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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

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BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of TEXAS UTILITIES ELECTRIC COMPANY, et al.	)	Docket Nos. 50-445 50-446
(Comanche Peak Steam Electric Station, Units 1 and 2)	)	(Application for Operating Licenses)

TESTIMONY OF NANCY H. WILLIAMS
IN RESPONSE TO CASE QUESTIONS OF
FEB. 22, 1984 TO CYGNA ENERGY SERVICES

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NUCLEAR REGULATORY COMMISSION

Docket No. 50-445&446 Official Exh. No. Cygna April '84 #1
In the matter of Comanche Peak
Staff ________ IDENTIFIED 4/24/84
Applicant ________ RECEIVED 4/24/84
Intervenor ________ REJECTED ________
Cont'g Off'r ________
Contractor ________ DATE 4/24/84
Other Cygna Witness Williams
Reporter Sg

April 12, 1984

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PDR ADOCK 05000445
G PDR

1. Question: Please state your name, current business position.

Answer: I am Nancy H. williams, Project manager, Cygna Energy Services, 101 California Street, Suite 1000, San Francisco, California

2. Question: What is the purpose of the testimony being presented at this time?

Answer: During hearings of February 20 through February 24, 1984 in this proceeding, Board Exhibit No. 1 "Independent Assessment Report," Volumes 1 and 2 were introduced into evidence. During those same hearings I testified in support of the report and was cross-examined by parties to this proceeding. At the conclusion of that set of hearings, it was agreed that intervenor CASE would provide Cygna with its cross-examination questions in writing. Attached hereto as "Attachment 1" is a copy of the written questions submitted to Cygna by CASE.

Subsequent to receipt of "Attachment 1," Cygna formulated its responses and informally circulated those responses to the Board and the parties in a document entitled "Testimony of Nancy H. Williams in Response to CASE Questions of February 22, 1984 to Cygna Energy Services" and dated March 18, 1984. As a result of conferences between Cygna and CASE on March 21, 1984, March 27, 1984 and April 3, 1984, correspondence from CASE, and guidance provided in the Board's "Memorandum (Clarification of Open Items)" dated March 15, 1984 Cygna has reformulated its responses to the questions contained in Attachment 1.

Attached hereto and incorporated herein are copies of Cygna's responses to the CASE questions mentioned in Attachment No. 1.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of TEXAS UTILITIES ELECTRIC COMPANY, et al. (Comanche Peak Steam Electric Station, Units 1 and 2)	Docket Nos. 50-445 50-446 (Application for Operating Licenses)

CERTIFICATE OF SERVICE

I hereby certify that copies of the foregoing Testimony of "Nancy H. Williams in Response to CASE Questions of February 22, 1984 to Cygna Energy Services" in the above-captioned matter were served upon the following persons by overnight delivery (*), or deposit in the United States mail, first class, postage prepaid, this 12th day of April, 1984.

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Donna Oldag

Donna Oldag

From CASE Witness Jack Doyle to CYGNA
Summary of Cross-Examination Questions 2/22/84

BRIEF SUMMARY OF GENERIC PROBLEMS

<u>Omitted from Calculations and Omitted from Checklists</u>

1. Cinched up U-bolts:

   o Not in compliance with Cygna criteria

   o Not in compliance with NRC criteria

   o Stresses of unknown quantity due to pre-stress, thermal and design loads

   o Effects on pipe not shown on calculations

   o Not in compliance with Board Notification.

2. Local effects on tube walls:

   o Punching shear

   o Effect on welds

   o Resultant effect due to wall flexibility on moment at tube weld.

3. Dead weight of structure not included in calculations.

4. Weight of support masses as they affect pipe stress.

5. Inaccurate conclusions as relate to KL/R for pinned columns:

   o If a column fixed at its base and free at the top has an effective K of 2.0 cutting at some point up from the base and adding a pin does not address the problem.

6. 16-inch pipe with about 20 kip load along 3 1/2-inch length induces high bearing stresses which require pads. This is not addressed.

   o ASME Code against flattening.

7. Clip angle 4x4x1/2 which supports U-bolt not addressed (critical to maintaining stability):

   o Section modulus .04 in cube

   o Moment arm at least 2 inches

   o 1100# load exceeds Code allowables.

   o Pre-tensioning to obtain a clamping force required could exceed

this (not including thermal constraint and design loads)

o Clamping force with no margin of safety for single degree system (not point contact or line contact) is force/coefficient of friction or about 4 times what is required for clamping force.

8. There is no documentation in calculations to support the conclusion that flair weld is stronger than fillet weld--no calculations, therefore why did Cygna accept this statement?

   o Flair weld strength depends on radius of flair (depth).

9. The reduction of weld capacity in the calcuation is based on 135°. Actual tangental angle is 150.3°. Therefore, an error exists. Did Cygna take note of this?

   o More stress in weld than stated.

   o Wide/thin ratio induces cracking as well as the 1:4:1 ratio width to depth.

10. Changing from flair weld to fillet weld induces flange bending. Has this been addressed by Cygna?

11. Effects of out-of-plane seismic excitation of support handware not included in calculation. Did Cygna address this point?

    o Additional loads on support

    o Additional loads on pipe

12. Restraint of rotation by the pipe because of coupling effect of hardware on both sides of a pipe:

    o Load increase in 1 of 2 snubbers/struts

    o Alteration of dynamics of pipe system during seismic event

13. In Note 2 following page PS-01-4 of 4, Cygna decided to eliminate their stiffness criteria based on their knowledge that a report existed to address the problem (but without personal knowledge of what was contained

in the document in detail). Why didn't Cygna consult with their experts--for example, Eric von Strijgeren (who was the editor on a paper by T.Y. Chow, C.H. Chen and O. Bilgen)--in reference to deviations from generic stiffnesses in pipe supports and the effects on piping systems.

o Third paragraph introduction et. seq. (CASE Ex. 884)

14. In Note 1, same source, did Cygna consider the additive effects of self weight excitation if the stiffness is considered from node point to hard point as opposed to the stiffness of the frame independent of hardware, local effects, base plate and anchor bolts?

   o Spring rate of base plate/anchor bolts (particularly bearing-type joints) can be considerable (observation of base plate II finite analysis).

