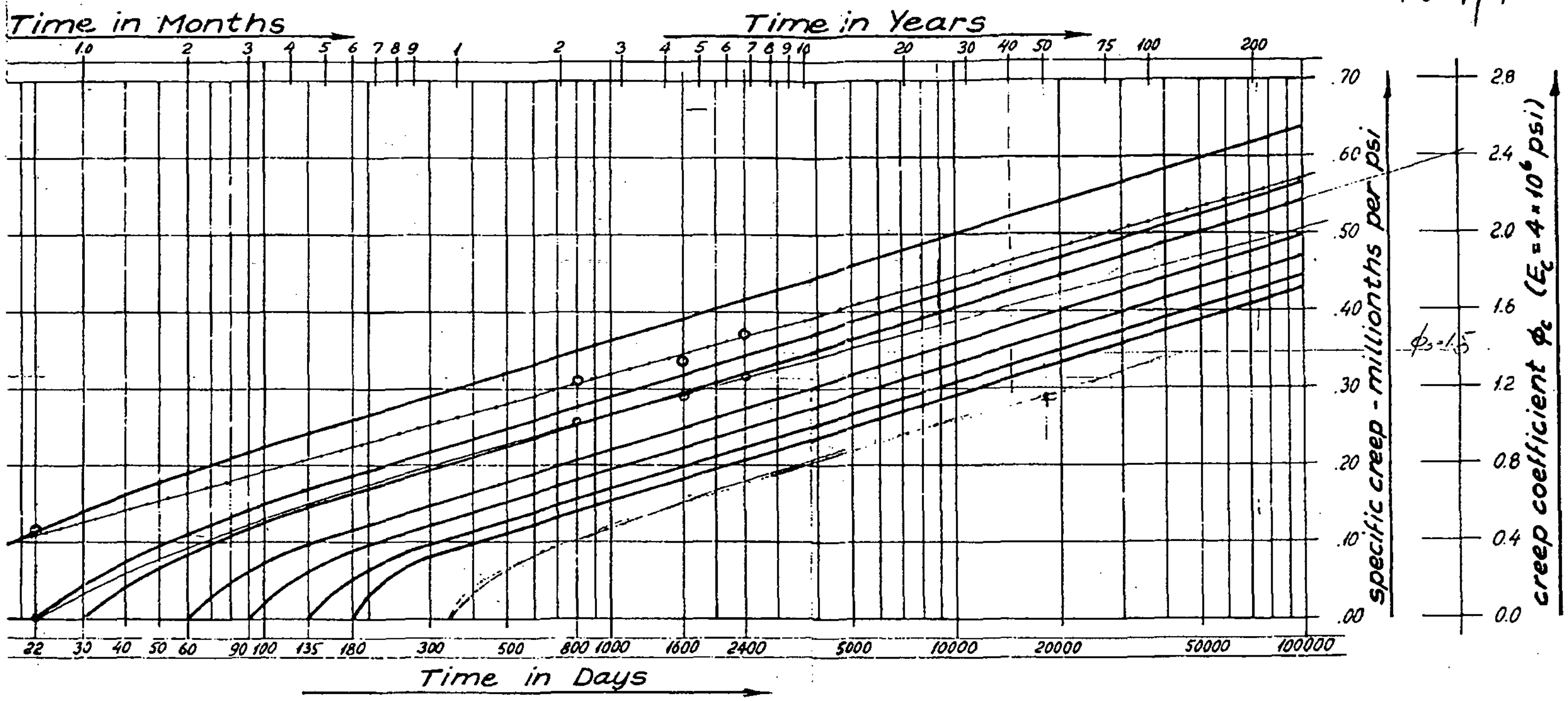
Differential creep @ top fibre, average 2×6 beams, V/S ≈ 1.85

CONCRETE AGE →	22 DAYS	800 DAYS	1600 DAYS	2400 DAYS
$\varepsilon_{TOT.(D+L)} - \varepsilon_{SHRINK}$ $\times 10^6$	650	1650	1810	1975
$\varepsilon_{TOT.(D)} - \varepsilon_{SHRINK}$ $\times 10^6$	100	230	320	405
$\Delta\varepsilon_{TOTAL}$ (L) $\times 10^6$	550	1420	1490	1570
$\Delta\varepsilon_{elast.}$ (L) 1830 p.s.i. $\times 10^6$	545	545	545	545
$\Delta\varepsilon_{creep}$ (L) $\times 10^6$	5	875	945	1025
CORRECTION FOR AGE-LOADING $\times 10^6$	-5	-5	-5	-5
$\Delta\varepsilon_{CREEP}$ (L)	0	870	940	1020
*OTHER NON-$\varepsilon_{cr(BASIC)}$ IN $\Delta\varepsilon_{cr}$ VALUES	—	60	65	100
$\Delta\varepsilon_{CREEP}$ (L) $\times 10^6$	0	810	875	920
$\phi_{1.85} = \Delta\varepsilon_c / \varepsilon_{elast.}$	0	1.490	1.610	1.690
$\phi_{1.85} - \phi_{BASIC}$	—	0.65	0.65	0.65
Equivalent ϕ_{BASIC}	—	0.84	0.96	1.04
$\Delta\varepsilon_c/_{p.s.i.}$ $\times 10^6$		0.250	0.286	0.310

*OVERALL EFFECT OF NON-CREEP PART ~ 2/3 EFFECT @ C.G.S (SEE STRAIN DIAG)

Basic Creep for 5000 psi Concrete

1.01.7) 45.



— Total creep @ cgs ...., Port Hueneme Tests 1); age @ loading: 8 days ;

— Differential creep, top fibre, —"— —"— —"— 1); —"— @ —"— : 22 —"— ;

— Estimated basic creep 2) for loading ages: 8 & 22 days and 1, 2, 3, 4½ & 6 months

SCHUPACK & ASSOCIATES
CONSULTING ENGINEERS - STAMFORD, CONN.
Job: Oyster Creek Unit 2
By: C.S. Date: May 10, 1968

CRYSTAL RIVER	MADE R.S	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D. T.K.	
SCHUPACK & ASSOCIATES REPORT	CP. DPN. / ENG.	4203-00 (WORK ORDER) — 23 (REV.)
ON CREEP AND SHRINKAGE VALUES	REV. CH. APP. DATE	4-24-69

SUMMARY OF THE "REPORT ON RECOMMENDED CONCRETE CREEP AND SHRINKAGE VALUES FOR COMPUTING PRESTRESSING LOSSES" BY SCHUPACK AND ASSOCIATES, CONSULTING ENGINEERS STAMFORD, CONNECTICUT. DATED JUNE 1968

JOB: OYSTER CREEK NUCLEAR STATION-UNIT 2

TYPE TENDONS: GROUTED TENDONS

CONCRETE SHRINKAGE AND CREEP OVER A 50 YEAR PERIOD AT AN AVERAGE TEMPERATURE OF 85°F ± 10°F TO ESTIMATE PRESTRESS LOSSES

SHRINKAGE STRAIN = 0.0001 INCH/INCH

$= 100 \times 10^{-6}$

A UNI-AXIAL CREEP STRAIN $= 0.4 \times 10^{-6}$ /PSI

WALL

STRESS IN MERIDIONAL DIRECTION = -783.8 PSI

STRESS IN HOOP DIRECTION = -1465.0 PSI

NET CREEP STRAIN IN THE MERIDIONAL DIRECTION

$= 0.4 \times 10^{-6}(-783.8 + \frac{1}{6} \times 1465)$

$= 0.4 \times 10^{-6}(-783.8 + 244.2)$

$= 0.4 \times 10^{-6}(-539.6)$

$= 216 \times 10^{-6}$

NET CREEP STRAIN IN THE HOOP DIRECTION

$= 0.4 \times 10^{-6}(-1465 + \frac{1}{6} \times 783.8)$

$= 534 \times 10^{-6}$

CRYSTAL RIVER	MADE RS	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D. [illegible]	
SCHUPACK & ASSOCIATES REPORT	SQ. CP. / CP. DPN. / ENG.	WORK ORDER 4203-00 · SIZE 212 · DRAWING · REV. 24
ON CREEP AND SHRINKAGE VALUES	REV. CH. APP. DATE	4-24-69

