FLORIDA POWER AND LIGHT COMPANY

TURKEY POINT UNIT 3

DOCKET NO. 50-250

PRELIMINARY DRAFT OF

CONTAINMENT DOME REPORT

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1.0 INTRODUCTION

This report describes the Turkey Point Unit 3 containment dome, delamination of the dome concrete during post tensioning of tendons, the subsequent investigation and analysis of this phenomena, and the repair and test program. When about two thirds of the dome tendons had been tensioned, it was noted that concrete cracking and sheathing filler leakage was developing and that in some areas of the dome, the concrete felt springy when walked on. The dome was struck with a sledge hammer and, in some areas, it sounded as if it were hollow. The concrete was locally removed in some of these areas and shallow (approximately 1/2" to 4") delamination planes were found running almost parallel with the surface, but eventually intersecting it. A full investigation was begun to determine both the extent and cause of the delaminations, and to cover the following:

1. Construction Procedures
2. Core Sampling
3. Materials Properties
4. Analysis of Loads During Construction

As a result of the investigation, it has been determined that inadequate contact in the meridional construction joint together with unbalanced post-tensioning loads, were the major cause of the delaminations. After the post-tensioning was complete, there was no evidence that the dome was not capable of indefinitely resisting the applied loads. From detensioning there was no detectable loss in the tendon forces due to the delaminations.

Concrete replacement procedures have been prepared and will include modifications to the original placement procedures shown to be desirable during the analysis of the delamination causes.

The completed dome will meet performance requirements and the adequacy will be demonstrated during structural tests.

The firm of T. Y. Lin, Kulka, Yang and Associate, the consultant in the design of the containment, has participated in the investigation program and the concrete replacement method selection.

2.0 DOME AND CONSTRUCTION DESCRIPTION

2.1 DOME DESCRIPTION

The containment is described in FSAR, Section 5.1.2 and shown in FSAR Figure 5.1-1 (2 sheets).

The dome design geometry and dimensions are shown in Figure 2-1.

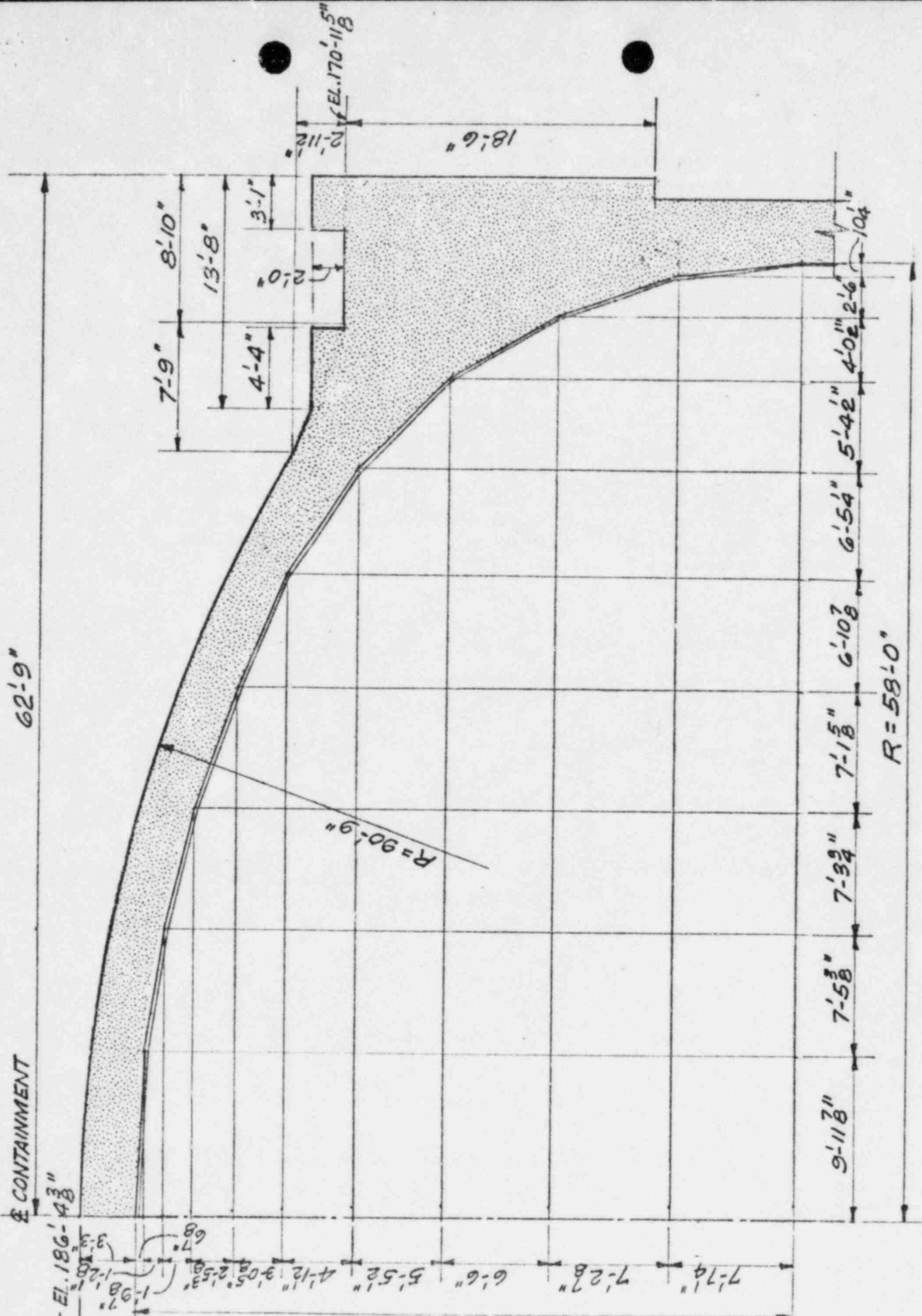
2.2 CONSTRUCTION DESCRIPTION

Locations of construction joints and dates of concrete placement are shown on Figure 2-2. Concrete placed between October 21, 1969 and March 3, 1970 inclusive consists of the top portion of the dome and the construction blockouts. These locations are where delaminations (discussed later) were found. A work stoppage of seven weeks duration resulted in the time lapse between the two largest pours.

Expanded metal was used to form the construction joints. The concrete was placed with buckets and pumps and vibrated for consolidation. Some of the concrete was pumped through aluminum pipe, a practice subsequently discontinued.

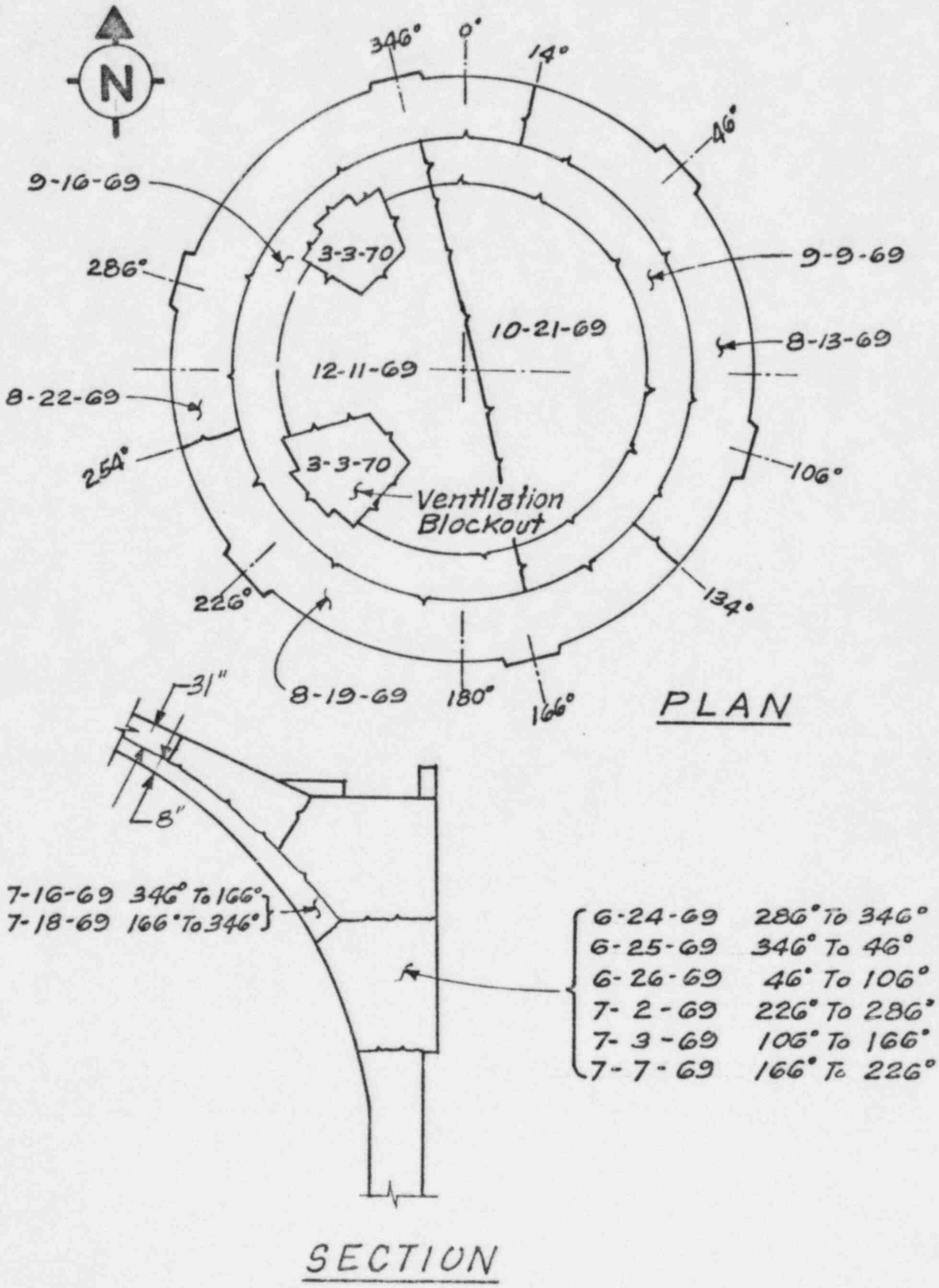
A white pigmented concrete curing compound meeting ASTM C-309 was applied on all exposed surfaces. However, a rainstorm occurred shortly after coating the east half of the dome, placed October 21, 1969, and washed away most of the curing compound. A work stoppage the next day, October 22, 1969 and lasting seven weeks, prevented reapplication of a curing compound.

The dome post-tensioning tendons are composed of 3 groups oriented as shown on Figure 2-3. The tendons are arranged in five layers. The tendons in Group I are in a single layer and are spaced approximately 1'-6" from center to center, whereas the tendons in Groups 2 and 3 are in 2 layers for each group spaced approximately 3 ft. from center to center of tendons in a layer. Tensioning of tendons ultilized conventional equipment and techniques. Sheathing filler pumps, with a pressure capability of between 200 and 250 psi, were used to inject the sheathing filler for tendon corrosion protection.



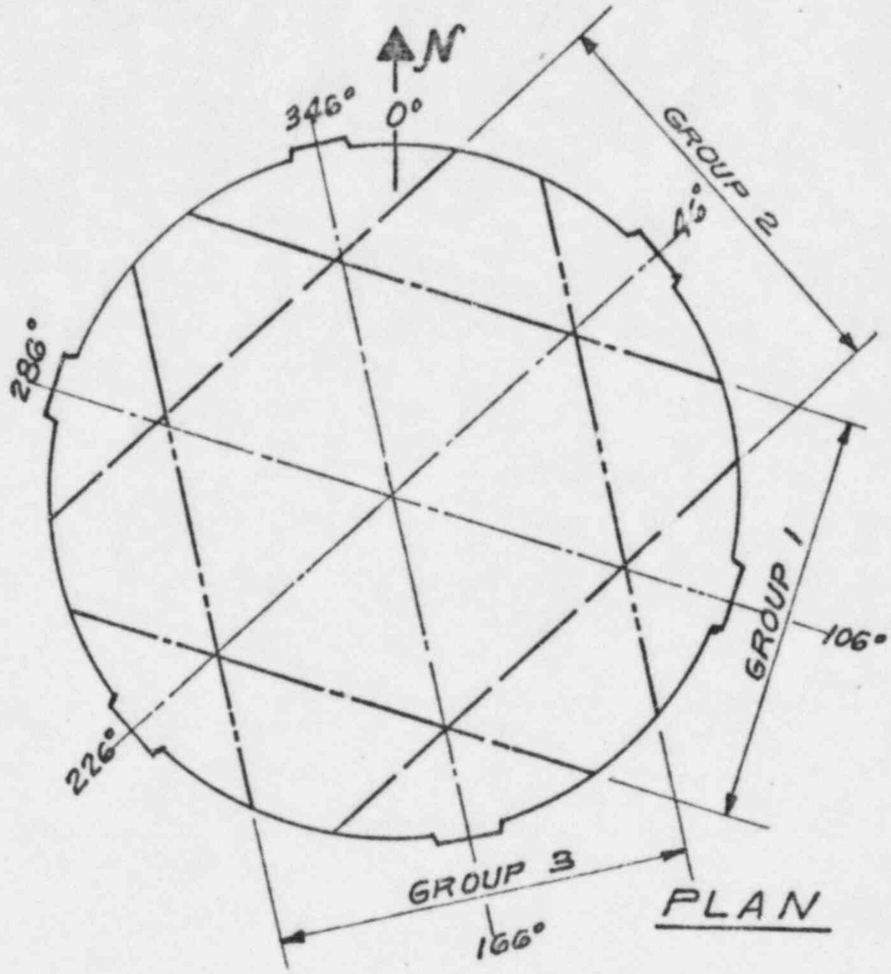
CONTAINMENT DOME GEOMETRY

FIG. 2-1

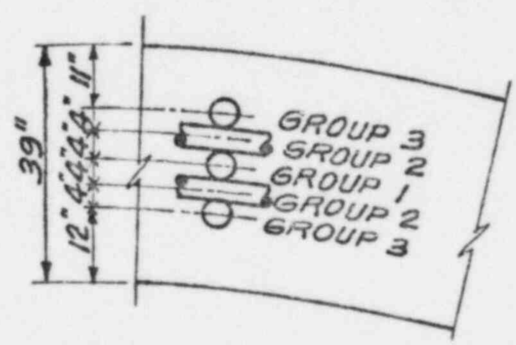


CONSTRUCTION SEQUENCE

FIG. 2-2



PLAN



SECTION

TENDON LAYOUT

FIG. 2-3

3.0 FIELD OBSERVATIONS AND INVESTIGATION

3.1 INITIAL OBSERVATIONS

On June 17, 1970, when 110 out of 165 dome tendons had been tensioned, sheathing filler was observed leaking from a crack in the dome surface. Nine sheaths had been filled on June 16, 4 were filled on June 17, and this work was considered to be the source of the sheathing filler leak.

The leakage location was at azimuth 216 degrees and a radius of 35' from the dome center. A small amount of concrete was chipped away adjacent to the crack. A crack plane parallel to the surface (delamination) was found within an inch or so of the surface. There was evidence of sheathing filler flow on the surfaces created by the delamination.