15. Was thermal lockup considered for anchors which restrain pipe radial growth?

   o Induces frame moments

16. The base plate analysis is based on distribution of shear relative to load path/stiffness for all bolts in the pattern. Did Cygna address this problem?

   o With oversized holes and the inability to eliminate construction tolerances (location of the bolts combined with localtion of the bolt holes), it is not possible for all of the bolts in the system to be active. (See CASE Exhibit 906).

   o The stiffness of the bolts is such that deflection cannot be counted on as a means to achieve full pattern participation

   o Even if deflection could result in full activity, the first bolts deflecting would receive the larger portion of the load in an ideal symmetrical and systems.

- For non-symmetrical system and systems of variable stiffness, the inactivity of a number of the bolts will alter the accuracy of the computerized analysis.

17. Has Cygna verified the statement: "No 2-inch topping"?
    - This affects the calculations for Hiltis relative to embedment, since a non-monolithic shear plane has been established.
18. The base plate analysis performed without including stiffeners alters the stiffness matrix of the base plate and consequently the distribution of moments and tension to the bolts. Beyond this point, stiffeners remain unqualified. Has Cygna addressed this?

The preceeding questions are the primary areas in which I will be cross-examining Cygna witnesses. (Additional questions may be triggered by Cygna witnesses' answers.)

In addition, CASE has not yet received all of the documents which it requested from Applicants' on the Cygna report. Therefore, additional questions may be triggered from these documents (if and when they are supplied).

From CASE witness Jack Doyle
Summary of Cross-Examination Questions 2/22/84

MATRIX OF EXHIBITS AND DOCUMENTS

CASE Exhibit	Concerns
891	1, 3, 4, 5, 6, 7, 11, 13, 14, 15, 16
892	9, minor question relative to pad width diameter + $(Rt)^{1/2}$
893	8, 10, 14
894	1, 4, 5, 11, 14
895	14, 16
896	12, 14
897	1, 2, 3, 4, 5, 11, 14, 16
898	14, 15, 16, 18
899	14, 15, 16, 18
900	14, 15, 16
901	Has minimum weld violation (walk-down)
902	Has support completely rebuilt on CMC and then calculated

This matrix has been compiled to the best of our ability due to time constraints. (It is from notes, etc.)

CYGNA	
JOB NO:	84042
DATE LOGGED:	3/4/84
LOG NO.:	#20
FILE:	15.3 Gen. Info ASLB
CROSS REF. FILE	15.3 Gen. Info Leg/ASLB

BRIEF SUMMARY OF CROSS-EXAMINATION QUESTIONS BY CASE WITNESS MARK WALSH TO CYGNA

1. Appendix E of Cygna Report

Section DC-2.4.4. What was the yield point used for A500 Grade B tube steel?

2. Observation Record PS-02-01: The Applicants did not consider shear cone interaction of adjacent bolts.

3. PI-01-01. There has been no detailed computer analysis performed to consider the concentrated loads (valves, etc.) and their effect on dead weight and seismic. Also, the seismic analysis will not be linerally proportional.

4. PI-02. Is there an error in the table shown?

5. CTS-00-03: See CASE Exhibit 889, sheet 129. F_{bx} = should be 21.2, not 23.2 or 22. The length is 6' not 5.5'.

See CASE Exhibit 890: 1) Why was only 1/2 SEE considered? 2) Why was 4% damping used; not consistent with FSAR? 3) Assumed cable tray was rigid when lumping the mass; this resulted in not combining the dynamic effects of the cable tray itself to the support; did not include effect on welds. 4) The validity that the cable trays have the capacity to transfer a load around a corner when one run of cable tray has no axial restraint, as shown on drawing 2323 EI-0601-01. (NOTE: We only have a 36"x48" drawing; please let us know when you want to look at it.). 5) What documentation did Cygna see that justified the hangers' receiving a lateral load around corners that resist the axial load from the tray segment that contains no axial restraints;

how did Cygna evaluate it? It appears the axial loa. has not been taken into account. 6) CASE Exhibit 902. Did not consider base plate flexibility.

6. CTS-00-05: In the description, it discusses a channel bent about its weak axis. The resolution does not consider this problem nor does the document CASE requested on discovery; see CASE Exhibit 907. On CMC 88306, are the originator and approver the same person?

7. CTS-00-06. What is the "significant design margin" as shown in the resolution?

8. CTS-00-07: The analysis that included the beam element did not consider prying action and the flexibility of the base plate to determine the center of compression.

9. WD-03-01: What documentation was there that "accept as is" was valid? Were there calculations to support this?

10. WD-07-02: What documentation did Cygna see that showed the temperature indicator would be installed at a later date?

11. Pipe stress checklist, note 3, item a: 1) What is the basis for considering that the effects were negligible? 2) What pipe stress run did Cygna look at, since the inclined load was used in the design of support RH-1-010-003-S22R?

WALSH TO CYGNA (3)

12. Cable Tray Check List: CTS-11, Item 6, problem 4. This was not discussed in CTS-00-07.

The preceeding questions are the primary areas in which I will be cross-examining Cygna witnesses. (Additional questions may be triggered by Cygna witnesses' answers.)

In addition, CASE has not yet received all of the documents which it requested from Applicants' on the Cygna report. Therefore, additional questions may be triggered from these documents (if and when they are supplied).

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #1
Exhibit No.: 891, 894, 897

1.0 CASE Question

Cinched up U-bolts:

- o Not in compliance with Cygna criteria
- o Not in compliance with NRC criteria
- o Stresses of unknown quantity due to pre-stress, thermal and design loads
- o Effects on pipe not shown on calculations
- o Not in compliance with Board Notification

2.0 Cygna Interpretation

N/A

3.0 Response

Section 4.1.2 of the Cygna review criteria document, DC-2, states the following:

"A gap shall be provided to accommodate radial expansion and construction tolerances. The maximum total gap allowed in the restrained direction is 1/8". In unrestrained directions, the support design shall allow clearances for the most severe thermal plus seismic movements of the pipe. Proper installation tolerances shall be provided where thermal movement cannot be accommodated within the specified gap minus 1/16"."

This criteria is intended to apply only to pipe supports which do not require physical contact with the pipe to insure that the require restraining forces are developed. Supports which require physical contact with the pipe in order to develop the proper restraining forces, such as pipe clamps and cinched U-bolts, cannot have gaps and therefore are not required to satisfy the conditions of DC-2, Section 4.1.2.

