IMPELL CORPORATION

CALCULATION/PROBLEM COVER SHEET

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Client: Tugco   Project: CPSES

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Noted Within

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Method:

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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 FSAR and SEP criteria for exclusion of thermal loading are therefore satisfied

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 anchorage 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 susceptibility of steel-to-concrete anchorages to thermal loading.

3.1 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 therefore 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.

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 will be shown to be SIGNIFICANT. (SEE Section 4.1).

3.2 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 used 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 4.0 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 4.2 of this calc.). 2

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C) ACCIDENT THERMAL LOADS

FOR THE CONDITIONS WHERE MAXIMUM THERMAL GROWTH COULD OCCUR AT ANY TIME DURING THE ACCIDENT, THE MOST CRITICAL CASE[1] IS THE 0.908 FT^2-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 4.0 OF THIS CALCULATION.

4.0 RESULTS

4.1 System Slack

The available slack in the system can be estimated 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.

∴ A reasonable estimate of the slack available for free growth is 0.28

4.2 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:

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

This expansion is insignificant, especially in consideration of the total slack available in the system.

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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 75 at t=10 with linear variation (BTU/HR·FT^2·°F) [8]

ρ = 490 LB/FT^3 [4]

c = 0.11 BTU/LB·°F [5]

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

					Thermal Load Evaluation
1	TRH	2/5/87	[illegible]	1/6/87	IMPELL CORPORATION
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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

$$177 - 120 = 59^\circ F$$

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

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

$$\Delta = 0.19''$$

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. IN ADDITION, IT WILL BE SHOWN IN THE NEXT SECTION THAT THE ANCHORAGES QUALIFY DURING THE WORST THERMAL LOAD WHICH MAY OCCUR AT ANY TIME DURING AN ACCIDENT, EVEN WITHOUT CONSIDERING THE SLACK AVAILABLE IN THE SYSTEM.

GIVEN THESE WORST-CASE CONDITIONS, ACCIDENT 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.

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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 occurring 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:

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

$$\Delta = 0.56''$$

THIS CONDITION REPRESENTS THE WORST CASE THERMAL LOAD SINCE THE CABLE TRAY HAS AN UNLIMITED AMOUNT OF TIME TO HEAT UP DURING THE ACCIDENT CONDITION. IT HAS BEEN SHOWN IN SECTION 4.1 THAT AT LEAST 0.28 INCHES OF SLACK EXISTS IN A 40 FOOT STRAIGHT TRAY RUN. AS AN ADDITIONAL CONSERVATISM, HOWEVER, THIS AVAILABLE SLACK WILL NOT BE CONSIDERED FOR THE SLB ACCIDENT.

TO DEMONSTRATE THAT THERMAL LOADS NEED NOT BE EVALUATED, IT WILL BE SHOWN THAT THE STEEL TO CONCRETE ANCHORAGE IS CAPABLE OF WITHSTANDING THE TENSION AND SHEAR FORCES INDUCED BY A 0.56 INCH DISPLACEMENT OF THE CABLE TRAY.

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APPLYING THIS DISPLACEMENT HAS TWO ADDITIONAL IMPLICATIONS. FIRST, AS MENTIONED EARLIER, IT NEGLECTS THE EFFECTS OF GAPS WHOSE EXISTENCE HAS BEEN DOCUMENTED IN REFERENCE 9. SECOND, THIS APPROACH ASSUMES THAT NO RESTRAINT TO THERMAL GROWTH IS SUPPLIED BY THE SUPPORT; THE TRAY IS ASSUMED TO DISPLACE THE FULL 0.56 INCHES IN SPITE OF THE FORCE APPLIED TO IT BY THE LONGITUDINAL SUPPORT. THIS IS CONSERVATIVE BECAUSE IN ACTUALITY THE SUPPORT WILL RESTRAIN THE TRAY PRIOR TO THE FULL 0.56 INCH DISPLACEMENT.

TO IMPLEMENT THE PLAN DESCRIBED ABOVE, THE THERMAL DISPLACEMENT WAS APPLIED TO THE VERY SHORT BRACED CANTILEVER SHOWN IN FIGURE 5. THIS CONFIGURATION WAS CHOSEN BECAUSE A BRACED CANTILEVER IS THE STIFFEST TYPE OF LONGITUDINAL SUPPORT COMMONLY USED AT CPSES. FURTHERMORE, A VERY SHORT LENGTH (3'-0) AND STIFF SECTIONS (A 'T' SECTION MADE FROM TWO C6×8.2s FOR THE CANTILEVER, AND A ∠ 3×3×3/8 FOR THE BRACE) WERE ASSUMED. TWO BOLT ANCHORAGES WITH A TYPICAL ORIENTATION, AS SHOWN IN FIGURE 5, ARE ASSUMED. THIS SUPPORT CONFIGURATION WAS CHOSEN TO PROVIDE A "WORST-CASE" EVALUATION OF COMBINED SHEAR AND TENSION LOADING ON THE BOLTS.

THE SUPPORT WAS MODELLED IN DETAIL IN ACCORDANCE WITH THE MODELLING PROCEDURE SPECIFIED IN REFERENCE 6. INCLUDING THE MODELLING OF THE ECCENTRICITY BETWEEN THE TIER CG AND TRAY ℄.

TO DETERMINE THE CRITICAL [illegible] TRAY ALONG THE CANTILEVER, THE SUPPORT WAS ANALYZED USING THE IMPELL LINEAR ELASTIC [illegible] SUPERPIPE [27], ANCHORAGE REACTIONS WERE COMPARED FOR A GIVEN DISPLACEMENT AT THE POTENTIAL TRAY LOCATIONS MIDWAY ALONG THE CANTILEVER LENGTH AND NEAR THE CANTILEVER END BRACING LOCATION. THE SUPERPIPE ANALYSES WERE DOCUMENTED IN REFERENCES [25] AND [26]. THE RESULTS CLEARLY INDICATED THAT THE MAXIMUM ANCHORAGE REACTIONS WERE DEVELOPED AT THE BRACE ANCHORAGE POINT [illegible] TRAY WAS LOCATED NEAR THE CANTILEVER END. A LOCATION SLIGHTLY INSIDE THE END BRACING ATTATCHMENT POINT WAS CHOSEN TO MODEL THE TRAY ℄, AS SHOWN IN FIGURE 4.

SINCE THE DISPLACEMENT IS APPLIED IN THE LONGITUDINAL DIRECTION OF THE TRAY, THE CLIP STIFFNESS USED IN THIS DIRECTION WAS TAKEN FROM STATIC CLAMP TEST RESULTS (REF [24]) OBTAINED FOR THE STIFFEST LONGITUDINAL CLIP TYPICALLY FOUND AT CPSES. THIS STIFFNESS CAN BE OBTAINED FROM THE PLOT SHOWN IN FIGURE 6. A DIAGRAM OF THE CLIP IS SHOWN IN FIGURE 7. THE CLIP STIFFNESSES IN THE REMAINING DIRECTIONS ARE INPUT AS RIGID. ALL SUPPORT AND CLIP MODELING WAS DONE IN ACCORDANCE WITH THE PROCEDURES SPECIFIED IN REFERENCE 6. NOTE THAT FIGURES 6 & 7 ARE FROM REFERENCE 24.

THIS SUPPORT MODEL WAS THEN ANALYZED FOR THE FULL 0.56" LONGITUDINAL DISPLACEMENT AT THE TRAY LOCATION USING SUPERPIPE. SECTION 4.3 ASSESSES THE THERMAL GROWTH EFFECTS IF 1" HILTI KWIK BOLTS ARE ASSUMED FOR THE SUPPORT ANCHORAGE. A SIMILAR ASSESSMENT IS PERFORMED [illegible] 1" RICHMOND INSERTS. THESE BOLTS REPRESENT THE SMALLEST SIZES USED FOR CPSES LONGITUDINAL CABLE TRAY SUPPORTS.

					THERMAL LOAD EVALUATION
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4.3 "WORST CASE" SUPPORT EVALUATION FOR COMBINED SHEAR AND TENSION EFFECTS.

a) 1"Ø HILTI KWIK BOLTS

IN PERFORMING THE SUPERPIPE ANALYSIS, BASE ANGLE ANCHORAGE STIFFNESS VALUES [illegible] WHICH REALISTICALLY REPRESENTED THE ACTUAL STIFFNESS VALUES AT THE BASE ANGLE [illegible] THE [illegible] TRAY [illegible] THE DIRECTION OF TRAY GROWTH ASSUMED [illegible] FIGURE 8, THIS PLACES THE BRACE [illegible] ANCHOR BOLT IN MAXIMUM TENSION.

THE FOLLOWING STIFFNESSES WERE USED:

CANTILEVER ANCHORAGE	BRACE ANCHORAGE
K_Z = RIGID	K_Z = 40 KIP/IN
K_{XX} = 2800 KIP-IN/RAD	K_{XX} = 2800 KIP-IN/RAD

THE REMAINING ANCHORAGE DEGREES OF FREEDOM ARE CONSIDERABLY STIFFER AND THEREFORE CONSERVATIVELY ASSUMED TO BE RIGID. THE ANCHORAGE CONFIGURATION ASSUMED AND ANCHORAGE COORDINATE AXES ARE ILLUSTRATED IN FIGURE 8. THE SAME ANCHORAGE, WITH DIFFERENT ORIENTATION, IS USED FOR THE [illegible] MEMBERS.