DOME

STRESS IN THE MERIDIONAL DIRECTION = 1648 PSI
STRESS IN THE HOOP DIRECTION = 1579 PSI

NET CREEP STRAIN IN THE MERIDIONAL DIRECTION

$$= 0.4 \times 10^{-6} \left(-1648 + \frac{1}{6} \times 1579\right)$$
$$= 0.4 \times 10^{-6} (-1648 + 263)$$
$$= 0.4 \times 10^{-6} \times 1385$$
$$= 554 \times 10^{-6}$$

NET CREEP STRAIN IN THE HOOP DIRECTION

$$= 0.4 \times 10^{-6} \left(-1579 + \frac{1}{6} \times 1648\right)$$
$$= 0.4 \times 10^{-6} (-1579 + 275)$$
$$= 0.4 \times 10^{-6} (-1304)$$
$$= 522 \times 10^{-6}$$

CREEP LOCATION	GAI SPEC.	SCHUPACK RECOMM.	SCHUPACK RECOMM.*
VERTICAL	200×10^{-6}	216×10^{-6}	162×10^{-6}
HOOP	300×10^{-6}	534×10^{-6}	401×10^{-6}
DOME	350×10^{-6}	554×10^{-6}	416×10^{-6}

TYPE OF STRAIN	GAI SPEC.	SCHUPACK RECOMM.
SHRINKAGE STRAIN	120×10^{-6}	100×10^{-6}

* BASED ON UNI-AXIAL CREEP STRAIN 0.3×10^{-6} INCH/INCH PER PSI (VALUE OBTAINED BASED ON THE ASSUMPTION CONCRETE AGE AT POST TENSIONING IS 1 YEAR)

1.01.7/48

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. F.F.	
REACTOR BUILDING	BD. CF.	
REVISED PRESTRESS REQUIREMENTS	CF. DFN.	4203-00 212 — REV. 25
	ENG.	WORK ORDER / SIZE / DRAWING / REV.
	REV. CH. APP. DATE	4-25-69

WALL

VERTICAL DIRECTION

TOTAL FORCE REQD = 175,200

NO OF TENDONS USED = 144

FORCE / TENDON = 1216 KIPS

AREA OF STEEL REQD = 1216 / (153.6 × 0.95) [146.0]

= 8.337 □" [8.648]

AREA OF EACH WIRE = 0.0597 □"

∴ NO OF WIRES REQD = 8.337 / 0.0597 [8.648]

= 139.6 WIRES [144.8]

= 140 WIRES (SAY) [145]

HORIZONTAL DIRECTION

FORCE REQD / FT. = 824.9 KIPS/FT.

SPACING OF TENDON = 19½"

LET F FORCE REQD / TENDON

$\frac{F}{824.9} \times 12 = 19.5$

$F = \frac{19.5 \times 824.9}{12}$

= 19.5 × 68.74

= 1341 KIPS

AREA OF STEEL REQD = $\frac{1341}{0.95 \times 149.5}$

= 9.441 □"

NO OF WIRES REQD = 9.441 / 0.0597

= 158.1 = 159 (SAY)

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. W.J.L.	
REACTOR BUILDING	GQ. CF.	
DOME PRESTRESSING	CF. DFN.	WORK ORDER 4203-00 / SIZE / DRAWING -26- / REV.
	ENG.	
	REV. CH. APP. DATE	7-18-69

TENDON DESCRIPTION

STABILIZED WIRE

163 - 0.276" ϕ WIRE ; $A_s = 9.73 \square''$

f_s' = ULTIMATE STRENGTH OF PRESTRESSING STEEL = 240 KSI

LOSSES

AVERAGE AXIAL COMPRESSIVE STRESS IN DOME = 1.53 KSI (TAKEN FROM LEININGER CALCULATION PAGE#TMI-DT-68)

ELASTIC SHORTENING $= \frac{1.53}{2} \times \frac{29}{4}$

$= 5.55$ KSI

CREEP $= .350 \times 10^{-6} \times 29 \times 10^{6}$

$= 10.15$ KSI

SHRINKAGE $= .12 \times 10^{-6} \times 29 \times 10^{6}$

$= 3.48$ KSI

4% RELAXATION LOSS BASED ON $0.6 f_s'$ $= 0.6 \times 240 \times 0.04$

$= 5.76$ KSI

TOTAL LOSSES $= 5.55 + 10.15 + 3.48 + 5.76$

$= 24.94$ KSI

STRESS IN WIRES AFTER LOSSES (UNDER NORMAL WORKING CONDITION)

$= 0.7 f_s' - 24.94$

$= 0.7 \times 240 - 24.94$

$= 168.0 - 24.94$

$= 143.06$ KSI

$0.6 f_s' = 0.6 \times 240 = 144.0$ KSI

FORCE IN EACH TENDON AFTER LOSSES (UNDER NORMAL WORKING CONDITION)

$= 143.06 \times 9.73$

$= 1392$ KIPS

1.01.7/50

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. W.J.L.	
REACTOR BUILDING	BO. CF.	
	CF. DFN.	4203-00 (WORK ORDER)
DOME PRESTRESSING	ENG.	
	REV. CK. APP. DATE	7-18-69

UNDER ACCIDENT CONDITION, THE PRESTRESSING FORCE IS INCREASED DUE TO ELASTIC ELONGATION

STRESS BEFORE ACCIDENT = -1630 PSI D+F

STRESS AFTER ACCIDENT = 56 PSI D+F+1.5P+T_a

Δ STRESS = 1686 PSI

STRESS IN WIRE DUE TO ELASTIC ELONGATION

$= 1.686 \times \frac{29}{4}$

$= 12.22$ KSI ∠ $T_p = T_c$

TOTAL STRESS IN WIRES DURING ACCIDENT

$= 143.06 + 12.22$

$= 155.28$ KSI

ϕ FACTOR $= 0.95$

FINAL FORCE / TENDON $= 0.95 \times 155.28 \times 9.73$

$= 1435$ KIPS

HORIZ. SPACING OF TENDON

(SEE LEININGER PAGE # TMI-DT69)

$= 22.7 \times \frac{1435}{1055}$

$= 30.87''$

$= 30''$ (SAY)

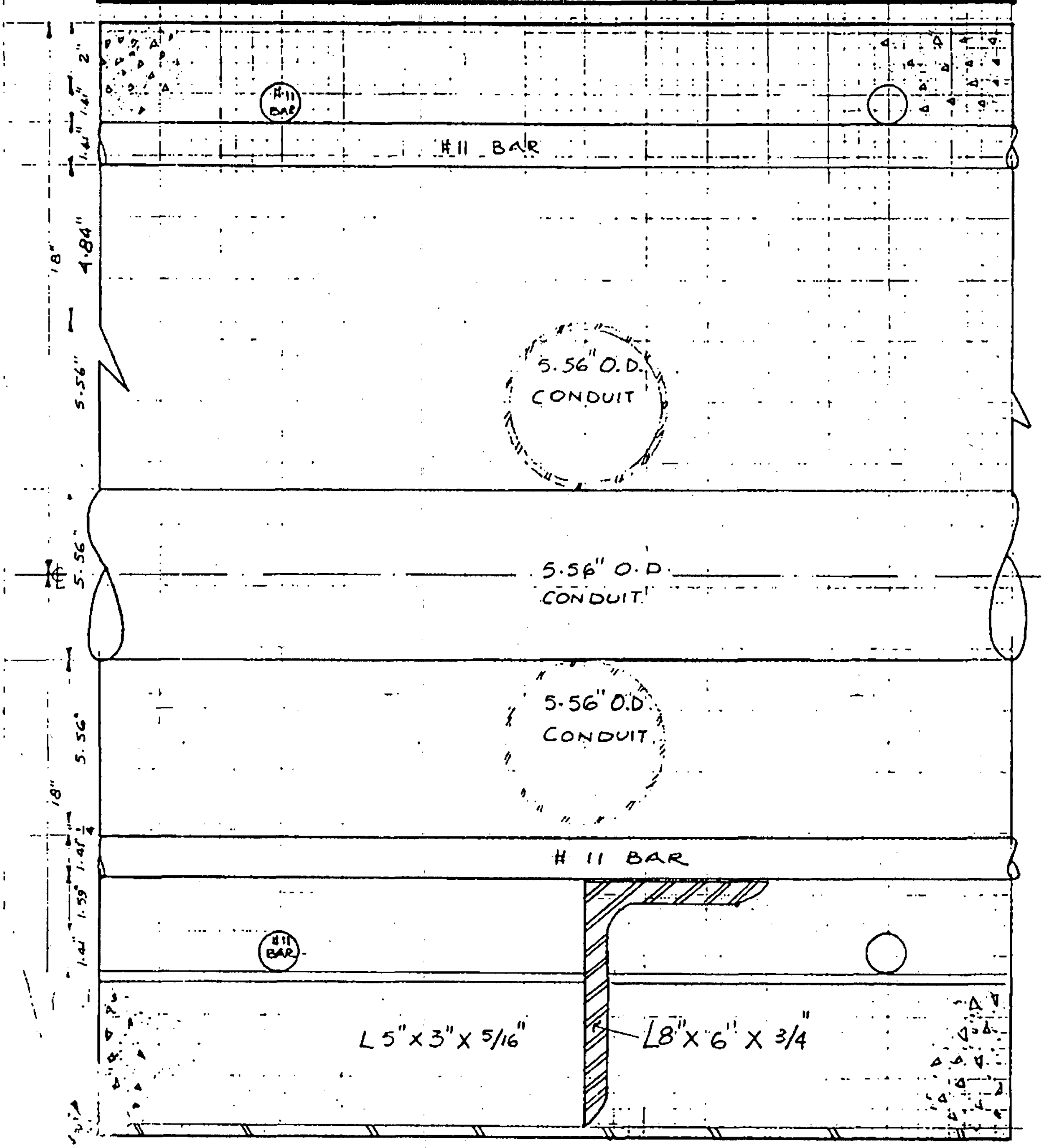
USE 123 TENDONS (41 TENDONS IN EACH OF 3 DIRECTIONS) WITH A CONSTANT HORIZONTAL SPACING OF 30".

