Impell Calculation M-27, "Thermal Load Evaluation", Rev. 1
(Replace Ref. 12, Complete Calculation)

IMPELL CORPORATION

CALCULATION/PROBLEM COVER SHEET

Calculation/Problem No: M-27

Title: Thermal Load Evaluation | 1 ⚠

Client: TUGCO **Project:** CPSES

Job No: 0210-040

Design Input/References:

Noted within

Assumptions:

Noted within

Method:

Noted within

Remarks:

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0	Original Issue	[signature]	5/29/86
1	Revised to include Expanded Numerical Evaluation.	[signature]	2-25-87

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1.0 INTRODUCTION

The question of thermal loading on cable tray supports has been raised by CYGNA in Issue No. 19 of the Independent Assessment Program: i.e. "For supports installed in the Reactor Building, the loads associated with a LOCA may be applicable, including pipe whip, jet impingment, and thermal loads." This statement is based upon the load combinations specified in Section 3.8.4.3 of the CPSES FSAR [1] for "Other Seismic Category I Structures." Several of these load combinations include the thermal loads associated with postulated thermal accident conditions.

The purpose of this calculation is to evaluate thermal load effects on cable tray supports and their anchorages and determine whether or not such loads need to be included in the design verification effort of cable tray systems at CPSES.

According to load combinations specified in the CPSES FSAR [1], which are consistent with those given in the NRC Standard Review Plan (SRP), thermal loading must be addressed "when present" and when the thermal loading could affect structural performance. Both the SRP and FSAR criteria specifically exclude thermal loads in cases where these loads are secondary and self-limiting in nature and where the materials are ductile. Given these conditions, thermal loads cannot lead to structural or system failure and thus do not need to be considered.

The exclusion of thermal loading in the qualification of the cable trays and support members is justified by the high level of ductility inherent in the mild carbon steel material. Furthermore, it will be shown that for the steel-to-concrete anchorages (the least ductile part of the cable tray system) the thermal displacements are self limited to values which preclude structural failure. The FSAE and SEP criteria for exclusion of thermal loading are therefore satisfied.

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2.0 PRECEDENT

Impell has performed a review of the FSARs and related design documents of other nuclear power plants — including Millstone 3, Braidwood, Byron, Catawba, and Diablo Canyon. None of these plants have considered thermal loads in the evaluation of cable tray systems. All of these plants are of the same vintage Westinghouse NSSS design as CPSES and share the same licensing position of not considering thermal loads in the evaluation of cable tray systems.

3.0 METHODOLOGY

The cable tray systems at CPSES Unit 1 are comprised of light gage steel trays and structural steel supports. Static tests of cable trays [2] show that the cable trays are indeed ductile by displaying a large nonlinear region on load-deflection curves before reaching the ultimate load. The cable tray supports are composed of structural steel sections. The inherent ductility of the mild carbon steel material ensures that the SRP/FSAR criteria are met (secondary, self-limiting loads and ductile materials) for the trays and supports and thermal loads need not be evaluated. It must be shown, however, that this conclusion can be applied to the case of steel-to-concrete ancharage details. Since the ductility of these connections could be governed by the behavior of concrete instead of steel, they might be susceptible to thermal loading even though the trays and supports are not. The following steps will be used to evaluate the susceptiblity of steel-to-concrete anchorages to thermal loading.

2.1 System Slack

A worst case condition will be considered, this being a straight run of cable tray with the maximum allowable spacing between longitudinal supports of 40 ft. Longitudinal supports are braced in the longitudinal direction, thereby restraining the thermal growth of the cable tray. By using the maximum allowable spacing of 40 ft. [10] between longitudinal supports the thermal expansion will be maximized.

Slack in the cable tray system will be utilized to relieve a portion of the thermal expansion. However, credit will only be taken for the minimal amount of slack which can confidently be assumed present even under worst case conditions.

This 40 ft. system would contain at least three tray splices, two tier-to-tray connections and two support-to-structure connections. Typically, the average bolt hole for anchorages is 1/8" [9] oversized and other bolt holes are 1/16" oversized [4]. Assuming these bolt hole oversizes, the expected slack in the system is at least 0.28". (See Section 3.0 of this calc.)

2.2 Expected Thermal Growth

Thermal growth of three conditions needs to be considered: a) thermal growth during normal operation and shutdown conditions, b) maximum thermal growth during a postulated break inside containment in conjunction with a seimic event and c) maximum thermal growth that could occur at any time.

a) Operating Thermal Loads

During normal operation and shutdown conditions the maximum temperature change is relatively small (less than 50° Fahrenheit [10]) and since the temperature change occurs gradually, the building structures will heat up at the same rate as the cable tray systems. Therefore, an effective coefficient of expansion equal to the difference in the coefficients of expansion for steel [5] and concrete [11] (the difference is approximately 1.0×10^{-6}) can be used.

b) Accident Thermal and Seismic

The probability that a Loss of Coolant Accident (LOCA) occurs in conjunction with a seismic event is not a credible event, unless the seismic event initiates the LOCA. Given such circumstances, the maximum temperature rise during an earthquake must be determined. Once determined, the maximum temperature change would be ased to determine the thermal growth and thus the possible subsequent thermal loading. A worst case situation can be postulated based upon the following conservative assumptions:

1. The LOCA is initiated simultaneously with the onset of the Safe Shutdown Earthquake (SSE) so that the maximum time possible is allowed for temperature changes to occur.