The NRC Information Notice No. 83-80 identifies potential significant problems that may exist with the usage of specialized "stiff" clamps. Under certain conditions, these clamps may induce high local stresses in the pipe. Cygna did not encounter any "stiff" clamps during the Cygna IAP review.

As defined in the Independent Assessment Program Plan, review of the RHR System included design criteria, analyses, design and drawings. It did not include installation specifications, where torqueing requirements such as cinching, would normally be defined. Cinching was not required or defined in any of the documents reviewed by Cygna. Accordingly, cinching loads were not known and were not considered in the design assessment.

Loads on the pipe due to cinching were not assessed for the reasons discussed above. Pipe loads due to the zero gap were judged to be negligible. The conclusions in the IAP Draft Report are based on that engineering judgment.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #2
Exhibit No.: 897

1.0 CASE Question

Local effects on tube walls:

- o Punching shear
- o Effect on welds
- o Resultant effect due to wall flexibility on moment at tube weld

2.0 Cygna Interpretation

When tube sections are employed in the design of pipe supports, how were the following local effects considered:

a. Punching shear?

b. Effect on welds?

c. Resultant effect due to wall flexibility on moment at tube weld?

3.0 Response

Pipe support RH-1-062-002-S22R (CASE Exhibit 897) is designed using a tube section, TS 4" x 6" x 1/2", welded to a baseplate at one end and to a strut clevis at the other end. Punching shear and welding stresses are discussed below:

a. Punching shear stresses are within allowable for all supports reviewed by Cygna. This is evidenced by the punching shear check provided in Attachment D2-1.

Adequacy with respect to punching shear can also be determined by inspection through a simple comparison of fillet weld size and tube wall thickness. The basic relationship for this comparison is established by considering a unit length of weld and tube wall as a freebody and equating the allowable force in the weld to the allowable shear force through the thickness of the tube wall.

The allowable force in the weld is

$$P_w = F_w * 1 * .707 * t_w$$

The allowable shear force in the plate is

$$P_c = F_c * l * t_c$$

where t_c = tube wall thickness, inches
F_c = allowable weld shear stress, ksi (use 18 ksi)
t_w = fillet weld leg size, inches
F_w = allowable tube shear stress, ksi (use 0.4 * 31 ksi)

By equating P_w to P_c and substituting the proper values for allowable stress, the following relationship is established:

$$t_c = (18 * .707 * t_w)/(0.4 * 31)$$
$$t_c = 1.0\ t_w$$

Therefore, assuming the fillet weld is properly sized, if the tube wall thickness is equal to or greater than the fillet weld size, punching shear stresses in the tube wall will be satisfactory. For support RH-1-064-S22R, the tube thickness (1/2") is twice the attached fillet weld (1/4").

b. and c. Each welded connection in support RH-1-064-011-S22R is discussed below:

<u>Tube-to-Baseplate</u>

This connection is a standard beam-to-column detail, as evidenced by the AISC Manual, Part 4. Furthermore, the flare-bevel weld detail has been properly evaluated and sized by the designer.

<u>Tube-to-Clevis</u>

Attaching the strut clevis to the tube flange introduces no adverse effects into the connecting fillet weld.

The flexibility of the tube wall produces no significant additional loads on the weld. This welded connection compares favorably with certain standard weldments shown in Blodgett's Design of Welded Structures (see Attachments D2-1; D2-2 Figure 9; and D2-3, Figure 12). The connections shown in these attachments are more "flexible" than the tube-to-clevis detail in support RH-1-064-S22R, and are not evaluated for added weld stresses due to diaphragm action or plate flexibility.

c. AWS Section 10.5 specifically addresses stepped tube connections and the evaluation of tube wall capacity for the case where the connecting tube transmits both axial and bending loads to the tube wall. The design equation (Section 10.5.1) used in the evaluation is a function of both the ratio of the tube widths (Beta) and the tube wall thickness. It seems implicit that by satisfying the design equation the local stresses within the tube wall are within acceptable limits at the design load.

In addition it should be emphasized that the Beta parameter alone is not sufficient to evaluate the serviceability or strength of stepped tube connections. The Beta parameter must be considered in conjunction with the tube wall thickness. For example, a connection having a Beta = 0.4 will possess approximately the same ultimate moment capacity and punching shear capacity (as well as the same moment-rotation and axial load-deflection characteristics) as a connection having a Beta = 0.8, if the connection with Beta = 0.4 has a wall thickness one-third greater than the wall thickness of the connection with Beta = 0.8 (see Korol & Mirza paper, ASCE, Journal of the Structural Division, September 1982, Figures 7, 8, 11 and 12 and Tables 2 and 3). Thus, a tube (or clevis) welded to a 3/8"-thick tube for which Beta = 0.8 will behave approximately the same with respect to deflection, rotation, punching shear and ultimate moment as when the tube (or clevis) is welded to a 1/2"-thick tube wall for which Beta = 0.4.

Punching shear check for Support No. RH-1-062-002-S22R.

Reference: American Welding Society (AWS), D1.1, Section 10.5.

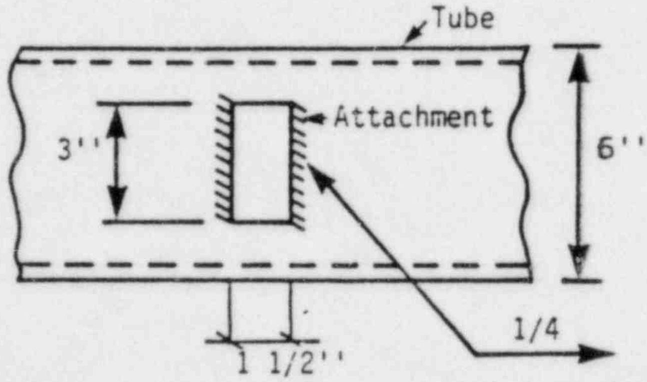


FIGURE D2-1

APPLIED AXIAL LOAD = 5092 LBS.

