February 1, 1982

ROBERT E. GINNA NUCLEAR POWER PLANT

CONTAINMENT VESSEL EVALUATION

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I. INTRODUCTION

As part of the Systematic Evaluation Program (SEP), Structural Mechanics Associates, Inc. (SMA) in support of Lawrence Livermore National Laboratory (LLNL) performed an evaluation to determine the capacity of the Ginna Containment to withstand combined seismic and LOCA load conditions. This evaluation is reported in Reference 1. In this work, concerns were expressed regarding the liner and concrete stresses in the region of the dome where liner insulation terminates (see containment configuration in Figure 1). Analysis results from the report indicate that accident temperatures would develop high liner compressive stresses in the uninsulated dome region and relatively low liner stresses behind the insulation. Since the liner in the dome is anchored to concrete by headed anchors (studs), this difference in liner compressive stresses would produce a shear force in the studs at the insulation termination interface. The main concern expressed in Reference 1 concentrated on whether these studs are able to withstand this induced shear force. In Reference 1 (page 18) the following was concluded: "Based on a shear capacity per stud of approximately 16.5 ksi, the force developed in the buckled liner exceeds the capacity of shear studs."

Gilbert/Commonwealth (G/C) was requested by Rochester Gas and Electric Corporation (RG&E) to review and evaluate Reference 1. The resulting preliminary G/C evaluation appears in Reference 2. Part of this evaluation addressed the differences between the accident pressure and temperature curves of the SMA report versus those from the Ginna FSAR. These differences are summarized below:

1. The accident pressure curve used in SMA report is different from that presented in the FSAR. Attached Figure 2 shows a comparison between the SMA report pressure curve and the Double Ended (D.E.) break pressure curve appearing in FSAR Figure 14.3.4.2. The peak values from both curves are about the same: 68.5 psia (53.8 psig) from SMA and 67.2 psia (52.5 psig) from the FSAR. The FSAR curve

decays more rapidly than the SMA curve. As an example, at 5 minutes into the accident the FSAR value is about 56.5 psia (41.8 psig) compared with the SMA value of 65 psia (50.3 psig). The peak pressures from both the SMA curve and the FSAR curve are less than the Ginna Design Pressure of 60 psig.

The effect on liner stresses (based on an uncracked concrete model) of differences between the SMA and FSAR pressure curves is not very significant. Reference 1 gives a value of dome liner stress of 4 ksi tension resulting from the 53.8 psig peak accident pressure. This is small relative to the 56 ksi liner compressive stress which is produced by the peak accident temperature of 285° F, as reported by SMA.

2. The SMA accident temperature curve is different from the temperatures which would be associated with the FSAR D.E. break pressures. This latter temperature curve was calculated by G/C and is based on the saturated steam temperature for the D.E. break pressure, assuming that the containment pressure is equal to the sum of the vapor pressure and the air pressure. A comparison of the two curves is shown in Figure 3. Similar to the pressure case, both curves exhibit about the same peak values: 280° F (FSAR) and 285° F (SMA). However, the FSAR curve decays more rapidly.

   In the heat transfer analysis performed by SMA, the film coefficient was not used. This is expected to produce conservatively high temperature and correspondingly high compressive stresses in the liner. G/C performed a heat transfer analysis (Reference 4) incorporating a film coefficient and the resulting maximum liner temperature was 250° F at approximately 5 minutes after a LOCA. The SMA analysis incorporated a temperature in the liner of 285°F.

Reference 2 also addressed the conclusions in the SMA report (Reference 1) regarding the integrity of the studs. First, the stud capacity of 16.5 ksi given in the report did not appear in the reference quoted*. This reference (Table 15) gives a shear capacity of 16.56 kips for a 5/8 inch diameter stud and 23.86 kips for a 3/4 inch diameter stud. There may have been an error in the SMA report in that the value was intended to be 16.5 kips. However, this value applies to 5/8 inch diameter studs, but 3/4 inch diameter studs were used for the dome liner based on the available information. Also, the report did not indicate whether a liner-stud interaction analysis was performed. The conclusions in the SMA report are apparently based on the conservative assumption that a single stud would carry all of the unbalanced liner force generated at the insulation termination interface. In reality, Reference 2 points out, a series of studs would deform in response to the unbalanced force which would redistribute the liner stresses and results in a much reduced value for the maximum stud force.

Because of the differences in the loads discussed in items 1 and 2 above and due to the questions regarding the stud integrity, G/C performed the analyses outlined in Section II. The results and conclusions of these analyses are presented in Sections III, IV, and V. Comparisons are made with the SMA results in Reference 1.

Subsequent to this work, on January 15, 1982 G/C received for evaluation a second draft of the SMA report (Reference 9). There are significant differences between the two SMA report drafts regarding the magnitudes of the accident pressure and temperature loads. Also the possible liner and stud integrity problems identified in the first draft (Reference 1) were sustained. Section VI addresses these items.

*"Embedment Properties of Headed Studs", TRW Nelson Stud Division, 1977.

II. OBJECTIVES

At RG&E'S request, G/C performed an independent analysis including the following items:

A. Obtain liner and concrete stresses for accident temperature and accident pressure conditions using the information presented in the FSAR. Compare the results with the SMA analysis results in Reference 1.

B. Conduct a liner-stud interaction analysis to evaluate the stud integrity. This analysis will use the SMA buckling and post buckling liner capacities in conjunction with both SMA and G/C liner stresses.

III. LINER AND CONCRETE STRESSES

A. Analysis Basis

The stresses in the liner and concrete were calculated by the computer program KSHEL1 (Reference 3), which is based on small deformation, elastic thin shell theory.

B. Model

In the stress analysis, the containment was modeled as an axisymmetric shell of revolution with a wall thickness of 42.375 inches in the cylinder and 30.375 inches in the dome (see Figure 1). A three-layer model was used. The inner layer represents the 0.375 inch steel liner. The middle and outside layers comprise the concrete. The model was set up so that the non-linear accident temperature distribution through the shell thickness could be approximated by specifying the appropriate thermal conductivities in each layer. The boundary conditions at the base of the cylinder were taken as spring supported in the radial direction with no meridional moment resistance. No movement was allowed in the vertical direction at the shell base. The radial stiffness of tension rods which attach the cylinder to the base slab was calculated to be 48.33 kips/in./in.

C. Loads

1. Accident Temperature

In order to determine the most severe temperature distribution in the shell wall, G/C conducted a heat transfer analysis (Reference 4) considering the film coefficient effect. Figure 4 shows part of the analysis results. It indicates that the uninsulated steel liner would reach a peak

temperature of 250° F at about 5 minutes into the accident. Accordingly, the associated temperature distribution in the wall at this time was used in the stress analysis.

2. Accident Pressure

The peak pressure does not occur concurrently with the peak accident temperature. Since the liner stress is tensile for pressure loads and compressive for temperature loads, considering peak values concurrently does not produce the most severe liner stress for the combined accident pressure and accident temperature case. Therefore, the pressure value corresponding to the maximum temperature (5 minutes into the accident) is determined to be 56.5 psia (41.8 psig) from the D.E. break pressure curve provided in FSAR Figure 14.3.4.2, and this was used in the liner stress analysis.

D. Results

Figures 5 to 12 provide the curves which compare G/C and SMA results for liner and concrete stresses. It is seen that the trends of the results are very similar to each other. However, there are differences in values. Generally, G/C obtained comparatively lower stresses. This is expected since the pressure and temperature values used in the G/C analysis are less than those used in the SMA analysis. Table 1 lists the values of the meridional and circumferential (hoop) stresses for different locations and loading cases. From the table, the compressive stress in the uninsulated dome liner for combined accident temperature and accident pressure loads is 42.7 ksi which is about 20% less than the SMA value of 52.1 ksi. The maximum concrete stress is 4.2 ksi compression from the G/C analysis versus 6.0 ksi compression from the SMA analysis.

IV. STUD ANALYSIS

A. Analysis Basis

In the containment stress analysis, there are two assumptions involved which result in conservatively high values for the liner compressive stress. First, the concrete is assumed to remain uncracked during accident loading. In reality, dome concrete cracking occurred under the SIT pressure of 69 psig, and additional dome cracking is expected to occur under the accident load combination. This gives an effective stiffness of the reinforced concrete which is somewhat less than its uncracked value. This, in turn, produces less restraint on the liner, which results in lower compressive stress in the liner under the accident temperature condition. Also, it has been calculated that a liner panel buckles at around 25.5 ksi (Reference 1, p. 18) and then develops a maximum post-buckling stress of 29 ksi. Hence, the liner stress is limited by this value.

From the G/C results in Figures 5 and 6, the minimum compressive stress in the liner behind the insulation for the accident temperature condition is 1.6 ksi compression. The unbalanced liner stress in the region is 29-1.6 = 27.4 ksi, and this develops a significant shear force acting at the insulation termination interface. The unbalanced force is transmitted to the liner and studs behind the insulation. The shear force and displacement developed in the studs largely depend on the value of unbalanced liner stresses, the material properties of liner and the characteristics of stud force-displacement relationship. Based on the equilibrium and compatibility relations at each stud, a set of simultaneous equations are derived, from which the stud displacements and forces are solved by iteration on the stud displacements (Reference 5).