2. The maximum seismic loading occurs at the end of the SSE (i.e. at t = 10 seconds) concurrent with the maximum thermal effect.

3. The LOCA which occurs is the Double-Ended Pump Suction Guillotine (DEPSG); i.e. the one producing the most rapid changes in temperature [1]. (Figure 1)

4. The cable trays are exposed to the LOCA environment over 100% of their surface area (inside and out).

Given these worst-case conditions, temperature changes in the cable trays can be evaluated through the following equation [8]:

$$T_t = (T_o - T_a)\, e^{\frac{-2ht}{\rho c L}} + T_a \qquad (1)$$

where: T_t = tray temperature at time, t (°F)
T_o = initial tray temperature (°F)
T_a = ambient temperature at time, t (°F)
h = heat transfer coefficient at time, t (BTU/HR·FT²·°F)
ρ = tray material density (LB/FT³)
c = tray specific heat (BTU/LB·°F)
L = tray thickness (FT)

Equation 1 has been evaluated numerically at one-second intervals with the results given in Section 3.0 of this calculation. Knowing the tray temperature at ten seconds after the earthquake is initiated, the expected thermal growth for this condition can be calculated (See Section 3.0 of this calc.).

c) Accident Thermal Loads

As for the condition where maximum thermal growth could occur anytime, the most critical case [1] is the 0.908 FT² - split Steam Line Break (SLB) at 70% power. The SLB temperature profile (Figure 3) shows the maximum temperature change inside Containment. Since there is no heat transfer data for the SLB, it will be assumed conservatively that the cable tray temperatures change at the same rate as the vapor inside Containment. Also, the temperature change of the internal concrete structures (Figure 4) will be taken into account. The results appear in Section 3.0 of this calculation.

2.3 Restrained Growth

Since thermal loading can only occur when thermal growth is restrained, only the net growth; i.e. the total growth minus the slack in the system, will determine if thermal loading exists and to what extent if it does exist.

3.0 RESULTS

3.1 System Slack

The slack in the system is determined as follows:

two support-to-structure connections :	2 x 1/8" =	0.25"
three tray splices :	3 x 1/16" =	0.1875"
two tray-to-tier connections :	2 x 1/16" =	0.125"
total slack =		0.5625"

Half of this total will be used assuming that the bolts are centered in the bolt holes.

∴ The amount of slack available for free growth is 0.28"

3.2 Expected Growth

a) Operating Thermal Loads

During normal operation the temperature change is relatively small (less than 50°F) and the coefficient of thermal expansion can be taken as the difference between concrete and steel (1.0×10^{-6} in/in °F). With these conditions, the thermal expansion can be shown to be small even for the worst case of a 40 ft. straight tray run:

$$\Delta = \alpha \Delta T L$$
$$= (1.0 \times 10^{-6} \text{ in/in°F})(50°F)(40 ft)(12 in/ft)$$
$$= 0.02 \text{ in}$$

This expansion is insignificant when compared to the total slack available.

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b) Accident Thermal and Seismic

The tray temperature is determined for the critical time interval of 0 to 10 seconds. The DEPSG controls with the ambient temperature profile shown in Figure 1. The tray thickness, L, will be for the lightest trays (16 gage material) used at CPSES, those that are the fastest to change temperature. The following data is used with Equation 1 to find the tray temperature at one second intervals:

$$T_t = (T_o - T_a)\, e^{\frac{-2ht}{\rho c L}} + T_a$$

T_o = 120° F (initial tray/ambient temperature)

h = 5 at t=0 and 15 at t=10 with linear variation (BTU/HR·FT²·°F) [8]

ρ = 490 LB/FT³ [4]

c = 0.11 BTU/LB·°F [5]

L = 0.005292 FT (16g Galv. Tray) [12]

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Equation 1 evaluated at 1-sec intervals as follows:

t (sec)	T_o (°F)	T_a (REF. Fig. 1) (°F)	h (BTU/LB·°F)	T_t (°F)
1	120.0	153	12	120.7
2	120.7	179	19	122.8
3	122.8	194	26	126.3
4	126.3	204	33	131.1
5	131.1	212	40	137.2
6	137.2	219	47	144.4
7	144.4	225	54	152.4
8	152.4	230	61	161.1
9	161.1	234	68	170.1
10	170.1	238	75	179.3

The maximum 10-second change in tray temp is

$179 - 120 = 59°F$

The thermal growth would be:

$$\Delta = \alpha \Delta T L$$

where: Δ = thermal growth
α = coefficient of thermal expansion
ΔT = change in temperature
L = length of tray

$$\Delta = (6.8 \times 10^{-6} \text{ in/in}\cdot°F)(59°F)(40 \text{ Ft})(12 \text{ in/FT})$$ [5]

$$\Delta = 0.19''$$

Clearly, the slack in the system, 0.28", is more than enough to accommodate the entire thermal growth, 0.19", without generating any thermal loads in the system. The vibrations caused by the seismic excitation will ensure that slippage does occur and that the system slack will be mobilized as necessary.