℄ OF LAST TENDON IS AT A RADIUS 50'-0".

TANGENT POINT FOR ALL TENDONS IS AT A RADIUS OF 55'-3"

1.01.7|51

FLORIDA POWER CORPORATION	MADE R.S. / CHK'D. F.L. / SQ. CP.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
CRYSTAL RIVER UNIT # 3	CP. DFN.	4203.00 — WORK ORDER / SIZE / DRAWING / REV.
REACTOR BUILDING	ENG.	
CROSS SECTION THROUGH THE DOME	REV. CH. APP. DATE	8-22-69



This copy to: R. Shanmugasundaram

1.01.7 | 52

GILBERT ASSOCIATES, INC.
READING

August 28, 1969

COPY

Chicago Bridge and Iron Company
P. O. Box 13308
Memphis, Tennessee 38113

Attn: Mr. P. Narbut
Engineering Coordinator

RE: Crystal River - Unit No. 3
Florida Power Corporation

Dear Mr. Narbut:

The stiffener angles for the dome, as shown on your design sketches, produce some interference with the tendon conduits and reinforcing bars. We wish to call this to your attention so that adjustments can be made before you transmit your drawings for approval.

The horizontal leg of C-1, C-2 and C-3 (L 8 x 6 x 3/4) causes an interference and if this leg were turned 180° so that the horizontal leg is projected toward the ℄ of the vessel the interference could be avoided.

The horizontal legs of C-4 (2 L's 8 x 6 x 3/4 with a 10" x 1/2" ℄) also cause an interference and we ask that this stiffener be revised so that any projection normal to the surface of the dome does not exceed eight (8) inches.

Enclosed please find a sketch showing the preceding, and in the event that you should have any questions please feel free to contact us.

Very truly yours,

A. G. Benyo

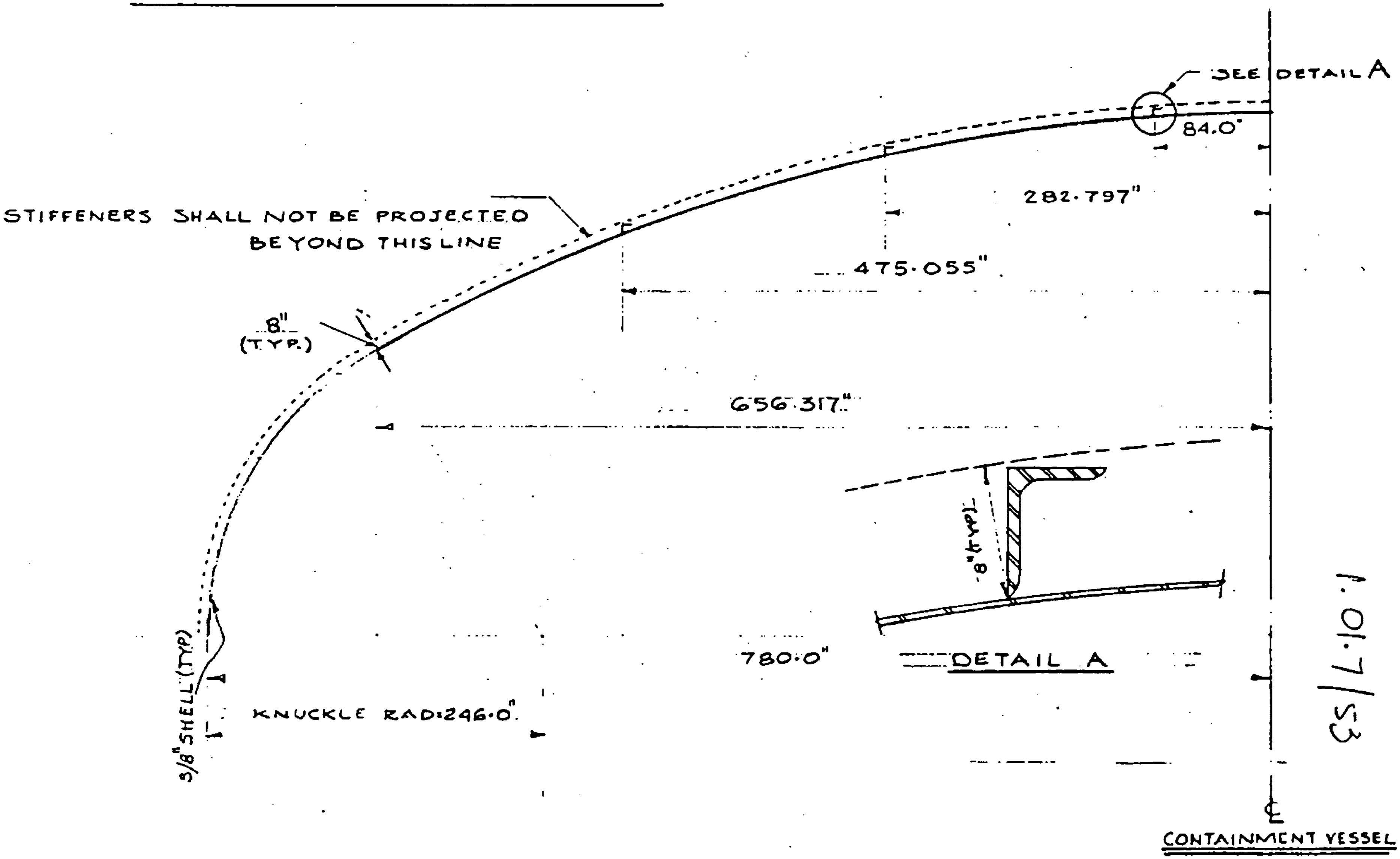
A. G. Benyo
Structural Engineer

gu
Enclosures
cc: W. O. May (2)(w/encl)
R. B. McKnight (w/encl)
W. R. Dreyer (w/encl)
E. R. Hottenstein (2)(w/encl)
R. Villforth
S. N. Dobreff
R. Shanmugasundaram
Dept. File

FLORIDA POWER CORPORATION

CRYSTAL RIVER UNIT #3

CONTAINMENT VESSEL - SPHERICAL HEAD



GAI-R.S.

1.01.7/53

To: Mr. S.N. Dobreff.
FROM: R. Shanmugasundaram
SUBJECT: Crystal River - Reactor Building
Changes required based on the
Final Revised Prestress Calculations.
GAI-4203

Based on the final revised prestress calculations (please see my memo dated April 10 '69), the revised final requirements are compared with the initial requirements which the Prescon, the prestress contractor has followed so far in lating out of the tendons.