THE HIGHEST LOADS FROM THE SUPERPIPE ANALYSIS [21], WHICH WERE FOUND AT THE BRACE ANCHORAGE, WERE THEN USED TO QUALIFY THE ANCHOR BOLTS USING THE CDC NON-LINEAR PROGRAM BASEPLATE II [28]. AS SHOWN IN FIGURE 8, A TWO BOLT BASE ANGLE WAS MODELED SINCE ONE BOLT BASE ANGLES ARE NOT USED FOR LONGITUDINAL SUPPORTS. THE MANUFACTURER'S SPECIFIED MINIMUM CENTER-TO-CENTER SPACING WAS ASSUMED AND THE LOADS WERE ASSUMED TO ACT HALFWAY UP THE FREE LEG, WITH THE ATTACHMENT LOCATION DIRECTLY IN LINE WITH ONE OF THE BOLTS. BOTH OF THESE ASSUMPTIONS MAXIMIZED THE TENSION GENERATED IN THE CRITICAL BOLT.

TO ANALYZE THE FIGURE 8 MODEL IN BASERII, A NON-LINEAR TENSION STIFFNESS WAS INPUT FOR THE 1" HILTI ANCHOR BOLTS. THE LOAD DISPLACEMENT CURVE FOR TENSION WAS TAKEN FROM MANUFACTURER'S TEST DATA (REFERENCE 17) AND IS SHOWN IN FIGURE 9. A BI-LINEAR TENSION LOAD-DISPLACEMENT CURVE WAS GENERATED USING THE AVERAGE RECORDED SLIP AND ULTIMATE DISPLACEMENTS FROM THE FIVE TEST SAMPLES SINCE NO LOAD-DISPLACEMENT CURVES WERE PRESENTED IN REFERENCE [14]. THE CURVE CORRESPONDS TO AN EMBEDMENT DEPTH OF 4 1/2". EMBEDMENT DEPTHS AT CPSES FOR 1" HILTIS ARE TYPICALLY MUCH GREATER. A CONSTANT BOLT SHEAR STIFFNESS OF 111 KIP/IN, TAKEN FROM REFERENCE [13] WAS USED.

THE FOLLOWING REACTIONS, TAKEN FROM THE REFERENCE [21] SUPERPIPE ANALYSIS, WERE APPLIED TO THE BASE R II ATTACHMENT LOCATION:

$$F_x = 5690 \text{ lb} \qquad M_x = 8940 \text{ lb-in}$$
$$F_y = 1020 \text{ lb} \qquad M_y = 11820 \text{ lb-in}$$
$$F_z = 5360 \text{ lb} \qquad M_z = 8940 \text{ lb-in}$$

THE ABOVE LOADS WERE APPLIED SIMULTANEOUSLY IN A SINGLE LOAD CASE. DIFFERENT LOAD COMBINATIONS WERE CREATED BY REVERSING LOAD DIRECTIONS IN ORDER TO MAXIMIZE THE LOADING ON THE CRITICAL BOLT. THE RESULTING MAXIMUM LOADS WERE [17]:

$$T_{max} = 8411 \text{ lb}$$
$$|S_{max\,x}| = 2849 \text{ lb}$$
$$|S_{max}|_y = 2601 \text{ lb}$$
$$S_{max} = \left[(S_{max\,x})^2 + (S_{max\,y})^2\right]^{1/2} = 3858 \text{ lb.}$$

THE BOLTS WERE THEN QUALIFIED USING THE FOLLOWING INTERACTION EQUATION FROM REFERENCE [13] AND

AVERAGE ULTIMATE LOADS FROM REFERENCE [14]
BASED ON 4 1/2" EMBEDMENT.

$$\frac{T_{MAX}}{T_{ULT}} + \frac{S_{MAX}}{S_{ULT}} \leq 1.0$$

$T_{ULT} = 16019$ lb [14]
$S_{ULT} = 26879$ lb [14]

$$\frac{8411}{16019} + \frac{3858}{26879} = 0.67 \leq 1.0$$

AS AN ADDED CHECK, THE SHEAR AND TENSION DISPLACEMENTS OF THE 1" ϕ HILTIS WILL BE COMPARED TO THE AVERAGED SHEAR AND TENSION ULTIMATE DISPLACEMENTS FROM REFERENCE [14], ΔS_{ULT} AND ΔT_{ULT}, RESPECTIVELY. THE TENSION DISPLACEMENT FROM ALL LOAD COMBINATIONS FROM BASEPLATE II [17], ΔT, IS THE MAXIMUM TENSION DISPLACEMENT FROM ALL LOAD COMBINATIONS. THE SHEAR DISPLACEMENT FROM BASEPLATE II [17], ΔS, IS THE SRSS COMBINATION OF THE MAXIMUM ANCHORAGE ΔX AND ΔY FOR BOTH BOLTS AND ALL LOAD COMBINATIONS.

$\Delta S = 0.035$ IN $\qquad \Delta S_{ULT} = 0.320$ IN

$$\frac{\Delta S}{\Delta S_{ULT}} = 0.109$$

$\Delta T = 0.220$ IN $\qquad \Delta T_{ULT} = 0.543$ in

$$\frac{\Delta T}{\Delta T_{ULT}} = 0.405$$

THE EQUILIBRIUM TENSION DISPLACEMENT IS INDICATED ON THE LOAD DISPLACEMENT CURVE IN FIGURE 9. FOR MORE TYPICAL CPSES EMBEDMENT DEPTHS FOR 1" HILTIS (ie 8 TO 10 INCHES), THE EQUILIBRIUM TENSION DISPLACEMENT WOULD BE A FAR SMALLER PERCENTAGE OF THE ULTIMATE.

THE LINEAR STIFFNESS ASSUMED IN SHEAR IS APPROPRIATE SINCE THE APPLIED SHEAR LOAD REMAINS FAR BELOW THE ULTIMATE.

THE K_Z AND K_{XX} ANCHORAGE STIFFNESSES ORIGINALLY ASSUMED IN THE SUPERPIPE ANALYSIS ARE VERIFIED IN ADDITIONAL LOAD CASES DOCUMENTED IN REFERENCE [17]. THE SAME PROCEDURE USED IN REFERENCE [16] TO DEVELOP ANCHORAGE STIFFNESSES WAS USED TO VERIFY THE ASSUMED STIFFNESS. THE F_Z AND M_X LOAD COMPONENTS ARE APPLIED TO THE BASE ~~PL~~ II BASE ANGLE ATTACHMENT LOCATION AND APPLIED AS SEPARATE LOAD CASES (SEE FIGURE 8 FOR LOCAL COORDINATE ORIENTATION).

THE ROTATION STIFFNESS CAN BE DETERMINED BY REVIEWING THE DISPLACEMENTS OF THE NODES ADJACENT TO THE LOADING:

FOR LOAD CASE #6 [17], LOADING OF $M_x = 8.94$ KIP-IN

$$\Delta Z_{11} = -.0009329'' \qquad \Delta Z_{12} = .003870'' \quad ; \quad \Delta = .004803''$$

$$\angle = \sin^{-1} \frac{.004803''}{1.5''} = .003202 \text{ RAD}$$

$$K_{xx} = \frac{8.94 \text{ KIP-IN}}{.003202 \text{ RAD}} = 2792 \text{ KIP-IN/RAD}$$

THE TRANSLATION STIFFNESS FOR PULL-OUT LOADING CAN BE DETERMINED DIRECTLY BY REVIEWING THE DISPLACEMENT AT THE LOAD POINT:

FOR LOAD CASE #5 [17] LOADING OF $F_Z = 5.36$ KIP

$$\Delta Z_{11} = .2154''$$

$$K_Z = \frac{5.36 \text{ KIP}}{.2154 \text{ IN}} = 25 \text{ KIP/IN}$$

THERMAL [illegible] EVALUATION

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IMPELL CORPORATION

JOB NO [illegible]

CALC NO [illegible]

FOR THE BRACING MEMBER ATTACMENT REACTIONS, THE VERIFIED ANCHORAGE STIFFNESS ARE COMPARABLE TO THE ASSUMED ANCHORAGE STIFFNESS BUT WITH THE PULL-OUT STIFFNESS, K_Z, BEING SOMEWHAT SOFTER THAN WHAT WAS ASSUMED.

	VERIFIED	ASSUMED
K_Z	25 KIP/IN	40 KIP/IN
K_{XX}	2792 KIP-IN/RAD	2800 KIP-IN/RAD

FOR THE CANTILEVER ATTACHMENT REACTIONS THE VERIFIED ANCHORAGE STIFFNESS ARE COMPARABLE TO THOSE ASSUMED.

	VERIFIED	ASSUMED
K_{XX}	2792 KIP-IN/RAD	2800 KIP-IN/RAD

FOR THE CANTILEVER ATTACHMENT, THE F_Z COMPONENT CAUSES COMPRESSION IN THE BOLTS AND THEREFORE THE MODELLING OF A RIGID STIFFNESS IN THIS DIRECTION IS APPROPRIATE.

2	+P1	5/5/87	DWS	5/5/87	THERMAL PIPE SUPPORT [illegible]
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b) 1" RICHMOND INSERTS

A SIMILAR SUPERPIPE ANALYSIS WAS PERFORMED WITH RICHMOND INSERTS ASSUMED AT THE ANCHORAGES. THE KZ AND KXX STIFFNESS VALUES USED FOR THIS ANALYSIS WERE CORRESPONDINGLY INCREASED DUE TO THE HIGHER STIFFNESS OF THIS BOLT TYPE.