On June 22, 1970, a small bulge in the dome surface was noticed at azimuth of 296 degrees and radius of 25 feet. The concrete was broken through in one small spot with a hammer and a delamination was discovered at about ½" depth. The exploratory chipping was expanded laterally and towards the center of the dome, revealing that the delamination became thicker as the dome center was approached. This stage of chipping was stopped at about 15 feet radius, at which point the separated layer was about 4" thick.

The initial investigation to determine the extent of the concrete separation below the surface was performed by soundings with a Swiss hammer and a steel sledge hammer. The steel hammer was found to be more effective in finding separations deeper into the concrete, and is considered reliable up to a depth of about 10 inches.

Sonic investigations with a V-scope were considered. The pulse velocity technique does not lend itself to a concrete mass with large numbers of embedded conduits and a liner plate on the underside of the dome.

Moreover, the presence of an intentional construction joint 8 inches from the liner plate further diminishes the reliability of the pulse velocity technique. The reflection method of ultrasonic examination used in metals has not been perfected for a heterogeneous mass such as concrete. A method of sonic induced vibratory resonance of concrete surfaces was tried but proved unsuccessful.

3.2 DOME CONCRETE CORING AND REMOVAL (BEFORE DETENSIONING)

In order, to estimate the depth and extent of the delaminations 65-4" diameter concrete cores were removed from the Unit 3 containment dome prior to destressing the tendons. The percentages of cores to various depths are as follows:

77% to the 1st layer of tendons
71% to the 2nd layer
22% to the 3rd layer
17% to the 4th layer
11% to the 5th layer

A summary of the information obtained from coring is given in Table 3-1. To help visualize the extent and depth of the delaminations inferred from coring, Figures 3-1 and 3-2 have been included. Figure 3-1 shows the core locations together with the depth to the delaminations and the core hole depth. Figure 3-2 is an estimate, from coring information, of the depth and area extent of the delaminations.

Concrete in an area approximately 7' x 7', with its northwest corner near core 23A, was removed to determine the condition of the meridional construction joint. The concrete was removed to a depth of from 12" to 15" so that the difficulty of concrete removal could also be determined.

The following is a summary of the information obtained from both coring and the 7' x 7' concrete removal area:

(1) The depth and extent of the delaminations has considerable symmetry about the meridional construction joint with major delaminations occurring on the south side of the dome.

(2) The delaminations appear to have originated at the meridional construction joint and then progressed away from the joint getting closer to the surface with eventual outcropping or termination at a circumferential construction joint.

(3) The adequacy of the meridional construction joint varied throughout the joint because of the small voids and other evidence of lack of proper consolidation found. Also sheathing filler was found on the joint to within about 6" of the concrete surface.

(4) Some of the core holes show multiple delaminations with gaps between delaminated surfaces of as great as 1".

(5) Many of the core holes had sheathing filler in them after coring, indicating that the delamination plane is continuous over areas other than those immediately around the sheath which was the source of sheathing filler.

3.3 DETENSIONING OF TENDONS

The tendons were detensioned to allow safe concrete removal from around them and so that the replaced concrete will assist the remaining concrete in resisting the prestressing forces.

All but two tendons, out of 165, were tensioned and therefore detensioned. The liftoff readings for tensioning and detensioning verify that the delaminated dome did indeed withstand the prestressing loads for over two months without greater than the normally anticipated losses in tendon forces.

The predicted prestressing force loss, with assumptions given in the FSAR, but for the period that the dome was prestressed, is calculated to be approximately 13% of the minimum ultimate strength of the tendons. The average actual loss was less than this value as shown in Figure 3-3. The effective prestress at the time of detensioning was therefore equal or greater than calculated. The delaminations did not result in a detectable effect on the prestressing forces. Further, the full prestressing force did not result in continuing delaminations attributable to the forces.

3.4 RESULTS OF INSTRUMENT READINGS DURING DETENSIONING

(Later)

3.5 DOME CONCRETE REMOVAL AND SURFACE PREPARATION

The dome concrete removal procedure is given in Specification No. 5610-C-60 (Proprietary). (This section to be completed after concrete removal.)

3-1 (Sheet 1)
TURKEY POINT UNIT #3
CONTAINMENT STRUCTURE DOME
Coring Log Summary (Before Detensioning)

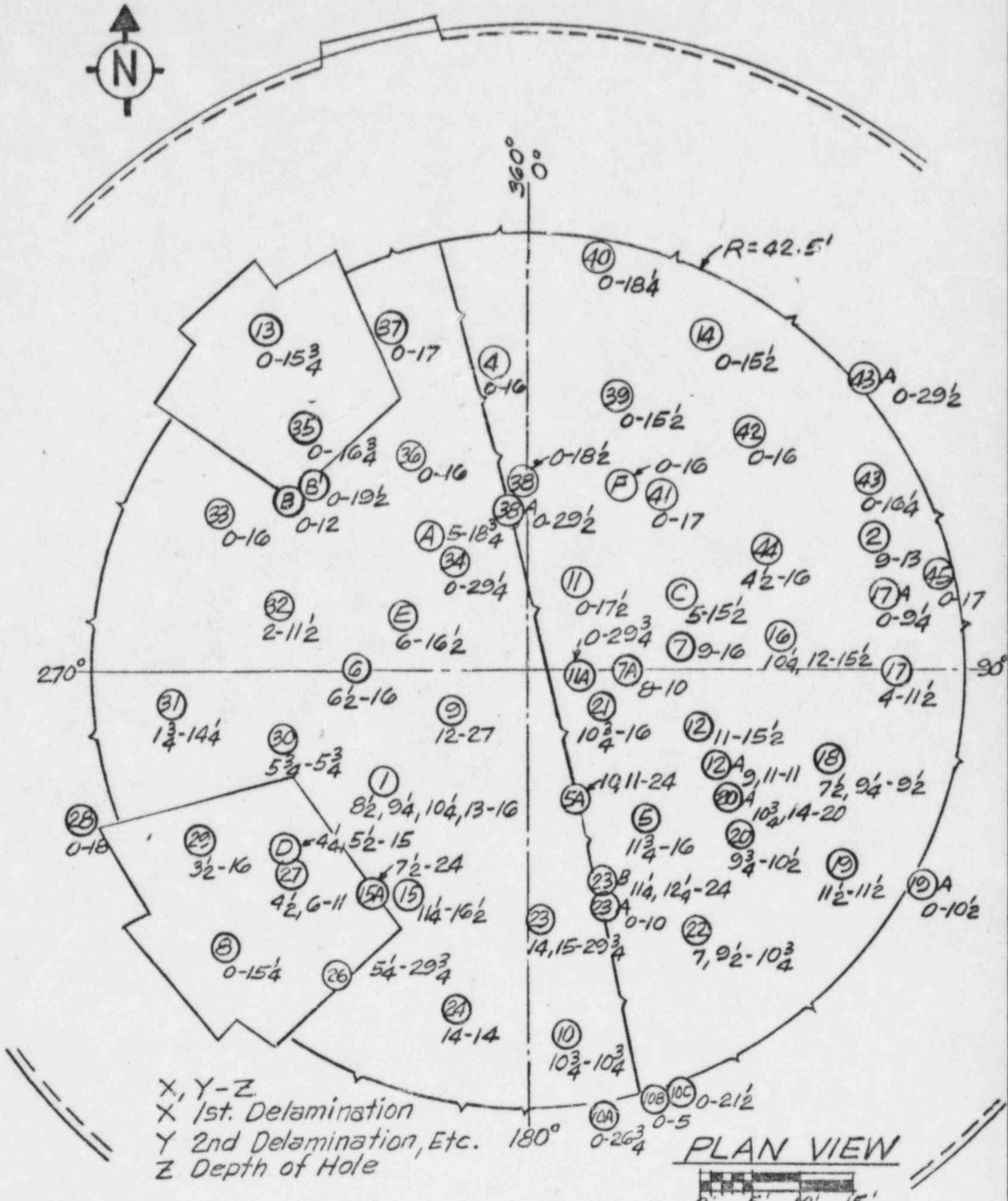
Hole No.	Azimuth	Radius	Depth to Delamination & Separation Distance (in.)	Depth of Hole (in.)	Sheathing Filler Present at Delamination	Examined with Boroscope	Photograph Taken	Comments
1	233°-00'	17'-10"	(8½,¼) (9¼,1/8) (10½,1/8) (13,½)	16	Yes	No	Yes	
2	70°-00'	36'-3"	(9,)	13	Yes	Yes	No	Hit Sheath
3								
4	354°-30'	30'-2"	None	16	-	No	No	
5	142°-00'	18'-8"	(11 3/4,½)	16	Yes	Yes	Yes	Hit Sheath
5A	158°-50'	13'-11"	(10, ¼) (11¼,½)	24	Yes	Yes	No	Hit Sheath, on C.J.
6	270°-00'	16'-10"	(6½, 3/4)	16	No	Yes	Yes	
7	83°-00'	15'-4"	(9,½)	16	Yes	No	No	
7A	91°-00'	10'-0"	(8,½)	10	No	Yes	Yes	Hit Sheath
8	227°-30'	39'-10"	None	15¼	No	Yes	Yes	
9	243°-00'	8'-5"	(12,¼)	27	Yes	Yes	No	
10	173°-30'	35'-3"	(10 3/4,)	10 3/4	Yes	No	No	Hit Sheath
10A	170°-03'	43'-10"	None	26 3/4	-	No	No	Hit Sheath
10B	163°-04'	43'-0"	None	5	-	No	No	On C.J.
10C	160°-19'	43'-0"	None	21½	-	Yes	No	Hit Sheath, on C.J.
11	29°-30'	9'-8"	None	17½	-	Yes	No	Hit Sheath
11A	98°-15'	5'-3"	None	29 3/4	-	No	No	
12	109°-30'	18'-0"	(11,½)	15½	Yes	Yes	Yes	
12A	116°-50'	20'-7"	(9,), (11,)	11	Yes	Yes	No	
13	322°-30'	41'-9"	None	15 3/4	-	No	No	
14	28°-30'	37'-0"	None	15½	-	Yes	No	
15	208°-00'	24'-10"	(11½, 1)	16½	Yes	Yes	Yes	Hit Sheath, on C.J.
15A	215°-20'	26'-1"	(7½,1)	24	Yes	Yes	No	

3-1 (Sheet 2)
TURKEY POINT UNIT #3
CONTAINMENT STRUCTURE DOME
Coring Log Summary (Before Detensioning)

Hole No.	Azimuth	Radius	Depth to Delamination & Separation Distance (in.)	Depth of Hole (in.)	Sheathing Filler Present at Delamination	Examined with Boroscope	Photograph Taken	Comments
16	83°-30'	25'-0"	(10¼,3/4) (12,¼)	15½	Yes	Yes	Yes	
17	90°-09'	35'-10"	(4,)	11½	Yes	Yes	No	
17A	78°-20'	36'-3"	None	9¼	-	No	No	Hit Sheath
18	106°-51'	30'-11"	(7½,3/4) (9¼,½)	9½	No	Yes	Yes	
19	121°-51'	36'-2"	(11½,)	11½	Yes	No	No	
19A	118°-00'	43'-0"	None	10½	--	Yes	No	On C.J.
20	127°-30'	26'-4"	(9 3/4, 3/4)	10½	Yes	No	No	
20A	122°-15'	23'-1"	(10 3/4, ½) (14, 1/8)	20	Yes	No	No	
21	114°-06'	8'-3"	(10 3/4, 1/8)	16	No	No	No	
22	147°-11'	30'-6"	(7,½) (9½, 1)	10 3/4	No	Yes	Yes	
23	177°-15'	24'-6"	(14,) (15,)	29 3/4	Yes	Yes	Yes	
23A	161°-57'	24'-1"	None	10	-	No	No	On C.J.
23B	161°-37'	21'-10"	(11¼,1) (12¼,¼)	24	Yes	Yes	No	(2nd Delam. East Side on C.J.)
24	191°-15'	33'-8"	(14,)	14	Yes	No	No	
25								
26	210°-54'	35'-11"	(5¼,¼)	29 3/4	Yes	Yes	No	
27	230°-03'	30'-1"	(4½, 1/8) (6,3/4)	11	Yes	Yes	Yes	1st Delam. East Side
28	251°-42'	45'-11"	None	18	-	No	No	
29	243°-37'	36'-4"	(3½, ¼)	16	No	Yes	No	
30	254°-40'	25'-2"	(5 3/4,)	5 3/4	Yes	No	No	
31	264°-40'	34'-9"	(1 3/4, ¼)	14¼	Yes	Yes	No	Hit Sheath
32	284°-54'	25'-3"	(2, 1)	11½	No	Yes	Yes	
33	297°-06'	33'-7"	None	16	-	Yes	No	
34	326°-00'	12'-10"	None	29¼	-	No	No	

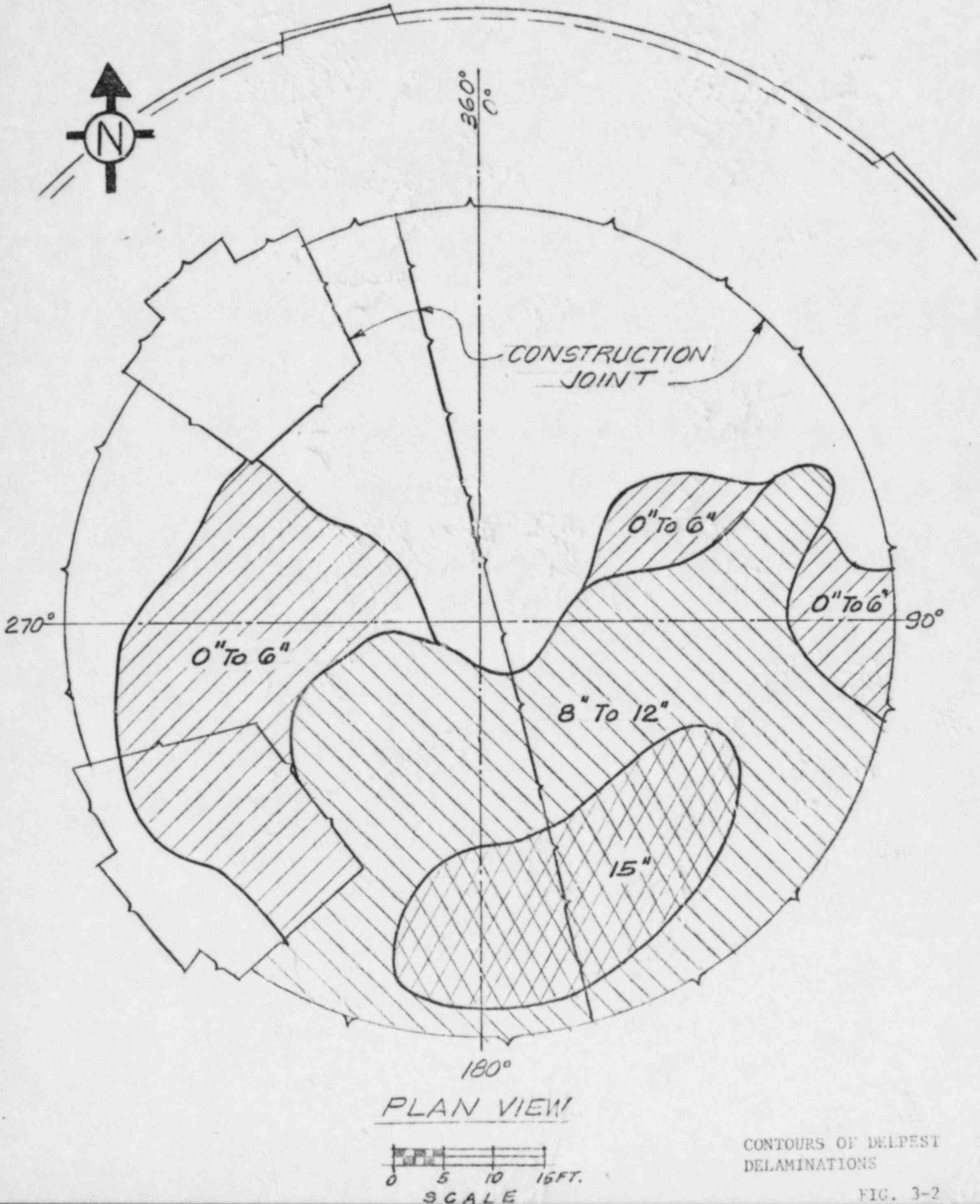
3-1 (Sheet 3)
TURKEY POINT UNIT #3
CONTAINMENT STRUCTURE DOME
Coring Log Summary (Before Detensioning)