PRESTRESS FORCE REQUIRED/FT

Cylindrical Wall	As specified Initial Requirements	Final Requirements
Vertical	457.0 KIPS 11%	414.6 KIPS
Hoop	829.0 KIPS .5%	824.9 KIPS

(163 - 0.276"Φ stabilised wires)
NO. OF TENDONS REQUIRED

Cylindrical Wall	Initial Requirements	Final Requirements
Vertical	144 12%	128
Hoop	282 3.2%	273

NO OF WIRES REQUIRED (IN EACH TENDON) KEEPING THE NUMBER OF TENDONS SAME AS OF THE INITIAL REQUIREMENTS

CYLINDRICAL WALL	INITIAL REQUIREMENTS	FINAL REQUIREMENTS
VERTICAL	163	145
HOOP	163	159

ALL THE ABOVE CALCULATIONS ARE BASED ON THE STABILISED WIRES WITH 4% RELAXATION LOSSES AND SHRINKAGE AND CREEP LOSSES AS GIVEN IN GAI CONTRACT SPECIFICATION. INITIAL EFFECTIVE PRESTRESS IS LIMITED TO 0·6 OF THE ULTIMATE STRENGTH OF PRESTRESSING STEEL AND IS ALLOWED TO INCREASE BEYOND THE ABOVE LIMIT ONLY DURING 1.5 ACC. PRESSURE.

THE FOLLOWING IS THE COMPARISON BETWEEN THE GAI SPECIFICATIONS AND SCHUPACK & ASSOCIATES RECOMMENDATIONS ON LOSSES DUE TO CREEP AND SHRINKAGE

TYPE OF LOSSES	GAI SPEC.	SCHUPACK RECOMM.*
CREEP STRAIN		
VERTICAL	200×10^{-6}	162×10^{-6}
HOOP	300×10^{-6}	401×10^{-6}
DOME	350×10^{-6}	416×10^{-6}
SHRINKAGE STRAIN	120×10^{-6}	100×10^{-6}

* VALUE OBTAINED BASED ON THE ASSUMPTION CONCRETE AGE AT POST TENSIONING IS 1 YEAR.

IN CASE OF DOME PRESTRESSING, THE FINAL SPACING HAS NOT BEEN WORKED OUT FOR CRYSTAL RIVER.

THEREFORE, THE FOLLOWING DECISIONS ARE TO BE MADE.

(1) LOSSES DUE TO, ELASTIC SHORTENING, CREEP AND SHRINKAGE FOR CRYSTAL RIVER CONCRETE.

(2) FINALISATION OF THE NO OF TENDONS IN THE WALL.

R. SHANMUGASUNDARAM
STRUCTURAL ENGINEER.

C.C. D.K. CRONEBERGER
D.A. SKILTON.

CRYSTAL RIVER	MADE A40 / CHK'D. TP. / EQ. CP.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BLDG.	CP. DPN.	
DESIGN CHECK ON PRESCON	ENG.	WORK ORDER / SIZE / DRAWING / REV.
ANCHOR COMPONENTS CALCS	DATE 11-10-69	1.01.7/56a

Check on Prescon calculations for bearing plate stresses. Prescon calculations make no allowance for square plate with a central circular hole. The following analysis is approximate; Prescon should provide more accurate calculations, which must make allowance for some of the loads etc. which occur.

Reference: Roark - "Formulas for Stress and Strain" page 221 formula 16 (free edge) & page 222 formula 21 (fixed inner edge)

Assumptions:

SIZE. actual bearing plate = 24"×24"×3".
assume bearing plate 24" ϕ × 3"

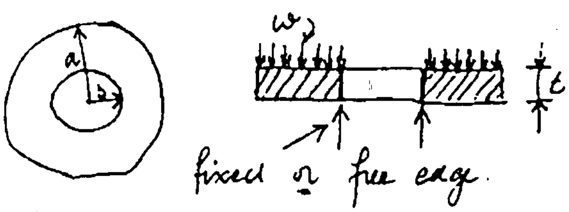
HOLE inner hole ϕ = 11"
outer ϕ of shims = 13½"
assume edge of hole at ϕ = 11" is area of maximum stresses.

PRESSURE at 70% GUTS average bearing pressure = 3400 #/□"
assume pressure acting uniformly under the 24"ϕ circular bearing plate is 3400 #/□" [note: this gives a < tendon force than 70%]

POISSONS RATIO assume $\nu = 1/3$

$$\therefore m = \frac{1}{\nu} = \frac{1}{1/3} = 3$$

From the above $a = 12"$ $b = 5.5"$ $t = 3"$ $m = 3$ $w = 3400$ #/□"



CRYSTAL RIVER	MADE: [illegible]	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BLDG.	CHK'D. [illegible]	
DESIGN CHECK ON PRESCON	ENG.	WORK ORDER / SIZE / DRAWING / REV.
ANCHOR COMPONENTS CALCS	DATE 11-10-69	1.01.7/57

Refer to letter of Oct. 20 1969

(A) INNER EDGE FREE:

from Roark

$$S = \frac{3w}{4mt^2(a^2-b^2)}\left[4a^4(m+1)\log\frac{a}{b} + 4a^2b^2 + b^4(m-1) - a^4(m+3)\right]$$

$$= \frac{3\times3400}{4\times3\times3^2(12^2-5.5^2)}\left[4\times12^4(3+1)2.3026\log_{10}\frac{12}{5.5} + 4\times12^2\times5.5^2 + 5.5^4(3-1) - 12^4(3+3)\right]$$

$$= \frac{3\times3400}{4\times3\times3^2\times17.5\times6.5}\left[7.65\times10^5\times\log_{10}2.18 + 1.74\times10^4 + 1.83\times10^3 - 1.245\times10^5\right]$$

$$= 8.3\times10^{-1}\left[7.65\times0.338\times10^5 + 17.4\times10^3 + 1.83\times10^3 - 124.5\times10^3\right]$$

$$= 8.3\times10^{-1}\left[259\times10^3 + 19.23\times10^3 - 124.5\times10^3\right]$$

$$= 8.3\times10^{-1}\left[278.23\times10^3 - 124.5\times10^3\right]$$

$$= 8.3\times10^{-1}\times153.73\times10^3$$

$$= \underline{127,500}\ \#/\square''$$

(B) INNER EDGE FIXED:

from Roark

$$S = \frac{3w}{4t^2}\left[\frac{4a^4(m+1)\log\frac{a}{b} - a^4(m+3) + b^4(m-1) + 4a^2b^2}{a^2(m+1) + b^2(m-1)}\right]$$

$$= \frac{3\times3400}{4\times3^2}\left[\frac{4\times12^4(3+1)2.3026\log_{10}\frac{12}{5.5} - 12^4(3+3) + 5.5^4(3-1) + 4\times12^2\times5.5^2}{12^2(3+1) + 5.5^2(3-1)}\right]$$

using arithmetic from above

$$= 283\left[\frac{259\times10^3 - 124.5\times10^3 + 1.83\times10^3 + 17.4\times10^3}{576 + 60.5}\right]$$

$$= \frac{283\times153.73\times10^3}{636.5}$$

$$= \underline{68,300}\ \#/\square''$$

SUMMARY: From the above, the approx. max. stress in the bearing plate is midway between the two values, ie 97,900 #/□". This is greater than the min. ultimate for the C1045 steel. As this analysis is not exact, Prescon's calcs should allow for friction at back of plate; edge restraint of the bearing plate; friction and load transfer along trumpet; non-uniform pressure distribution; as well closer approximations for diameter of hole diameter

CRYSTAL RIVER	MADE DJ	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
	CHK'D. F.P.	
REACTOR BLDG.	SQ. CP.	
"INFORMAL" DRL QUESTION 2	CP. DPN. / ENG.	WORK ORDER / SIZE / DRAWING / REV.
FPC LETTER 10-23-69.	DATE 10-24-69	1.01.7/58

Level p for prestressing wire to be used in the containment structure.

Ⓐ VERTICAL TENDONS:

no tendons = 144

℄ radius = 65'-0 3/8 + 1'-9" = 66'-9 3/8" r. = 798 3/8" r.

thickness of wall = 3'-6" = 42"

A_s = 9.75 □"/ tendon

$$\therefore \; p = \frac{A_s}{bd} = \frac{9.75 \times 144}{2\pi\, 798^{3/8} \times 42}$$

$$= 0.00667$$

say 0.0067

Ⓑ HOOP TENDONS

Tendons spaced alternately at 13" & 26" crs.

ave. spacing = $\frac{13+26}{2}$ = 19.5"

$\therefore$ b = 19.5" d = 42"

$$\therefore \; p = \frac{9.75}{19.5 \times 42} = 0.0119$$

Ⓒ DOME TENDONS

no. of tendons = 3 × 41 = 123

at 60'-0" radius circle periphery = 2π 60 × 12 = 4520" b.

d = 3'-0" = 36"

$$\therefore \; p = \frac{123 \times 9.75}{4520 \times 36}$$

$$= 0.00737$$

say 0.0074

CRYSTAL RIVER	MADE DG CHK'D. FP.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.	
REACTOR BLDG.	SQ. CP. CP. DPN.		
"INFORMAL" PRL QUESTION 9	ENG.	WORK ORDER / SIZE / DRAWING / REV.	
FPC LETTER 10-23-69	DATE 10-24-69		1:01.7/59

Density of reinforcement in Test Block (Prescon) and Buttress.