CANTILEVER ANCHORAGE	BRACE ANCHORAGE
KZ = RIGID	KZ = 300 KIP/IN.
KXX = 8500 KIP IN/RAD	KXX = 8500 KIP IN/RAD

THESE STIFFNESS VALUES WILL AGAIN BE CONFIRMED PERFORMING A BASER II ANALYSIS USING THE FIGURE 8 CONFIGURATION AND BOLT STIFFNESS VALUES CORRESPONDING TO RICHMOND INSERTS.

THE CRITICAL REACTIONS FROM THE SUPERPIPE ANALYSIS (DOCUMENTED IN REFERENCE [22]) WERE AGAIN FOUND AT THE BRACE MEMBER ATTACHMENT ANCHORAGE.

TO ANALYZE THE FIGURE 3 MODEL IN BASEPL II, A NON-LINEAR TENSION STIFFNESS WAS INPUT FOR THE 1" RICHMOND INSERTS. THE LOAD DISPLACEMENT CURVE FOR TENSION WAS TAKEN FROM REFERENCE [19] AND IS SHOWN IN FIGURE 10. THE CURVE CHOSEN FROM REFERENCE [19] WAS ONE THAT SHOWED A TYPICAL REPRESENTATION OF THE BEHAVIOR OF THE 1" RICHMOND INSERTS IN TENSION. A CONSTANT BOLT SHEAR STIFFNESS OF 435 KIP/IN TAKEN FROM REFERENCE [13] WAS USED.

THE FOLLOWING REACTIONS TAKEN FROM THE REFERENCE [22] SUPERPIPE ANALYSIS, WERE APPLIED TO THE BASEPL II ATTACHMENT LOCATION:

$F_x = 7120$ lb	$M_x = 14300$ lb-in
$F_y = 1350$ lb	$M_y = 2630$ lb-in
$F_z = 7050$ lb	$M_z = 14300$ lb-in

AS BEFORE, DIFFERENT COMBINATIONS OF THE ABOVE LOADS WERE PUT INTO BASEPL II TO MAXIMIZE THE LOADING ON THE BOLTS. THE RESULTING MAXIMUM BOLT LOADS WERE [18]:

$$T_{max} = 20135 \text{ lb}$$
$$(S_{max})_x = 3586 \text{ lb}$$
$$(S_{max})_y = 3566 \text{ lb}$$
$$S_{max} = \left[(S_{max\,x})^2 + (S_{max\,y})^2 \right]^{1/2} = 5057 \text{ lb}$$

THE BOLTS WERE THEN QUALIFIED USING THE FOLLOWING INTERACTION EQUATION FROM REFERENCE [13] AND

AVERAGE ULTIMATE LOADS FROM REFERENCE [19]:

$$\left(\frac{T_{MAX}}{T_{ULT}}\right)^{4/3} + \left(\frac{S_{MAX}}{S_{ULT}}\right)^{4/3} \doteq 1$$

$T_{ULT} = 42600$ lb [19]
$S_{ULT} = 45200$ lb [19]

$$\left(\frac{20135}{42600}\right)^{4/3} + \left(\frac{5057}{45200}\right)^{4/3} = 0.42 \leq 1$$

IN ADDITION, THE BOLTS MUST BE QUALIFIED USING THE FOLLOWING INTERACTION EQUATION FROM REFERENCE [13]:

$$\left(\frac{T_{MAX}}{T_{BOLT}}\right)^{2} + \left(\frac{S_{MAX}}{S_{BOLT}}\right)^{2} \leq 1$$

WHERE T_{BOLT} AND S_{BOLT} ARE THE ULTIMATE TENSION AND SHEAR CAPACITIES OF THE BOLT CONSERVATIVELY CHOSEN TO BE MADE OF A307.

$$T_{BOLT} = 36350 \text{ lb} \quad [20]$$

THE SHEAR VALUE, S_{BOLT}, IS CONSERVATIVELY TAKEN AS THE ALLOWABLE SHEAR LOAD ON A 1" Ø A307 BOLT IN SINGLE SHEAR. NOTE, THIS IS A DESIGN ALLOWABLE AS OPPOSED TO AN ULTIMATE CAPACITY.

$$S_{BOLT} = 7850 \text{ lb} \quad [4]$$

$$\left(\frac{20135}{36350}\right)^{2} + \left(\frac{5057}{7850}\right)^{2} = 0.72 \leq 1.0$$

AS AN ADDED CHECK THE MAXIMUM SHEAR AND TENSION DISPLACEMENTS OF THE 1" Ø RICHMOND INSERTS WILL BE COMPARED TO AVERAGE SHEAR AND TENSION ULTIMATE DISPLACEMENTS FROM REFERENCES [19]. THE PROCEDURE TO OBTAIN THE SHEAR AND TENSION DISPLACEMENTS FROM [illegible] I IS THE SAME PROCEDURE DESCRIBED IN SECTION 4.3a.

$\Delta S = 0.010$ $\qquad\qquad \Delta S_{ULT} = 0.294$ in

$$\frac{\Delta S}{\Delta S_{ULT}} = 0.034$$

$\Delta T = 0.01868$ $\qquad\qquad \Delta T_{ULT} = 0.163$ in

$$\frac{\Delta T}{\Delta T_{ULT}} = 0.115$$

THE EQULIBRIUM TENSION DISPLACEMENT IS INDICATED ON THE LOAD DISPLACEMENT CURVE IN FIGURE 10. FIGURE 10 IS A TYPICAL CURVE WITH AN ULTIMATE DISPLACEMENT OF 0.146". EVEN COMPARING ΔT, 0.01868", TO THE MINIMUM ΔT_{ULT}, 0.134", FROM REFERENCE [19] SHOWS THAT IF THE MINIMUM ΔT_{ULT} IS USED FOR THE CURVE, THE EQULIBRIUM TENSION DISPLACEMENT REMAINS FAR BELOW THE MINIMUM ΔT_{ULT}.

THE LINEAR STIFFNESS ASSUMED IN SHEAR IS SHOWN TO BE APPROPRIATE SINCE THE APPLIED SHEAR LOAD REMAINS FAR BELOW THE ULTIMATE.

THE K_Z AND K_{XX} ANCHORAGE STIFFNESS ORIGINALLY ASSUMED IN THE SUPERPIPE ANALYSIS ARE VERIFIED BY ADDITIONAL LOAD CASES DOCUMENTED IN REFERENCE [18]. THE F_Z AND M_{XX} LOAD COMPONENTS ARE APPLIED TO THE BASE PL. II BASEANGLE ATTACHMENT AND APPLIED AS SEPERATE LOAD CASES (SEE FIGURE 8 FOR LOCAL COORDINATE ORIENTATION). THE PROCEDURE TO OBTAIN THE STIFFNESSES IS THE SAME PROCEDURE AS DESCRIBED IN SECTION 4.3a.

ROTATIONAL STIFFNESS, K_{XX}:

FOR LOAD CASE # 6 [18], LOADING OF $M_X = 14.3$ KIP-IN

$$\left.\begin{array}{l} \Delta Z_{11} = -.001898'' \\ \Delta Z_{12} = .0006247'' \end{array}\right\} \Delta = .002523''$$

$$\angle = \sin^{-1} \frac{.002523''}{1.5''} = .001682 \text{ RAD}$$

$$K_{XX} = \frac{14.3 \text{ KIP-IN}}{.001682 \text{ RAD}} = 8502 \text{ KIP-IN/RAD}$$

TRANSLATIONAL STIFFNESS, K_Z:

FOR LOAD CASE # 5 [18], LOADING OF $F_Z = 7.05$ KIP

$$\Delta Z_{11} = .02360''$$

$$K_Z = \frac{7.05 \text{ KIP}}{.02360 \text{ IN}} = 299 \text{ KIP/IN}$$

FOR THE BRACING MEMBER ATTACHMENT REACTIONS, THE VERIFIED ANCHORAGE STIFFNESSES ARE COMPARABLE WITH THOSE ASSUMED.

	VERIFIED	ASSUMED
K_z	299 KIP/IN	300 KIP/IN
K_{xx}	8502 KIP-IN/RAD	8500 KIP-IN/RAD

FOR THE CANTILEVER ATTACHMENT REACTIONS, THE VERIFIED ANCHORAGE STIFFNESS COMPARABLE WITH THOSE ASSUMED.

	VERIFIED	ASSUMED
K_{xy}	8502 KIP-IN/RAD	8500 KIP-IN/RAD

FOR THE CANTILEVER ATTACHMENT, THE F_z COMPONENT CAUSES COMPRESSION IN THE BOLTS AND THEREFORE THE MODELLING OF A RIGID STIFFNESS IN THIS DIRECTION IS APPROPRIATE,

4.4 "Worst Case" Support for Shear Alone

In the previous section, the braced cantilever was intended to represent a 'worst case' condition for combined shear and tension bolt deformation. For bolt shear alone, a "worst case" [illegible] to a concrete [illegible].

The direct clamping of trays to concrete is rarely employed, particularly in the Reactor Building. Due to the high axial compressive force mobilized by the stiff support, any available system slack will be mobilized by slippage. The equilibrium displacement at the support point will also be significantly reduced by the tray axial flexibility.

The bolt shear displacement is estimated by again conservatively assuming a 40' tray run and neglecting axial tray flexibility. Tray slack will be assumed mobilized by slippage at the high axial loads.

$$\Delta_{bolt} = \Delta_{TOTAL} - \Delta_{SLACK}$$
$$\Delta_{bolt} = 0.56 - 0.28 = 0.28 \text{ in.}$$

$\Delta_{ultimate,\ shear,\ 1'' \text{ Hilti}} = 0.320 \text{ in} > 0.28 \text{ in}$

* $\Delta_{ultimate,\ shear,\ 1'' \text{ Richmond}} = 0.294 \text{ in} > 0.28 \text{ in.}$

Therefore both bolts have adequate shear displacement capacity to accomodate the "worst case" condition

* The average test value presented is based on an "initial spalling" of concrete rather than a loss of structural capacity to resist load. A significantly greater margin against failure therefore actually exists.