Given these worst-case conditions, thermal loading can not develop concurrently with seismic loading. Therefore, the load combination of SSE plus LOCA need not be considered for the steel-to-concrete anchorages or any other part of the cable tray systems.

c) Accident Thermal Loads

The maximum temperature change occurs for a $0.908 FT^2$ - split Steam Line Break (Figure 3). From Figure 3 it can be seen that the maximum temperature change is 212°F, occuring approximately 90 seconds into the SLB. At the same time, the temperature of the internal concrete structures (Figure 4) has increased approximately 39°F. It will be assumed conservatively that the cable tray temperatures change at the same rate as the vapor inside containment since no heat transfer data is available for the SLB. The relative temperature change would be 212°F - 39°F = 173°. Given this data the thermal growth would be:

$$\Delta = (6.8 \times 10^{-6}\ in/in°F)(173°F)(40\ Ft)(12\ in/Ft)$$

$$\Delta = 0.56''$$

At least half of the thermal expansion would be taken up by the system slack (0.28" or more). Considering the inherent flexibility of the trays, clips and supports, it appears unlikely that significant thermal loading would actually develop. For the purpose of this evaluation, it will be conservatively assumed that there is no flexibility in the system and the entire thermal loading must be accommodated by the steel-to-concrete anchorages. It will also be assumed that the displacement is transferred to the anchorage as pure shear (i.e, the least ductile failure mode for the anchorages).

The net restrained thermal growth is obtained by subtracting the system slack from the total growth:

$$\Delta_{net} = 0.56 - 0.28 = 0.28 \text{ inches}$$

The anchorage devices used for CPSES cable trays are 3/4", 1", and 1 1/4" Hilti Kwik and Super Kwik bolts at various embedment depths, and 1" and 1 1/2" Richmond Inserts. The ultimate shear displacements for Hilti Kwik bolts are taken from tests conducted for Hilti by Abbot A. Hanks, Inc. [14]. These tests were authorized by the manufacturer and used to determine design allowables. Test results from three specimens in 4000 psi concrete were averaged to determine ultimate shear displacements:

Diam. (in)	Embedment (in)	Ultimate Displacement (in)
3/4	3 1/4	0.30
	6 1/4	0.33
	9 1/4	0.28
1	4 1/2	0.32
	8 1/2	0.52
	10 1/2	0.48
1 1/4	5 1/2	0.38
	8	0.32
	10 1/2	0.42

The range of embedments tested covers all embedments used at CPSES for cable tray hangers. In all cases, the restrained growth is self limited to values which preclude bolt shear failure. In the single case where the ultimate displacement is equal to the restrained growth, (i.e., 3/4" diam. @ 9 1/4" embedment) the averaged ultimate displacement calculated is affected by one singularly low test specimen.

Super Kwik bolts were not included in the referenced test program. These types have an additional wedge mechanism, increasing the bolt capacity. Ultimate shear displacements are expected to be comparable, if not correspondingly greater, for these bolts.

Ultimate shear displacements for 1" Richmond Inserts are derived by averaging two specimens of test data in 3220 psi concrete [15]. The calculated ultimate displacement is 0.48". Again, net thermal displacements are shown to be self limited to values below the ultimate shear displacement of the anchorage. The 1 1/2" Richmond Inserts were not included in the tests. However, due to the increased bolt diameter a larger concrete spall and higher capacity will be mobilized at failure. Therefore ultimate shear displacements again are expected to be comparable, if not correspondingly greater, for these bolts.

The effect of thermal expansion on a more realistic system (i.e., including the effects of support flexibility) is demonstrated in Figure 5. The thermal load for a rigid restraint completely restraining the net thermal growth is determined:

$$P = \frac{\Delta A E}{L}$$

where: P = thermal load
A = tray cross sectional area
Δ = net growth
L = length of tray run
E = elastic modulus

Using the greatest tray cross sectional area to maximize thermal load –

$$P = \frac{(0.28\,in)(1.787\,in^2)(29.5\times10^3\,ksi)}{(480\,in)} = 30.7\ kips$$

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The line of complete restraint and free expansion is drawn on the graph of Figure 5. Also drawn is the line representing the stiffness, as determined from testing [7], of a typical CPSES longitudinal cable tray support. The support used is a T-channel cantilever braced to resist out-of-plane loading. The support is shown in Figure 6.

Figure 5 shows the point of equilibrium between the axial force in the cable tray and the shear force in the bolts. The equilibrium displacement is shown to be approximately 0.12 inches. These results indicate that when support flexibility is considered the equilibrium displacement yields an even larger margin against anchorage failure.