B. Model

The stud-liner structure is represented by a simplified one dimensional system as shown in Figure 13. The width of the liner panel is taken to be 24 inches, which is the stud spacing. One end of the liner is fixed approximately 16 feet from the buckled panel, which is the location of embedded channels at the shell springline (see Figure 1). At the other end a force exists which corresponds to the post-buckling stress 29 ksi. This condition results in movement of the studs in the adjacent panels with the largest displacement occurring adjacent to the panel where post-buckled stress is applied (Stud #1). The steel liner is represented as a linear elastic material with Young's modulus $E = 29{,}000$ ksi. The force-displacement relationship for the studs is obtained from the experimental results provided in Reference 6. From this reference, the force-displacement relationship can be represented by the empirical equation $Q = Q_u\ (1-e^{-18\Delta})^{2/5}$ (see Figure 14). In this equation Δ is the stud displacement; Q is the corresponding stud force; and Q_u is the ultimate stud capacity which is calculated to be 31.1 kips from Equation (3) of Reference 6 for a 3/4 inch diameter headed stud.

C. Loads

Using the post-buckling stress of 29 ksi from the SMA report, three different initial stress cases for the insulated liner portion are investigated: (1) 8 ksi, which is the value used in the SMA report for the accident temperature condition, (2) 1.6 ksi, which is the G/C result for the accident temperature condition, and (3) a varying liner panel stress for the $P_a + T_a$ condition (G/C results) which represents the calculated variation in panel stress starting at the end of the insulation and extending toward the springline.

D. <u>Results</u>

Figure 13 presents the comparison of the stud analysis results corresponding to three initial stress cases. The maximum stud force and stud displacement for each case are listed in Table 2 along with the ultimate stud capacity and the allowable stud displacement. The allowable displacement of a stud is limited by Table CC-3730-1 of the ASME Code (Reference 7) to 50% of its ultimate displacement for displacement limited loads. The load on the studs under consideration are classified as displacement limited loads since they are primarily due to the accident temperature condition. A value of 0.30 inches was used for the ultimate displacement of the 3/4 inch diameter by 3 inch long studs specified for the dome liner. This value is based on the test results presented in Reference 8. Therefore, an allowable displacement of 0.15 inches appears in Table 2.

It is noted that the maximum stud displacements for all three cases are within the Code allowable. Among them, the case of 1.6 ksi initial liner stress has the greatest values for stud displacement (0.098 inches) and force (28.9 kips).

V. CONCLUSIONS

A. For the liner and concrete stesses, the trend of the results from the G/C and the SMA analyses is similar; however, the magnitude of the stresses from the G/C analysis is lower. The G/C analysis used an accident pressure-time history from the FSAR along with the associated accident temperature-time history. In the SMA report new time histories appear for these two conditions. There is some difference between the G/C and SMA pressure and temperature profiles. However, the significant differences in the liner stresses occurs primarily because a film coefficient was used in the G/C heat transfer analysis, but it was not included in the SMA analysis. Thus, even though the peak air temperature under accident condition is approximately the same from both the G/C (280° F) and the SMA (285° F) analyses, a lower liner and concrete temperatures resulted from the G/C analysis. The effect of that is G/C obtained a maximum liner stress of approximately 48 ksi compression versus the SMA value of approximately 58 ksi (Figure 6). Also a maximum compressive concrete stress of approximately 4900 psi resulted from the G/C analysis versus 6800 psi from SMA (Figure 8). The difference of 10 ksi in liner stress is not significant because this stress is limited by the buckling strength of approximately 26 ksi. The G/C concrete stress is less than the 5000 psi design strength. The SMA value of 6800 psi is conservatively high due to the absence of a film coefficient in the heat transfer analysis.

B. The conclusions of the SMA report regarding the structural integrity of the studs and the liner are not valid because the analysis neglects the interaction of the studs and the liner in transmitting the force in the buckled panel. The more realistic stud analysis performed by G/C indicates that the stud displacements and forces are within acceptable limits using either G/C or SMA liner stresses. The maximum stud force of 28.9 kips is less than its ultimate capacity of 31.1 kips, and the maximum stud

displacement is 0.098 inches which is within the ASME allowable of 0.15 inches. Therefore, it is concluded that stud integrity is maintained under accident temperature and accident pressure conditions.

VI. DECEMBER, 1981 SMA REPORT

The accident pressure and temperature-time histories in the December, 1981 draft of the SMA report (Reference 9) are significantly different from those appearing in the previous draft dated August, 1981 (Reference 1). The peak values are compared below along with the G/C values reported herein.

Report	Peak Air T_a		Peak P_a		Max. Liner T_a		P_a @ Max. Liner T_a
	T_a (°F)	Time (Secs)	P_a (psia) (psig)	Time (Secs)	T_a (°F)	Time (Secs)	(psia) (psig)
SMA December, 1981 (Ref. 9)	417	35	86.2 (71.5)	94	267	380	69 (54.3)
SMA August, 1981 (Ref. 1)	285	60-150	68.5 (53.8)	120	Used 285°F	120	68.5 (53.8)
G/C	280	6-100	67.2 (52.5)	10	250	300	56.5 (41.8)

The peak air temperature reported in Reference 9 has increased to 417° F versus 285° F from Reference 1 and 280° F calculated by G/C. However, the new maximum liner temperature of 267° F is in the neighborhood of the 285° F and 250° F values used in the analyses discussed herein.

The peak accident pressure of 71.5 psig from Reference 9 is significantly greater than both the 53.8 psig value from previous SMA report draft and the peak value of 52.5 psig from Figure 14.3.4-2 in the FSAR. The 71.5 psig pressure exceeds the Design Basis Accident Pressure of 60 psig. However, the containment structure is designed for 90 psig internal pressure simultaneous with accident temperature as specified by load combination (a) appearing in Section 5.1.2.3 of the FSAR.

The effect on containment integrity of the increased accident pressure, temperature, and seismic loads is evaluated for FSAR load combination (c) in Reference 9. The conclusion of the evaluation are that in the cylinder and dome portions of the containment, the liner and concrete stresses are acceptable. In the base knuckle region of the cylinder a maximum shear stress of 21 ksi is reported, which exceeds locally a code (unspecified) allowable of 19.2 ksi for the 32 ksi minimum yield strength liner used for Ginna. The SMA report concludes that "yielding of the liner in the knuckle may possibly occur in a very localized area". However, this conclusion does not seem to follow from the results in light of the fact 19.2 ksi represents the allowable shear stress and not the value corresponding to the onset of yield. Nevertheless, the 32 ksi yield strength is a minimum specified value, and the actual values for liner plate maaterial are historically much greater.

The report concludes that the safety factor is 1.0 against seismic overturning (of the containment shell). Actually, Figure 5-7 in the report shows that under D + SSE + P_a the compressive stress at the base of the wall is nearly zero. However, there is additional resistance to overturning provided by the tendons, which should be considered in the stability analysis.

The concern expressed in the previous draft of the SMA report regarding liner and stud integrity at the insulation termination interface in the dome still exists. However, this concern is not warranted for the same reason discussed in Section IV herein; namely, the unbalanced liner force caused by panel buckling must be considered in a liner-stud interaction analysis. It is extremely conservative to assume that the unbalanced force is entirely resisted by one stud or a circumferential line of studs located at this interface. The interaction analysis performed by G/C is discussed in the previous sections. There it is concluded that using either the liner stresses from the SMA analysis (Ref. 1) or those obtained by G/C, the stud displacements are within the ASME Code acceptance limits. The maximum stud displacement from

the interaction analysis was 0.098 inches which is within its allowable value of 0.15 inches. This value of stud displacement came from an initial stress of 1.6 ksi compression for all the panels behind the insulation and for a stress in the buckled panel of 29 ksi compression. These values can be compared with those now reported by SMA in Reference 9 which are 5.8 ksi compression for the insulated panels adjacent to the buckled panel and 25.6 ksi compression for the buckled panel. From this comparison it is seen that an initial unbalanced panel stress of 27.4 ksi exists for the interaction analysis performed previously versus 19.8 ksi from Reference 9. This imbalance can be used as a comparative measure of the resulting stud displacements, i.e. a greater stud displacement will be produced by the condition with the greater initial stress imbalance. Therefore, by inspection, the panel stresses resulting from the revised SMA report (Reference 9) will produce stud displacements less than those obtained previously from the interaction analysis, which were acceptable.

REFERENCES

1. "Systematic Evaluation Program for Robert E. Ginna Nuclear Power Plant Combined Loads Evaluation", prepared for Lawrence Livermore National Laboratory by Structural Mechanics Associates, August, 1981.

2. "Review of Combined Loads Evaluation Report" by Gilbert/Commonwealth letter 13N1-GR-T3387, November, 1981.

3. "Computer Program for the Stress Analysis of Axisymmetric Thin, Elastic Shells" by Arturs Kalnins, 1976.

4. "HEATING 5 - An IBM 360 Heat Conduction Program", Code Documentation, W. D. Turner, D. C. Elrod, and I. I. Siman-Tov, Union Carbide Corp., Nuclear Division.

5. "Some Structural Considerations in the Design of Nuclear Containment Liners" by J. M. Doyle, Nuclear Engineering and Design, Volume 16 (1971), pp. 294-300, 1971.

6. "Shear Strength of Stud Connections in Lightweight and Normal Weight Concrete" by J. G. Ollgaard, R. G. Slutter and J. W. Fisher, AISC Engineering Journal, pp. 55-64, April, 1971.

7. ASME Boiler and Pressure Vessel Code, Section III - Division 2, "Code for Concrete Reactor Vessels and Containments", 1980 Edition.

8. "Design Data - Nelson Concrete Anchor", TRW report.

9. "Systematic Evaluation Program for Robert E. Ginna Nuclear Power Plant Combined Loads Evaluation", prepared for Lawrence Livermore National Laboratory by Structural Mechanics Associates, December, 1981.