① 4'×4'×4' PRESCON TEST BLOCK

From page 15 of Prescon's "Tentative Test Schedule and Test Program", is taken the following quantity of rebar.

MK 1000	no = 5	size = 1¼"	l = 4×3'-9" = 15'-0"	W = 5×15×4.303	= 324#
MK 1001	no = 12	size : 1¼	l = 2×2'-4"+3'-9" = 8'-5"	W = 12×8.42×4.303	= 435#
MK 400	no = 2	size : ½"	l = 4×3'-9" = 15'-0"	W = 2×15×.668	= 20#
MK 401	no = 10	size : ½"	l = 2'-4"+3'-9"+2'-4" 8'-5"	W = 10×8.42×.668	= 56#
MK 402	no = 2	size : ½"	l = 4×3'-9" = 15'-0"	W = 2×15×.668	= 20#
				Total	= 855#

∴ density of rebar [#/cu. yd.] = $\frac{855 \times 27}{4 \times 4 \times 4}$

= 361 #/cu.yd.

② Using BMI drg no. E421-031 and adjusting for 17% larger tendon on CR3, the total rebar per anchorage in the buttress is: (note tendon anchorages @ 39" crs)

#10 R12 @ 12" crs horiz	l = 22'-0/2 = 11'-0"	W = 4.303 × 11 × 39/12	= 154#
#10 R17 - 4 per tendon	l = 12'-0"	W = 4.303 × 12 × 4	= 207#
#10 R18 - 1 per tendon	l = 13'-0"	W = 4.303 × 13 × 1	= 56#
#10 13 per buttress - say 5/anch	l = 3'-3"	W = 4.303 × 3.25 × 5	= 70#
#5 spiral - 2/tendon	l = 57'-9"	W = 1.043 × 2 × 57.75	= 121#
		Total	= 608#/anch.

add 17% for larger tendon = 608 × 1.17 = 710#

say anchorage is 2.5' × 3.25' × 4' = 32.5 cu. ft

= 1.2 cu yds.

∴ density = $\frac{710}{1.2}$ = 592 #/cu yd > 361 #/cu yd → OK

Density of rebar in buttress > density of rebar in concrete test blocks.

04-4762-016

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: FLORIDA POWER CORP.	FILING CODE: 1.01.7A
	PROJECT: CR 3	W.O. 04 4203016 PAGE 1 OF 11
SYSTEM: REACTOR BUILDING		ORIGINATOR: DATE 5/10/77
CALCULATION FOR: SP-395 TENDON SURVEILLANCE / PRESTRESS LOSSES		REVIEWER: D. B. DATE 10/28/77

RESULTS

INDEX

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT FLORIDA POWER CORP	FILING CODE 1.01.7A
	PROJECT CR3	W.O. OR 4203016 / PAGE 2 of 11
SYSTEM REACTOR BUILDING		ORIGINATOR [signature] DATE 5/10/77
CALCULATION FOR SP-395 TENDON SURVEILLANCE / PRESTRESS LOSSES		REVIEWER D. Bird DATE 10/28/77

RESULTS

OBJECTIVE

DERIVE CURVES OF ESTIMATED PRESTRESS FORCE IN HOOP, VERTICAL, AND DOME TENDON WIRES AT TIMES OF TENDON SURVEILLANCE OVER THE PLANT LIFE. REQUIRED BY CR3 TECH. SPEC. 4.6.1.6.

TO ACCOUNT FOR TENDONS WITH LESS THAN 163 WIRES, THE CURVES WILL BE FOR ONE WIRE.

REFERENCES

1) CR3 REACTOR BUILDING DOME DELAMINATION - FINAL REPORT - DECEMBER 10, 1976 SECTION 4.4. C PRESTRESS LOSSES.

2) INSERVICE SURVEILLANCE OF UNBONDED POST TENSIONING SYSTEMS IN PRESTRESSED CONCRETE CONTAINMENT STRUCTURES. J. V. ROTZ, BECHTEL - PAPER PRESENTED AT UNIV. OF YORK, ENGLAND, 8-12 SEPTEMBER 1975.

3) SCHUPACK & ASSOC. BASIC CREEP FOR 5000 psi CONCRETE. APPENDIX 5J - TMI-2 PSAR

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Florida Power Corp	FILING CODE 1.01.7 A
	PROJECT CR3	W.O. # 9200-016 — PAGE 3 OF 11
SYSTEM Reactor Building		ORIGINATOR [signature] DATE 5/10/77
CALCULATION FOR SP-395 Tendon Surveillance		REVIEWER D.B. DATE 10/28/77

DISCUSSION

THE FIRST DRAFT OF SP-395 INCLUDED TENDON FORCE/TIME PLOTS BASED ON ORIGINAL DESIGN CRITERIA. THOSE CALCULATIONS WERE INFORMALLY REVIEWED BY D. SKILTON.

DURING THE DOME REPAIR THE BASIS OF TENDON PRESTRESS LOSSES WERE REEVALUATED AND INCLUDED IN REFERENCE 1). THIS INFORMATION GAVE PRESTRESS LOSSES AT THE TIME OF THE SIT (WITHIN A FEW MONTHS) AND AT 40 YEARS. AS THIS WAS PRESENTED TO THE NRC AND IS PUBLIC RECORD, IT PROPERLY SHOULD BE THE BASIS FOR DEVELOPMENT OF WIRE FORCE/TIME CURVES.

RESULTS

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Florida Power Corp	FILING CODE 1:01.7A
	PROJECT CR3	W.O. 04 4203-016 PAGE 4 OF 11
SYSTEM Reactor Building		ORIGINATOR [signature] DATE 5/16/77
CALCULATION FOR SP-395 Tendon Surveillance		REVIEWER D. Bus DATE 10/20/77

RESULTS

METHOD

PRESTRESS LOSSES CONSIDER FOUR LOSSES FROM THE INITIAL LOCK OFF OF THE TENDON AT 70% OF GUTS (240 KSI) WHICH IS 168 KSI. THOSE LOSSES ARE:

1) ELASTIC SHORTENING
THIS IS A FUNCTION OF TENDON STRESSING SEQUENCE. THE VALUES PRESENTED IN REF 1) ARE THE AVERAGE. THE FIRST TENDON STRESSED WOULD EXPERIENCE TWICE THE VALUE AND THE LAST WOULD EXPERIENCE NONE. THEREFORE THIS HAS IMPUT INTO THE MAX., AVG., AND MIN. EXPECTED LOSSES.

2) CREEP
CREEP IS TIME DEPENDENT, AND THE VALUES CALCULATED FOR ONE YEAR AFTER THE SIT USE REF. 1) VALUES CORRECTED BY REF 3).

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: FLORIDA POWER CORP	FILING CODE: 1:01.7A
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3) STEEL RELAXATION

STEEL RELAXATION IS TIME DEPENDENT WITH SMALL CHANGE FROM SIT TO ONE YEAR AFTER SIT. SEE REF 1) FOR VALUES.

4) SHRINKAGE

SHRINKAGE LOSSES IN REF 1) WERE CONSIDERED AS OCCURING FROM PLACEMENT OF CONCRETE TO TIME OF SIT ONLY. DUE TO VOLUME / THICKNESS NOT CONSIDERED TO OCCUR OVER PLANT LIFE - REF 1).