5.0 CONCLUSIONS

The preceding sections have presented an evaluation of thermal load effects on cable tray systems. The results show that cable tray systems satisfy the SRP/FSAR criteria for the exclusion of accident thermal load from design requirements – the loads are secondary and self-limiting in nature and all materials and connections have been shown to be sufficiently ductile to ensure failure will not occur. For cases where ductility was questioned (i.e., steel-to-concrete anchorages), a numerical evaluation was performed for a "worst case" configuration in the least ductile (shear) failure mode. The anchorage evaluation has shown that:

- For normal operating conditions the calculated thermal expansion has been shown to be insignificant. The maximum accident thermal expansion which could conceivably occur along with a seismic event will not exceed the minimum expected slack in the system. Therefore these load combinations need not be considered.

- The maximum thermal loadings for accident conditions alone do generate sufficient expansion to overcome system slack. However, the displacements have been shown to be self-limited to values which cannot cause failure of the steel-to-concrete anchorage. When a minimal amount of support flexibility is included, the margin against failure is shown to significantly increase.

6.0 REFERENCES

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[10] "General Instructions for Cable Tray Hanger Analysis for Comanche Peak Steam Electrical Station No. 1 and 2", Rev. R2, December 20, 1985, by Ebasco Services Inc.

[11] Raymond J. Rourk and Warren C. Young, <u>Formulas for Stress and Strain</u>, 5th Edition, McGraw-Hill, New York, 1975

[12] Theodore Baumeister, Eugene A. Avallone and Theodore Baumeister III, <u>Mark's Standard Handbook for Mechanical Engineers</u>, McGraw-Hill, New York, 1978

[13] Impell Project Instructions PI-07, "Design Verification of Base Plates, Base Angles and Embedment Plates" Rev. 3

[14] Abbot A. Hanks File No. H2189-S1, Report No. 8784, dated January, 1974.

[15] Transmittal, V. Svetlichny (Richmond) to J. Ramuta (Impell) Bulletin #6 test reports on EC Type Inserts, dated 9/17/86

7.0 FIGURES

DESIGN VERIFICATION	
CLIENT TU ELECTRIC	
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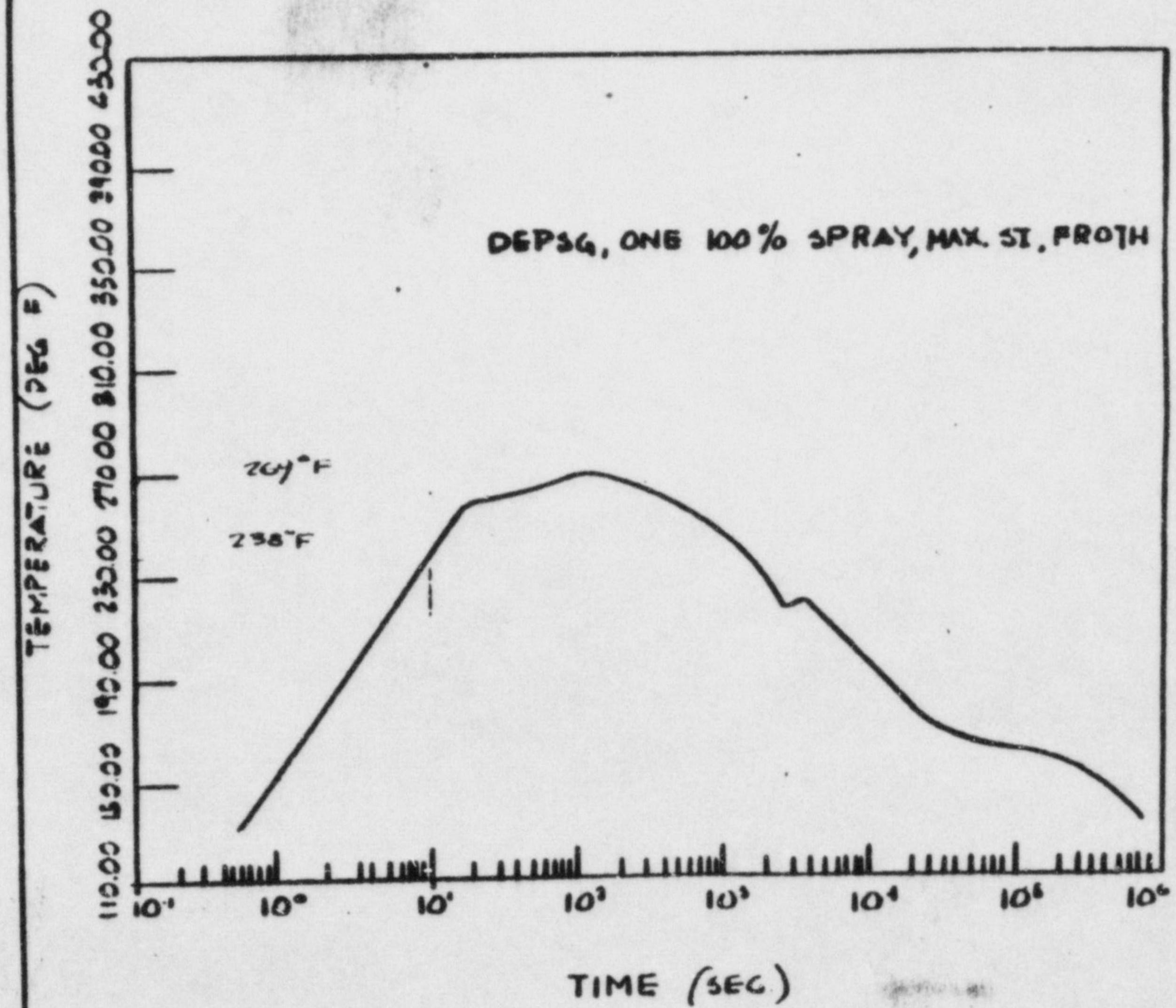