					THERMAL LOAD EVALUATION
2	DWS	5/5/87	[illegible]	5/5/87	IMPELL CORPORATION — JOB NO 0210-041 — CALC NO M-27 — PAGE 29 OF 47
REV	BY	DATE	CHECKED	DATE	

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. THE ANCHORAGE EVALUATION HAS SHOWN THAT:

1. 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.

2. FOR A SUPPORT INTENDED TO DEMONSTRATE "WORST CASE" COMBINED SHEAR AND TENSION BOLT DEFORMATIONS, THE MAXIMUM THERMAL EXPANSION WHICH MAY OCCUR DURING AN ACCIDENT EVENT WAS SHOWN TO BE LIMITED TO VALUES WHICH WOULD NOT CAUSE STRUCTURAL FAILURE. THIS WAS SHOWN TO BE TRUE EVEN FOR A LONGITUDINAL CABLE TRAY SUPPORT WITH AN UPPER BOUND STIFFNESS AND A "WORST CASE" ARRANGEMENT OF BOLTS AT THE BASE ANGLE ANCHORAGE. THE VALIDITY OF THIS CONCLUSION WAS NOT DEPENDENT ON MOBILIZING SYSTEM SLACK.

					Thermal Load Evaluation
2	HAH	5/5/87	DWS	5/5/87	JOB NO [illegible]
1	[illegible]	2/23/87	HAH	2/24/87	CALC NO M 27
REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION

THE SHEAR AND TENSILE FORCES USED TO QUALIFY THE ANCHOR BOLTS FOR THE ACCIDENT THERMAL LOAD CASE WERE OVERESTIMATED FOR THE FOLLOWING REASONS:

- THE THERMAL DISPLACEMENT WAS BASED ON A 40 FOOT STRAIGHT RUN OF CABLE TRAY. INSPECTION OF THE REACTOR BUILDING SPAN LENGTH DRAWINGS SHOWS THIS LENGTH IS UNREALISTIC DUE TO THE CLOSELY SPACED CABLE TRAY BENDS IN THIS BUILDING.

- IN SPITE OF THE DOCUMENTED SLACK WHICH EXISTS IN THE CABLE TRAY SYSTEMS, IT WAS ASSUMED THAT THE THERMAL DISPLACEMENT OF THE SUPPORT WOULD BE THE SAME AS THE THERMAL DISPLACEMENT OF THE TRAY.

- THE THERMAL DISPLACEMENT WAS APPLIED TO A "WORST CASE" SUPPORT IN A LOCATION WHICH MINIMIZED THE FLEXURAL DEFORMATION OF THE SUPPORT AND MAXIMIZED REACTION FORCE AND MOMENT AT THE ANCHORAGE.

- ALTHOUGH THE AXIAL FLEXIBILITY OF THE TRAY WOULD REDUCE THE EQUILIBRIUM DISPLACEMENT AT THE SUPPORT POINT, THE ENTIRE THERMAL DISPLACEMENT WAS APPLIED TO THE CLAMP.

- IN PERFORMING SEPARATE SUPERPIPE ANALYSES FOR ANCHORAGES WITH BOTH HILTI KWIK BOLTS AND RICHMOND INSERTS, FINITE STIFFNESS VALUES WERE ASSUMED FOR THE LOCAL KY AND KXX ANCHORAGE DIRECTIONS. THESE ASSUMED STIFFNESS VALUES WERE LATER CONFIRMED BY ANALYZING THE ANCHORAGE FOR APPLIED LOADS IN THESE TWO DIRECTIONS WITH THE NON-LINEAR PROGRAM BASE II. ANCHORAGE STIFFNESS VALUES FOR THE REMAINING DEGREES OF FREEDOM WERE ASSUMED RIGID.

- A CRITICAL ANCHORAGE CONFIGURATION WAS ASSUMED IN THE BASE II QUALIFICATION. USING THE MEMBER ANCHORAGE REACTIONS CALCULATED BY SUPERPIPE, THE BOLT DISPLACEMENTS AND SHEAR/TENSILE LOADS WERE SHOWN TO BE FAR BELOW ULTIMATE VALUES. THE EQUILIBRIUM LOCATIONS ARE INDICATED IN FIGURES 9 AND 10 FOR THE BOLT TENSION DIRECTIONS. DUE TO THE LARGE AMOUNT OF DEFORMATIONAL CAPACITY REMAINING IN THE BOLTS, THERMAL DISPLACEMENTS SIGNIFICANTLY LARGER THAN THOSE CALCULATED COULD BE ACCOMODATED WITHOUT FAILURE.

					THERMAL LOAD EVALUATION
2	RKH	5/5/87	DWS	5/5/87	IMPELL CORPORATION — JOB NO 0210-071 — CALC NO M-27 — PAGE 32 OF 47
REV	BY	DATE	CHECKED	DATE	

3. FOR A SUPPORT INTENDED TO DEMONSTRATE A "WORST CASE" CONDITION FOR BOLT SHEAR ALONE, THE MAXIMUM DISPLACEMENT WAS ONCE MORE SHOWN TO BE LIMITED TO VALUES WHICH WOULD NOT CAUSE FAILURE. FOR THIS CASE THE AVAILABLE SLACK WAS ASSUMED MOBILIZED BY THE HIGH AXIAL TRAY LOADS

IT HAS BEEN THEREFORE BEEN SHOWN THAT THE CPSES CABLE TRAY SYSTEMS SATISFY THE SRP/FSAR CRITERIA FOR THE EXCLUSION OF THERMAL LOADS FROM DESIGN CRITERIA.

					THERMAL LOAD EVALUATION		
2	TRA	5/5/87	DWS	5/5/87	IMPELL CORPORATION	JOB NO 0210-041 CALC NO M-27	PAGE 33 OF 47
REV	BY	DATE	CHECKED	DATE			

6.0 REFERENCES

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[2] STATIC TESTS OF CABLE TRAYS AND FITTINGS FOR CPSES BY CC&L · REPORT # A-719-86, JUNE, 1986

[3] CPSES UNIT I FIELD VERIFICATION METHOD, TNE-FUM-CS-001, MARCH 20, 1986

[4] AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC), 7th EDITION

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[6] IMPELL PROJECT INSTRUCTIONS PI-02 "DYNAMIC ANALYSIS OF CABLE TRAY SYSTEMS" REV. 5

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[8] FRANK P. INCROPERA AND DAVID P. DEWITT, FUNDEMENTALS OF HEAT TRANSFER, JOHN WILEY & SONS, NEW YORK, 1981

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[12] Theodore Baumeister, Eugene A. Avallone and Theodore Baumeister III, Mark's Standard Handbook for Mechanical Engineers, 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. H2139-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

[16] IMPELL CALCULATION M-04: "BASE ANGLE STIFFNESSES", REV. 1

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HILTI

[18] BASER II OUTPUT 87/05/02 . 15.59.16
RICHMOND

[19] TEST REPORT: "SHEAR AND TENSION LOADING OF RICHMOND INSERTS 1½-INCH TYPE EC-6W 1-INCH TYPE EC-2W" APRIL 19, 1984

[20] ASTM SPECIFICATION A 307-78.

[21] SUPER PIPE RUN 87/05/02 . 15.49.14 HILTI

[22] SUPERPIPE RUN 87/05/02 16.30.37 RICHMOND

[23] NOT USED

[24] C.C.L. REPORT # A-717-86: "TEST REPORT FOR MONOTONIC AND CYCLIC TESTS FOR TRAY CLAMPS FOR CPSES", JUNE 27, 1986; APPENDIX C-4.