TABLE 1

COMPARISON OF LINER AND CONCRETE
STRESSES FROM G/C AND SMA ANALYSES

Location	Load Case	SMA Meridional (ksi)	SMA Hoop (ksi)	G/C Meridional (ksi)	G/C Hoop (ksi)
Cylinder Concrete:	Pressure (P_a)*	+0.4	+0.8	+0.3	+0.6
Inner Surface	Temperature (T_a)	-0.9	-0.9	-0.6	-0.6
(Section A-A)	$P_a + T_a$	-0.5	-0.1	-0.3	-0.0
Dome Concrete:	Pressure (P_a)*	+0.5	+0.5	+0.4	+0.4
Inner Surface	Temperature (T_a)	-6.5	-6.4	-4.6	-4.6
(Section B-B)	$P_a + T_a$	-6.0	-5.9	-4.2	-4.2
Cylinder Steel	Pressure (P_a)	+2.4	+5.2	+2.1	+4.5
Liner	Temperature (T_a)	-8.0	-8.0	-4.0	-4.3
(Section A-A)	$P_a + T_a$	-5.6	-2.8	-1.9	0.2
Dome Steel	Pressure (P_a)	+3.9	+4.0	+3.3	+3.3
Liner	Temperature (T_a)	-56.0	-56.0	-46.0	-46.0
(Section B-B)	$P_a + T_a$	-52.1	-52.0	-42.7	-42.7

*Not shown on figures

Sign Convention: - Compressive stresses
+ Tensile stresses

TABLE 2

MAXIMUM STUD FORCE AND STUD DISPLACEMENT
BUCKLED PANEL STRESS = 29 ksi

	INITIAL STRESS IN LINER BEHIND INSULATION		
	8 ksi for T_a (SMA)	1.6 ksi for T_a (G/C)	Varying Liner Stress for $T_a + P_a$ (G/C)
Max. Stud Force	26.8 kips	28.9 kips	28.4 kips
Ultimate Stud Capacity	31.1 kips	31.1 kips	31.1 kips
Max. Stud Displacement	0.065 in.	0.098 in.	0.088 in.
Allowable Stud Displacement	0.15 in.	0.15 in.	0.15 in.

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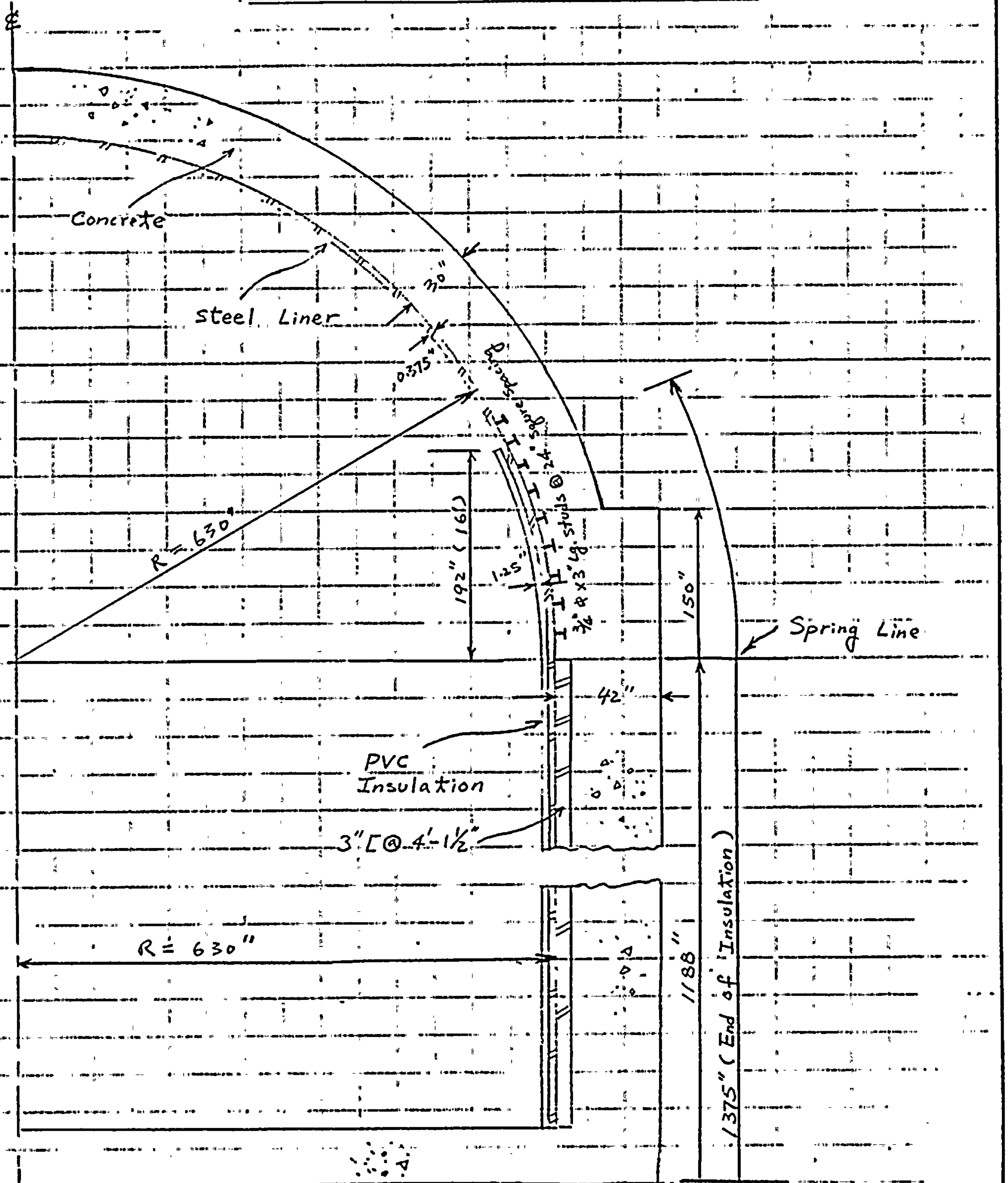
	ORIGINATOR S. S. HSIEH			
	DATE 1/11/82			

FIG. 1: GINNA CONTAINMENT STRUCTURE



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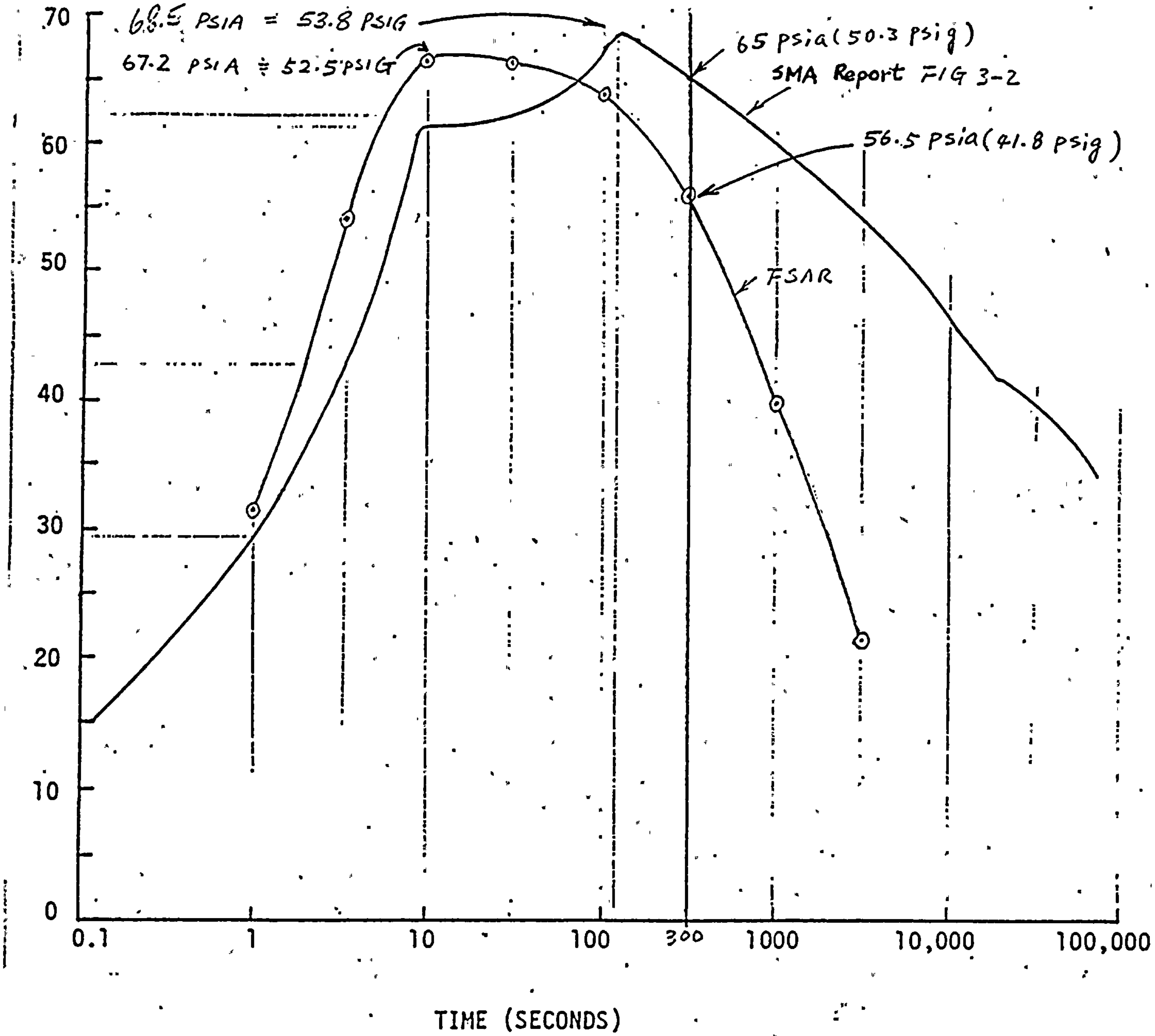