Figure 1

COMANCHE PEAK S.E.S
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CONTAINMENT VAPOR
TEMPERATURE TRANSIENT -
DEPSG

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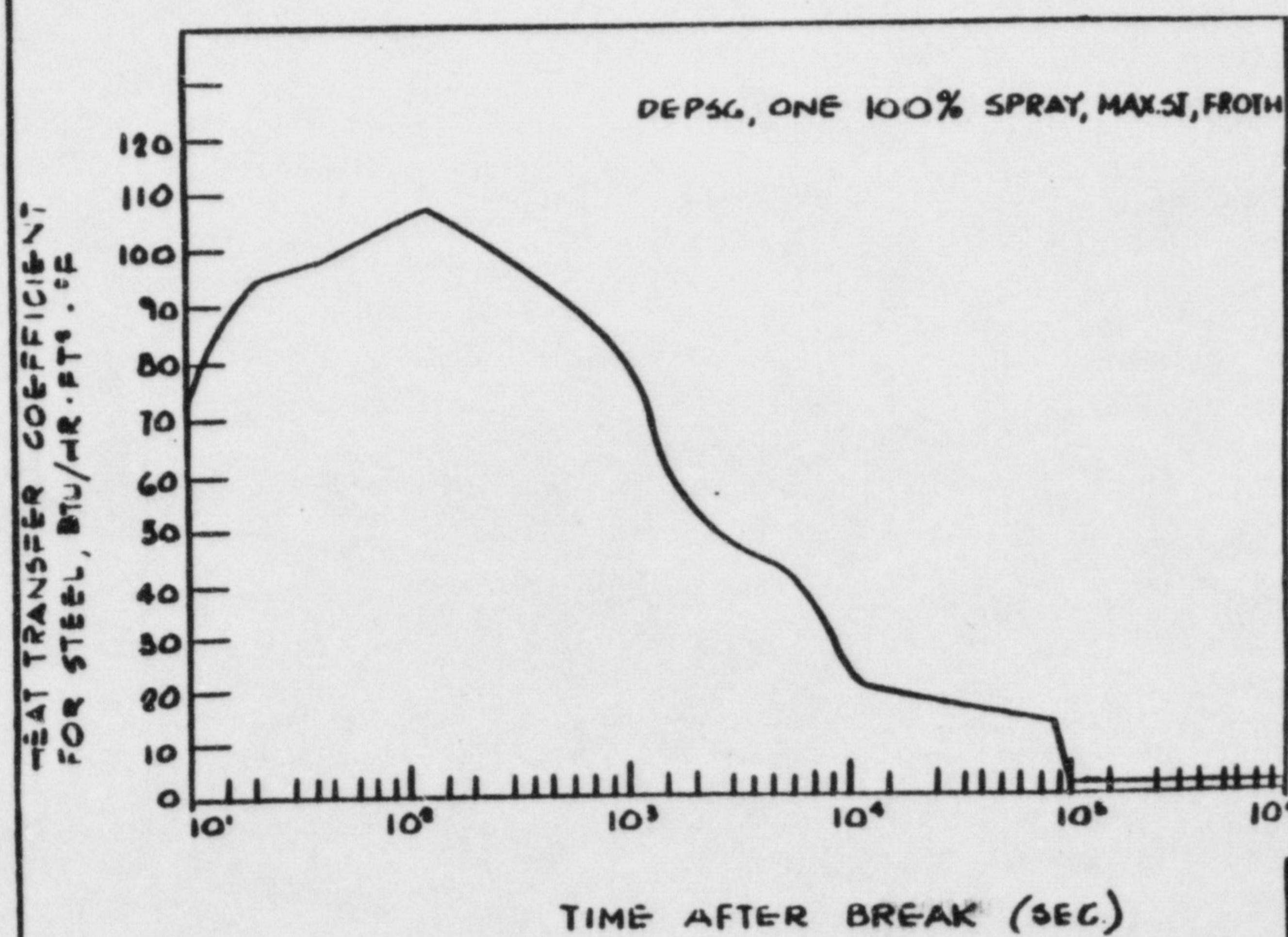