[25] IMPELL SUPERPIPE OUTPUT 87/05/05 . 14.33.53

[26] IMPELL SUPERPIPE OUTPUT 87/05/05 . 13.42.15

[27] SUPERPIPE, V.21A 10/16/86, IMPELL CORPORATION STANDARD PROGRAM

[28] BASEPLATE II, V 2.6 JULY 86, COMPUTER PROGRAM

7.0 FIGURES

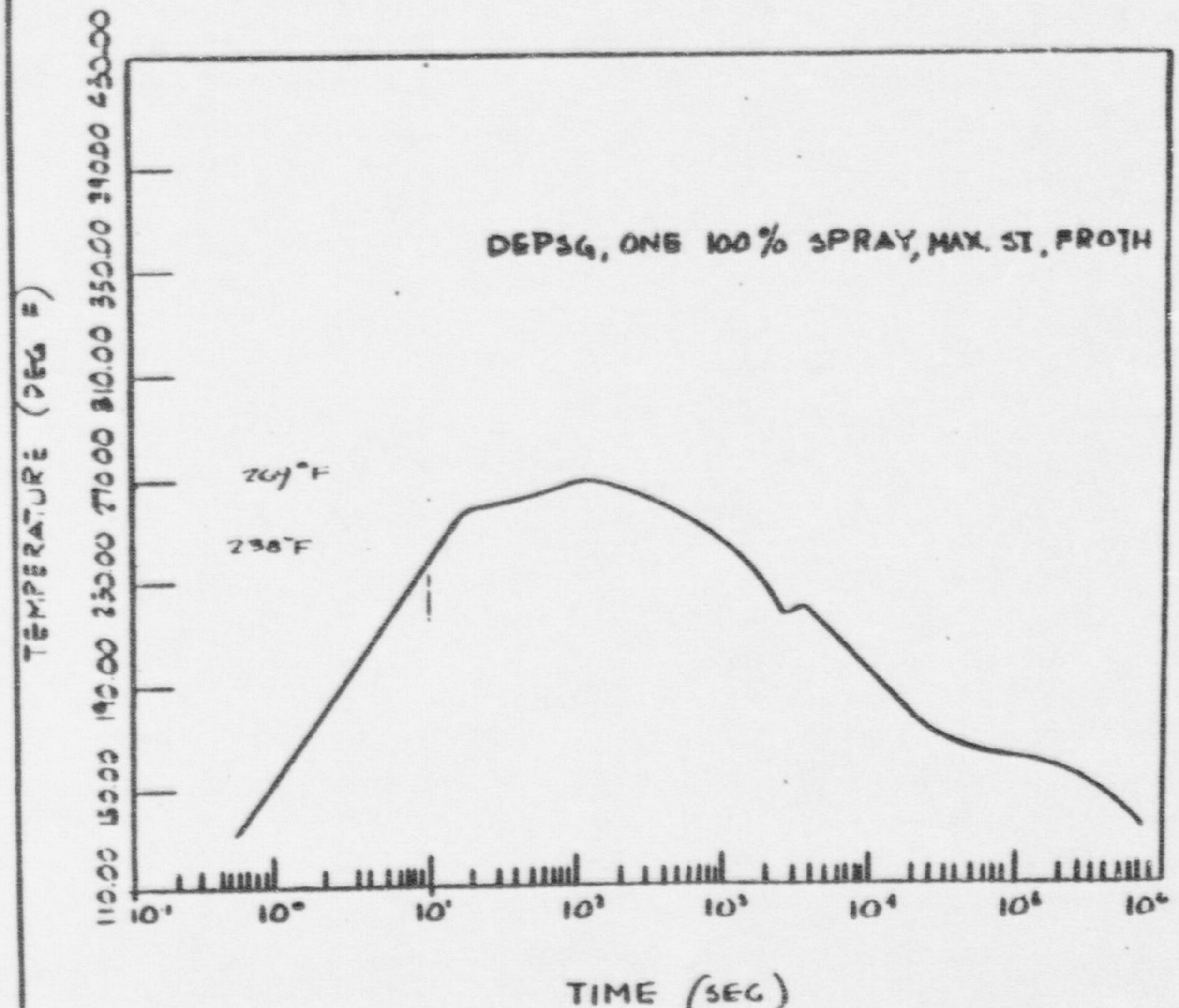

Figure 1

CLIENT TU ELECTRIC	
JOB NO. 0210-041	
CALC/PROB. NO. M-27	
BY: TRA	DATE: 2/5/87
CHKD: RTK	DATE: 7/6/87

COMANCHE PEAK S.E.S.
FINAL SAFETY ANALYSIS REPORT
UNITS 1 and 2

CONTAINMENT VAPOR
TEMPERATURE TRANSIENT -
DEPSG

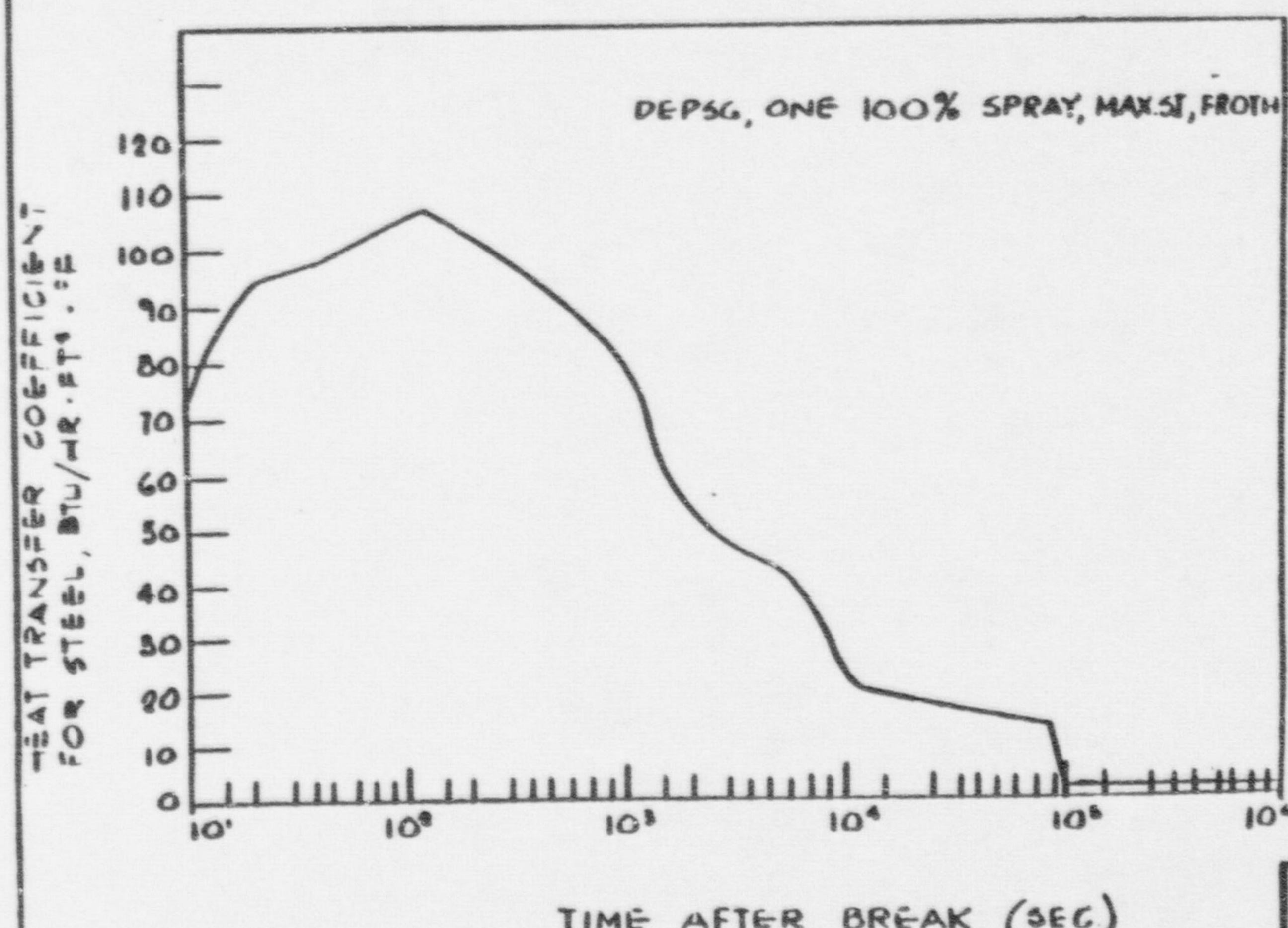

Figure 2

COMANCHE PEAK S E S
FINAL SAFETY ANALYSIS REPORT
UNITS 1 and 2

STRUCTURAL HEAT TRANSFER COEFFICIENT - DEPSG

DESIGN
CLIENT TU ELECTRIC
JOB NO. 0210-041
CALC. NO. M-27
BY [illegible] DATE 2/5/87
CHKD [illegible] DATE [illegible]

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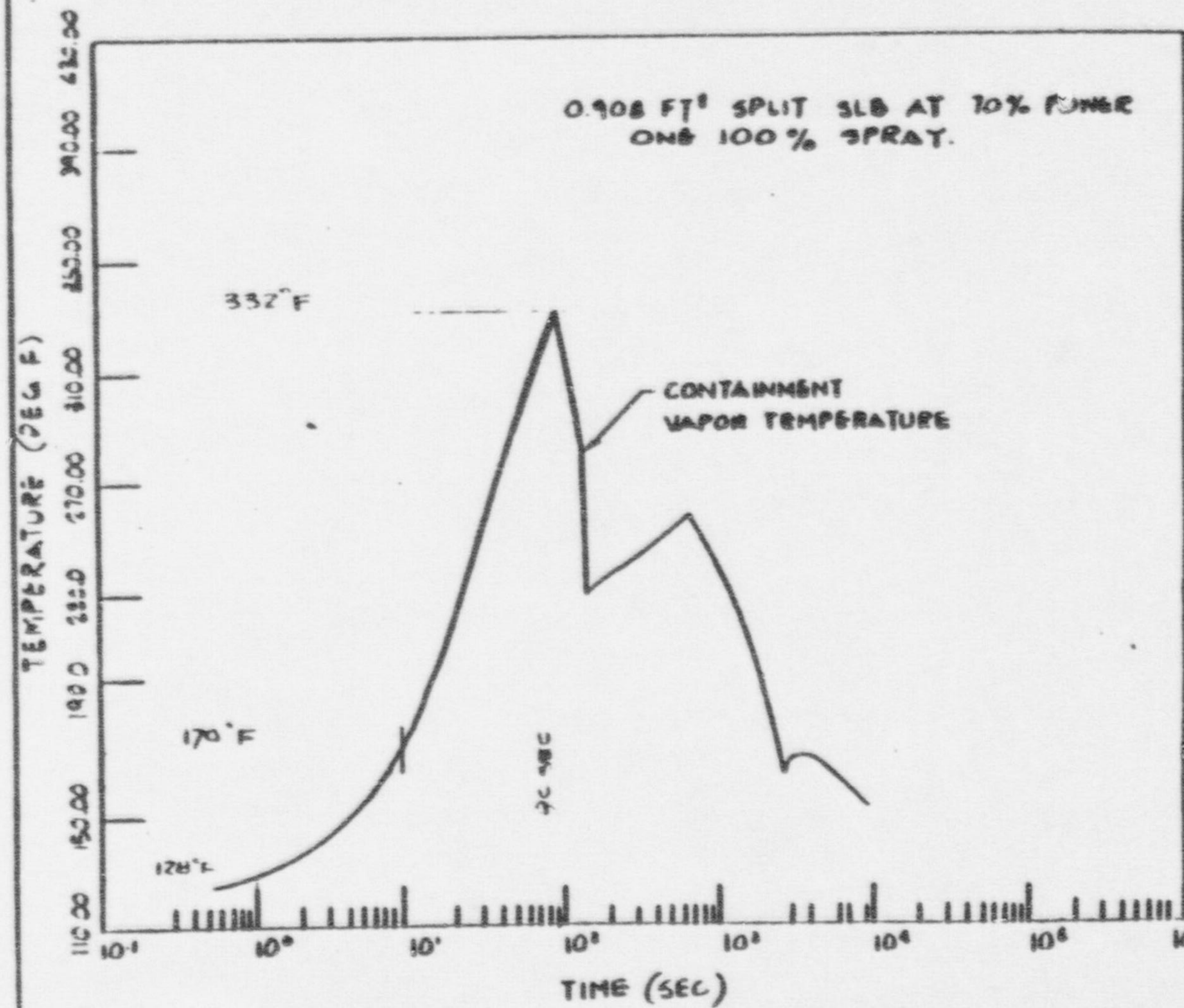