Since the attachment is not a tube and only welded on the 3" side, the calculation of f_a in the following equation for Acting V_p (AWS Section 10.5.1) will be conservatively high, because the loads shared by the 1-1/2" sides of the tube are being neglected.

$$\text{Acting } V_p = \tau \left(\frac{f_a \sin \theta}{K_a} + \frac{f_b}{K_b}\right)$$

where $f_a = 5092/(3+3)\ t_b = 849/t_b$

$f_b = 0$

$\theta = 90$ degrees

$\tau = t_b/t_c$

$B = b/D$

$K_a = 1.0$

Acting $V_p = 1698$

Basic $V_p = F_y/(0.6\gamma)$ where $\gamma = D/2t_c$

$= 31350/(0.6)(6)$ $= 6/2(1/2) = 6$

$U = (f_a + f_b)/0.6\ F_y$ (see Note 1, Table 10.5.1)

$f_a = 849/t_b = 849/(1/4) = 3395$ psi

(Note: f_a is conservatively calculated using t_b of 1/4", i.e., the weld size).

$U = (3395 + 0)/(.6)(31350) = 0.18$

Since U is less than 0.44, $Q_f = 1.0$; and, since beta (0.5) is less than 0.6, $Q_b = 1.0$.

Allowable $V_p = Q_b Q_f$ (Basic V_p)

$= (1)(1)(8708$ psi$) = 8708$ psi

CYGNA

This is considerably greater than the Acting A_p = 1698 psi.

Design margin = (8708/1698) - 1 = 4.12 = 412%

OK.

of 38.4 kips for a weld size of $\omega = \frac{3}{16}''$ and angle length of $L_v = 10''$ slightly exceeds the reaction. The corresponding (Field) Weld B, using $\omega = \frac{1}{4}''$, also is satisfactory. Since the beam's required web thickness is 0.21" while the actual web thickness is 0.25", the indicated 3" x 3" x $\frac{5}{16}''$ is all right.

If the beam is made of A36 steel, this connection's capacity will be reduced in the ratio of 0.25/0.29 of actual to required web thickness. The resulting capacity of 33.1 kips is less than the reaction. The next larger connection with apparently sufficient capacity shows that (Shop) Weld A's capacity is 47 kips, using same angle section but an angle length of $L_v = 12''$. Applying the multiplier of 0.25/0.29 reduces the capacity of the connection to 40.5 kips, which exceeds the end reaction.

5. SINGLE-PLATE OR TEE CONNECTION ON BEAM WEB

In the previous design of the field weld, connecting a pair of web framing angles to the supporting column or girder, it was assumed that the reaction (R) applied eccentric to each angle, resulted in a tendency for the angles to twist or rotate. In doing so, they would press together at the top and swing away from each other at the bottom, this being resisted by the welds. These forces are in addition to the vertical forces caused by the reaction (R); see Figure 10.

However, in both the single-plate web connection and the Tee-section type, this portion of the connection welded to the column is solid. Thus, there is no tendency for this spreading action which must be resisted by the welds. These vertical field welds to the column would be designed then for just the vertical reaction (R); see Figure 11.

In the shop weld of the single plate to the web of the beam, Figure 12, this double vertical weld would be designed for just the vertical reaction (R). There is not enough eccentricity to consider any bending action.

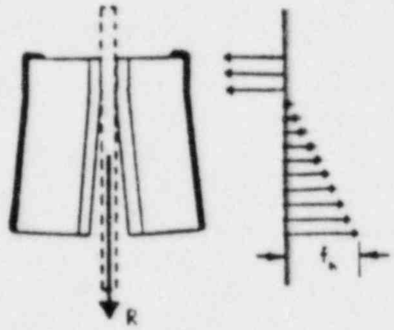


FIG. 10—Double-web framing angle.

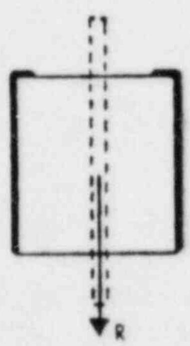


FIG. 11—Single plate or Tee.

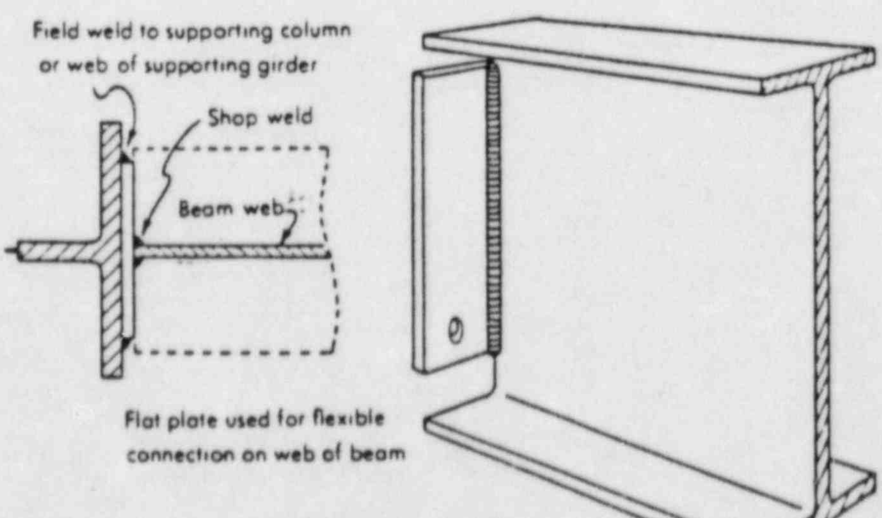


FIG. 12—Flat plate used for flexible connection on web of beam.