FIGURE 2 .. ACCIDENT PRESSURE LOAD TRANSIENT IN THE CONTAINMENT

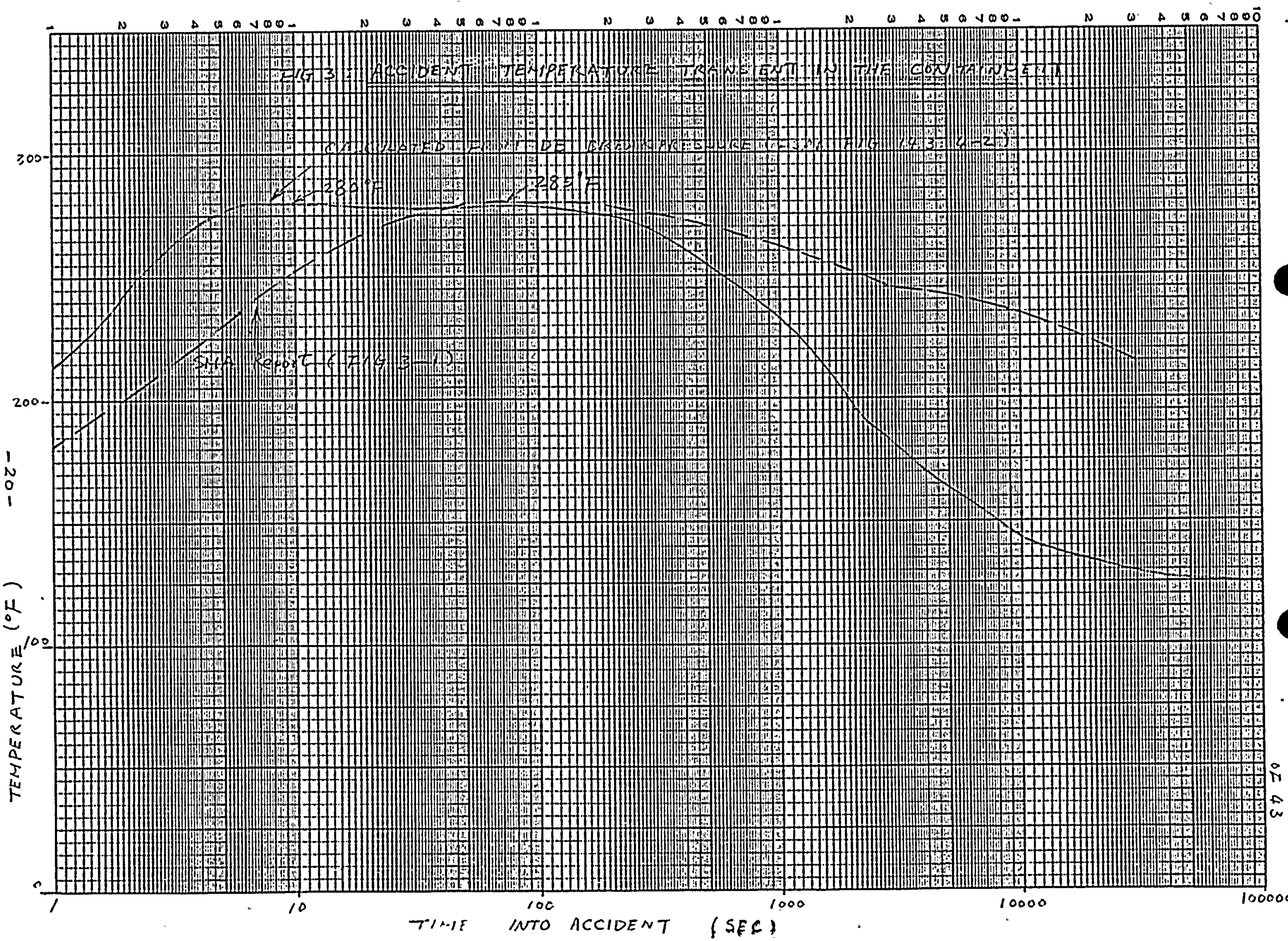

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FIG. 4: ACCIDENT TEMPERATURE DISTRIBUTION IN THE STEEL LINER

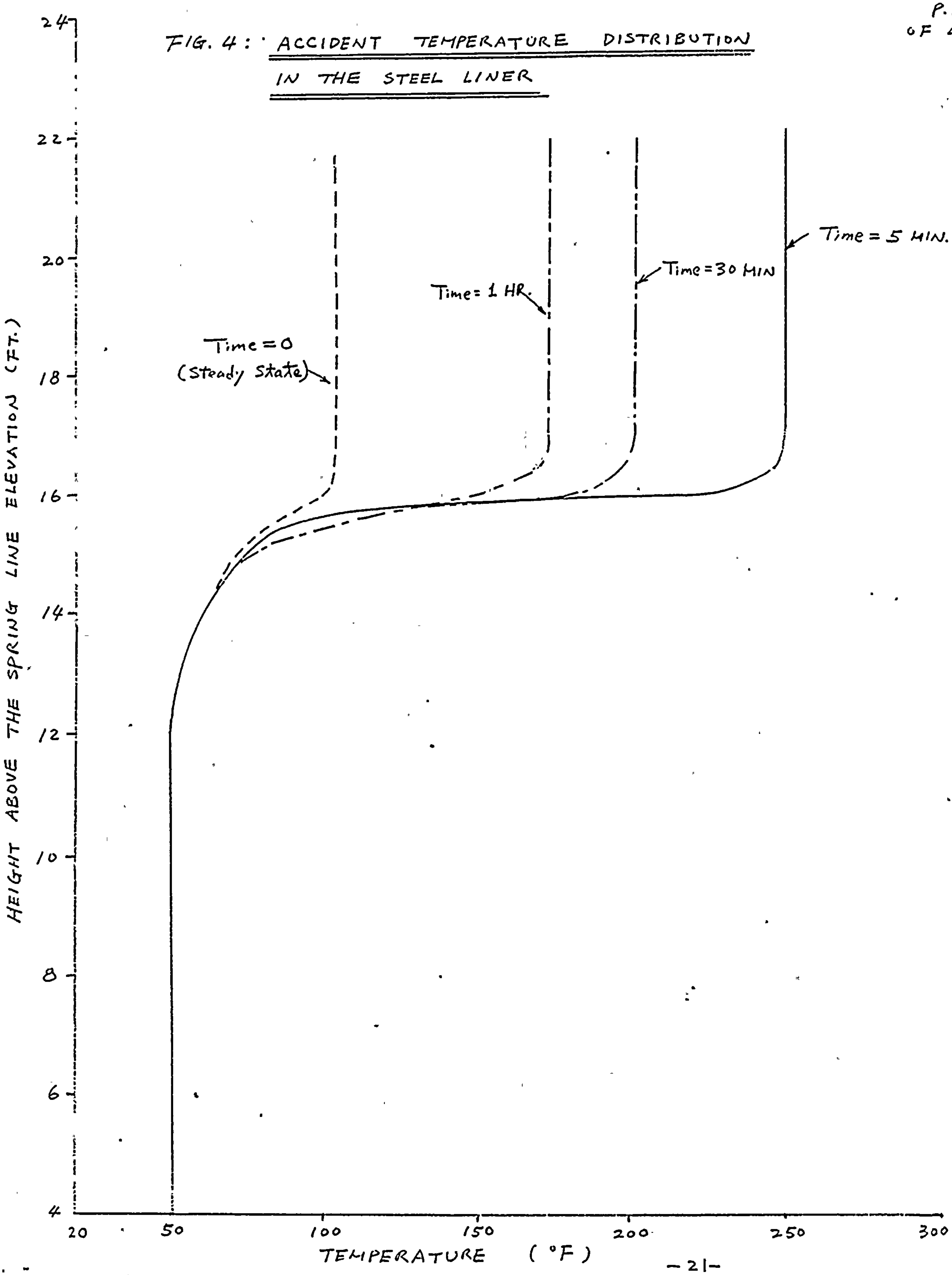

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FIG. 5: MERIDONAL STRESSES IN LINER FOR Ta