Figure 3

COMANCHE PEAK S E S
FINAL SAFETY ANALYSIS REPORT
UNITS 1 and 2

CONTAINMENT VAPOR
TEMPERATURE TRANSIENT
0.908 FT² SPLIT SLB

Pg 40
of
47

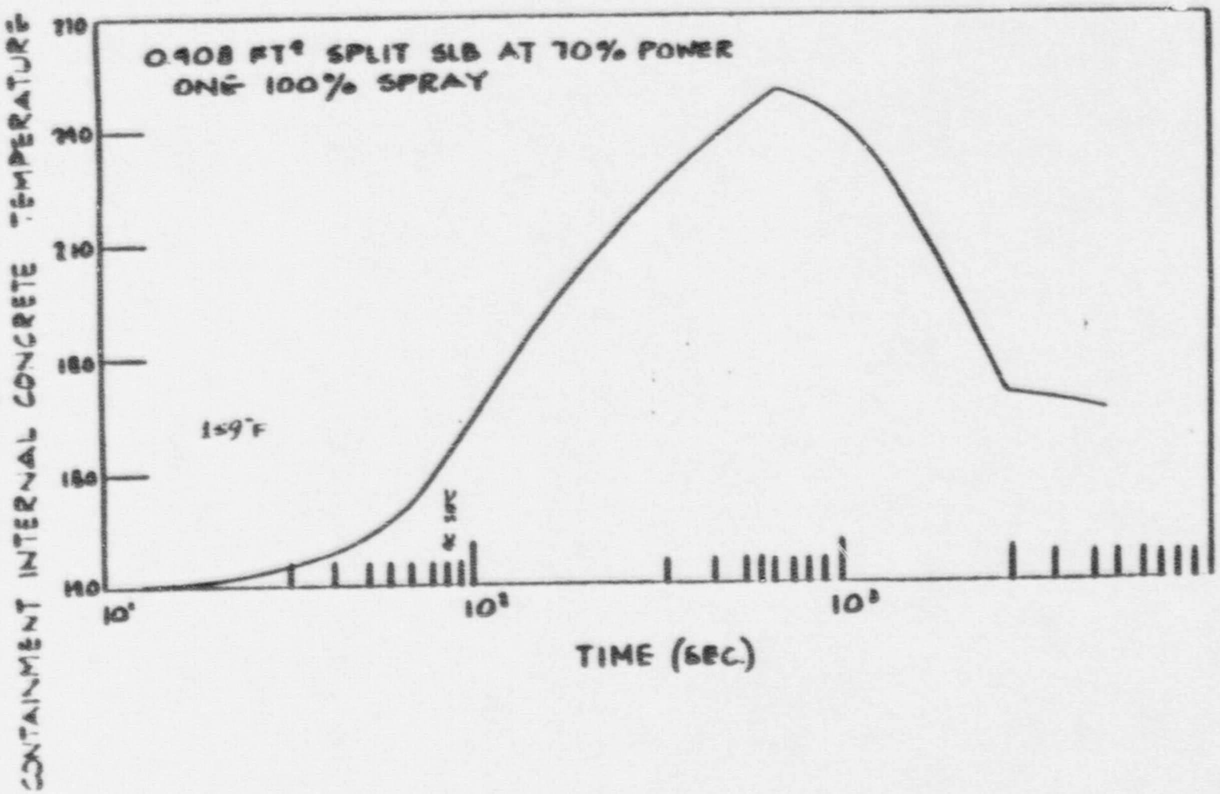
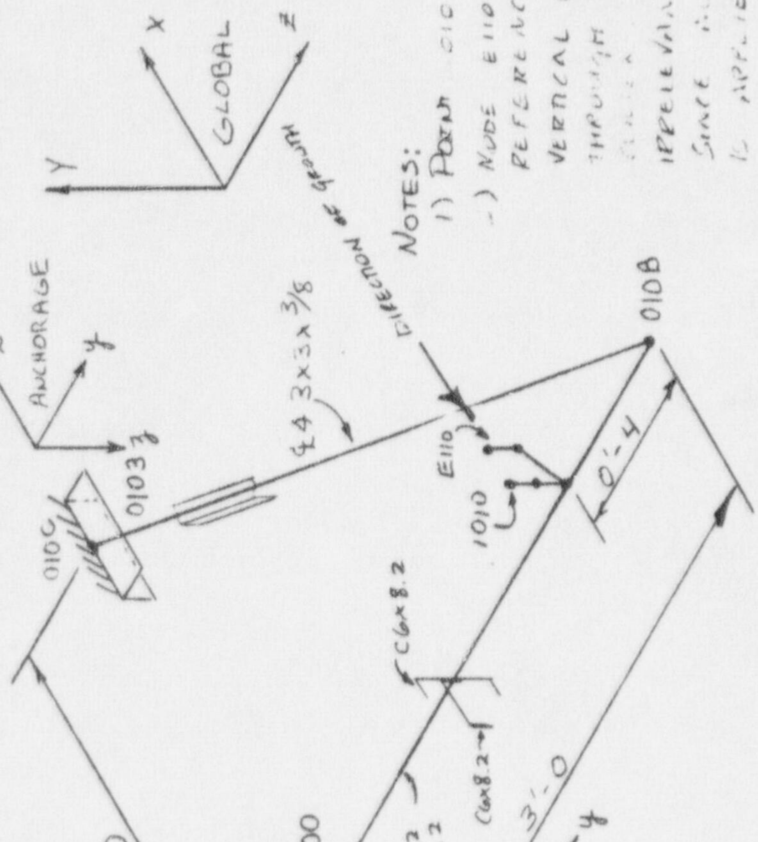


Figure 4

COMANCHE PEAK S.E.S.
FINAL SAFETY ANALYSIS REPORT
UNITS 1 and 2

CONTAINMENT INTERNAL CONCRETE TEMPERATURE TRANSIENT 0.908 FT² SPLIT SLB

DESIGN ...
TU ELECTRIC
0210-041
M-27
2/5/87
2/6/87



NOTES:
1) POINT 010 IS ON TRAY ℄
2) NODE E110 IS SPECIFIED IN REFERENCE 6 TO ALLOW VERTICAL LOAD TRANSFER THROUGH THE CHANNEL ... IRRELEVANT IN THIS CASE SINCE NO VERTICAL LOAD IS APPLIED.