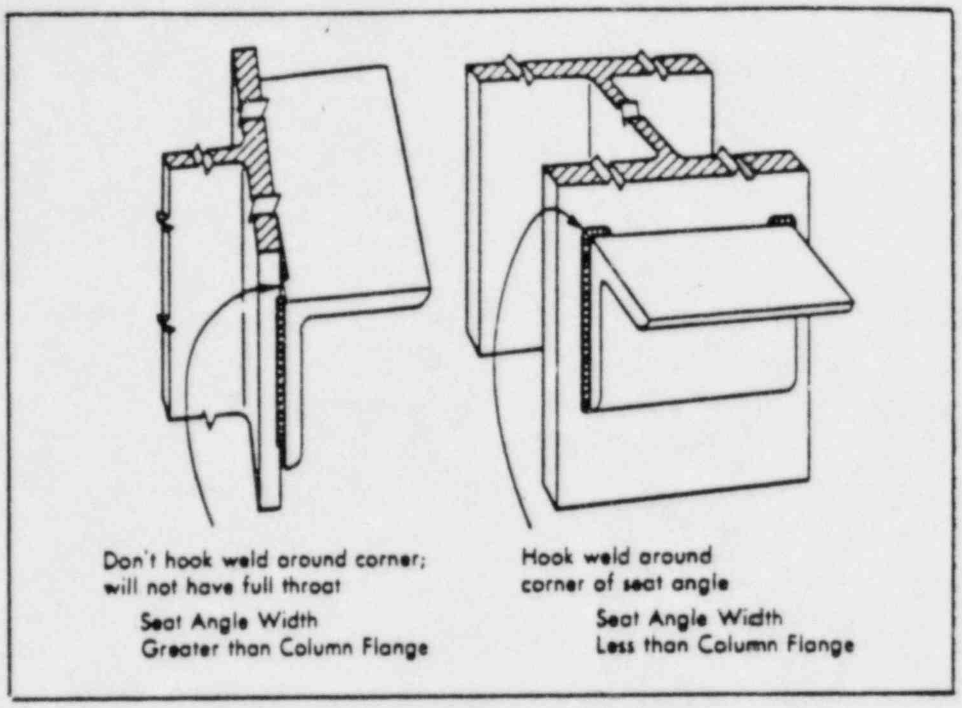


FIGURE 8

5. HORIZONTAL STABILITY

A flexible top angle is usually used to give sufficient horizontal stability to the beam. It is not assumed to carry any of the beam reaction. The most common is a 4" x 4" x ¼" angle, which will not restrain the beam end from rotating under load. After the beam is erected, this top angle is field welded only along its two toes. For beam flanges 4" and less in width, the top angle is usually cut 4" long, for beam flanges over 4" in width, the angle is usually cut 6" long.

In straight tension tests of top connecting angles at Lehigh University, the 4" x 4" x ¼" angle pulled out as much as 1.98" before failure, which is about 20 times greater than usually required under normal load conditions.

Notice in the following figure, that the greatest movement or rotation occurs in the fillet weld connecting the upper leg of the angle to the column. It is important that this weld be made full size.

This test also indicated that a return of the fillet weld around the ends of the angle at the column equal to about ¼ of the leg length resulted in the greatest strength and movement before failure.

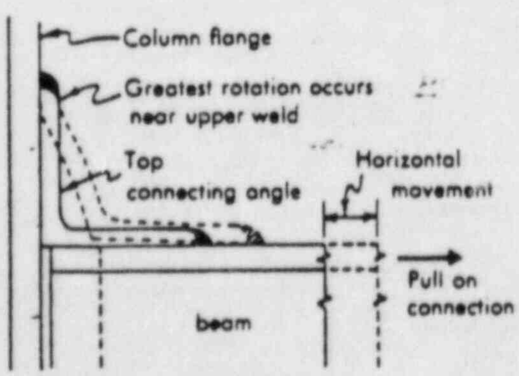


FIGURE 9

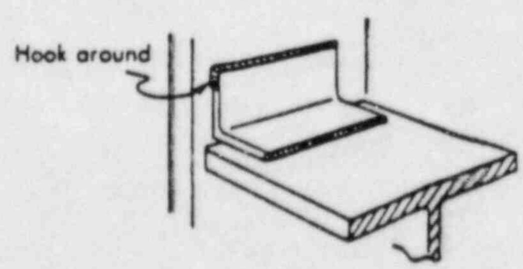


FIGURE 10

Problem 1

Design a flexible seat angle to support a 12" WF 27# beam, having an end reaction of R = 30 kips. Use A36 steel, E70 welds.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #3
Exhibit No.: 891, 897

1.0 CASE Question

Dead weight of structure not included in calculations.

2.0 Cygna Interpretation

N/A

3.0 Response

General purpose structural design codes specify that dead load shall be considered in the design of structures. The significance of the various components of dead load in the design of a structure varies with the type of structure. In the case of a piping system, dead load is considered in the design of pipe supports. The dead load included in the design of a pipe support consists of the piping dead weight and the weight of all material attached to or integral with the piping, such as insulation, valves, etc. Since the dead weight of the pipe support itself is generally very small compared to the piping dead load, thermal load and seismic load for which the support is designed, it can usually be neglected. Cygna believes that neglecting this specific component of dead load (i.e., support dead weight), except in the case of very unusual supports, is consistent with industry practice.

With respect to the specific supports cited, the total dead weight of the support in CASE Exhibit 891 and 897 is 715 lbs and 82 lbs, respectively. This amounts to 4% and 2% of the design load for these supports. These percentages will be even smaller when compared to the support capacities.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #4
Exhibit No.: 891, 894, 897

1.0 CASE Question

Weight of support masses as they affect pipe stress.

2.0 Cygna Interpretation

What is the effect of support weights (masses) on the pipe stress analysis?

3.0 Response

Standard industry practice is not to include support masses in the analysis of pipe stresses. This practice, which was employed on the RHR system Train B at Comanche Peak, produces satisfactory results for the following reasons: 1) support weights are relatively small; 2) support stiffnesses are relatively high; 3) support damping is typically higher than piping system damping; 4) standard analysis techniques are structured to envelope minor variations such as those associated with support masses.

The importance of each item is discussed in detail below. In order to help place these discussions in perspective, the following basic equation of motion may be useful.

$$M\ddot{x} + C\dot{x} + Kx = -Ug \qquad (1)$$

where

M	= mass
C	= damping
K	= stiffness
$\ddot{x}$	= acceleration
$\dot{x}$	= velocity
x	= displacement
$-Ug$	= input motion

Equation (1) describes the response of a system (left hand side of the equation) to a particular input motion. If the input motion is set to zero and system damping is small, the response tendencies of the system can be calculated as,

$$f = \frac{1}{2\pi} \sqrt{\frac{K}{M}} \qquad (2)$$

where

f = fundamental system response (frequency)

π = 3.1416

Equation (2) links the system response (f) to basic system characteristics expressed in terms of stiffness and mass. From this equation, it can be seen that an increase in stiffness will tend to increase the frequency, while an increase in mass will decrease the frequency.

Standard response spectrum techniques are founded on Equations (1) and (2), such that the system response can be directly related to accelerations plotted on a response spectrum. Damping effects are normally included in this process by developing sets of response spectra for various standard damping values.

Relative Support Weights

Except for particularly unbalanced and massive support configurations, which were not observed in the RHR reviewed by Cygna at Comanche Peak, support masses are small relative to the piping system masses that drive the overall response.