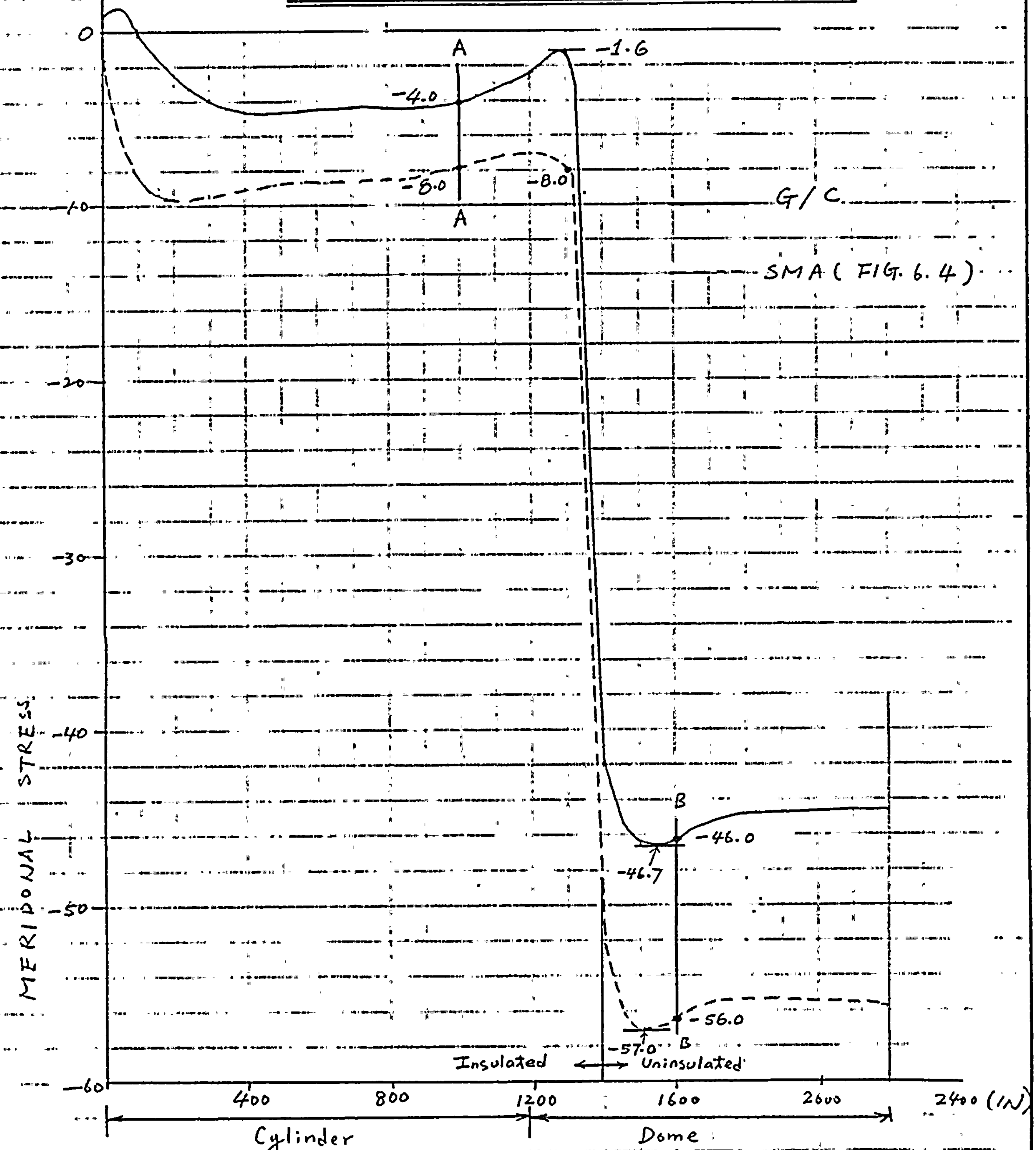

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FIG. 6: CIRCUMFERENTIAL STRESSES IN LINER FOR Ta

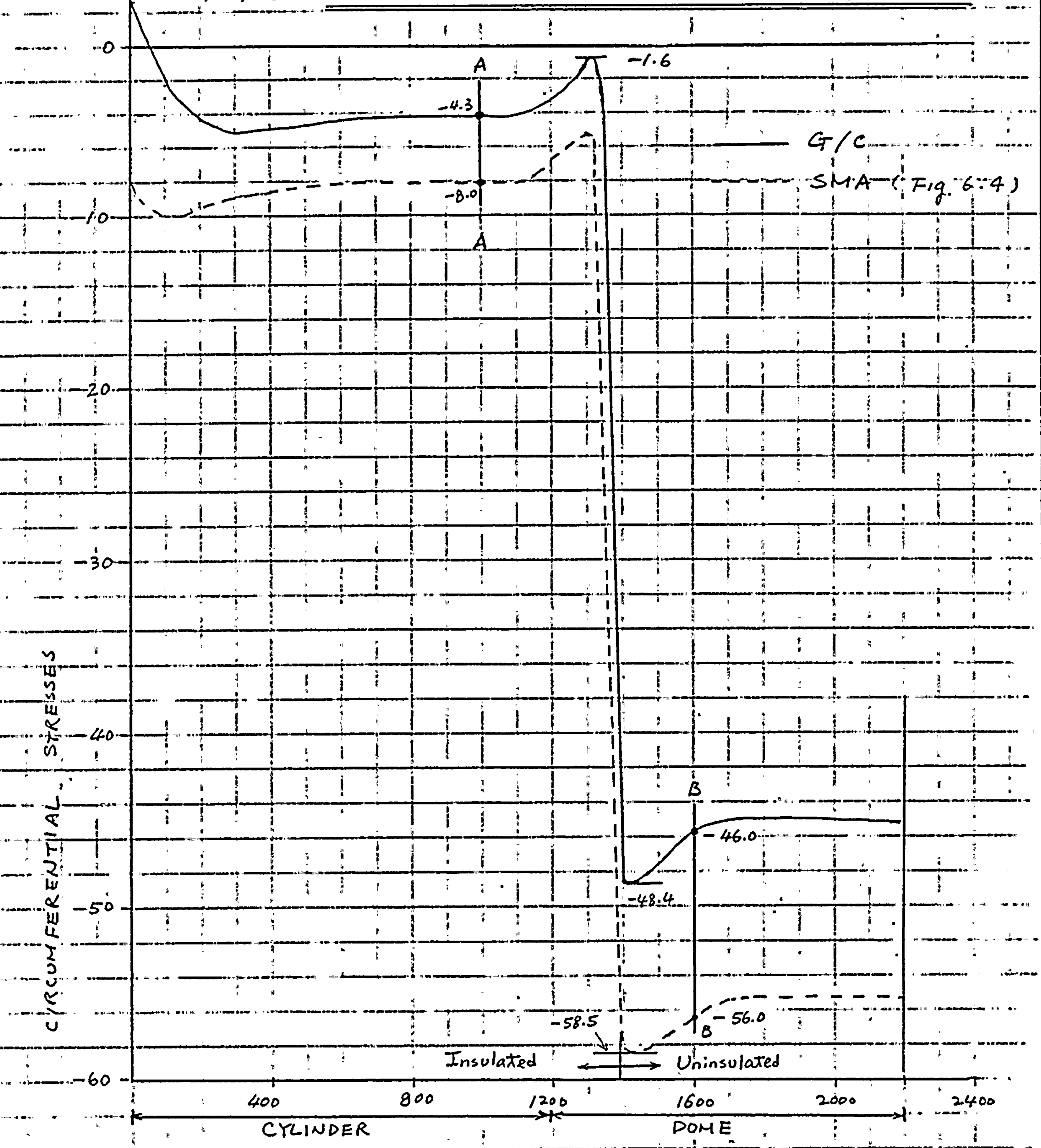

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FIG. 7: MERIDONAL STRESSES AT INNER SURFACE OF CONCRETE FOR Ta

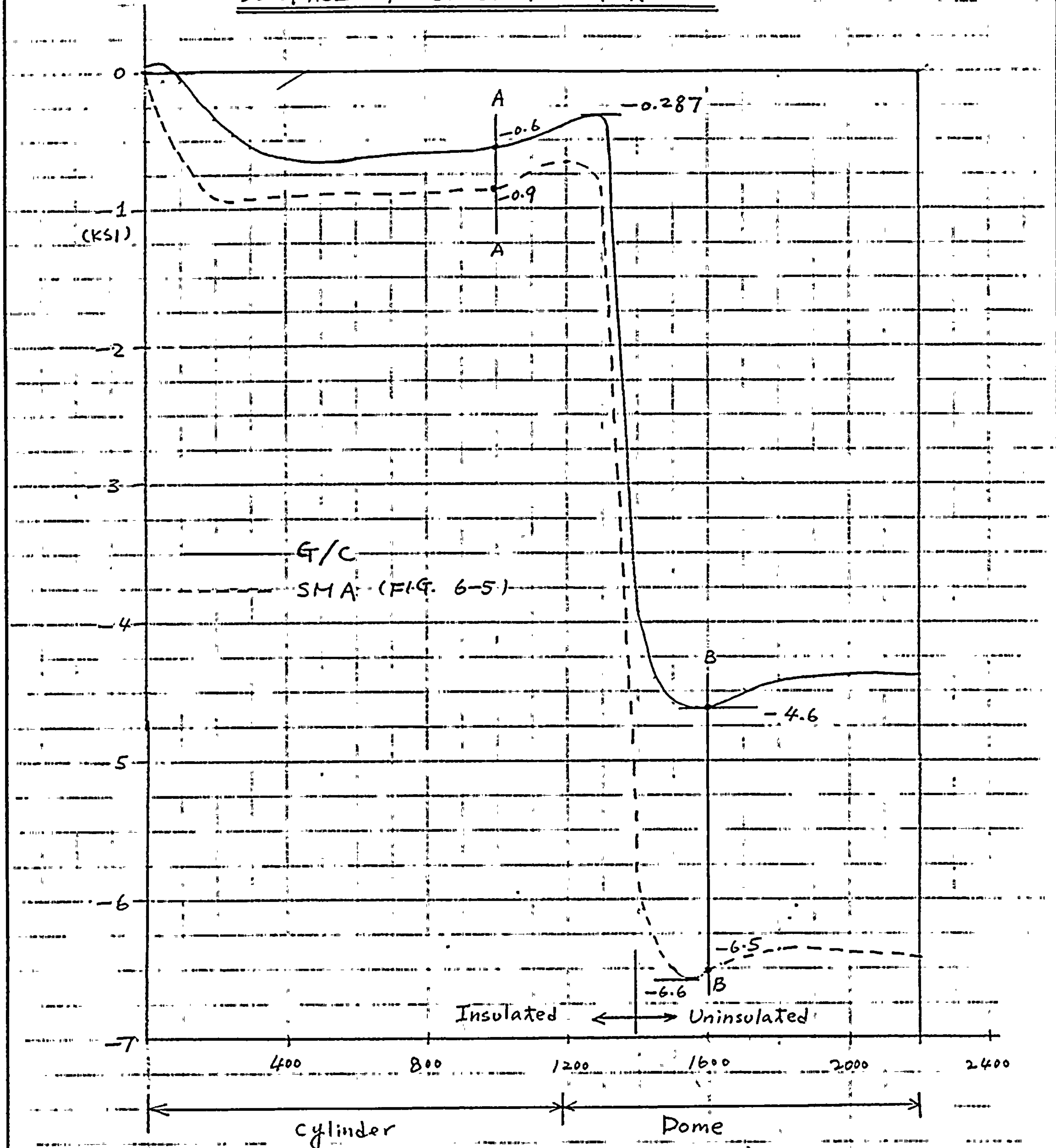

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FIG. 8: CIRCUMFERENTIAL STRESSES AT INNER SURFACE OF CONCRETE FOR Ta:

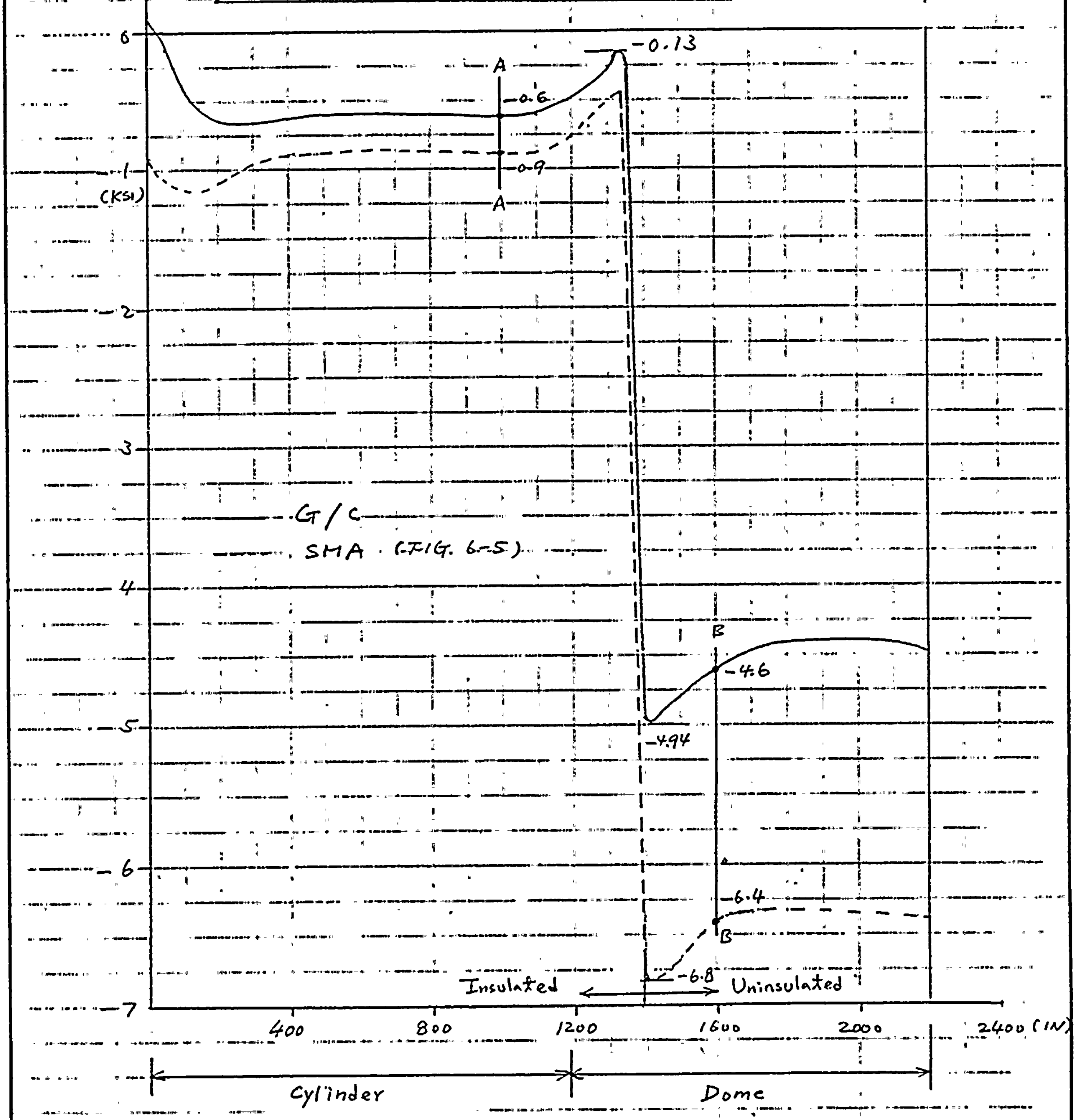

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FIG. 9: MERIDONAL STRESSES IN LINER FOR P_a

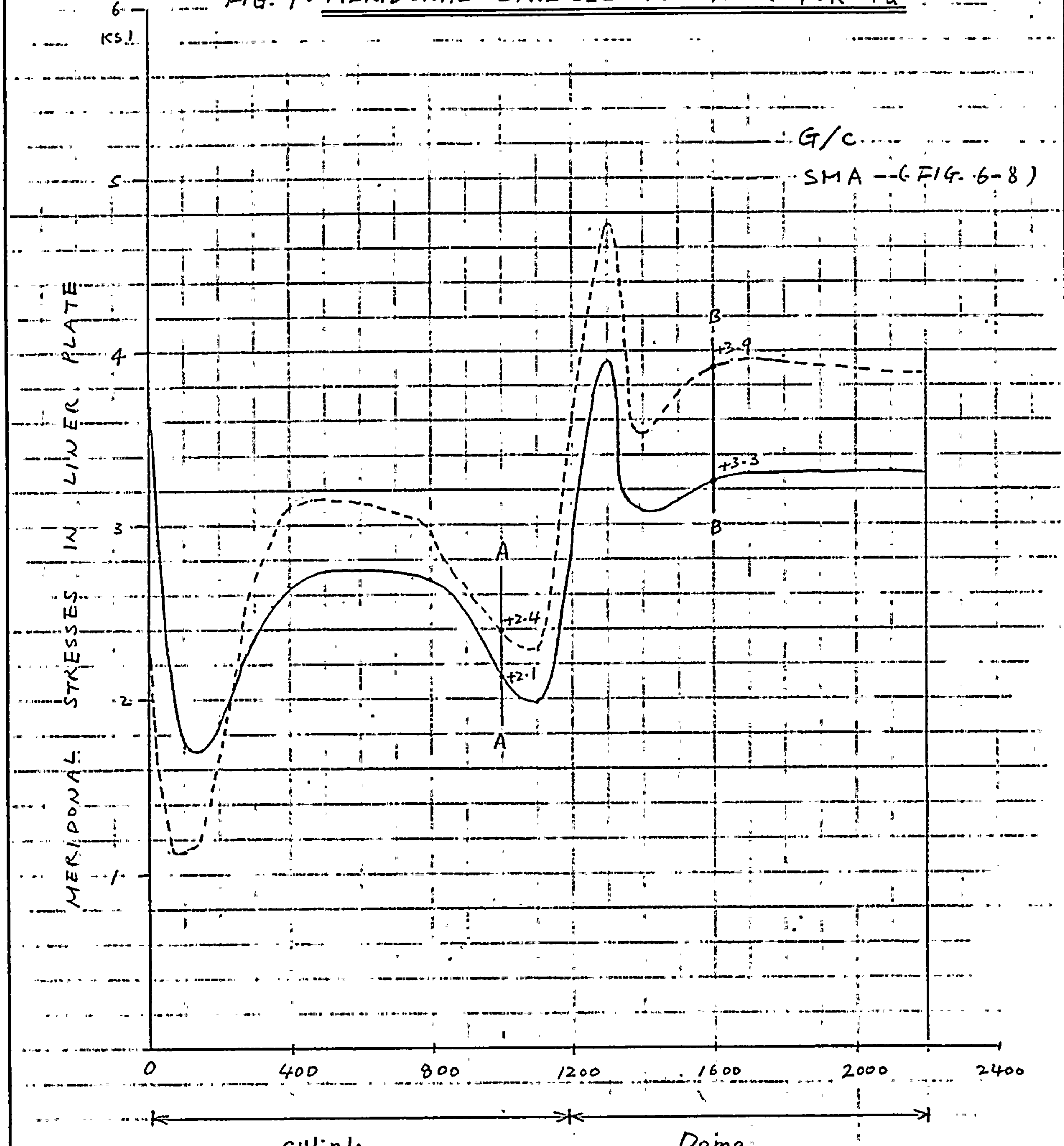

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FIG. 10: CIRCUMFERENTIAL STRESSES IN LINER FOR P_a