Hole No.	Azimuth	Radius	Depth to Delamination & Separation Distance (in.)	Depth of Hole (in.)	Sheathing Filler Present at Delamination	Examined with Boroscope	Photograph Taken	Comments
35	317°-35'	32'-1"	None	16 3/4	-	Yes	No	
36	331°-00'	23'-8"	None	16	-	Yes	No	
37	338°-19'	35'-11"	None	17	-	No	No	
38	359°-13'	18'-10"	None	18½	-	No	No	Hit Sheath
38A	353°-40'	16'-4"	None	29½	-	Yes	No	Hit Sheath, on C.J.
39	18°-00'	28'-4"	None	15½	-	No	No	
40	10°-25'	40'-2"	None	18¼	-	No	No	
41	38°-30'	21'-6"	None	17	-	No	No	Hit Sheath
42	43°-40'	31'-9"	None	16	-	No	No	
43	61°-13'	38'-5"	None	16¼	-	No	No	
43A	49°-20'	43'-0"	None	29½	-	No	No	On C.J.
44	64°-08'	26'-4"	(4½, ½)	16	No	No	No	
45	77°-45'	40'-6"	None	17	-	No	No	
A	324°-00'	16'-5"	(5, ½)	18 3/4	No	Yes	-	
B	305°-18'	28'-6"	None	12	-	Yes	Yes	On C.J.
B'	310°-30'	27'-7"	None	19½	-	No	No	On C.J.
C	62°-50'	17'-0"	(5,)	15½	No	Yes	No	
D	233°-46'	29'-3"	(4¼,) (5½,)	15	No	Yes	No	
E	294°-21'	13'-4"	(6,)	16½	No	Yes	No	
F	27°-30'	20'-5"	None	16	-	No	No	



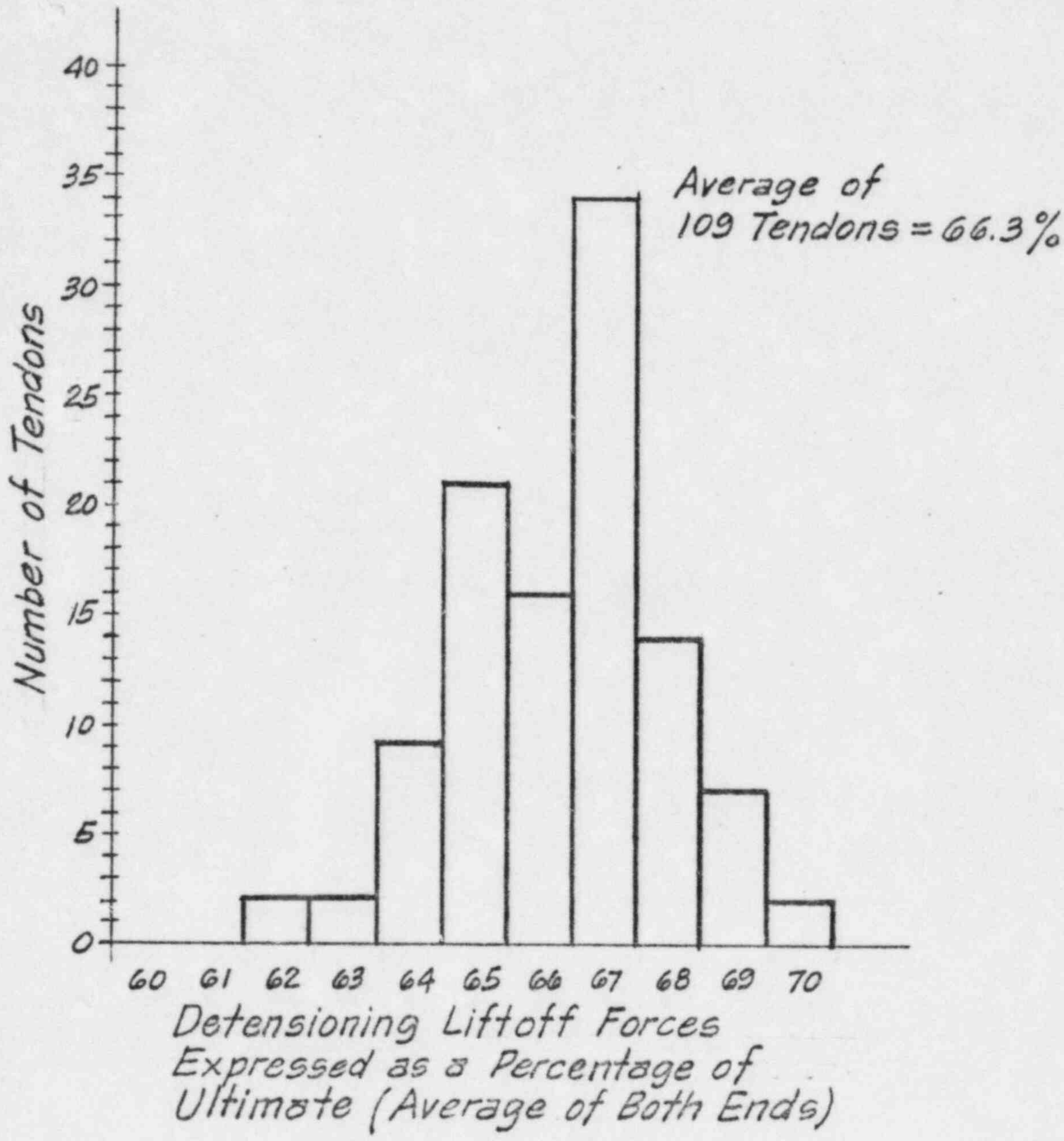
DOME CORING RESULTS

FIG. 3-1



CONTOURS OF DEEPEST
DELAMINATIONS

FIG. 3-2



TENDON LIFTOFF VALUES

FIG. 3-3

4.0 MATERIALS INVESTIGATION

An extensive study was made to recheck the adequacy of the Turkey Point concrete to perform its intended function. The study involved documenting the physical and chemical properties of constituent materials, together with standard testing of specimens prepared with the concrete design mixes and tests not normally required. To establish a comparative basis, information on other concretes within Bechtel's experience are also included.

Table 4-1 shows the concrete design mixes for Turkey Point and other structures. The 2P5 mix is applicable for concrete placed before October 21, 1969. The delaminated dome concrete was formulated to the 2P6 design mix.

Table 4-2 shows the chemical and physical tests for the cements. The Turkey Point Cement conforms to the requirements of Type II cement. Low heat of hydration cement (combined limit of 58% on tri-calcium silicate and tri-calcium aluminate) was not specified for Turkey Point. (The ASTM limit of 58% is optional and applies only when specifically requested by the user). However, control of the concrete placement temperature at 70 F was specified and "Retardwell" was used to slow down the rate of hydration of the cement.

Table 4-3 shows the properties of the fine and coarse aggregate for Turkey Point. The coarse aggregate was specified as 1" minus since larger sizes were considered too absorbtive.

Tables 4-4 and 4-5 show the chemical analyses for both the water and ice used in the mixing of the concretes. In all cases the water and ice are suitable for their intended purpose.

Table 4-6 lists the air entraining agents used in the various concretes and Table 4-7 lists the water reducing agents. All are within specification requirements.

Table 4-8 shows the physical concrete properties based on initial testing which was performed to verify the adequacy of the design mixes for their intended use and to obtain design data for creep, shrinkage, etc. With the exception of the lower splitting tensile strength on Turkey Point all other properties are comparable. However all calculated tensile stresses are considerably lower than the values given in Table 4-8.

Table 4-9 shows the comparison of uniaxial compression strength for concrete cylinders cast during dome concrete placement and also for concrete taken from the delaminated area of the Turkey Point Dome. All cylinders sampled from concrete when cast show strengths exceeding 5000 psi.

Table 4-10 shows additional splitting tensile strength results for the various concretes. Comments are the same as for Table 4-8.

Table 4-11 shows the results of direct tensile tests on concrete taken from the dome. The average of 8 tests was 352 psi. As is common, the direct tensile strengths are less than those calculated from the results of the cylinder splitting tests.

Another series of tests were performed to determine the stress and strain values for uniaxial tension and compression given in Table 4-12 and 4-13. To provide evidence that a state of biaxial compressive strain would not lead to a condition more critical than that indicated by uniaxial compression tests, a series of tests were performed. The tests were made by placing a concrete cylinder with a membrane around it in a pressure chamber. The chamber could apply an essentially frictionless pressure load to the cylindrical surface while the ends remained free of load. There was a test technique problem in preventing oil from causing a premature failure due to penetration of the membrane or collapsing of subsurface voids and creating a longitudinal tension force. As shown in Table 4-14, it was difficult to cause a failure, resulting from radial pressure alone, however the results do show that the biaxial capability of the concrete strength is equal or greater than the uniaxial capability.

In both biaxial and uniaxial compression the failure mechanism was by formation of crack planes parallel to the applied loading direction for the Turkey Point and other concretes. When loading a cylinder in uniaxial compression, first cracking would occur in the longitudinal direction. Individual columns would then form and eventually fail in shear on inclined planes with resulting multiple failure surfaces. The texture of the compression failure surfaces were much closer to that of the dome delamination surface than were those resulting from uniaxial tension. Figure 4-1 shows a delaminated surface of a core together with both a tension and compression failure specimen. This fact, together with the knowledge that multiple delaminations existed in the dome, confirms the conclusion that the dome delamination resulted from large compression forces essentially parallel to the surface. These large forces resulted in a concrete strain failure on surfaces parallel to the dome. The testing was for short duration loads, and strengths for long term loads are typically lower. Therefore, it is reasonable to assume that the delaminations occurred because of long term loads that caused relatively widespread compressive stresses of approximately .75 to .85 f'_c parallel to the surface. (For the Turkey Point concrete .75 f'_c is typically equal to 4500 psi).

Petrographic analyses of concrete have been performed by 2 independent laboratories, namely: Erlin Associates of Northbrook, Illinois through Pittsburgh Testing Laboratory and by Dr. Richard C. Mielenz, Vice President of Research and Development, Master Builders Company of Cleveland, Ohio.

The result of their examinations shows the concrete to be a competent material with:

(1) A low water-cement ratio.
(2) A good air void system.
(3) No sign of alkali-carbonate reaction.
(4) A good distribution of sound coarse aggregate.
(5) No sign of metallic aluminum or hydrogen gas formation due to pumping through aluminum pipe.

TABLE 4-1

TABULATION OF DESIGN MIX QUANTITIES
(For 1 Cubic Yard)

	JOB & JOB NUMBER					
	TURKEY POINT 5610	TURKEY POINT 5610	CONTAINMENT A	CONTAINMENT B	CONTAINMENT C	CONTAINMENT D
Mix Number (28 Day Strength)	2P5* (5000 PSI)	2P6* (5000 PSI)	(5000 PSI)	(5000 PSI)	(5000 PSI)	(5000 PSI)
Cement	588#	635#	526#	479#	580#	611#
Fly Ash	-	-	93#	85#	-	-
Sand	1290#	1254#	1210#	1227#	1080#	1131#
3/4" Aggregate	1660#	1632#	937#	996#	1100#	893#
1½" Aggregate	-	-	1032#	1095#	1100#	954#
Water (W/C Ratio)	279# (0.47)	260# (0.40)	246# (0.48)	229# (0.48)	265# (0.46)	225# (0.37)
Water Reducing Agent	12.65 OZ.	13.5 OZ.	11 OZ.	24 OZ.	12 OZ.	13 OZ.
Air Entraining Agent	1.5 OZ.	1.6 OZ.	10.5 OZ.	4 OZ.	10 OZ.	4.6 OZ.

* Mix 2P5 lost approximately 1500 PSI strength when the pump was used. Adopted new mix 2P6 with an extra ½ sack of cement to offset this loss.

TABLE 4-2

CHEMICAL & PHYSICAL TESTS OF CEMENT IN ACCORDANCE WITH ASTM C-150

AVERAGE OF USER TESTS RUN ON THE SITE DURING THE APPROX. PERIOD OF TIME THE DOME WAS POURED (EXCEPT FOR CALVERT CLIFFS). ALL USER TESTS RUN BY INDEPENDENT TESTING LABORATORIES.

ASTM TYPE I' REQUIREMENTS LISTED IN PARENTHESIS

	Chemical Properties									Physical Properties											
	SiO_2 (21%) (Min)	Al_2O_3 (6%) (Max)	Fe_2O_3 (6%) (Max)	MgO (5%) (Max)	SO_3 (2.5%) (Max)	Loss on Ign. (3%) (Max)	Insol. Res. (.75%) (Max)	C_3A (8%) (Max)	C_3A & C_3S (58%) (Max)	Specific Wagner (1600 CM^2/G) (Min)	Surface Blaine (2800 CM^2/G) (Min)	Soundness -Autoclave Expansion- (0.80%) (Max)	Time of Setting Gillmore Initial (60 Min) (Min)	Gillmore Final (10 Hr) (Max)	Vicat (45 Min) (Min)	Air Content % by Vol. (12% Max.)	Comp. Strength 3d (1000 PSI Min)	7d (1800 PSI Min)	28d (3500 PSI Min)	Tensile Strength 3d (125) (PSI) (Min)	7d (250) (PSI) (Min)
TURKEY PT. #5610 TYPE II CEMENT	21.7	4.13	3.16	1.04	2.12	1.16	.22	5.57	*70.9	2070	3862	0.007%	2 Hr.	5 Hr. 13 Min.	1 Hr. 26 Min.	7.5%	3322	4456	6300	337 Result of 2 tests. This data not requested.	412
CONTAINMENT A TYPE II CEMENT	22.9	4.24	4.05	2.29	1.95	1.02	.25	4.39	47.0	--	3500	.03%			2 Hr. 40 Min.	8.0%	1517	2257	4291		
CONTAINMENT B TYPE II CEMENT	21.8	4.98	4.41	2.00	2.22	1.15	.10	5.74	54.3	--	3367	.024%	3 Hr. 55 Min.	5 Hr. 56 Min.		8.05%	2383	3375			
CONTAINMENT C TYPE II CEMENT	22.2	4.3	3.6	2.46	2.27	1.23	.13	5.3	58.7	--	3398	.08%			2 Hr. 33 Min.	7.3%	2253	3380	5493		
CONTAINMENT D TYPE II CEMENT	21.75	4.3	4.25	3.05	2.3	0.8	0.25	4.2	52	-	3655	0.06%	-	-	2 Hr. 32 Min.	7.1%	1715	2923	4224	308	38-

* Each User Test Value & Avg. of all Tests exceed maximum. Since moderate heat of Hydration Cement was not requested and the thermal co-efficient of expansion is low in comparison to other mixes, this is an acceptable value.