In order to test this effect on Comanche Peak, Cygna performed an analysis of a segment of piping within our scope of review, using the ANSYS code. As illustrated in Attachment D4-1, the main piping from the RHR pump to the heat exchanger was studied. Branch lines, including the safety injection lines, were omitted to make the model more manageable for this test. Basically, the test model contains about 95 feet of main piping

with 16 supports. This model was analyzed with and without support masses using the same standard analysis techniques employed on Comanche Peak. The only difference between the two analyses was support masses, which are listed below:

Support Masses

Support Number	Weight (lbs)
RH-1-010-003	72
RH-1-010-004	42
RH-1-010-002	11
RH-1-010-001	87
RH-1-064-010	41
RH-1-064-004	77
RH-1-064-011	25
RH-1-064-003	15
RH-1-064-005	26
RH-1-064-009	24
RH-1-064-002	27
RH-1-064-006	50
RH-1-064-007	56
RH-1-064-008	122
RH-1-064-001	31
RH-1-010-005	30

The results of this test are contained in Attachments D4-2 (calculation package), D4-3 (computer output without support masses), and D4-4 (computer output with support masses). A summary of the system frequencies and pipe stresses at the most massive support (RHR-1-064-008) is provided below:

Frequencies

Mode No.	Without Support Mass (hertz)	With Support Mass (hertz)	Difference (%)
1	7.7	7.6	1.1 decrease
2	10.3	10.1	1.9 decrease
3	12.3	11.4	7.5 decrease
4	20.0	19.4	2.9 decrease
5	22.1	21.9	1.0 decrease
6	23.2	22.9	1.3 decrease
7	28.2	27.6	2.3 decrease
8	33.0	31.4	5.0 decrease

Pipe Stress (at Support RH-1-064-008)

Stress	Without(1) Support Mass (hertz)	With(2) Support Mass (hertz)	Difference (%)
σ_1	2.47	2.44	1.4 decrease
σ_3	-2.32	-2.29	1.4 decrease
SI	2.50	2.45	1.4 decrease
σ_E	2.48	2.45	1.4 decrease

where

σ_1 = maximum principal stress

σ_3 = minimum principal stress

SI = stress intensity = maximum of $\sigma_1-\sigma_2$, $\sigma_2-\sigma_3$, $\sigma_3-\sigma_1$

σ_E = equivalent stress = $\frac{1}{\sqrt{2}}\left[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right]^{\frac{1}{2}}$

(1) From computer output dated 4/10/84 @ 10:29 for element 11, node 4

(2) From computer output dated 4/10/84 @ 10:21 for element 13, node 14.

Interpreting these results, it can be seen that the added mass results in only a minor decrease in system frequencies. Pipe stresses actually decrease slightly, but the changes are negligible.

Pipe Stiffness

Pipe supports are normally designed to be rigid in their support direction relative to the attached piping. This design method tends to uncouple support response from overall system response.

In the off (non-support) direction, the support stiffness normally has no effect. Unless gaps are provided to uncouple the support mass in the off-direction, the mass will participate with the piping. The effect of this interaction has already been shown to be negligible in the "Relative Support Weights" discussion.

Support Damping

Damping directly associated with pipe supports is not considered on Comanche Peak. However, if support masses and stiffnesses are included in the analysis, then support damping should also be included.

As shown in Equation (1), the acceleration and displacement terms will tend to decrease for a given input motion as damping (velocity term) increases. USNRC Regulatory Guide 1.61 recommends damping values up to 7% for structures and 3% for piping under SSE loadings. Therefore, if support response is a significant contributor to overall system response, then the overall system damping will fall somewhere between the individual damping values for piping and supports.

Standard Analysis Techniques

There are many conservatisms built into the standard analysis techniques that are intended to simplify the analysis and focus on the most significant mechanisms. A few of these are briefly discussed below:

a. Low System Damping

Researchers have shown that piping systems exhibit damping values greater than those allowed by USNRC Regulatory Guide 1.61. For example, the Pressure Vessel Research Council (PVRC) has proposed the damping values shown in Attachment D4-5.

b. Modal Response Method

This method combines individual system responses (modes) without regard for direction (or signs). For example, even though responses may be either left or right, this technique assumes that all responses act to the right. A more refined analysis would circumvent this combination technique, but the costs are not practical for production analyses.

c. Spectra Broadening

Motions input to the piping analyses in the form of response spectra contain two significant conservatisms: (1) the rough (saw-toothed) spectra are broadened, usually ± 15%, and (2) the rough shape is enveloped by a smooth curve.