Figure 2

COMANCHE PEAK S.E.S
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STRUCTURAL HEAT TRANSFER COEFFICIENT - DEPSG

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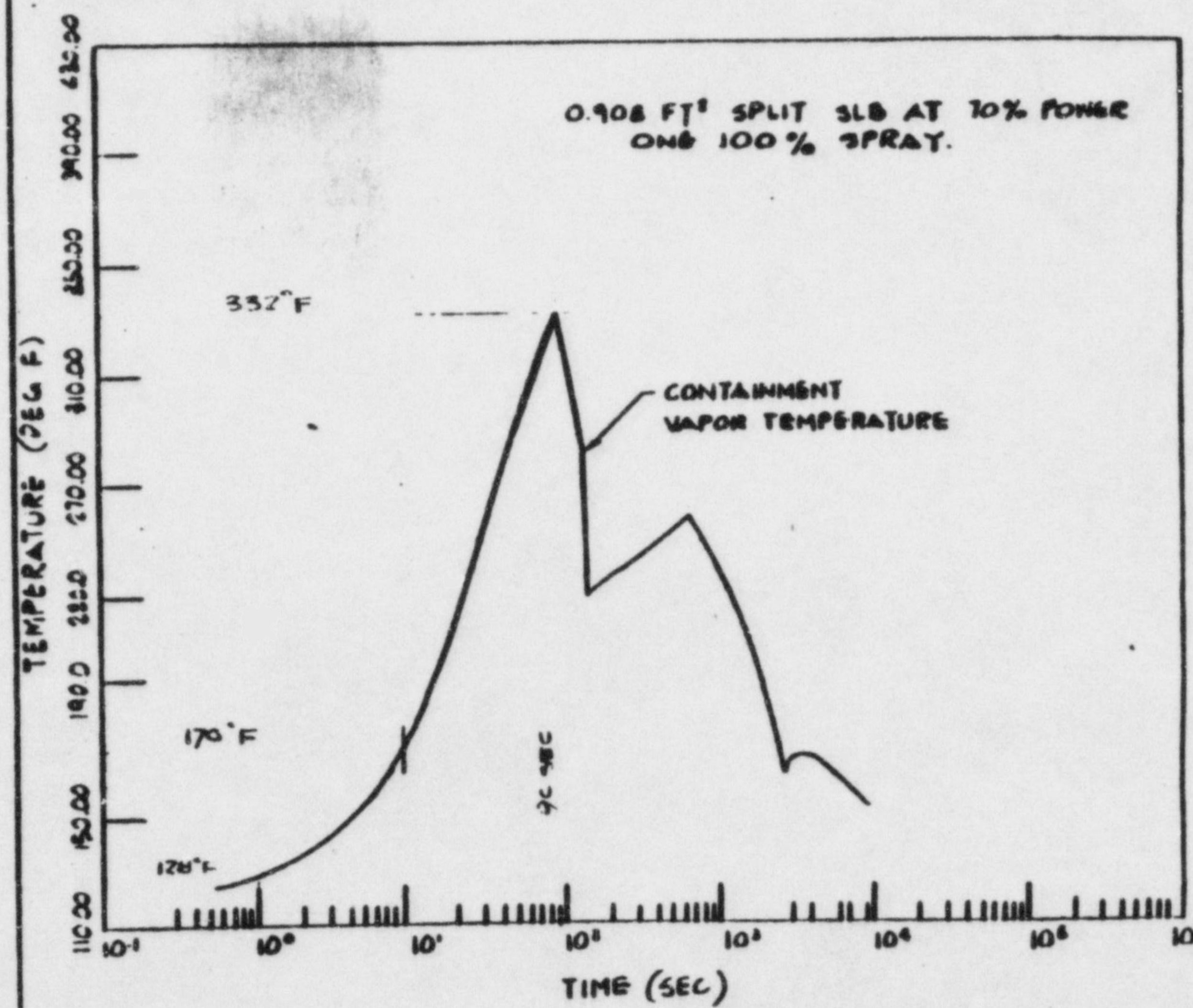

Figure 3

COMANCHE PEAK S E S
FINAL SAFETY ANALYSIS REPORT
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CONTAINMENT VAPOR TEMPERATURE TRANSIENT 0.908 FT² SPLIT SLB