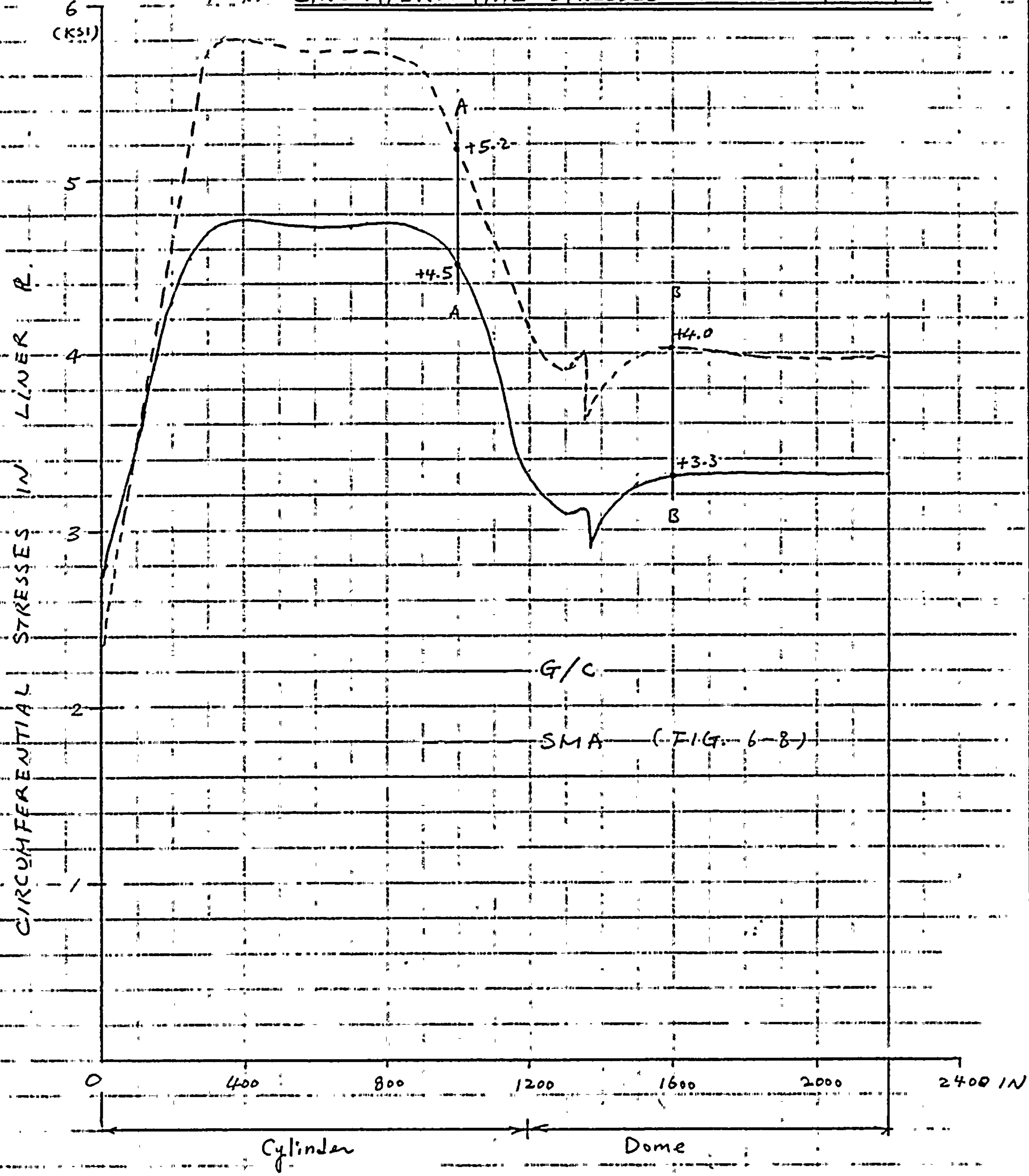

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FIG. 11: MERIDONAL STRESSES IN LINER FOR $P_a + T_a$

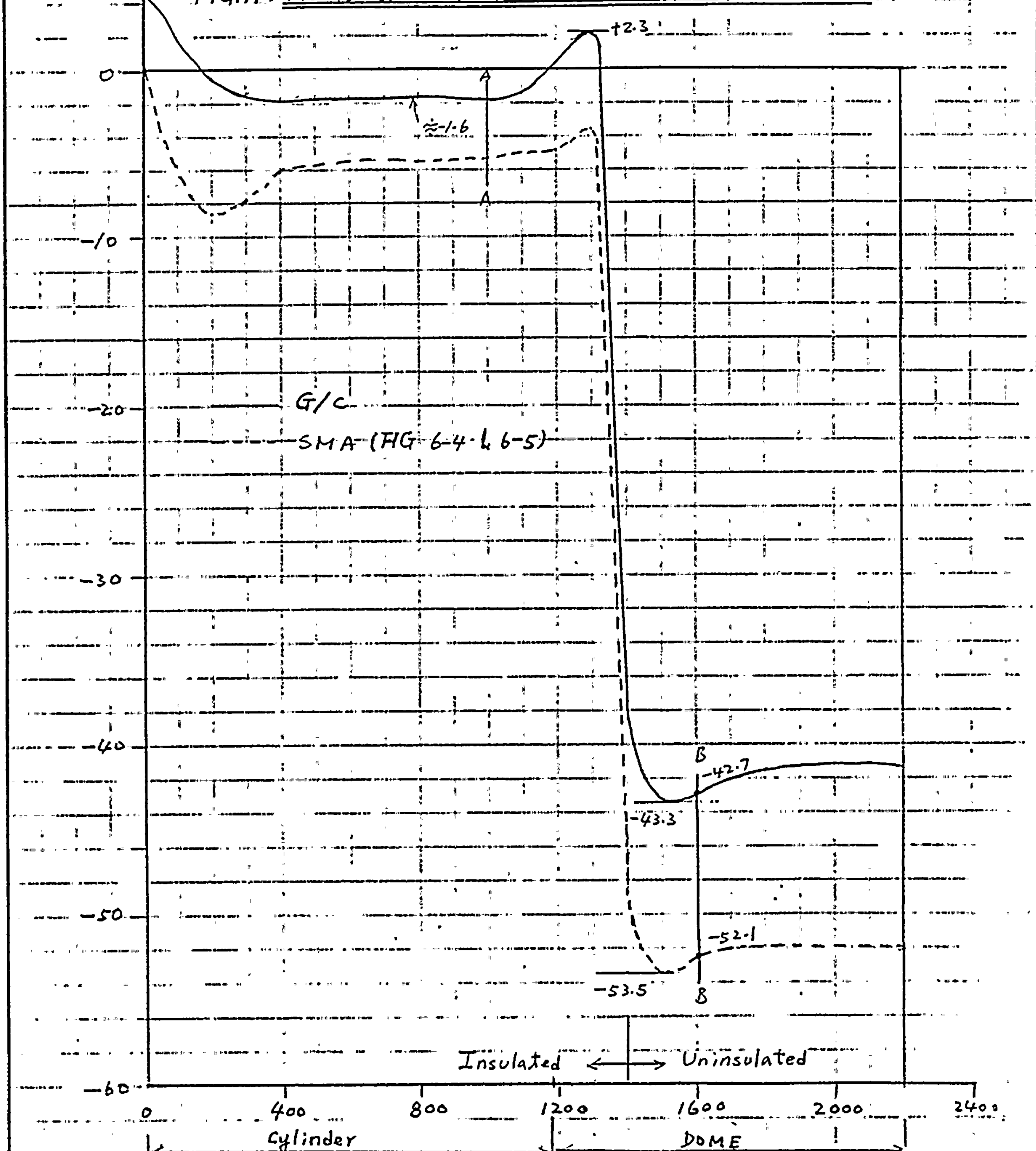

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FIG. 12: CIRCUMFERENTIAL STRESSES IN LINER FOR $P_a + T_a$