d. Ground Spectra

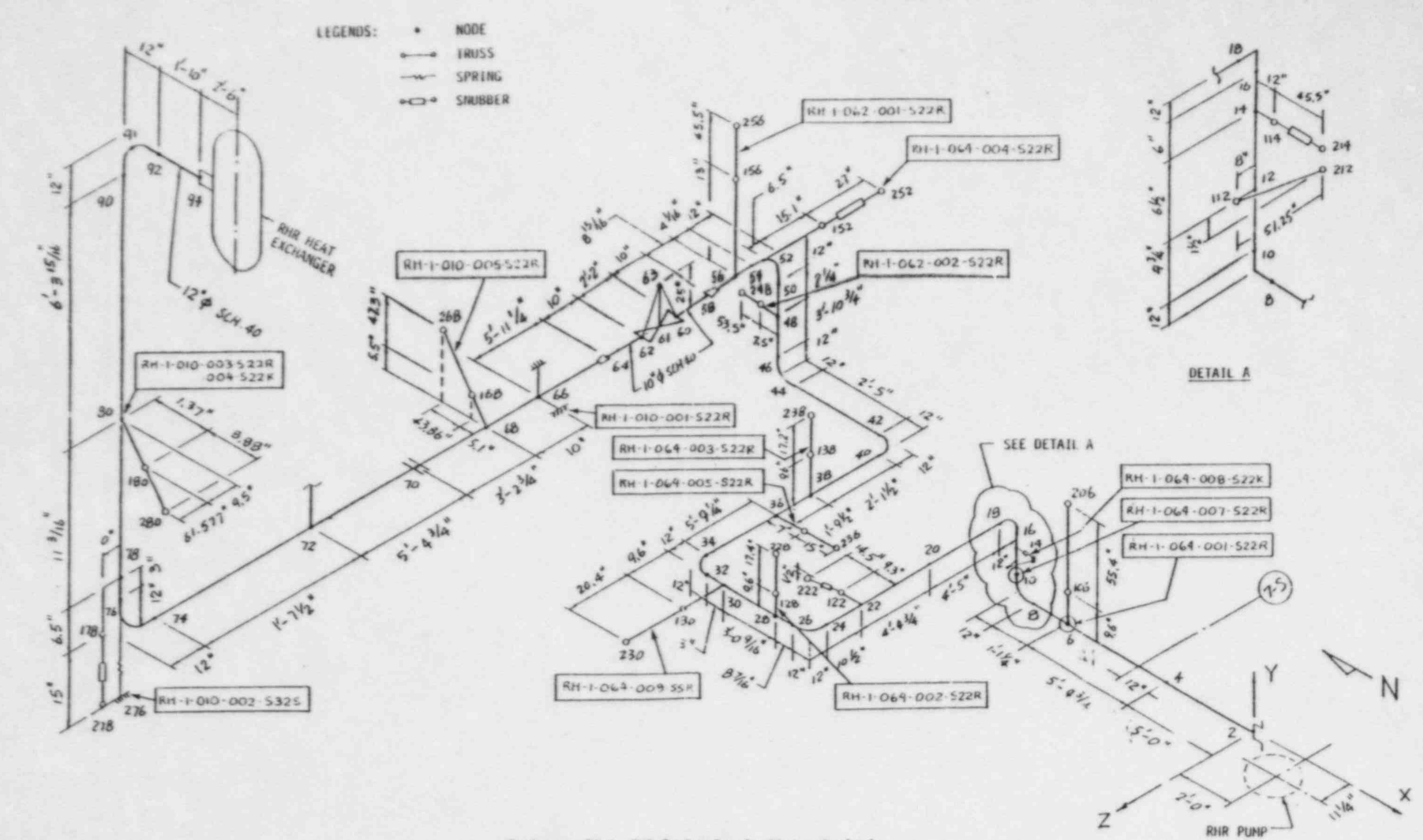
The shape of the ground spectrum is generically defined per USNRC Regulatory Guide 1.60. A site-specific spectrum would normally impose significantly less demands on the structures, systems and components. The peak ground accelerations are also based on conservative interpretations of the geotechnical conditions.

e. Elastic Analyses

Pipe systems have considerable inherent strength that is not tapped by the standard analysis/design techniques. Being constructed of steel, these systems exhibit strength well beyond the yield point. This is often defined as ductility. In Attachment D4-6 (Appendix A to Standard Review Plan, Section 3.5.3) the NRC

recognizes this fact. Although Attachment D4-6 is intended for impact or impulse loads, it shows that steel members in tension can resist strains up to 10 times yield and still perform their intended function.

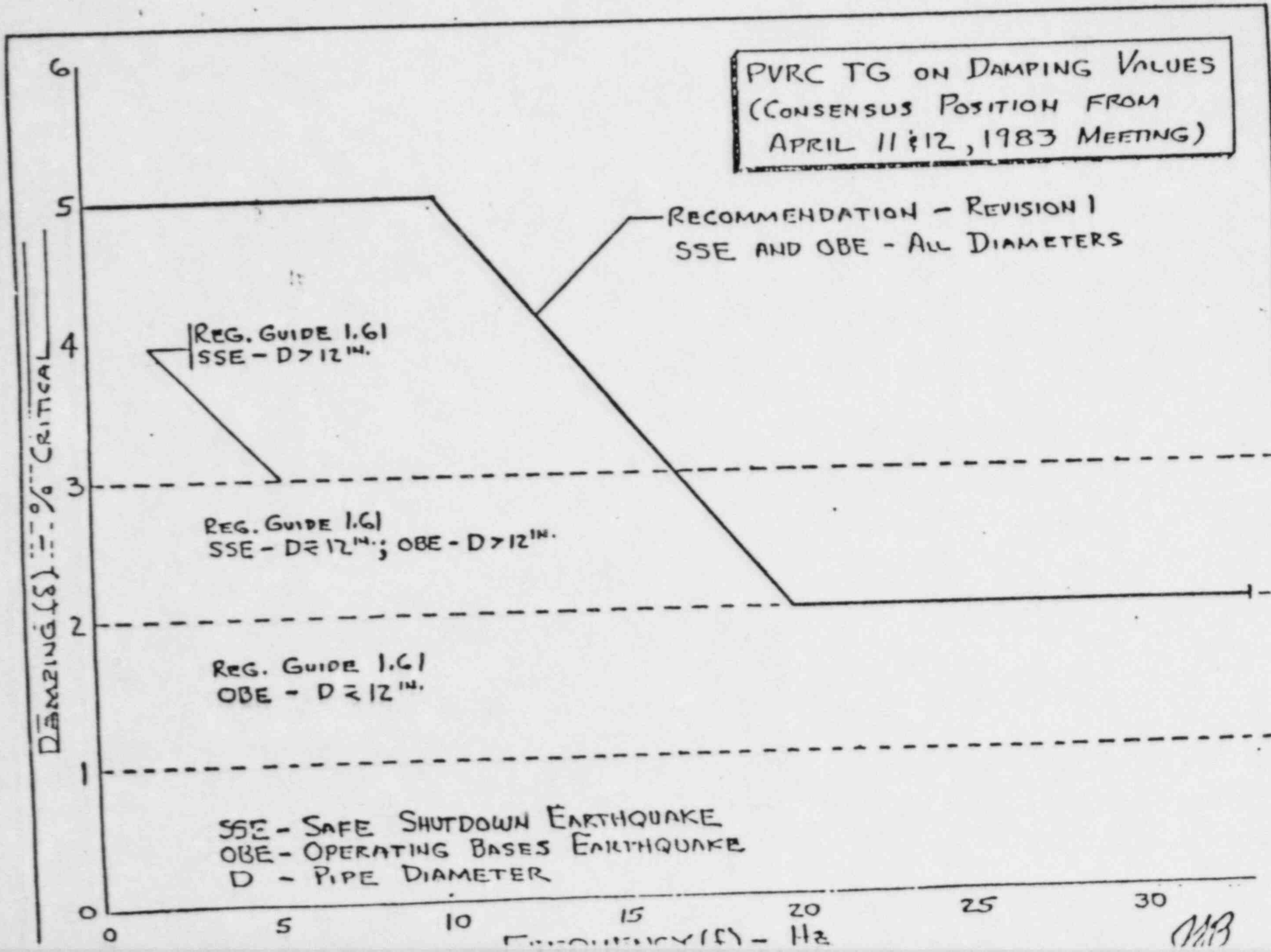
In conclusion, Cygna does not recommend that the conservatisms noted above be deleted from the Comanche Peak analyses. But, on the other hand, the presence of these conservatisms should be recalled whenever minor effects are considered, such as the effect of support masses on pipe stress analyses. Regarding the Comanche Peak practice of not including support masses in the piping analysis, Cygna considers this practice to be consistent with industry practice and with the degree of refinement of the analysis techniques. Furthermore, the test problem results show that support masses have a negligible effect on pipe stresses in a system similar to the one reviewed by Cygna.