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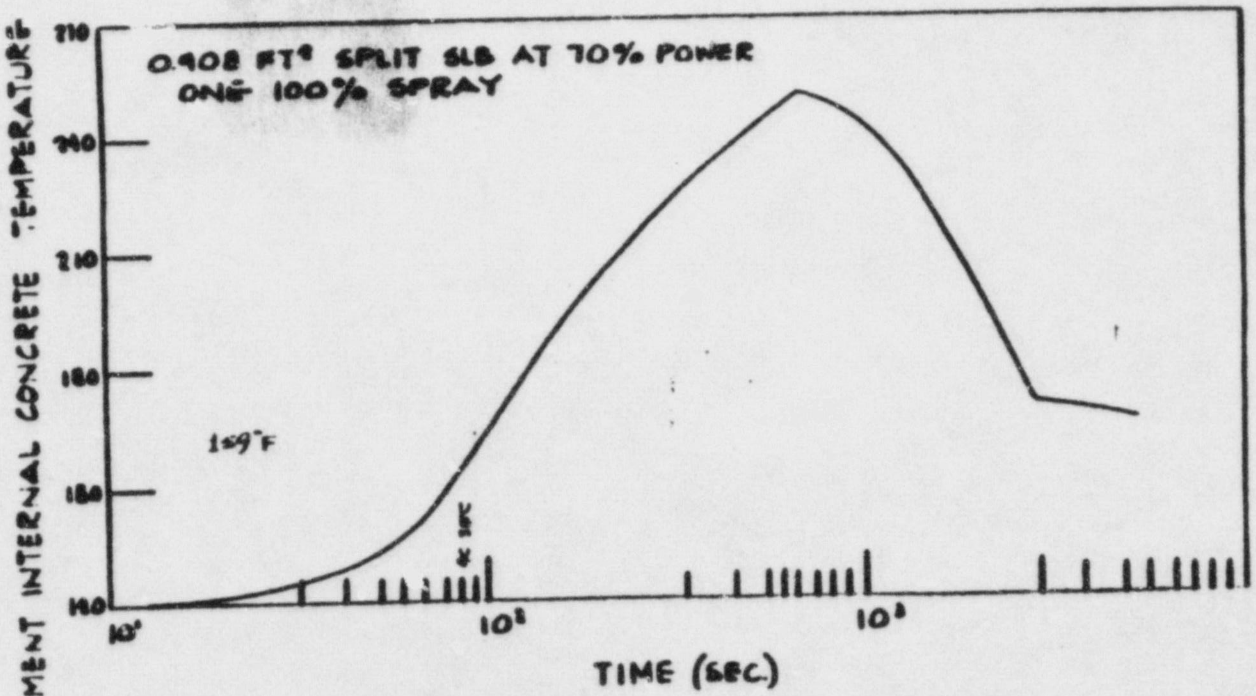

Figure 4

COMANCHE PEAK S.E.S.
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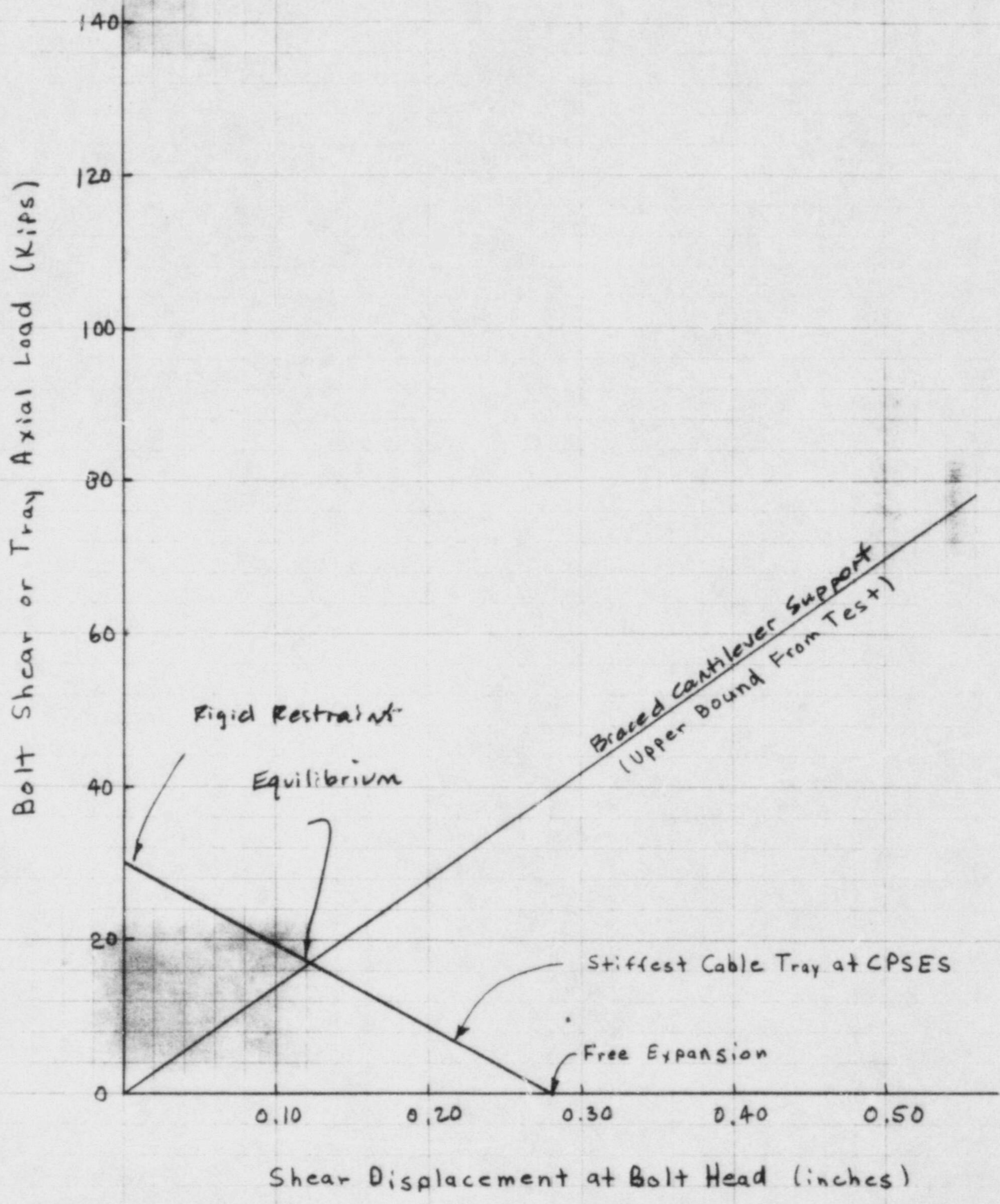
CONTAINMENT INTERNAL CONCRETE TEMPERATURE TRANSIENT 0.908 FT² SPLIT SLB