RESULTS

FILING CODE

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GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: Florida Power Corp	FILING CODE: 1:01.7A
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SYSTEM: Reactor Building		ORIGINATOR: DATE 5/10/77
CALCULATION FOR: SP-395 Tendon Surveillance		REVIEWER: D. Bias DATE 10/28/77

CALCULATIONS

From Dome Delamination Report (Page 4-3)

c. Prestress Losses

The calculated prestress losses (ksi) and effective prestress (ksi) are given below:

Present (SIT)	Elastic Shortening	Creep	Steel Relaxation	Shrinkage	(AVG.) Total Losses (KSI)
Vertical	3.6 (0-7.2)	3.9	2.2	2.9	12.6
Hoop	6.4 (0-12.8)	7.0	2.2	2.9	18.5
Dome	10.4 (0-20.8)	11.3	2.2	2.9	26.8

RESULTS: (165 ksi − losses) Effective Prestress (ksi)

One yr. after SIT	Elastic Shortening	Creep	Steel Relaxation	Shrinkage	Total Losses Min	Avg.	Max	Effective Prestress Max	Avg.	Min
Vert.	(0-7.2)	4.7	2.3	2.9	9.9	13.5	17.1	158.1	154.5	150.9
Hoop	(0-12.8)	8.4	2.3	2.9	13.6	20.0	26.4	154.4	148.0	141.6
Dome	(0-20.8)	14.9	2.3	2.9	20.1	30.5	40.9	147.9	137.5	127.1

40 yr.	Elastic Shortening	Creep	Steel Relaxation	Shrinkage	Total Losses Min	Avg.	Max	Effective Prestress Max	Avg.	Min
Vertical	3.6 (0-7.2)	9.1	3.4	2.9	15.4	19.0	22.6	152.6	149.0	145.4
Hoop	6.4 (0-12.8)	16.2	3.3	2.9	22.4	28.8	35.2	145.6	139.2	132.8
Dome	10.4 (0-20.8)	26.2	3.4	2.9	32.5	42.9	53.3	135.5	125.1	114.7

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Florida Power Corp	FILING CODE 1.01.7A
	PROJECT CR3	W.O. 04 4203-016 PAGE 7 OF 11
SYSTEM Reactor Building		ORIGINATOR [signature] DATE 5/10/77
CALCULATION FOR SP-395 Tendon Surveillance		REVIEWER D. N. Siso DATE 10/28/77

FORCE PLOTTED FOR ONE WIRE 7mm
AREA = 0.05965 SQ IN.

TOLERANCE ON JACKING TO BE ± 5%
APPLIED TO MAX & MIN. FORCE

PLOT UPPER, AVERAGE, LOWER.

One Yr After SIT	1.05 × Area × Eff. MAX.	Area × Eff. AVG	.95 × Area × Eff. MIN
Vert	9.90 K	9.22 K	8.55 K
Hoop	9.67	8.83	8.02
Dome	9.26	8.20	7.20

40 YR			
Vert	9.56	8.89	8.24
Hoop	9.12	8.30	7.53
Dome	8.49	7.46	6.50

RESULTS

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Florida Power Corp.	FILING CODE 1.01.7A
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SYSTEM Reactor Building		ORIGINATOR [signature] DATE 5/10/77
CALCULATION FOR SP 395 Tendon Surveillance		REVIEWER D. Bus DATE 10/28/77

RESULTS

To verify use of straight line plot on semi-log paper, calculate wire force for hoop tendon at ten years - average - and compare to plotted value:

10 YEAR

Elastic Short. (Avg.)	6.4 KSI
Creep $\frac{3.59}{6.19} \times 9.2 + 7.0$ =	12.3
Steel Relax. $\frac{3.3-2.2}{4} + 2.2$ =	2.5
Shrinkage	2.9
Total	24.1 KSI

Wire Force = $0.05965 \times (168 - 24.1) = 8.58^K$

Graph reading at 10 years = 8.50^K

$\% \text{ Diff.} = \frac{.08}{8.50} \times 100 = 0.9\%$

∴ Confirms use of semi log and straight line.

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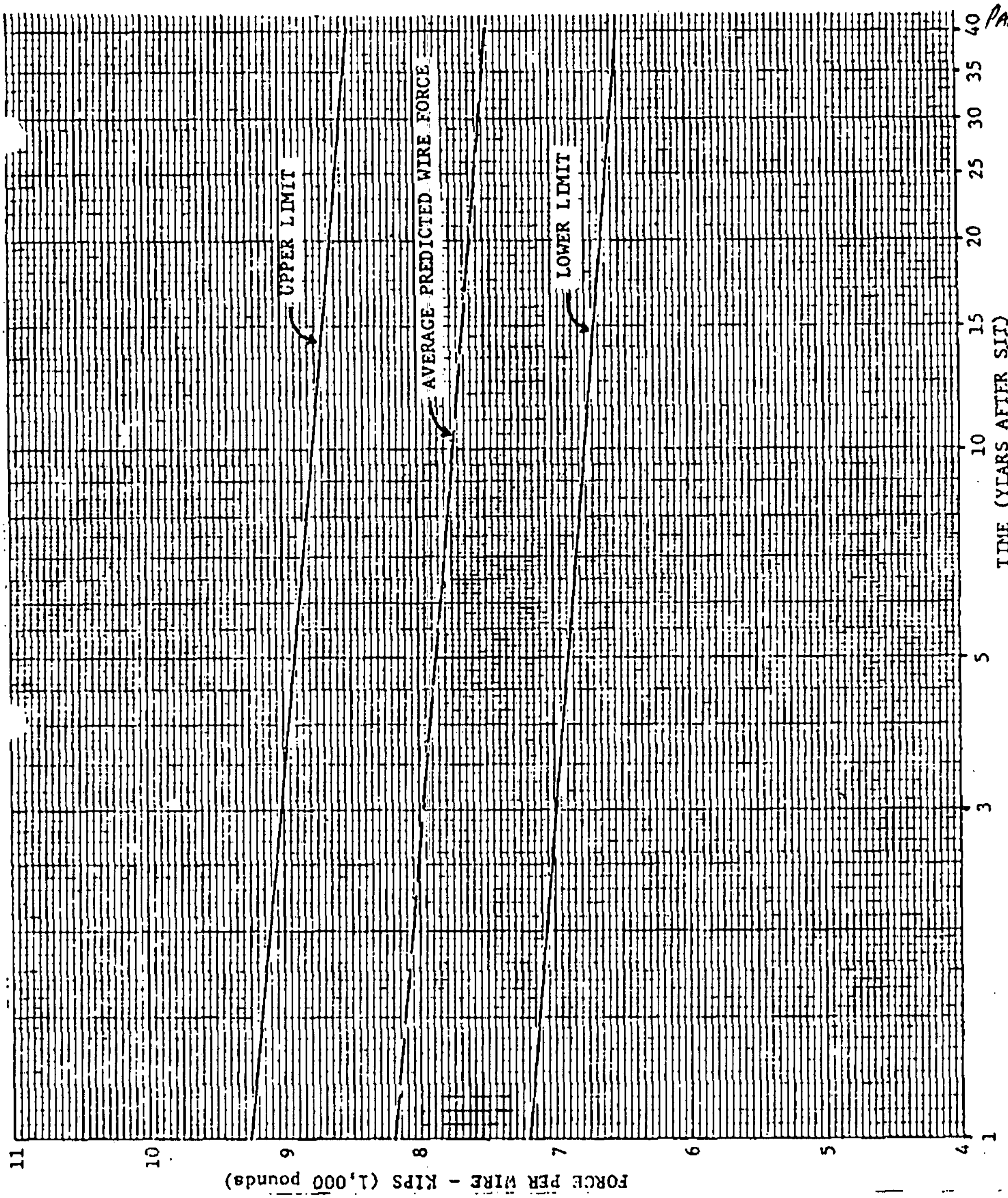