TABLE 4-3 (SHEET 1)

FINE & COARSE AGGREGATES TESTED IN ACCORDANCE WITH ASTM C-33

Read Table Down For Each Job (ASTM Req't at Right)		AVERAGE OF USER TESTS SUBMITTED BY EACH JOB FROM TESTS RUN ON THE SITE BY AN INDEPENDENT TESTING LAB WITH THE EXCEPTION OF THE PETROGRAPHIC TESTS. PETROGRAPHIC TESTS RUN BY BECHTEL CORPORATION'S GEOLOGY DEPARTMENT.					
TEST	ASTM TEST DESIGNATION	TURKEY PT. #5610	CONTAINMENT A	CONTAINMENT B	CONTAINMENT C	CONTAINMENT D	REQ'T. BY ASTM C-33
Los Angeles Abrasion	C-131	36.8%	42.8%	24.8%	32.8%	36.0%	Max. Loss = 50%
Clay Lumps Natural Aggregate	C-142	Fine Agg. - None Coarse Agg. "	Fine Agg. - None Coarse Agg. "	Fine Agg. - None Coarse Agg. "	Fine Agg. - None Coarse Agg. "	Fine Agg. - None Coarse Agg. "	Max. Friable Particles Fine Agg: 1% by WT. Coarse Agg: .25% by WT.
Material Finer than #200 Sieve	C-117	Fine Agg. 2.5% 3/4" Coarse Agg. 1.5%	Fine Agg. 2% 3/4" Coarse Agg. 1.1% $1\frac{1}{2}$" " " 1.0%	Fine Agg. 5.9% 3/4" Coarse Agg. 0.6% $1\frac{1}{2}$" " " 0.1%	Fine Agg. No Data 3/4" Coarse Agg. No Data $1\frac{1}{2}$" Coarse Agg. No Data	Fine Agg. 0.4% 3/4" Coarse Agg. 0.5% $1\frac{1}{2}$" " " 0.6%	Fine Agg: 3-5% Max. MFG Fine Agg: 5-7% Max. Coarse Agg: 1% Max. Crushed Coarse Agg: 1.5% Max.
Mortar Making Properties	C-87	Satisfactory	Satisfactory	Satisfactory	Satisfactory	Satisfactory	Not less than 95%
Organic Impurities	C-40	Lighter than STD.	Lighter than STD.	Lighter than STD.	Lighter than STD.	Lighter than STD.	Darker than STD.
Potential Reactivity (Chemical)	C-289	F.&C. Agg. - Innocuous	F.&C. Agg. - Innocuous	F.&C. Agg. - Innocuous	F.&C. Agg. - Innocuous	F.&C. Agg. - Innocuous	Reactive when S_c-R_c Plot falls to RT of Curve.
Sieve Analysis	C-136	F.&C. Agg. Sieve Anal - OK Fine Agg. F.M. = 2.51 3/4" Agg. F.M. = 6.8	F.&C. Sieve Anal - OK Fine Agg. F.M. = 2.51 3/4" Agg. F.M. = 7.26 $1\frac{1}{2}$" Agg. F.M. = 7.76	F.&C. Sieve Anal - OK Fine Agg. F.M. = 2.69 3/4" Agg. F.M. = 6.95 $1\frac{1}{2}$" Agg. F.M. = 7.86	F.&C. Sieve Anal - OK Fine Agg. F.M. = 2.68 3/4" Agg. F.M. = 6.44 $1\frac{1}{2}$" Agg. F.M. = 7.93	F.&C. Sieve Anal - OK Fine Agg. F.M. = 2.91 3/4" Agg. F.M. = 6.46 $1\frac{1}{2}$" Agg. F.M. = 7.96	As listed in tables for F&C Agg.
Soundness	C-88	F. Agg. 8% C. Agg. 8%	F. Agg. 6.4% C. Agg. 6.4%	F. Agg. 6.7% C. Agg. 6.7%	F. Agg. 3.9% C. Agg. 3.9%	F. Agg. 4.5% C. Agg. 4.5%	F. Agg. - Sodium Sul. - 10% Max C. " - " " 12% "
Specific Gravity and Absorption for Coarse Aggregate	C-127	3/4" Agg. S.G.=2.44 " " ABS=3.4%	3/4" Agg. S.G.=2.72 " " ABS=1.5% $1\frac{1}{2}$" Agg. S.G.=2.76 " " ABS=0.9%	3/4" Agg. S.G.=2.82 " " ABS=0.33% $1\frac{1}{2}$" Agg. S.G.=2.82 " " ABS=0.31%	3/4" Agg. S.G.=2.88 " " ABS=0.4% $1\frac{1}{2}$" Agg. S.G.=2.88 " " ABS=0.4%	3/4" Agg. S.G.=2.59 " " ABS=1.6% $1\frac{1}{2}$" Agg. S.F.=2.58 " " ABS=1.6%	Spec. Grav - No Limit. Absorption - " "

TABLE 4-3 (SHEET 2)

FINE & COARSE AGGREGATES TESTED IN ACCORDANCE WITH ASTM C-33

Read Table Down For Each Job (ASTM Req't at Right)

TEST	ASTM TEST DESIGNATION	AVERAGE OF USER TESTS SUBMITTED BY EACH JOB FROM TESTS RUN ON THE SITE BY AN INDEPENDENT TESTING LAB WITH THE EXCEPTION OF THE PETROGRAPHIC TESTS. PETROGRAPHIC TESTS RUN BY BECHTEL CORPORATION'S GEOLOGY DEPARTMENT.					REQ'T. BY ASTM C-33
		TURKEY PT. #5610	CONTAINMENT A	CONTAINMENT B	CONTAINMENT C	CONTAINMENT D	
Specific Gravity and Absorption for Fine Agg.	C-128	S.G. = 2.52 ABS = 3.8%	S.G. = 2.69 ABS = 1.8%	S.G. = 2.83 ABS = 0.5%	S.G. = 2.63 ABS = 0.5%	S.G. = 2.62 ABS = 0.7%	Spec. Grav - No Limit. Absorption - " "
Petrographic Analysis	C-295	Manufactured Fine Aggregate: Calcite - 80% Quartz - 20% Manufactured Coarse Aggregate: Calcite - 67% Quartz - 30% Chalcedony - 3% & Opal	Alluvial Fine Agg: Alluvial Glacial Limestone-100% Manufactured Coarse Agg: Crushed Quarried Dolomite - 100%	Manufactured Fine Aggregate: Crushed Dolomite-100% Manufactured Coarse Aggregate: - same -	Alluvial Fine Agg: Alluvial River Sand Quartz - 50% Feldspar - 35% Muscovite Magnitite Kyanite, Actinolite, Garnet & Expodite 15% Manufactured Coarse Aggregate: Quarried Dolomite-98% Talc, Quartz Magnitite & Hemotite 2%	Alluvial Fine Agg: Quartz - 86% Sandstone - 5.5% Siltstone - 3.0% Claystone - 5.5% Claystone comprised of Kaolin & Montmorillonite Expanding Clays Alluvial Coarse Agg: - same -	Method Only

TABLE 4-4

WATER ANALYSIS

	AVERAGE VALUE OF TESTS RUN BY INDEPENDENT TESTING LABORATORIES AND SUBMITTED BY EACH JOBSITE (ALL VALUES ARE EXPRESSED IN PPM EXCEPT pH)																					
	Change in Setting Time Compared to Distilled Water		Reduction in Strength Compared to Distilled Water	Fe	Ca	Mg	Na &K	Hardness As $CaCO_3$	Alkalinity as $CaCO_3$		Total Dissolved Solids	CO_3	HCO_3	SO_4	Cl	OH, As $CaCO_3$	CO_3, As $CaCO_3$	HCO_3 As $CaCO_3$	COLOR	Tur-bidity	pH	CO_2
	Initial	Final							P.	M.O.												
TURKEY POINT #5610				.365*	95.8	2.4*	285*	242	0	189	258	0*	250*	28*	19	0*	0*	205*	0	3	7.23	29*
CONTAINMENT A	7%*	0%*	4.6%* (7 Days)																			
CONTAINMENT B				.64	61.	16.	2.8	220	0	184	230	0	223	20	16		0	223			7.05	
CONTAINMENT C								14*		31*	64*				4.5*							
CONTAINMENT D	-	-	5%* (28 Days)								120*										7.78*	

* Result of 1 Test

TABLE 4-5

ICE ANALYSIS

	AVERAGE VALUE OF TESTS RUN BY INDEPENDENT TESTING LABORATORIES AND SUBMITTED BY EACH JOBSITE (ALL VALUES ARE EXPRESSED IN PPM EXCEPT pH)																					
	Change in Setting Time Compared to Distilled Water		Reduction in Strength Compared to Distilled Water	Fe	Ca	Mg	Na &K	Hardness As $CaCO_3$	Alkalinity as $CaCO_3$		Total Dissolved Solids	CO_3	HCO_3	SO_4	Cl	OH, As $CaCO_3$	CO_3, As $CaCO_3$	HCO_3 As $CaCO_3$	COLOR	Turbidity	pH	CO_2
	Initial	Final							P.	M.O.												
TURKEY PT. #5610				.01*	7*	0*	4.2*	20*	0	35	83	0*	24*	0*	9	0*	0*	20*	2*	2.3	7.42	4.3*
CONTAINMENT A																						
CONTAINMENT B																						
CONTAINMENT C				0.27*	2.8*	0.8*		5		10.7	30				1.5*						7.4*	
CONTAINMENT D	+8%*	+9%*	4%* (7 Days)								72										7.2	

* Result of 1 Test

TABLE 4-6

AIR ENTRAINING AGENT

JOB	BRAND NAME - SUPPLIER	Supplied in Accordance with ASTM Specification
TURKEY PT. #5610	"AIRECON" - UNION CARBIDE	C-260
Containment A	"SIKA AIR" - SIKA CHEMICAL CO.	C-260
Containment B	"M.B.V.R." - MASTER BUILDERS	C-260
Containment C	"SIKA AIR" - SIKA CHEMICAL CO.	C-260
Containment D	"AIRECON" - UNION CARBIDE	C-260

TABLE 4-7

WATER REDUCING AGENT

JOB	BRAND NAME - SUPPLIER	Supplied in Accordance with ASTM Specification
TURKEY PT. #5610	"RETARDWELL" - UNION CARBIDE	C-494 TYPE D
Containment A	"PLASTIMENT" - SIKA CHEMICAL CO.	C-494 TYPE D
Containment B	"POZZOLITH 8" IMPROVED - MASTER BUILDERS	C-494 TYPE D
Containment C	"PLASTIMENT" - SIKA CHEMICAL CO.	C-494 TYPE D
Containment D	"RETARDWELL" - UNION CARBIDE	C-494 TYPE D

TABLE 4-8
CONCRETE PHYSICAL CHARACTERISTICS

	Turkey Point*	Containment A	Containment B	Containment C	Containment D	Remarks
Comp. Strength psi 28D 180D 365D	*Mix 2P5 7780 7760 6790	 5900 8000 -	 7240 8620 8720	 5617 7925 7810	 6420	70°
†Splitting Tensile & Comp. psi	Comp. Tensile 7000 473 7000 390 7000 359	Comp. Tensile 7000 585 7000 560 7000 515	Comp. Tensile 5980 555 6140 525 5910 520	Comp. Tensile 6100 580 6100 563 6100 -	Comp. Tensile 5670 560 5450 560 - 485	†28 day test in accordance with ASTM 39 [illegible]96
Elastic & Creep Strain $x 10^{+6}$ in/in 1D 180D 14600D	 272 388 543	 277 372 387	 175 224 280	 245 332 405	 336 (Load applied at 28 days)	Reference to application of load at 180 days, 70° except as noted
Poisson Ratio	0.24	0.25	0.26	0.27	0.18 (28 days)	
"E" $x 10^{-6}$ psi Inst. 1D Sust. 180D Sust. 14600D	 6.2 3.9 2.8	 6.4 4.1 3.2	 8.9 6.7 5.4	 7.32 4.52 3.70		
Auto. Vol. $x10^{+6}$ 180D 365D	 -2 -4	 -15 -23	 -17 -32	 -120 -145	 -2 -	@70°F
Therm. Exp.$x10^{+6}$ in/in per °F	5.1	6.3	6.9	6.8	7.4	------
Spec. Heat BTU/"°F	0.268	No Data	0.257	No Data		------
Diff. Ft.^{2}/hr	0.0340	0.048	0.0513	0.0395	.067	
"E" $x 10^6$ Static psi 1D 180D 14600D	 4.6	 4.2 6.6	 7.3 8.3 8.8	 5.5 5.8 6.5	 4.9 (Load applied at 28 days)	Reference to application of load at 180 days, 70° except as noted

TABLE 4-9

COMPRESSIVE STRENGTH (UNIAXIAL COMPRESSION)

	Avg. 28 Day Comp. Strength For Test Cylinders From Dome Concrete (psi)	Comp. Strength For Cylinders Cored From Dome Concrete-40 hr. Water Cure Prior To Testing (psi)	Comp. Strength For Cylinders Cored From Dome Concrete-Tested In A Dry State (psi)
	ASTM C-39	ASTM C-42	-
Turkey Point (Unit 3)	6724 (24 Tests)	6020 6270 }Ave. 9Mos. old 4910 5570 5280 5840 AVG. = 5648	6810 } Ave. 9 Mos. old 6710 AVG.=6760
Containment A	#1 6968 (28 Tests) #2 7137 (32 Tests)		
Containment B	5510 (28 Tests)		
Containment C	#1 5493 (36 Tests)		
Containment D	6155 (7Tests) From wall; dome not poured		

TABLE 4-10

TENSILE STRENGTH (SPLIT CYLINDER TEST)

	Tensile Strength Of Cylinders Cored From Dome Concrete (psi)	Tensile Strength Of Test Cylinders Made From Same Mix As Dome Concrete (psi)
	ASTM C-496	ASTM C-496
Turkey Point (Unit 3)	743-W (7 mo.old) 740-W (10 mo. old) 753-W (12 mo. old) 652-D (9 mos. old) 719-D (9 mos. old) 562-D (9 mos. old) Avg. = 745 - W Avg. = 644 - D	390-W (20 days old) 360-W (20 days old) 746-W (28 days old) 473-W (73 days old) 785-D (28 days old) Avg. = 492 - W Avg. = 785 - D
Containment A		585-W, 615-W) 560-W, 620-W) all 28 days old 515-W, 595-W) Avg. = 580 - W
Containment B		575-W) 590-W) all 28 610-W) days old 590-W) Avg. = 590 - W
Containment C		580-W) 563-W) all 28 347-W) days old 345-W) Avg. = 459 - W
Containment D		560) 560) all 28 days 485) old Avg. = 535 - W

W = Tested Wet
D = Tested Dry

TABLE 4-11

TENSILE STRENGTH (UNIAXIAL TENSION TEST)

Job	Source	Cylinder Number	Tensile Strength (psi)	Maximum Tensile Strain ($x10^{-6}$in/in	Age	Remarks
Turkey Point	Test made on 2"x6" cylinders cored from 6x12 std. test cylinders made on jobsite with same design mix as in dome.	1185-2	329		Days 31	About 80% of aggregate broke through; 20% pulled off
		1185-1	347		31	About 50% of aggregate broke through; 50% pulled off
		1177-2	241		38	About 75% of aggregate broke through; 25% pulled off
		1177-1	394		38	About 100% of aggregate broke through
		1183-2	341		33	About 90-95% of aggregate broke through; 5-10% pulled off
		1183-1	392	112.5@ 371psi	33	About 90-95% of aggregate broke through; 5-10% pulled off
		1186-2	368	102.5@ 340psi	31	About 90-95% of aggregate broke through; 5-10% pulled off
		1186-1	405	112.5@ 402psi	31	About 90-95% of aggregate broke through; 5-10% pulled off
			AVG.=352			

TABLE 4-12

UNIAXIAL TENSION TESTS

Project	Age days	σ_z - Stress psi	ε_z - Strain μ in/in*	ε_ϕ - Strain μ in/in*
Turkey Point 6" x 12" Cyl.	28 28 28	+405) +370) +392[+] +400)	+105) +110) +108 +110)	-20) -20) -22 -25)
Turkey Point NX core-From Dome Concrete	11 mo 11 mo	+330) +365) +347	+80) +105) +93	-22) -25) -24
Containment A 6" x 12"	42 21	+490) +420) +455	+90) +125) + 108	-20) -16) -18
Containment B				
Containment C 6" x 12"	80 75 73	+404) +410) +408 +415)	+70) +75) +72 +72)	-15) -16) -16 -16)
Containment D 6" x 12"	28 28 28	+340) +300) +321 +324)	+76) +64) +69 +66)	-08) -08) -9 -12)

* Smallest strain at ultimate load. Some specimens continued to strain while maximum load was held.