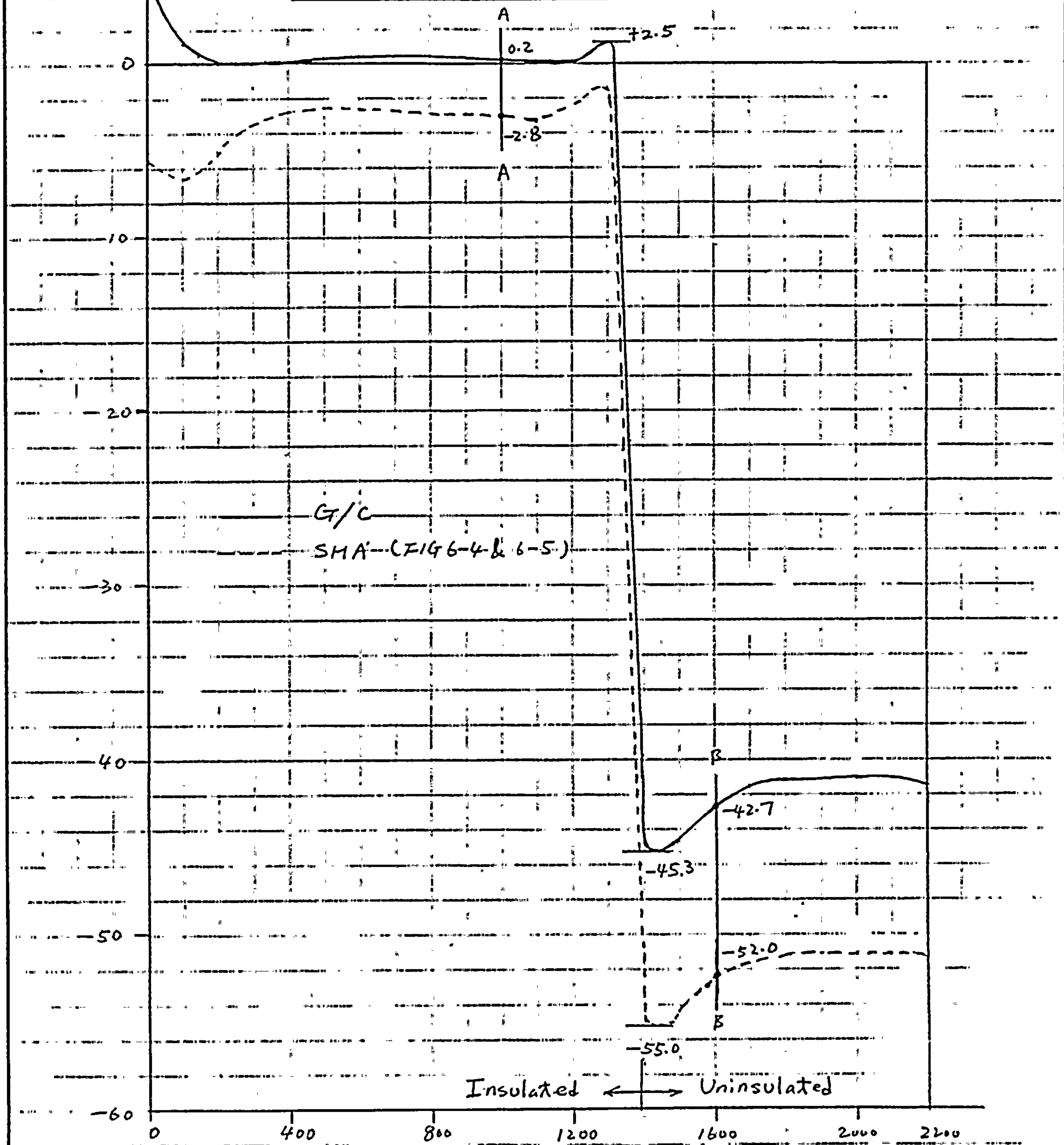

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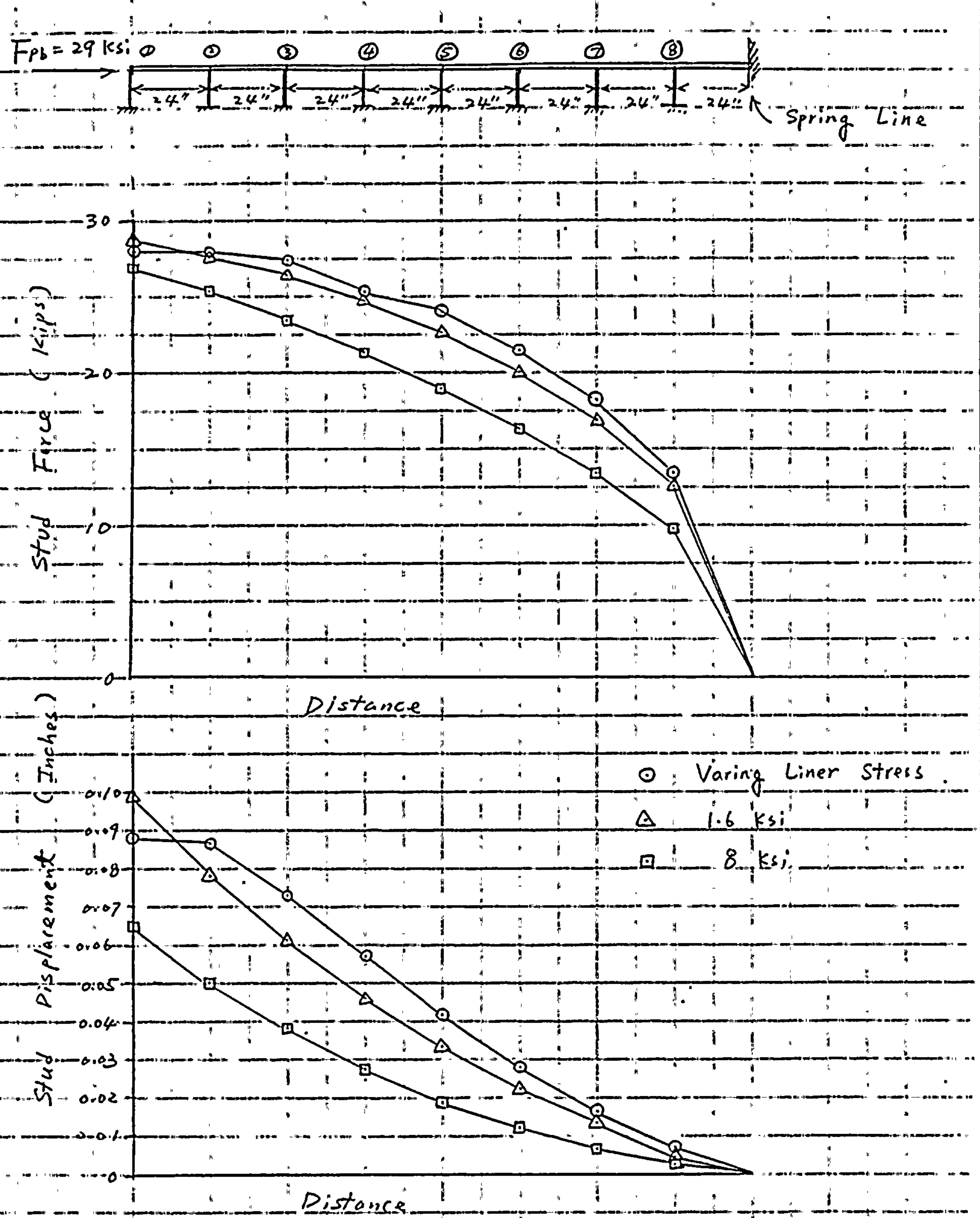
FIG. 13: THE MODEL AND RESULTS OF STUD ANALYSIS



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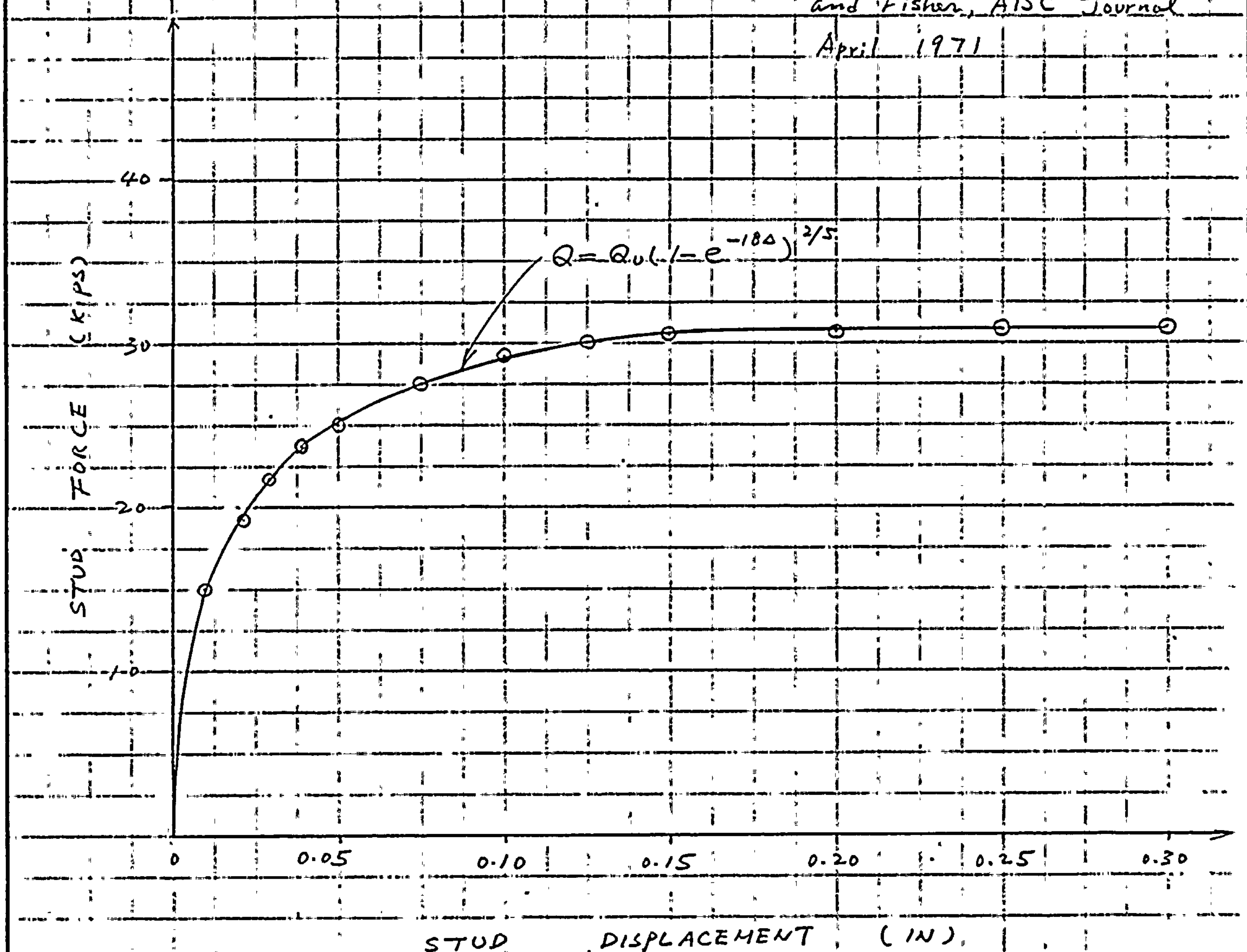
Gilbert Associates, Inc. Reading, Pennsylvania	SUBJECT GINNA LINNER			CISID :51	PAGE 40 OF PAGES 43
ANALYSIS/CALCULATION	REV. 0	1	2	3	
	MICROFILMED				
	ORIGINATOR S S HSIEH				
	DATE 1/26/82				

FIG. 14: Stud Force-Displacement Relationship

⊙ "Shear Strength of Stud Connectors in LW and NW Concrete" by Ollgarrd, Slutter and Fisher, AISC Journal April 1971



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