SUPPORT ANALYSIS MODEL

FIGURE 5

2	HAH	5/5/87	DWB	5/5/87	THERMAL LOAD EVALUATION
REV	BY	DATE	CHECKED	DATE	IMPELL CORPORATION

JOB NO 0210-041
CALC NO M-27

1903.22 TUGCO

SAMPLE NO. A-4C-02

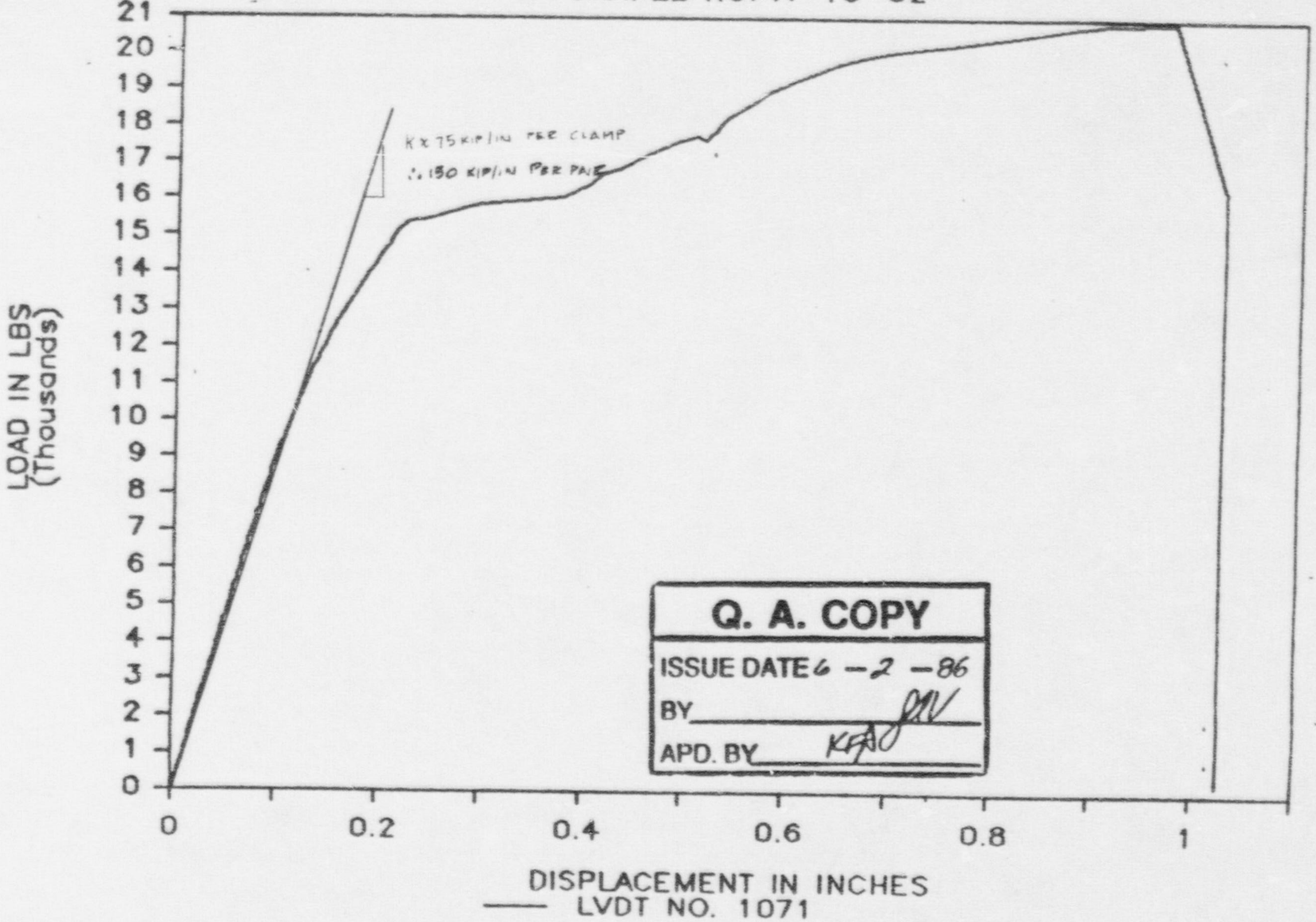
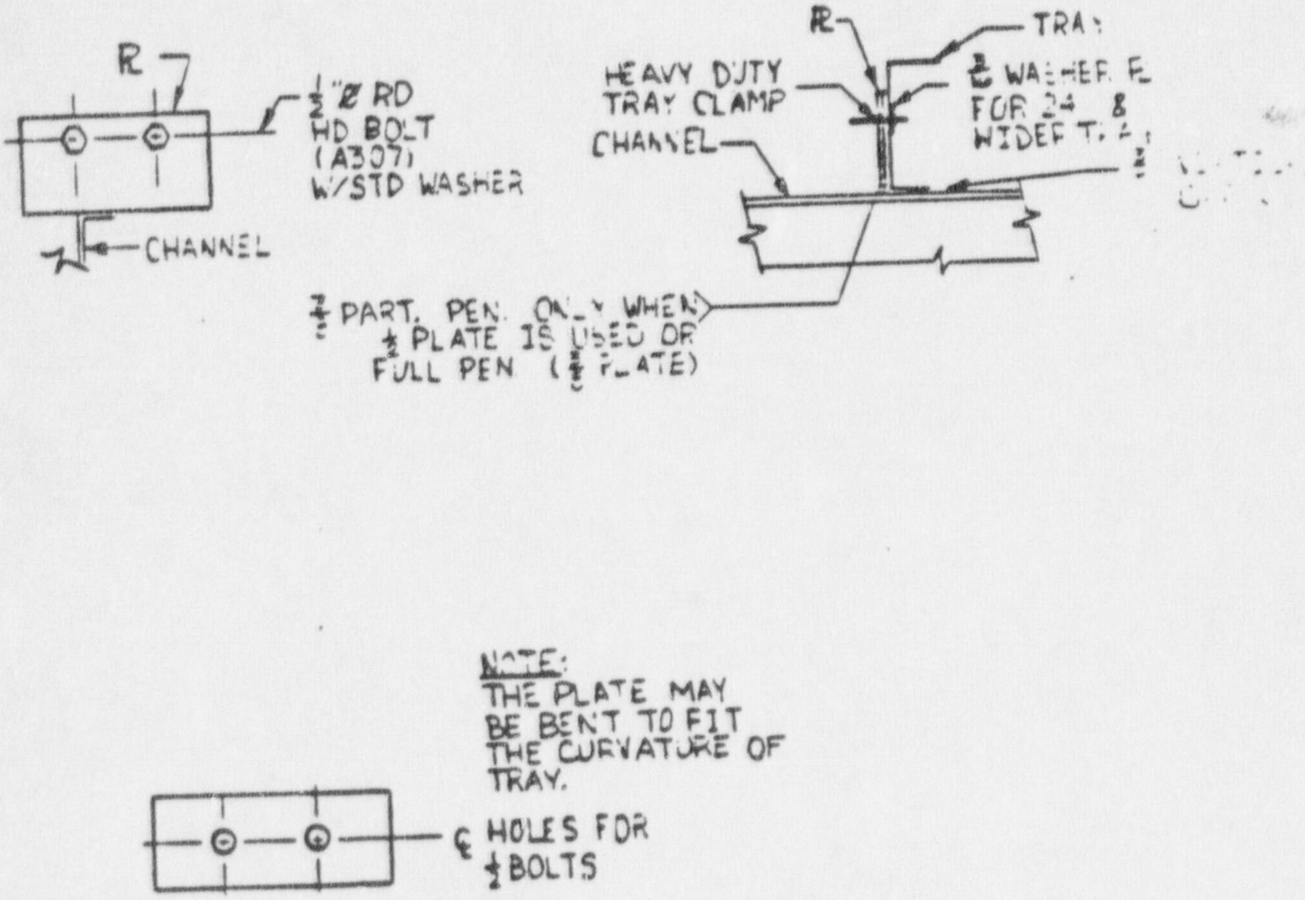