\+ Average

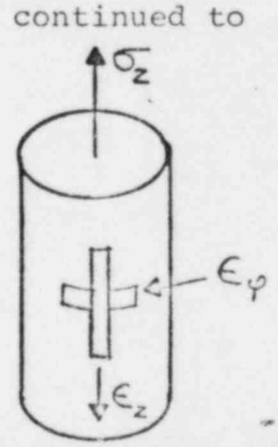


TABLE 4-13

UNIAXIAL COMPRESSION TESTS

Project	Age days	σ_z-Stress psi	ε_z-Strain μ in/in	ε_ϕ-Strain μ in/in
Turkey Point 6" x 12" cyl.	28 28	-5500 -6030	-2700 -2650	+450 @ 5000 psi +400 @ 5000 psi
Containment A 6" x 12"	21 21	-6700 -7000	-2000 -2000	+350 @ 6000 psi +475 @ 6000 psi
Containment B 6" x 12"				
Containment C 6" x 12"	61 56	-7050 -6600	-1850 -1850	+550 @ 6000 psi +400 @ 6000 psi
Containment D 6" x 12"	48 48	-6550 -6800	-1850 -1750	+550 @ 6000 psi +400 @ 6000 psi

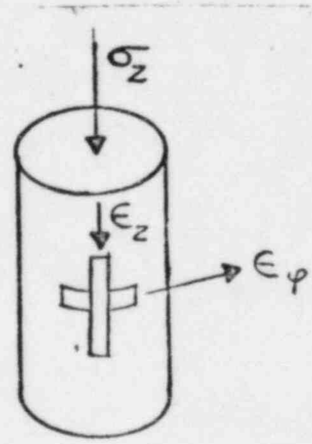


TABLE 4-14

BIAXIAL COMPRESSION TESTS

Project	Age days	σ_r-Stress psi	ε_z-Strain μ in/in	ε_ϕ-Strain μ in/in	Remarks
Turkey Point 6" x 12" cyl.	26	-6000	+545	-	Indicative of ultimate biaxial concrete stress
	25	-5300	+300	-	Possible failure due to hydraulic fluid in void
	25	-4700	-	-	Cracked near end due to failure of membrane
Turkey Point NX core From Dome Concrete	11 mo	-4700	-	-	Tensile break, possible fluid penetration
	11 mo	-6000	+1200	-	Good failure of concrete
Containment A 6" x 12"	45	-4600	+700	-	No failure No cracks
	28	-7500	+1200	-	No failure No cracks
Containment D 6" x 12"	40	-5550	+530	-1030	Membrane failure caused crack in concrete

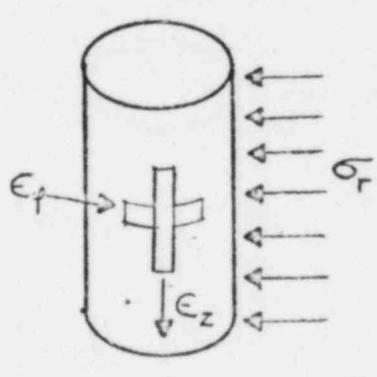


FIGURE 4-1
CONCRETE SPECIMENS

5.0 ANALYTICAL INVESTIGATION

In order to determine why the containment structure dome delaminated various analyses were performed. These analyses covered all the known items which could have caused the delaminations or been a contributing factor.

5.1 CRANE LOADING

A 50 ton Bay City truck crane was set up at the apex of the dome 1 month after completion of concrete placement and 4 months before the start of post-tensioning, for handling tendons and tendon installation equipment. The crane location is shown in Figure 5-1.

The crane loads were resisted by the outriggers. The dead load for a single outrigger is 13K. Considering the rated 5.5K lifted load at a radius of 70 ft. an outrigger downward load of 17.5K results. The dead load, together with a 50% impact factor on the lifted load, yields a maximum downward load of 39K. "Local Stresses In Spherical Shells From Radial Or Moment Loadings" by Bijlaard, Welding Research Council Bulletin No. 34 was used to estimate the stresses from the 39K concentrated load. In the analysis the shell was assumed to be 31" thick with the initial 8" pour neglected. A dome radius of 89' and a 2' diameter loading area was also assumed.

The predicted stresses are as follows:

Maximum Meridional Flexure:	± 86 psi
Maximum Hoop Flexure:	± 26 psi
Maximum Meridional Membrane:	- 11 psi

Due to the low magnitude of the calculated stresses the crane is not considered a significant contributor to delamination causes.

5.2 TEMPERATURE AND MOISTURE

An assumed worst temperature gradient (for compression on the outer surface) is shown in Figure 5-2.

Using the following formula the peak compressive stress on the outside face is predicted to be:

$$\sigma = \frac{\Delta T \sigma E}{1-\nu} = \frac{36(5.\times 10^{-6})\ (4.5\times 10^{+6})}{(1-.25)} = -1080 \text{ psi}$$

The stress distribution will be similar to the temperature gradient plot with a tendency to reduce to very small values within 4" from the surface.

To simulate a condition of wetting for a prolonged period of time, the following tests were performed. Four concrete specimens, approximately 10" x 10" x 4", removed from the dome were soaked in water. Using a Whittemore strain gage, 3 of the specimens were found to expand to a strain of 167 μ in/in after 14 hours of soaking. After 40 hours of soaking the specimens remained constant with an accumulated strain of 233 μ in/in. One of the 4 specimens had very little change in dimensions. Converting the strain to stress yields

$$\sigma = \frac{\varepsilon E}{(1-\nu)} = \frac{(233\times 10^{-6})\ (4.5\times 10^{6})}{.75} = -1400 \text{ psi}$$

if the specimen would have been fully restrained.

The stresses from temperature and moisture do not peak simultaneously since one tends to reduce the other. Both are primarily surface effects and they would not cause delaminations 15" in depth. However there is a possibility that these two items could have been a contributor in causing shallow delaminations.

5.3 SHEATHING FILLER PRESSURE

One of two pumps used for sheathing filling had a stall pressure of 250 psi with the other lower. With all vent valves closed the pressure in the sheaths would not have exceeded 150 psi due to head losses. In a few isolated cases the vent valves were closed, however with only a few tendons affected, this is not of concern. Since the lowest known temperature of the filler during pumping was 90 F, it was assumed that the filler had zero pressure at 85 F. Thermocouple measurements have indicated a temperature of 97 F at an 11" depth when interpolating between the inside and outside readings. Through past testing the filler pressure has been found to rise 8 psi for each 1 F change. Therefore it is possible that a 96 psi pressure could have existed in the sheathing. A finite element analysis was performed subjecting a portion of concrete with a 4" diameter hole 11" deep to a pressure of 100 psi. The analysis indicated that the peak radial stress would be 80 psi at the edge of the hole with the stress rapidly decreasing away from the hole. The radial tension is not high enough to case delaminations and a direct tension load of this kind would not have caused the multiple and shallow delaminations actually found above the sheaths.

5.4 RADIAL TENSION CAUSED BY PRESTRESSING

Since the tendons are not located on the outside surface, radial tension will exist near the outside face of the concrete. To estimate the magnitude of radial tension the analytical calculations were done in two parts and superimposed. The maximum radial tension should exist near the upper layer of tendons, which are 11" from the outer surface, because this is the maximum thickness of concrete without direct radial compression from the tendons.

The first part of the solution considered the effects of all tendons other than the first layer. Since the prestressing essentially loads the shell with a pressure of 100 psi then the pressure from all tendons other than the first layer will be 5/6 (100) = 83.3 psi. Due to displacement compatibility the following relationship must exist

$$\delta = C\frac{P_1R^2}{Et_1} = C\frac{P_2R^2}{Et_2} \quad \text{or} \quad P_1 = \frac{t_1}{t_2}\,P_2$$

Where P_1 is the tension in the top 11" of concrete, P_2 is the applied pressure, t_1 is the thickness of the top layer and t_2 is the total thickness. Then the radial tension is $P_1 = \frac{11}{39}$ (83.3) = 23.5 psi. A finite element analysis of a small portion of the dome was made to evaluate the local effects of the top layer of tendons. The analysis indicated that the peak radial tension was +68 psi occurring near the edge of the tendon sheathing void. The radial stress reduced greatly a few inches from the hole. Superimposing the two results lead to the radial tension distribution shown in Figure 5-3.

The analysis indicates that radial tension is not a major concern due to the magnitude and distribution. In addition a failure from radial tension should not lead to multiple delaminations close to the surface as were found in the investigation of the structure.

5.5 UNBALANCED LOADS FROM PRESTRESSING

A study was made to determine the force distribution on the dome due to the reported prestressing sequence. Each tendon group was divided into 2 zones giving a total of 6 zones. At various times, such as when 50% of the total tendons were tensioned, each zone was examined to determine the amount of normal pressure from the tensioned tendons within a particular zone. The normal pressures from each zone were then superimposed. Since the normal pressure from all the tendons being tensioned is approximately 100 psi, then the resulting pressure also indicates the percentage complete for a particular area. Figure 5-4 shows the results for 40, 50 and 60% completion of prestressing. When 50% of the total tendons were tensioned, one area had effectively 73.8% of its total load whereas another area only had 28.4%.

In order to determine the effect of these unbalanced loads an analysis was performed for a homogeneous containment structure dome. The analysis did not include the effects of concrete cracking or construction joints. The dome was analyzed for the most severe case when the prestressing was 50%

complete. The triangular areas shown in Figure 5-4 were further subdivided by using one large and three small circular areas as shown in Figure 5.5.

Solutions were obtained by loading a dome at the apex by loads distributed over the same areas as those shown in Figure 5-5. After obtaining this data a final solution was obtained by superimposing the effects of any loaded circular area which appreciably affected the location under consideration. The following table shows the maximum calculated stresses on the outside surface, together with the results of applying 100% of the prestressing load (100 psi pressure) distributed uniformly over the dome surface.

	Calculated Stresses (psi)	
	Unequal 50% Loading	Uniform 100% Loading
Meridional		
Membrane	- 727	-1389
Bending	- 974	- 300
Combined	-1701	-1689
Circumferential		
Membrane	- 876	-1450
Bending	- 660	- 200
Combined	-1536	-1650

As indicated above the bending stresses are great enough, so that when combined with membrane stresses, the combined stresses at 50% loading are slightly higher than the stresses under full uniform loading. These loads are considered to be a contributor.

5.6 CONSTRUCTION JOINTS

In the analysis of why the delaminations occured the construction joints deserved special attention because of the following:

(1) As shown by the coring results, the delaminations reached a maximum depth adjacent to the meridional construction joint.

(2) The delaminations appear to have some degree of symmetry about the meridional construction joint with a tendency to approach the surface as they progress away from this joint.

(3) Sheathing filler is present in the meridional construction joint indicating that separation existed.

To establish a base case for the dome stress distribution and magnitude, due to dome prestressing, a shell computer program was utilized. The program handles axisymmetric loads and uses a classical solution after the shell has been divided into small cone frustums. The results of this analysis for a homogeneous containment structure are given in Figures 5-6 and 5-7. The maximum combined meridional stress was found to be -1689 psi at the outer surface. And the maximum combined circumferential stress was found to be -1650 psi at the outer surface.

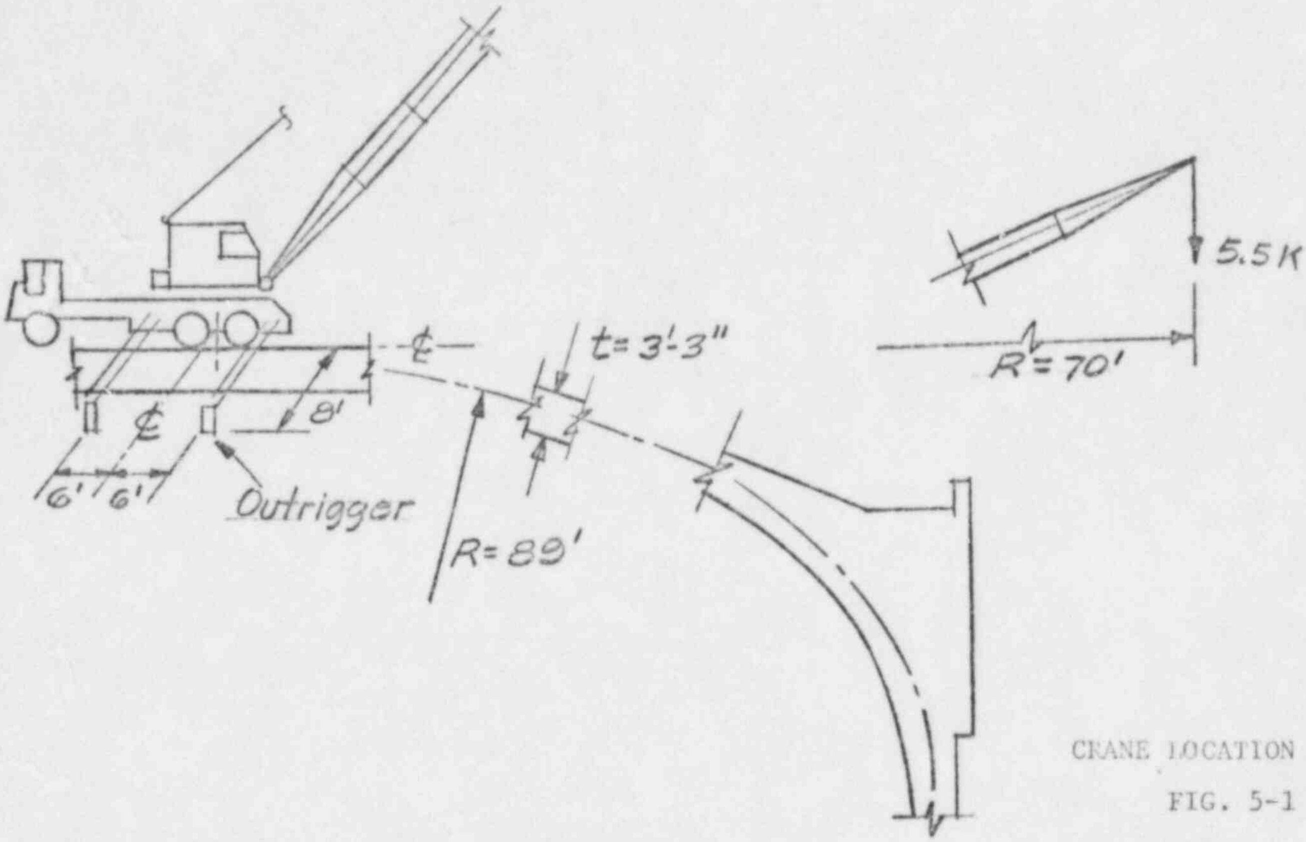
To determine the effects of the circumferential construction joints in conjunction with dome prestressing an analysis was performed using the shell program previously described. The construction joints were simulated by hinges. The line of thrust was through the center of the elements and therefore the results do not consider the effects of an eccentric thrust.