FIGURE 10
WIRE FORCE VS. TIME
DOME TENDONS

FC 1:01.7A
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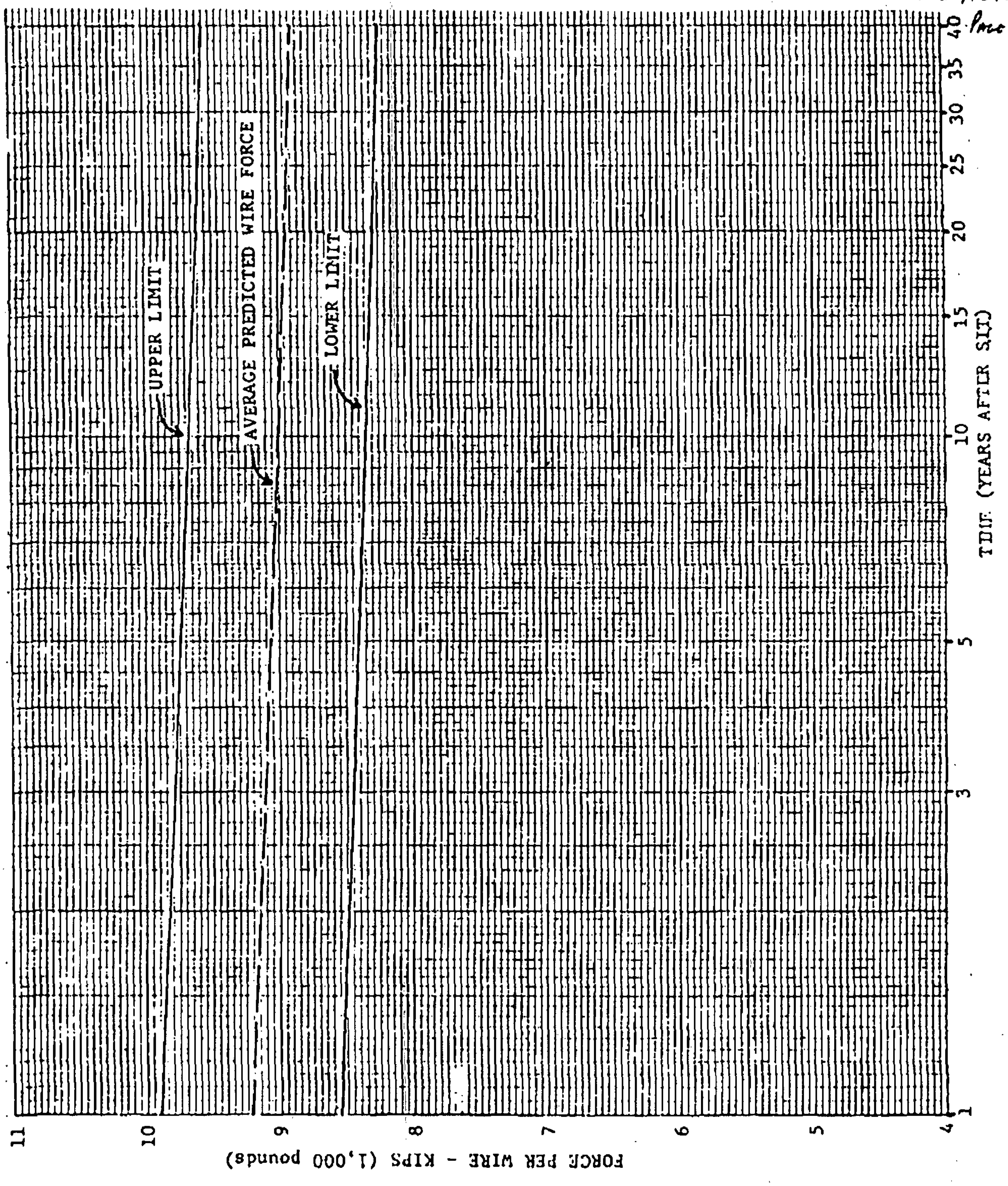

FIGURE 11
WIRE FORCE VS. TIME
VERTICAL TENDONS

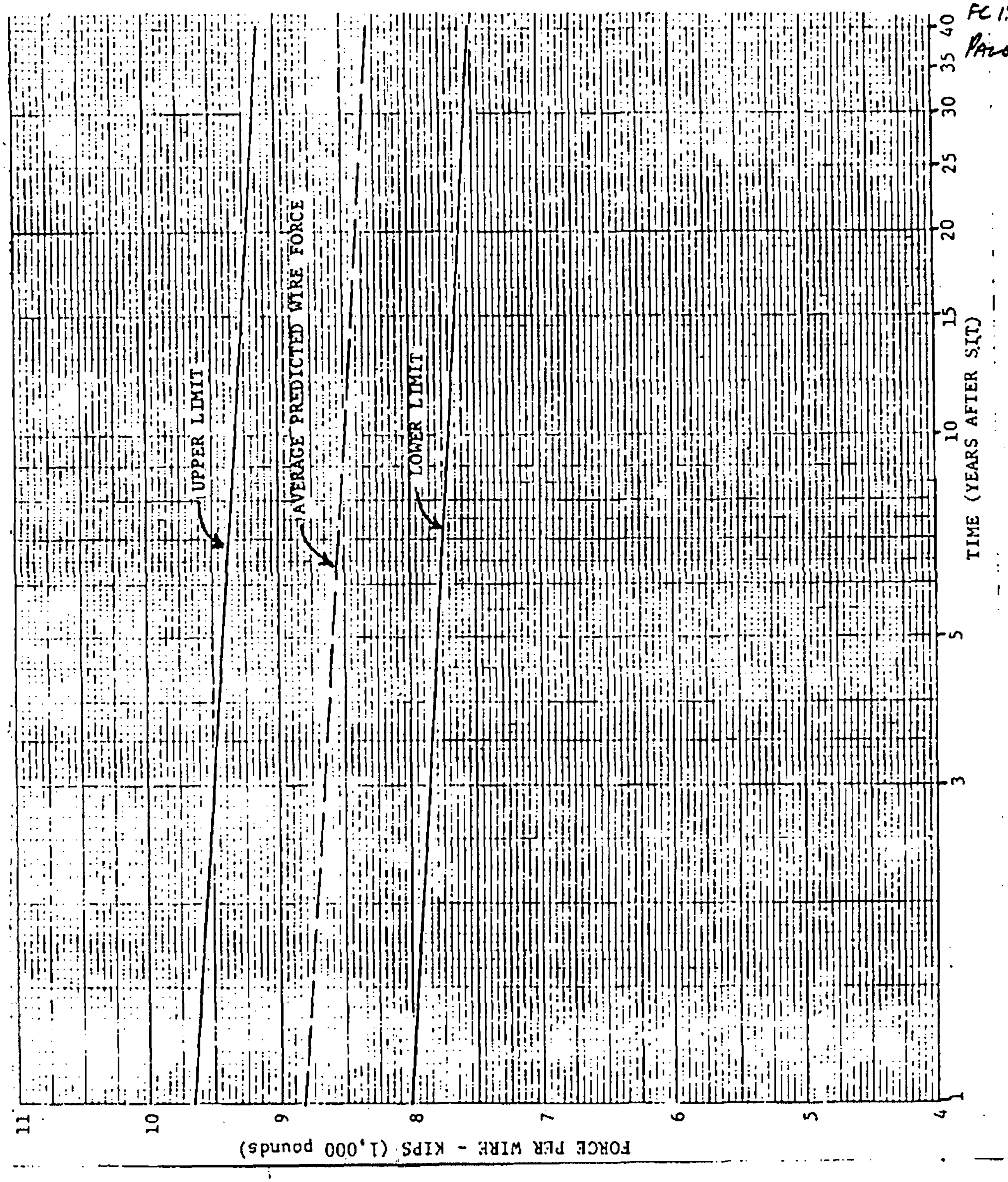

FIGURE 12
WIRE FORCE VS. TIME
HOOP TENDONS

SH. 12

memorandum Gilbert/Commonwealth

AUG. 23, 1977

to: S. N. Dobreff

from: M. R. Wardrop

subject: CR#3
Request for Approval of Design Tasks Requiring Verification

The following list of Design Tasks, which identifies safety related and non-safety related items and defines those which require Design Verification, has been prepared for Crystal River Unit #3 and is hereby submitted for your approval as required by Sections 5.1 and 5.5.1 of DCP 4.2.1:

Filing Code	W. O. Number Name of Task	Safety Class (1)	Verification Required (2)	Reason for Verification (3)
1:01.7A	04-4762-016 TENDON SURVEILLANCE PRESTRESS LOSSES	SR	YES	NRC

(1) SR = Safety Related NSR = Non-Safety Related
(2) Yes or No
(3) NRC = Nuclear Regulatory Commission GAI = Gilbert Associates, Inc.

Distribution:

F. J. Tomazic
J. C. HERR
T. D. BISS

M. R. Wardrop JCH
M. R. Wardrop
Project Structural Engineer

Approved by: [signature]

Date: 8-24-77

GAI 35 12/73

JEL

SH. 13

memorandum Gilbert/Commonwealth

August 26, 1977

to: Mr. J. E. Lisney

from: M. R. Wardrop

subject: CR#3 Tendon Surveillance
Design Review
W. O. 04-4762-016
Filing Code 1:01.7A

You have been designated as the Verifier for subject Design Review. Attached are the eleven (11) calculation pages, two (2) memorandums approving this task, and the Design Verification Record for this task.