FIGURE 6



3/16 WASHER PLATE
DETAIL

TYPE 'B' HEAVY DUTY CLAMP

FIGURE 7

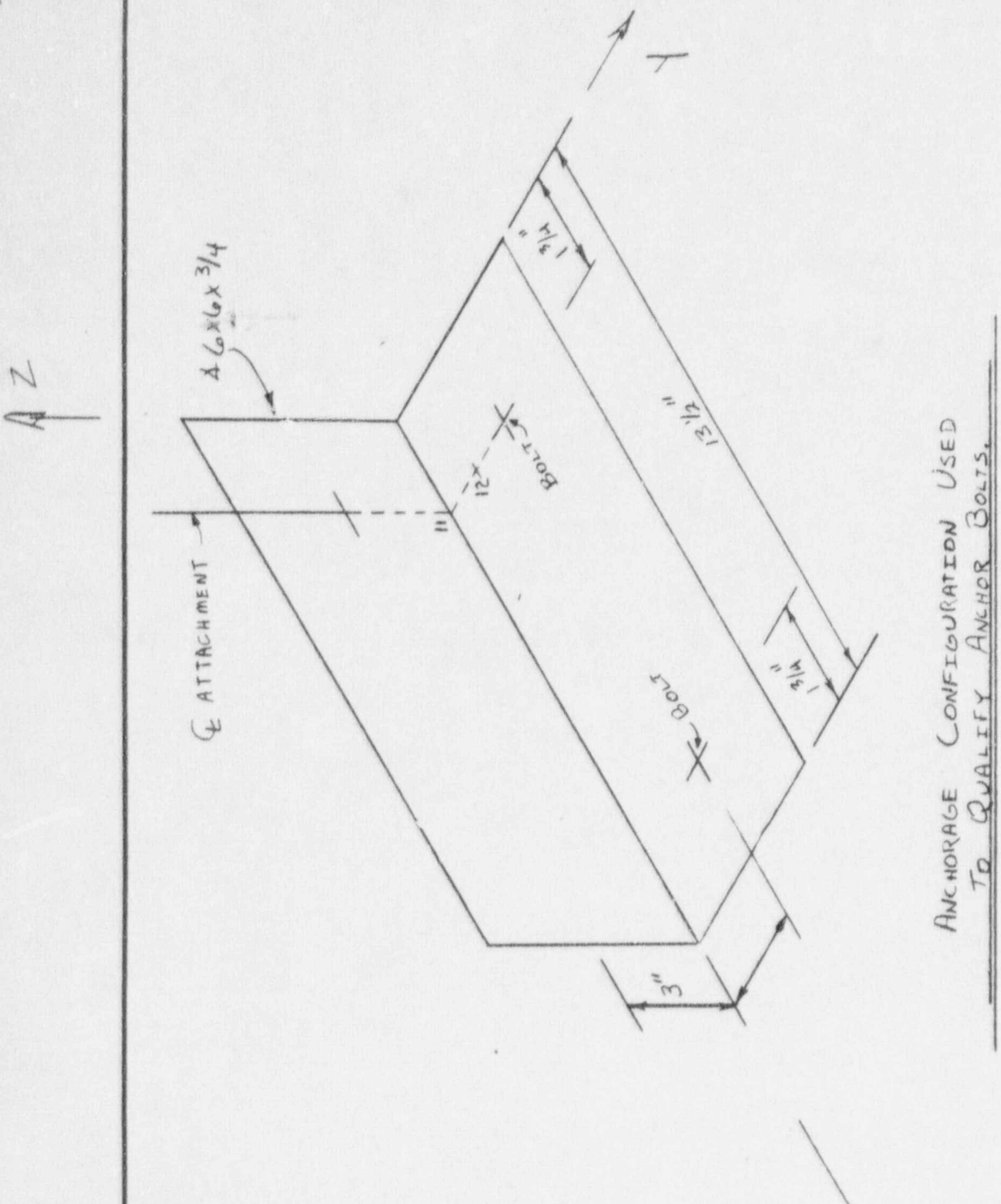


FIGURE 8

					THERMAL LOAD EVALUATION		
						JOB NO 0210-041	PAGE 45
2	TRA	5/5/87	DWS	5/5/87	IMPELL CORPORATION	CALC NO M-27	OF 47
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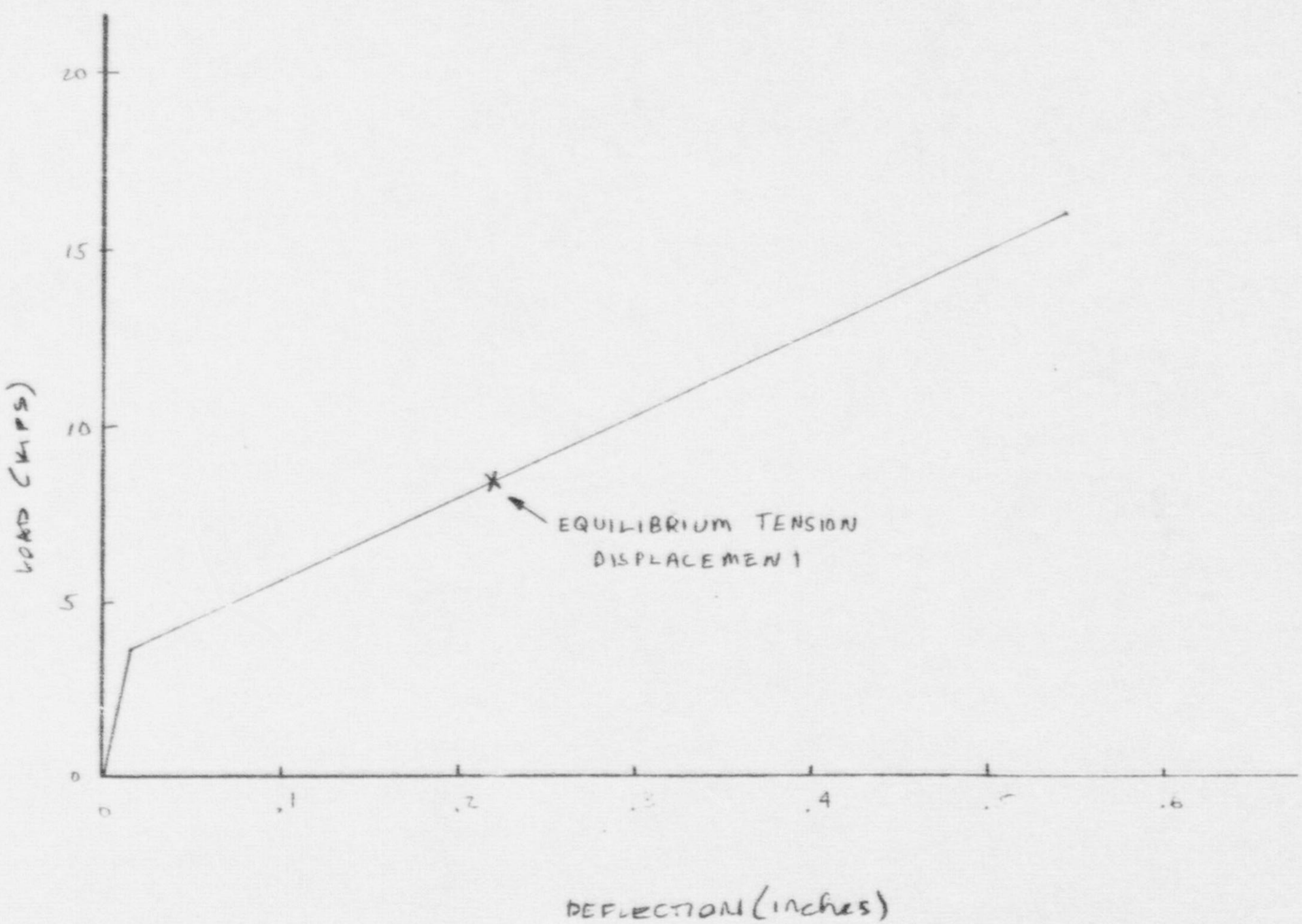
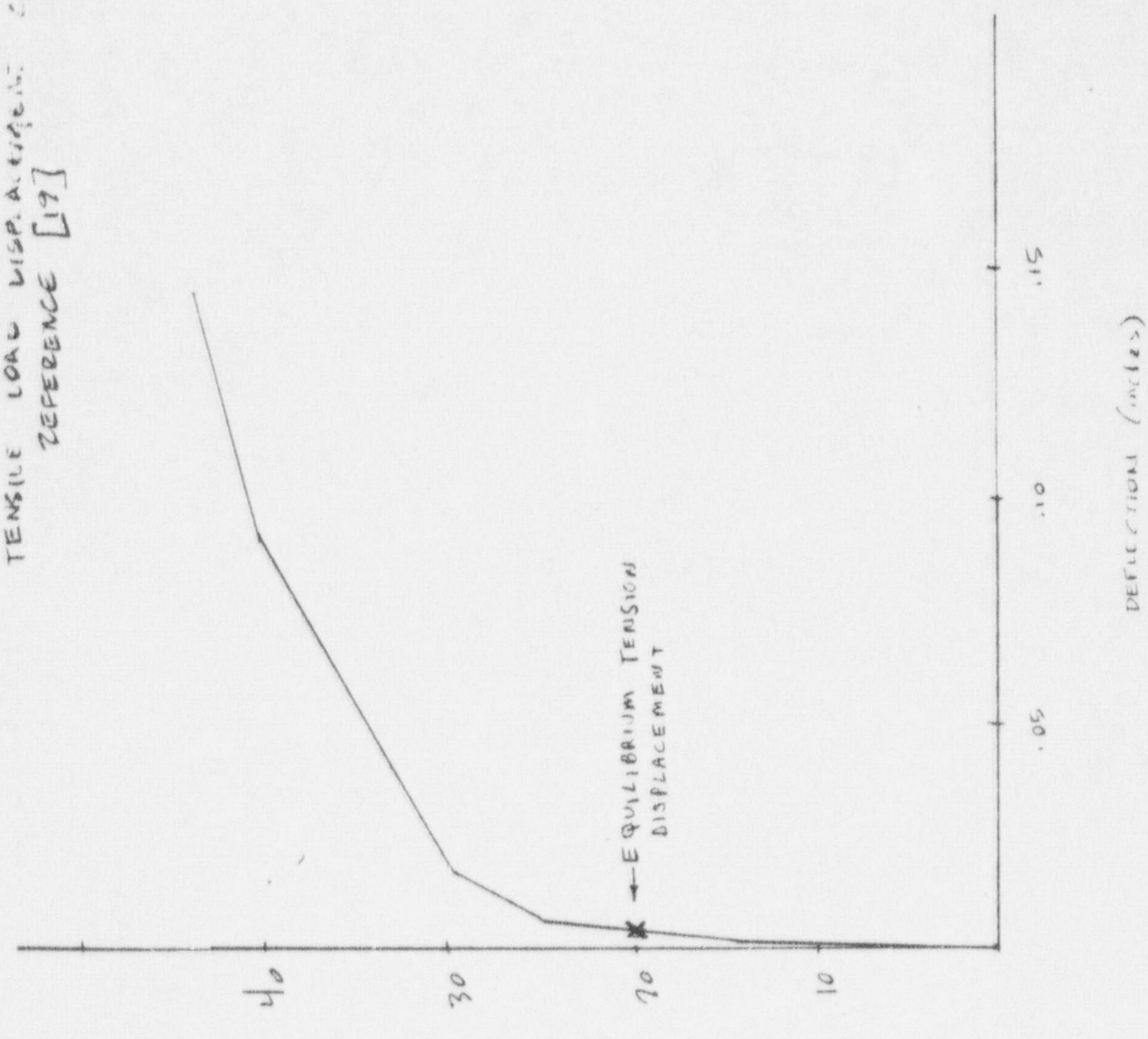
FIGURE 9 1" HILTI KWIK BOLT
TENSILE LOAD DISPLACEMENT CURVE
REFERENCE [1A]

4½" EMBEDMENT



REV	BY	DATE	CHECKED	DATE		
2	TAA	5/5/87	DWS	5/5/87	IMPELL CORPORATION	JOB NO 0210-041 CALC NO M-27	PAGE 46 OF 47

FIGURE 10 1" RICHMOND INSERT
TENSILE LOAD DISPLACEMENT CURVE
REFERENCE [17]



2	TRH	5/5/87	DWS	5/5/87	IMPELL CORPORATION	JOB NO 0210-041 CALC NO M-27	PAGE 47 OF 47
REV	BY	DATE	CHECKED	DATE			