Figures 5-8 and 5-9 show the distribution and magnitude of stress at the outside surface. The analysis indicates that this case is even less severe in the meridional direction, than the base case since the stress at the outside surface of the dome is -1620 psi. Due to the assumed hinges the circumferential stress increased considerably at a radius of 42 feet with a magnitude at the outer surface of -2600 psi. This stress increase could have been a local contributor to the delaminations.

As the field investigation progressed more evidence became available that many areas surrounding the meridional construction joint were not of the quality necessary to resist the applied loads without considerable redistribution of load. Figure 5-10 shows a condition which could have resulted in the formation of delaminations. Since expanded metal was used as a form for the joint, voids or soft spots could have resulted. As the structure was prestressed high compressive stresses would result at localized areas. As shown in Section 4 a high compressive stress will result in a strain failure in a plane parallel to the load. If the joint area was effectively reduced to 1/3 of the dome thickness then the resulting stress would be 4,500 psi, enough to cause failure. Figure 5-11 shows another case which could result in delaminations. In this case, the joint had poor tensile capability due to the lack of bond. When the prestressing loads occurred the joint would rotate due to the unbalanced loading. This condition would force the structure to carry high loads near the upper surface. Again high stresses would result and the strain failure would occur. Eventually equilibrium would be obtained.

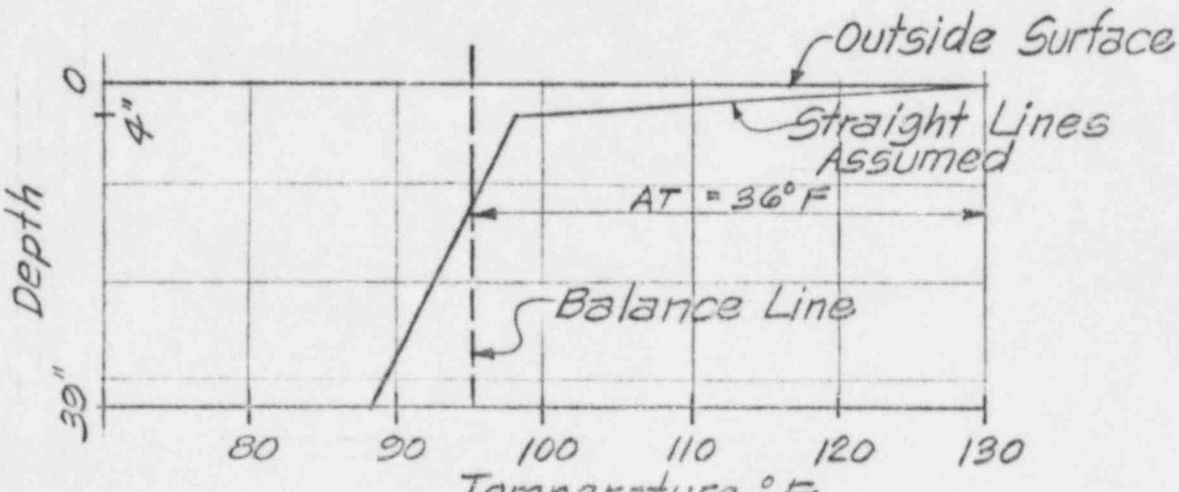
Two plexiglass domes were obtained to help visualize the phenomenon of the joint rotation. One of the domes was a hemisphere and the other was a hemisphere cut in half and then taped together so that only shear could be transmitted through the joint. Figures 5-12 and 5-13 show the split plexiglass dome before and after applying a load across the joint. Two plexiglass tabs were mounted normal to the surface, pointing inward, on each side of the simulated joint. The photographs indicate that as the load is applied the joint rotates, opening at the bottom.

An analysis was performed (using the shell program previously described) to simulate the effects of having the membrane force distributed over a small area near the surface with resulting eccentricity. In order to simplify the analysis a hemisphere was used with its equator as the construction joint. The geometry of the shell together with its comparison with the real shell geometry are shown on Figure 5-14. The shell was loaded with 100 psi in the area included by the 52° angle. The results which are also given in Figure 5-14 illustrate that both the reduced joint area and eccentricity of membrane load leads to large calculated stresses. This analysis illustrates how faulty construction joints lead to stresses high enough to be considered one of the main contributors in causing the delaminations.