The First Tendon Surveillance is scheduled for November, 1977. Therefore, your review should be completed by September 30, 1977.

M. R. Wardrop
M. R. Wardrop
Project Structural Engineer

MRW:km

cc: F. J. Tomazic
S. N. Dobreff
K. E. Nodland
J. C. Herr

GAI 35 12/75

SH. 14

memorandum Gilbert/Commonwealth

to: S. N. Dobreff

from: M. R. Wardrop

subject: Approval of Verifier and Scope of Verification CR#3

As required by DCP 4.2.1 and DP-0414-7, I request your approval of the sufficiency of the Verifier's independence from the Originator and the type and depth of Design Review as shown below:

Project: CRYSTAL RIVER UNIT #3
Design Task: CONTAINMENT BLDG. TENDON SURVEILLANCE PRESTRESS LOSSES
Filing Code 1.01.7A
W.O. 04.4762.016

Originator: J. C. HERR
Verifier: T. D. BISS
Description of Verification to be Performed:

To verify design concept and verify sufficiently the detail calculations as necessary to his satisfaction that the design intent is met. Checking of every arithmetical figure is not required.

Distribution

F. J. Tomazic
J. C. HERR
T. D. BISS
S. Dobreff

M. R. Wardrop
M. R. Wardrop
Project Structural Engineer

Approved By: S Dobreff

Date: 10/24/77

GAI 35 12/73

SH. 15

memorandum Gilbert/Commonwealth

October 24, 1977

to: T. D. Biss

from: M. R. Wardrop

subject: CR#3 Tendon Surveillance
Design Review
W. O. #04-4762-016
Filing Code: 1:01.7A

You have been designated as the Verifier for subject Design Review. Attached are the eleven (11) calculation pages, two (2) memorandums approving this task, and the Design Verification Record for this task.

The first Tendon Surveillance is scheduled to start in November, 1977. Therefore, your review should be completed as soon as possible.

This memorandum also serves to release Mr. J. E. Lisney from being the Verifier, as designated on August 26, 1977.

M. R. Wardrop

M. R. Wardrop

MRW:km

cc: F. J. Tomazic
S. N. Dobreff
J. E. Lisney
J. C. Herr

GAI 35 12/75

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME: Structural	DEPT. NO.: 0414	FILING CODE: 1:01.7A/16
	PROJECT NAME: Crystal River Unit 3	W.O. NUMBER: 041-4203-115	PAGE: R1(1-11)

SUBJECT: Tendon Surveillance - Creep rates

ORIGINATOR: D. Biss

DATE: 10/25/77

VERIFIER:

DATE:

Check creep loss values for 1 year after s.i.t.

Ref. Dome Repair Calc's Book II, section 1

at time of calculation (5/76) average age of concrete was ≈ 700 days

1 year after SIT: ≈ 700 days + 1½ year
: ≈ 1250 days
≈ 3.41 year

from time dependent creep curve
S_c : 0.17 μ"/"/psi

for Vertical tendons

Δf_{sv} = (0.17)(997 psi)(29 x 10^6 psi) = 4.9 ksi vs 4.7

for Hoop tendons

Δf_{sH} = (0.17)(1776)(29 x 10^6) = 8.8 ksi vs 8.4

for Dome tendons

Δf_{sD} = (0.17)(2873)(29 x 10^6) = 14.2 ksi vs 14.9

∴ values used in development of chart ~~to~~ are as accurate as can be predicted with the given input data.

FILING CODE

DESIGN CONTROL PROGRAM — DESIGN VERIFICATION RECORD

CLIENT	W.O.
FLORIDA POWER CORPORATION	04-4762-016
PROJECT: CRYSTAL RIVER UNIT #3	FILING CODE: 1:01.7A/17
DESIGN DOCUMENT: Calculations for SP-395 Tendon Surveillance/Prestress Losses/Page 1-11	REVISION AND DATE: Rev 0 5/10/77
ORIGINATOR: J.C. Herr	VERIFIER: T. D. Biss

DOES THE DESIGN DOCUMENT CONTAIN ASSUMPTIONS IDENTIFIED FOR SUBSEQUENT RE-VERIFICATION WHEN THE DETAILED DESIGN ACTIVITIES ARE COMPLETED (REF. QUESTION 2, BELOW)?

NO ✓ YES ____

VERIFIERS ATTESTATION

THE (DESIGN DOCUMENT HAS) (~~REVISIONS TO THE DOCUMENT OCCURRING SINCE THE PREVIOUS VERIFICATION HAVE~~) BEEN REVIEWED BY ME TO PROVIDE ASSURANCE THAT IT MEETS THE DESIGN INPUTS. ANY FINDINGS UNCOVERED DURING THE COURSE OF MY REVIEW HAVE BEEN DIRECTED TO THE ORIGINATOR AND RESOLVED. AS A PART OF THE REVIEW, THE NINETEEN QUESTIONS LISTED HEREUNDER HAVE BEEN ADDRESSED TO JUDGE WHICH ARE APPLICABLE. THE DESIGN DOCUMENT HAS BEEN EVALUATED AGAINST THE APPLICABLE QUESTIONS.

Theodore Daniel Biss — VERIFIER'S SIGNATURE

10/28/77 — DATE

1. WERE THE INPUTS CORRECTLY SELECTED AND INCORPORATED INTO DESIGN? (SEE DCP 4.9.1)
2. ARE ASSUMPTIONS NECESSARY TO PERFORM THE DESIGN ACTIVITY ADEQUATELY DESCRIBED AND REASONABLE? WHERE NECESSARY, ARE THE ASSUMPTIONS IDENTIFIED FOR SUBSEQUENT RE-VERIFICATIONS WHEN THE DETAILED DESIGN ACTIVITIES ARE COMPLETED?
3. ARE THE APPROPRIATE QUALITY AND QUALITY ASSURANCE REQUIREMENTS SPECIFIED?
4. ARE THE APPLICABLE CODES, STANDARDS AND REGULATORY REQUIREMENTS INCLUDING ISSUE AND ADDENDA PROPERLY IDENTIFIED AND ARE THEIR REQUIREMENTS FOR DESIGN MET?
5. HAVE APPLICABLE CONSTRUCTION AND OPERATING EXPERIENCE BEEN CONSIDERED?
6. HAVE THE DESIGN INTERFACE REQUIREMENTS BEEN SATISFIED?
7. WAS AN APPROPRIATE DESIGN METHOD USED?
8. IS THE OUTPUT REASONABLE COMPARED TO INPUTS?
9. ARE THE SPECIFIED PARTS, EQUIPMENT, AND PROCESSES SUITABLE FOR THE REQUIRED APPLICATION?
10. ARE THE SPECIFIED MATERIALS COMPATIBLE WITH EACH OTHER AND THE DESIGN ENVIRONMENTAL CONDITIONS TO WHICH THE MATERIAL WILL BE EXPOSED?
11. HAVE ADEQUATE MAINTENANCE FEATURES AND REQUIREMENTS BEEN SPECIFIED?
12. ARE ACCESSIBILITY AND OTHER DESIGN PROVISIONS ADEQUATE FOR PERFORMANCE OF NEEDED MAINTENANCE AND REPAIR?
13. HAS ADEQUATE ACCESSIBILITY BEEN PROVIDED TO PERFORM THE IN-SERVICE INSPECTION EXPECTED TO BE REQUIRED DURING THE PLANT LIFE?
14. HAS THE DESIGN PROPERLY CONSIDERED RADIATION EXPOSURE TO THE PUBLIC AND PLANT PERSONNEL?
15. ARE THE ACCEPTANCE CRITERIA INCORPORATED IN THE DESIGN DOCUMENTS SUFFICIENT TO ALLOW VERIFICATION THAT DESIGN REQUIREMENTS HAVE BEEN SATISFACTORILY ACCOMPLISHED?
16. HAVE ADEQUATE PRE-OPERATIONAL AND SUBSEQUENT PERIODIC TEST REQUIREMENTS BEEN APPROPRIATELY SPECIFIED?
17. ARE ADEQUATE HANDLING, STORAGE, CLEANING AND SHIPPING REQUIREMENTS SPECIFIED?
18. ARE ADEQUATE IDENTIFICATION REQUIREMENTS SPECIFIED?
19. ARE REQUIREMENTS FOR RECORD PREPARATION REVIEW, APPROVAL, RETENTION, ETC., ADEQUATELY SPECIFIED?