FIGURE 6.2.1-19

Figure 5: Maximum thermal loads at support anchorages



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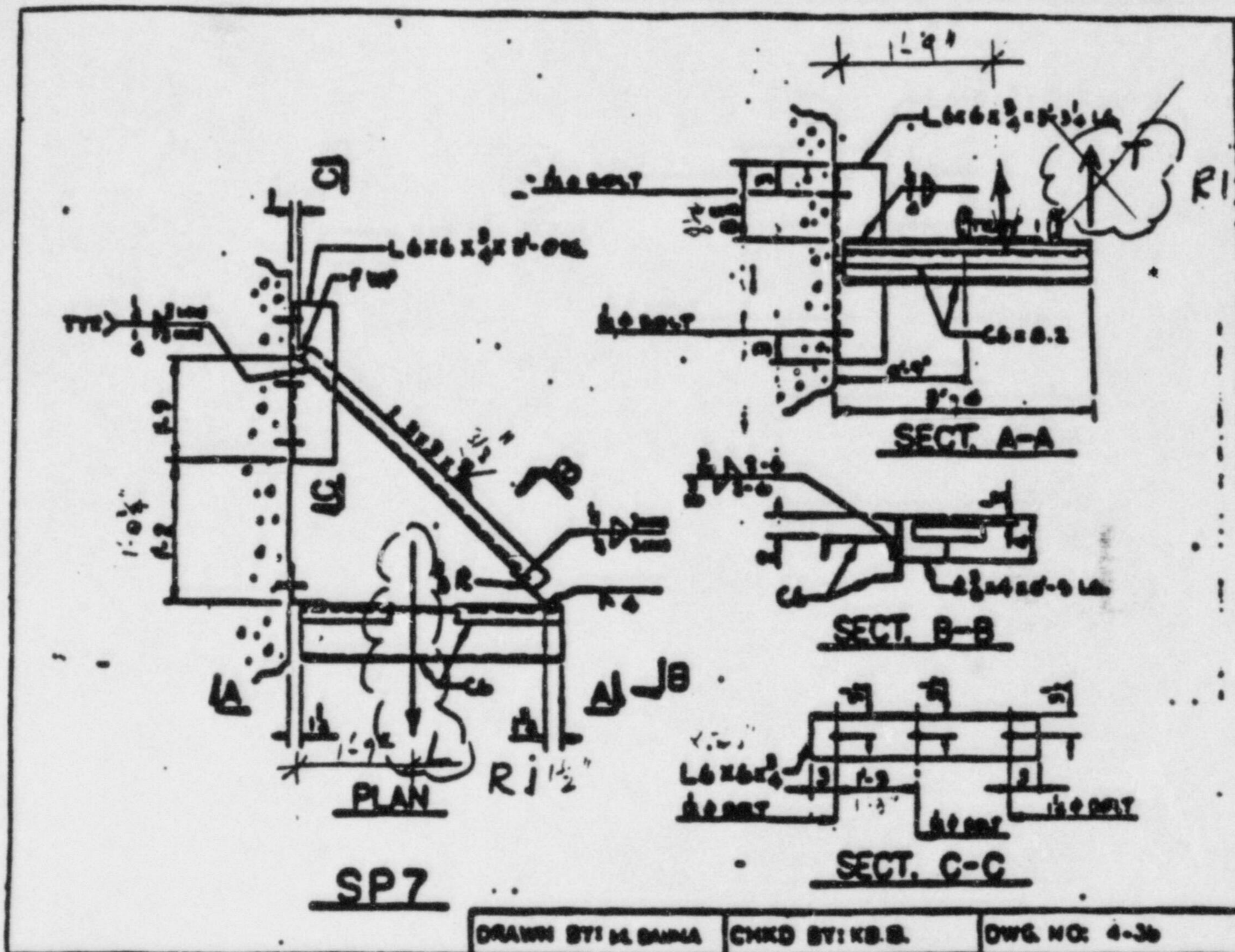

Figure 6

Tested Longitudinal Support

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