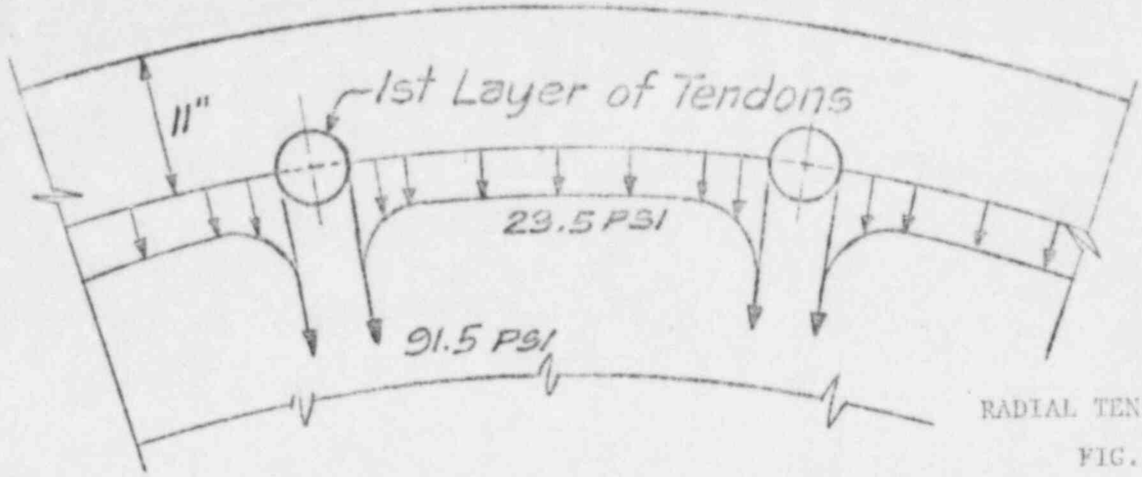
CRANE LOCATION

FIG. 5-1



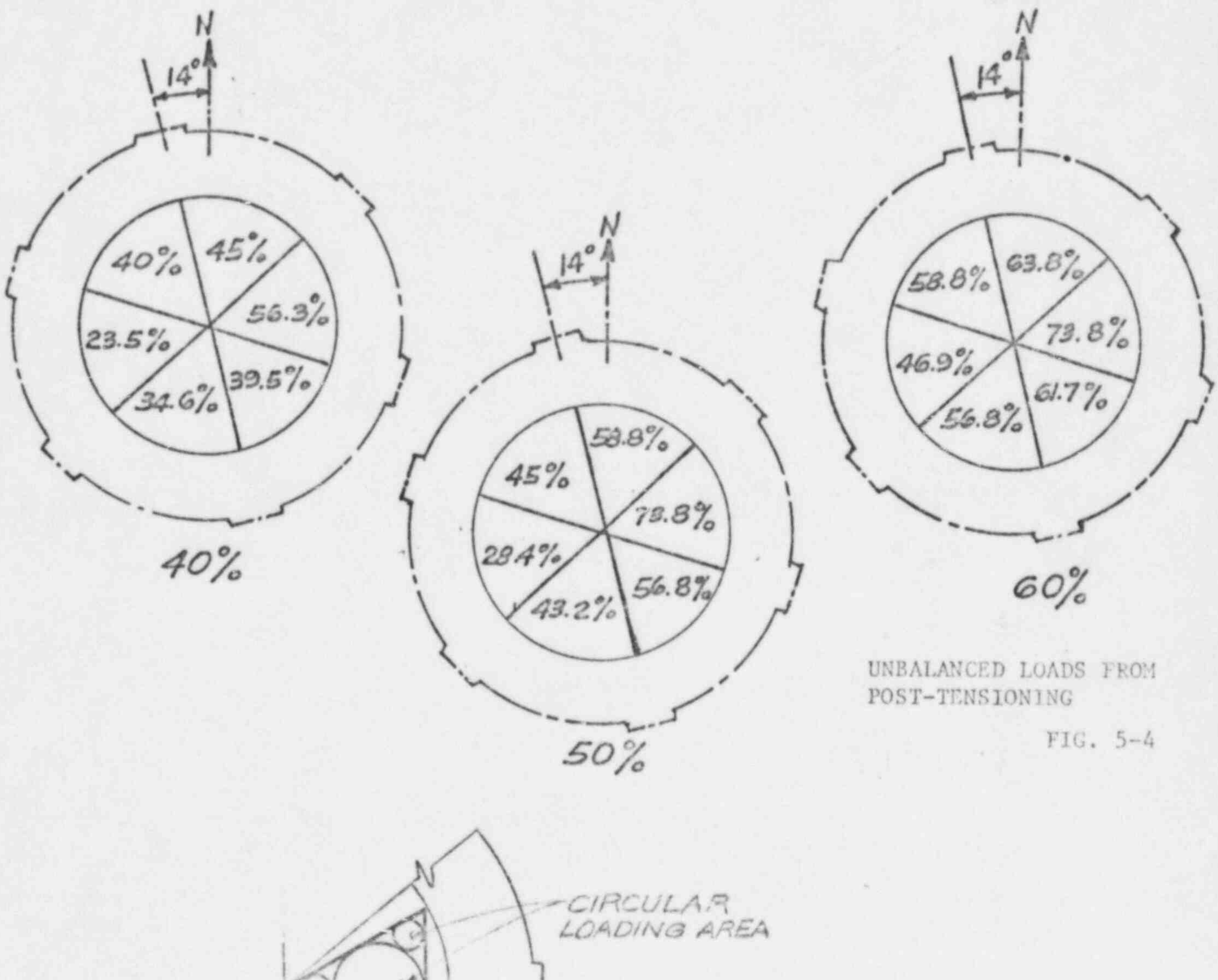
THERMAL GRADIENT

FIG. 5-2



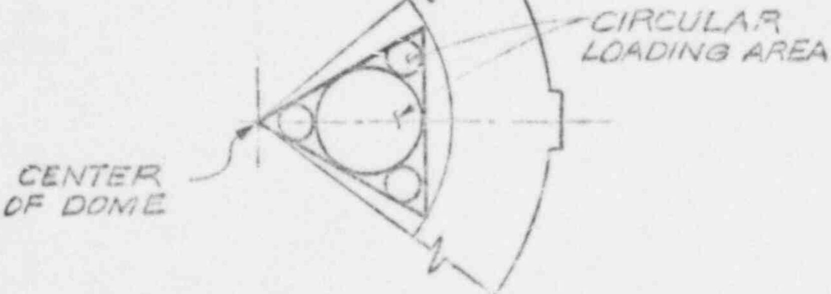
RADIAL TENSION

FIG. 5-3



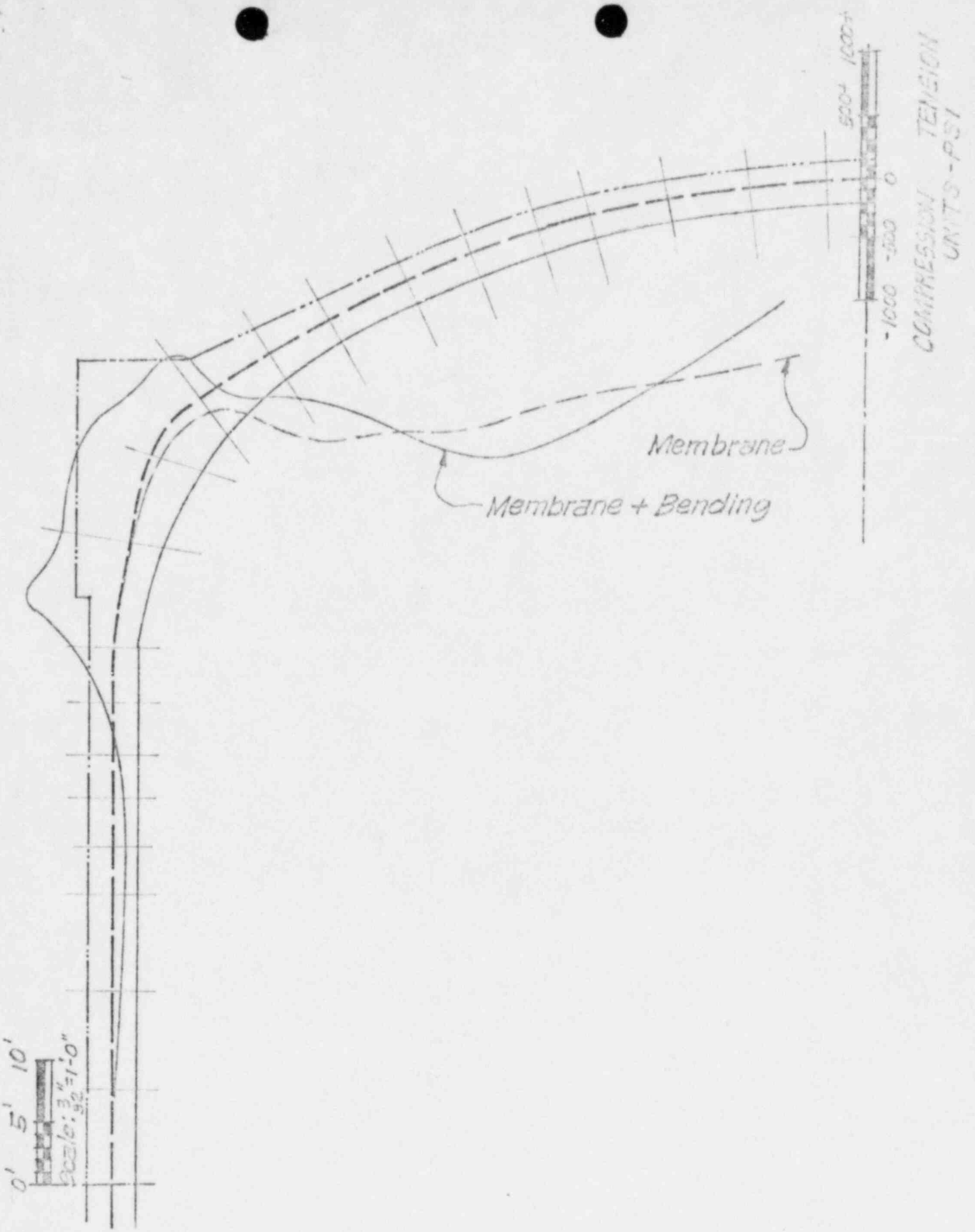
UNBALANCED LOADS FROM POST-TENSIONING

FIG. 5-4



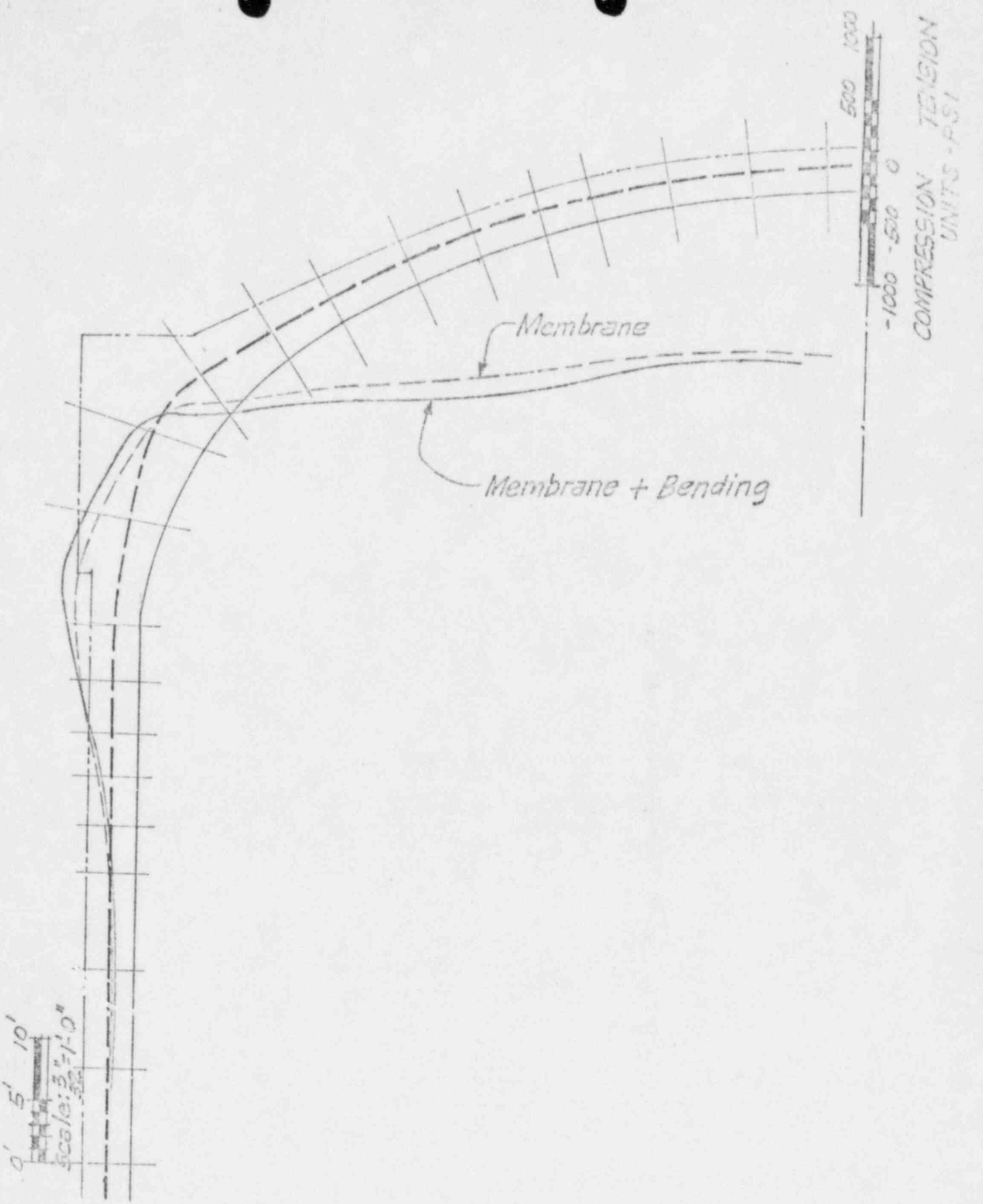
LOADING AREAS

FIG. 5-5



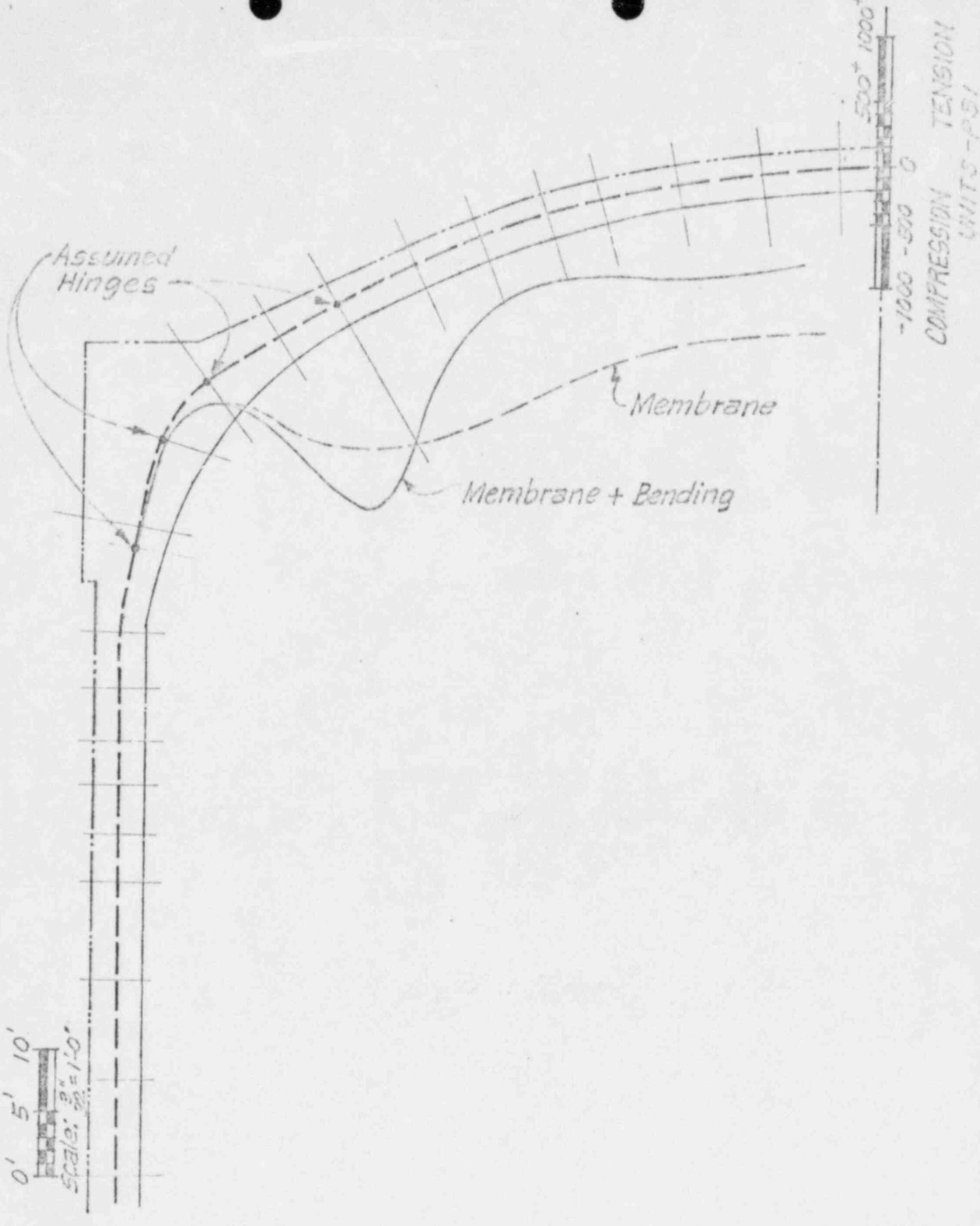
MERIDIONAL STRESS AT THE OUTER SURFACE FROM DOME POST-TENSIONING

FIG. 5-6



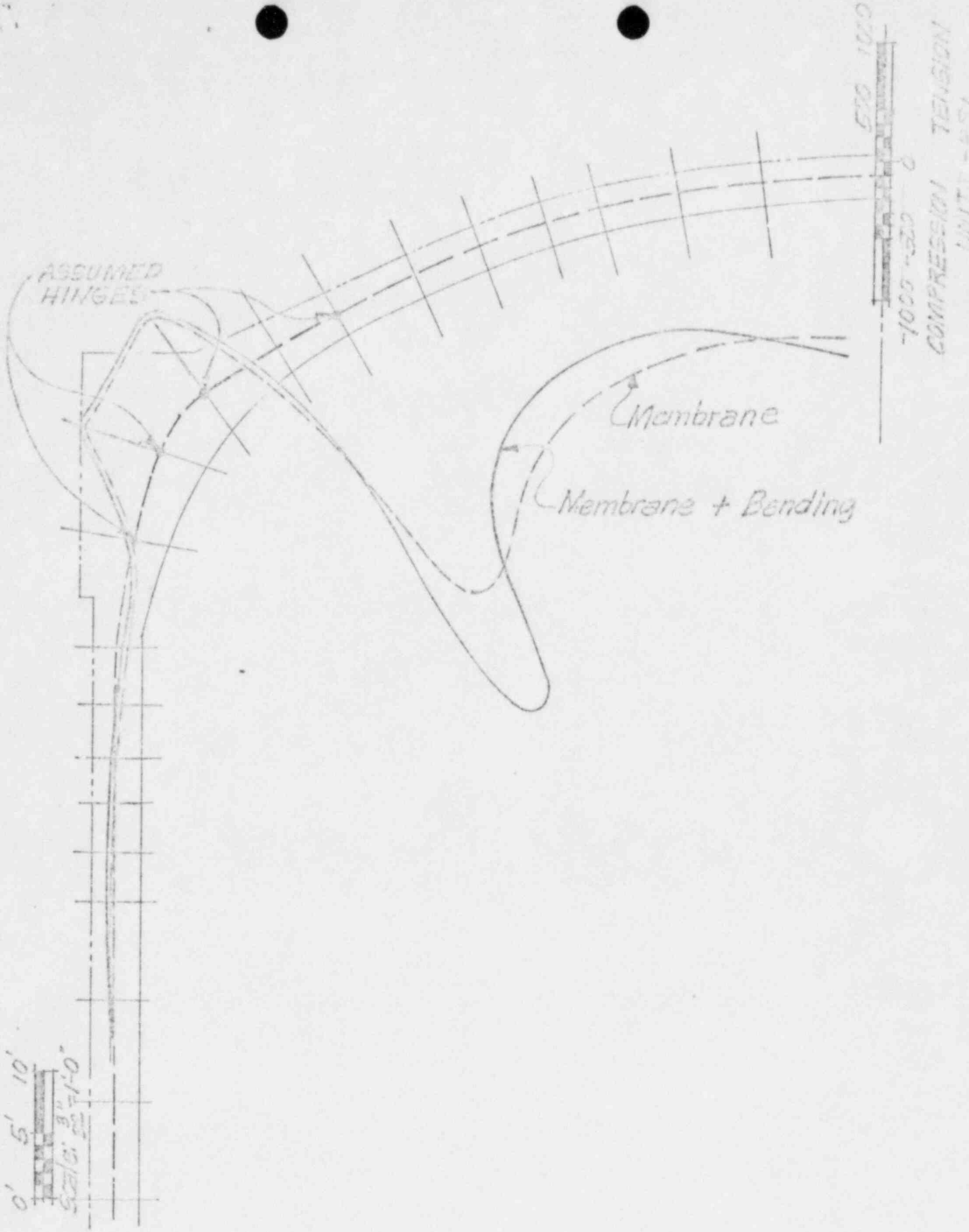
CIRCUMFERENTIAL STRESS AT THE OUTER
SURFACE FROM DOME POST-TENSIONING

FIG. 5-7



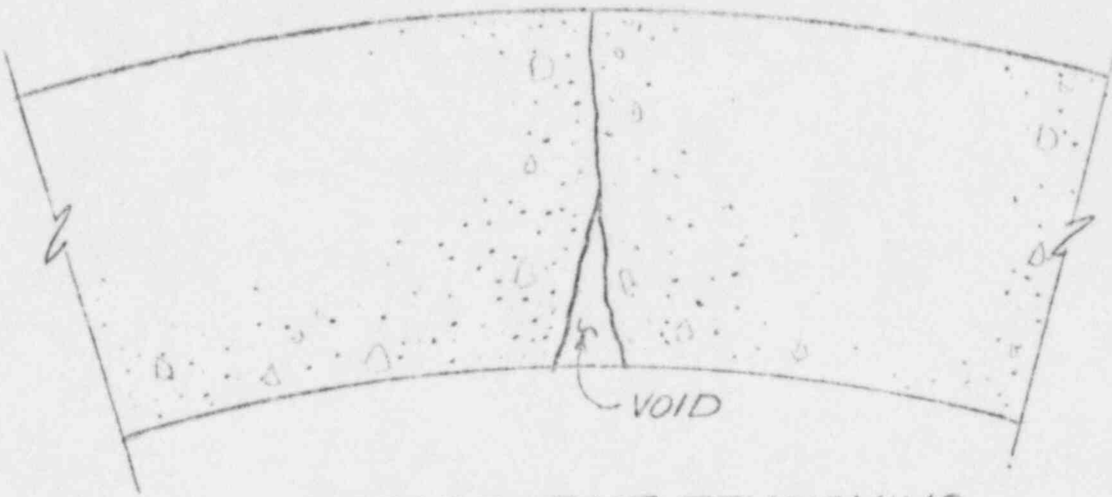
MERIDIONAL STRESS AT THE OUTER SURFACE FROM DOME POST-TENSIONING

FIG. 5-8

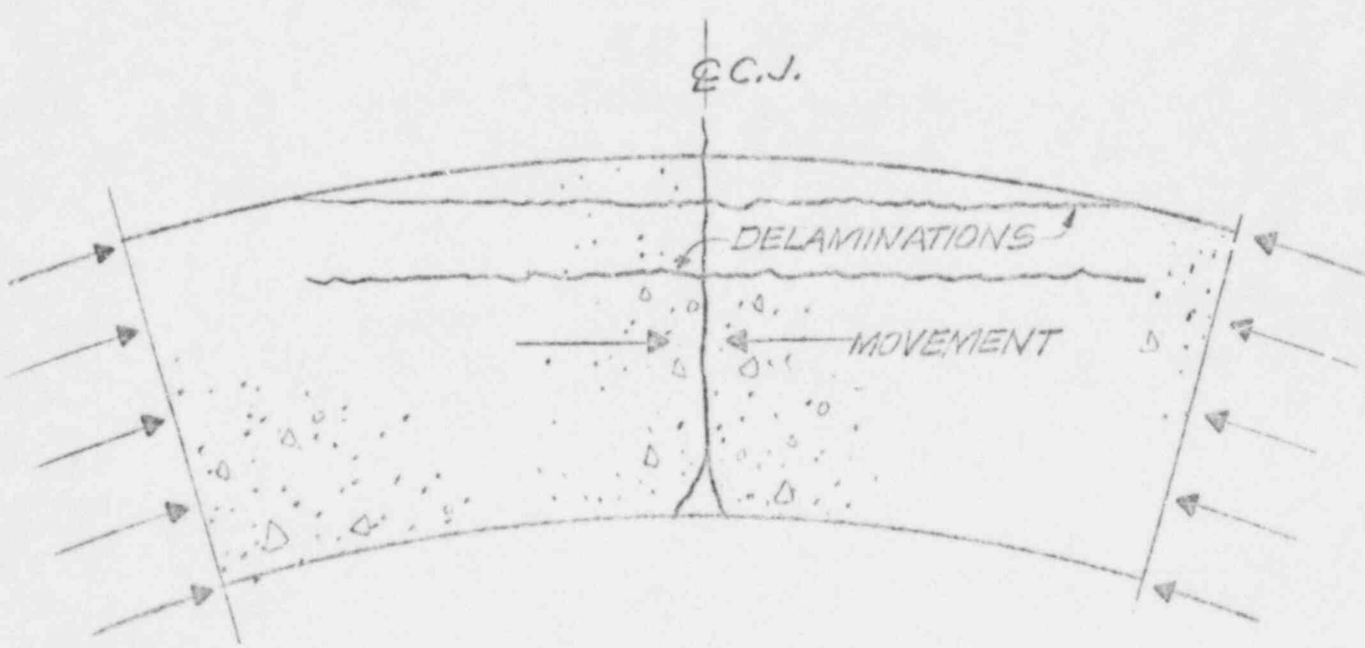


CIRCUMFERENTIAL STRESS AT THE OUTER SURFACE FROM DOME POST-TENSIONING

FIG. 5-9



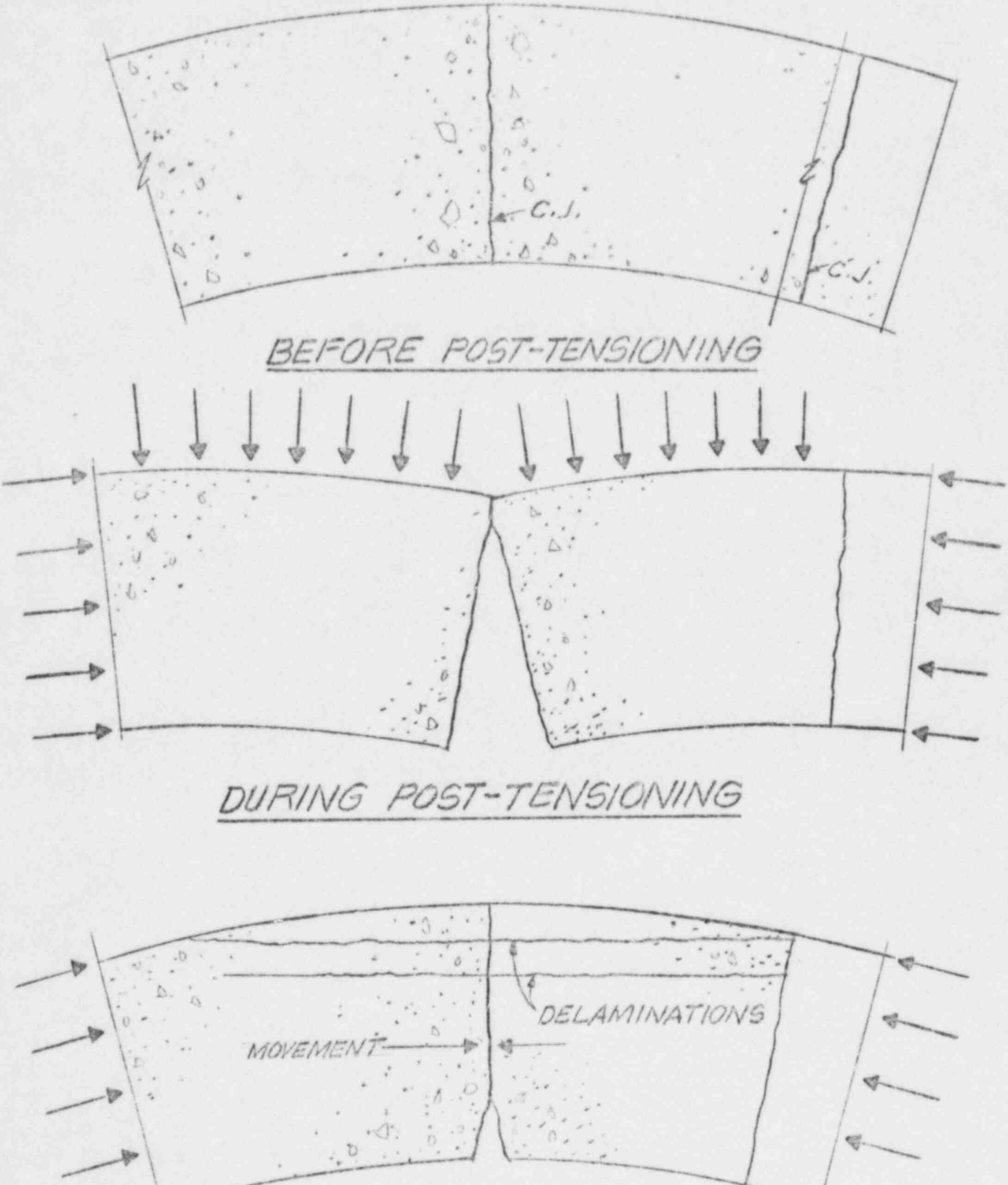
BEFORE POST-TENSIONING



AFTER POST-TENSIONING

NONUNIFORM BEARING CASE 1

FIG. 5-10



NONUNIFORM BEARING CASE 2

FIG. 5-11

FIGURE 5-12
PLEXIGLASS DOME
WITHOUT LOAD

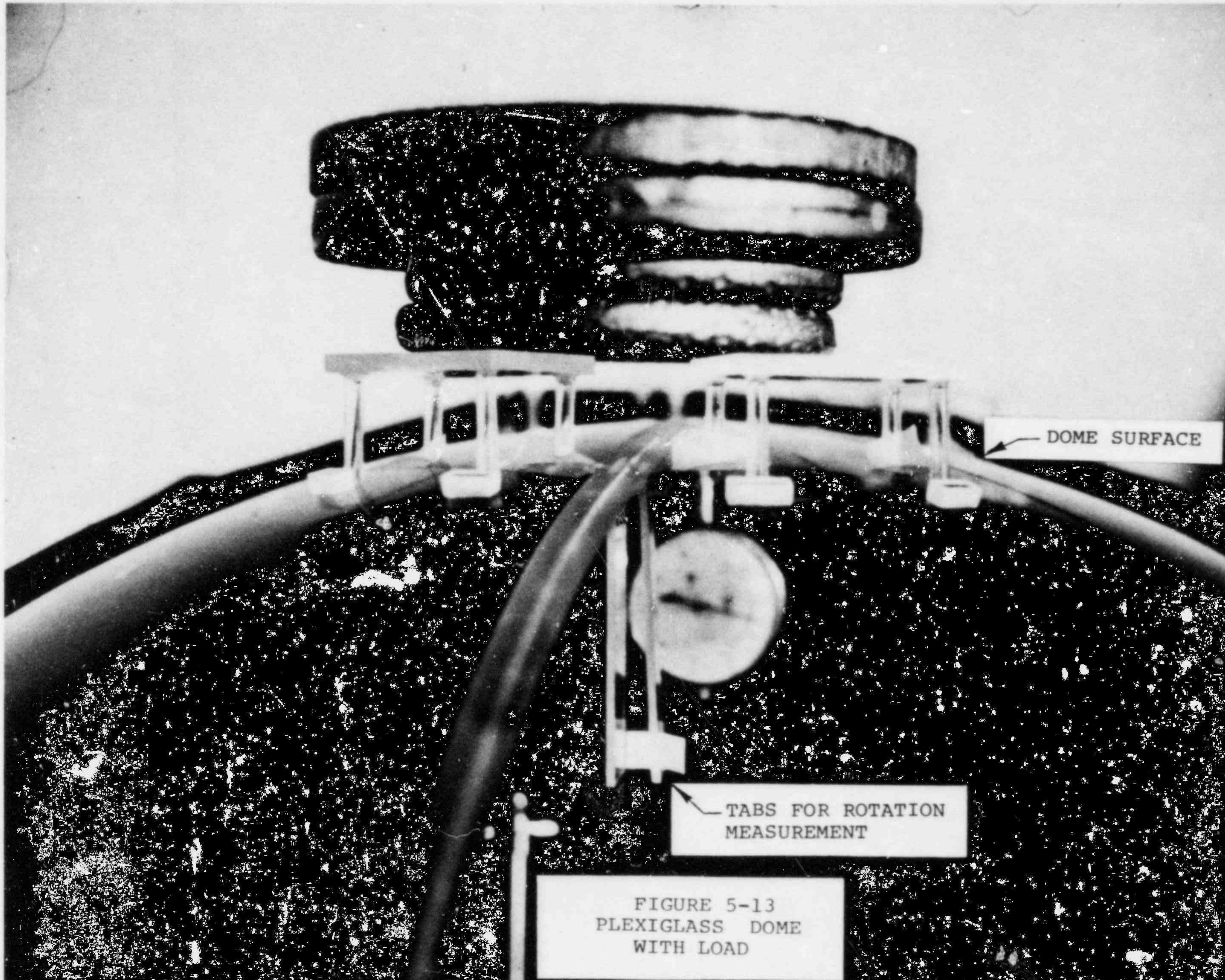
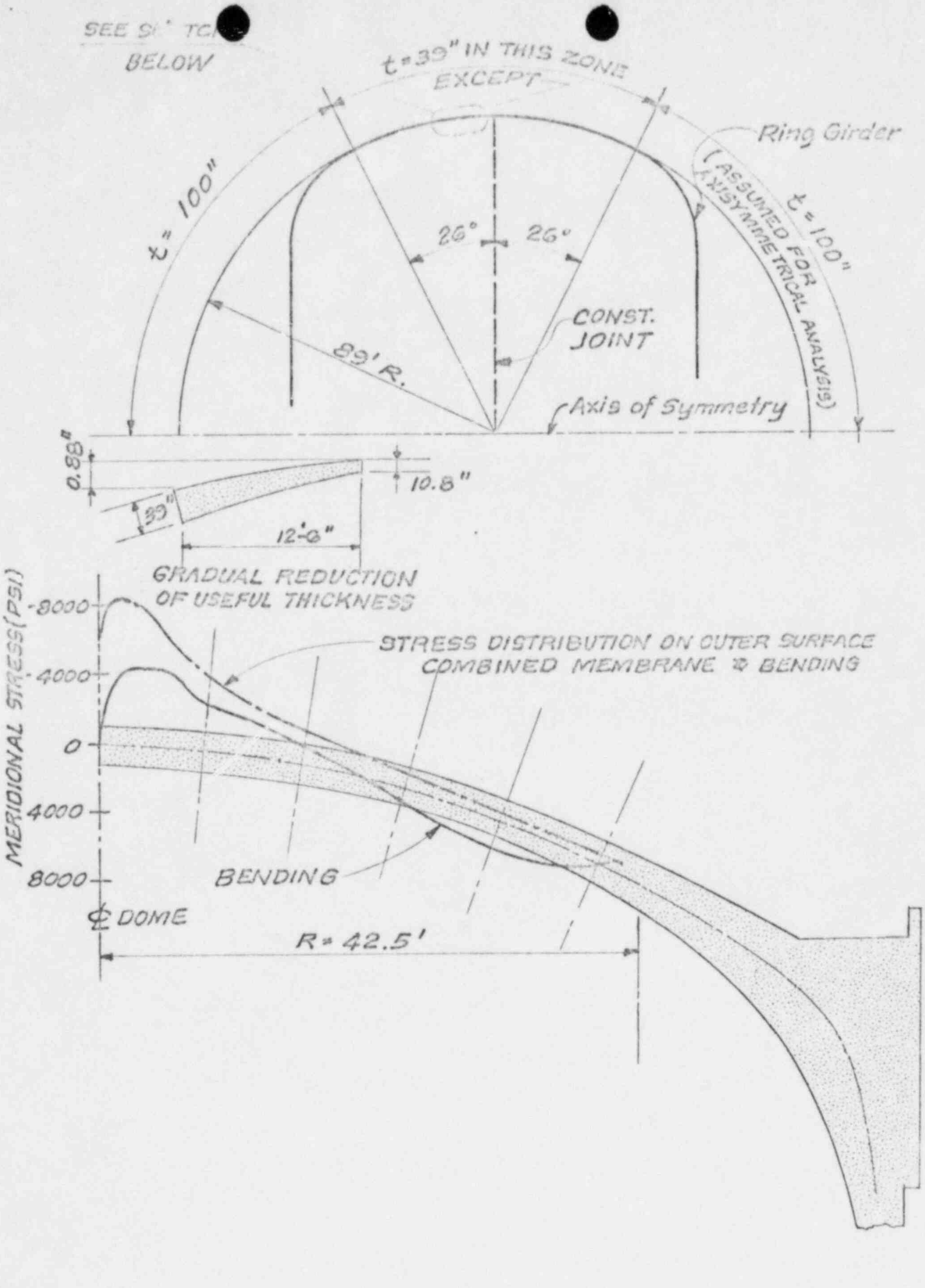


FIGURE 5-13
PLEXIGLASS DOME
WITH LOAD



AXISYMMETRICAL SIMULATION OF THE
MERIDIONAL CONSTRUCTION JOINT

FIG. 5-1

6.0 REPLACEMENT OF CONCRETE

After concrete removal and initial surface preparation, construction will be completed by replacing the concrete in accordance with Specification 5610-C-61 (Proprietary). The specification requires final treatment of the existing concrete surfaces in advance of and during the concrete replacement period.

The final surface treatment method specified was tested prior to use. Its purpose is to provide bonding of the new concrete to old with capabilities equal or better than methods specified in ACI 318-63.

The mix design quantities for the replaced concrete are those used for the existing concrete. Type II, moderate heat of hydration cement is specified as well as retarding and water reducing admixtures. Maximum temperatures for the concrete are specified.

Concrete placement utilizes the "preshrunk" method which requires that the concrete be revibrated in relatively shallow lifts.

The concrete must have, at the time of restressing, a cylinder compressive strength equal or greater than the 5000 psi minimum specified for the containment concrete. The bonding and contact at interfaces between new and original concrete must provide for force transfer such that the replaced concrete provides assistance to the original concrete (which has a proven capability for resisting the prestressing forces) in sustaining the prestressing forces. The capability for providing such assistance will be demonstrated by measurements for comparisons with other measurements such as the strain values given in the material property test results contained in this report.

A consultant on the concrete replacement plan was Mr. Lewis H. Tuthill, retired, formerly of the California Department of Water Resources, Division of Design and Construction.