Enclosure D4-1 RHR System-Sample Piping Analysis

ATTACHMENT D4-5 (Page 1 of 1)

ATTACHMENT Z



SKETCH SHEET
FORM 22577

APPENDIX A

STANDARD REVIEW PLAN SECTION 3.5.3

PERMISSIBLE DUCTILITY RATIO
FOR OVERALL DAMAGE PREDICTION

I. INTRODUCTION

In the evaluation of overall response of reinforced concrete and steel structural elements (e.g., missile barriers, columns, slabs, etc.) subjected to impactive or impulsive loads, such as impacts due to missiles, assumption of nonlinear response (i.e., ductility ratios greater than unity) of the structural elements is generally acceptable provided that the intended safety functions of the structural elements and those of safety-related systems and components supported or protected by the elements are maintained. The following summarizes specific positions for review and acceptance of ductility ratios for reinforced concrete and steel structural elements subjected to impactive and impulsive loads.

II. SPECIFIC POSITIONS

1. Reinforced Concrete Members

The technical position of the regulatory staff with regard to permissible ductility ratios is stated in Regulatory Guide 1.142. Prior to publication of Revision 1 of Regulatory Guide 1.142, the staff position regarding ductility will be provided to applicants on a case-by-case basis.

2. Structural Steel Members

a. For tension due to flexure

$$\mu_d \leq 10.0$$

b. For columns with slenderness ratio (l/r) equal to or less than 20

$$\mu_d \leq 1.3$$

Where l = effective length of the member

r = the least radius of gyration

For columns with slenderness ratio greater than 20

$$\mu_d \leq 1.0$$

c. For members subjected to tension

$$\mu_d \leq 0.5 \frac{e_u}{e_y}$$

Where e_u = Ultimate strain

e_y = Yield strain

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #5
Exhibit No.: 891, 894, 897

1.0 CASE Question

Inaccurate conclusions as related to KL/R for pinned columns:

- o If a column fixed at its base and free at the top has an effective K of 2.0, cutting at some point up from the base and adding a pin does not address the problem.

2.0 Cygna Interpretation

Does a stability problem exist for CASE Exhibits 891, 894 and 897?

3.0 Response

The stability characteristics of a structure under the action of compressive loads can generally be separated into three categories. These include rigid body modes of instability, Euler column buckling, and beam-column effects. For the purposes of discussion, the three support configurations in question (CASE Exhibits 891, 894 and 897) can all be idealized to the basic configuration shown in Figure 1, wherein the x component of reaction at A is provided by frictional clamping forces. For this basic configuration, the rigid body modes of instability generally account for the overall stability characteristics of the entire structure, while Euler column buckling and beam-column effects are confined to the individual members.

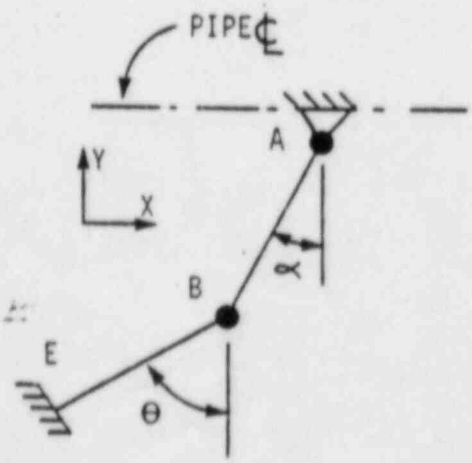


Figure 1

The rigid body mode of instability can be initiated in three ways: (1) when the clamping force at A is insufficient to develop the lateral (x) component of frictional force necessary to prevent sliding; (2) when the clamping force at A is insufficient to develop the resisting torque necessary to prevent the clamp from rotating; and (3) for the specific case of alpha equals theta, when the cantilevered member BC does not provide sufficient lateral stiffness at point B to prevent rigid body rotation of member AB.

Euler column buckling of member AB can occur for all values of alpha and theta given in the three exhibits. The correct value of K to be used in evaluating the stability of member AB is 1.0, since the member is pinned at both ends and can therefore only develop axial loads. Similarly, Euler column buckling can occur in member BC but only when alpha equals theta. The correct value of K to be used in evaluating the stability of this member is 2.0 since the member is fixed at one end and free at the other.

Beam-column effects account for the fact that the bending stresses produced by lateral loads on a column are amplified by the presence of the axial load. What this means is that the maximum stress in a laterally loaded column is not simply the sum of the axial stress and bending stress, but is in fact the sum of the axial stress and an amplified bending stress. This amplified bending stress is the product of the bending stress produced by the lateral load and an amplification factor which is given by the expression

$$\left(\frac{1}{1 - P/P_{cr}}\right)$$

where P is the axial load in the column and P_{cr} is the Euler buckling load for the column. Only member BC is influenced by the beam-column effect. Obviously beam-column effects have no influence on member BC when members AB and BC are either co-linear or perpendicular.

Each of the three CASE exhibits can now be briefly discussed with respect to each of these three categories of instability.

TABLE D5-1

	CASE Exhibits		
	891	894	897
Required Lateral Stiffness at point B (lbs/in)	800	40	N/A
Actual Lateral Stiffness at point B (lbs/in)	700,000	2,000	N/A

TABLE D5-2

CASE Exhibit	Type of Clamping Force Resistance	Required Clamping Force (lbs)	Required Bolt Torque (ft–lbs)
891	Sliding	150	2
	Rotation	107	1
894	Sliding	96	1
	Rotation	16	1
897	Sliding	N/A	N/A
	Rotation	122	1

Euler Buckling of member BC has been accounted for in the calculation and is not a problem. Member AB is a pre-qualified component and as such is stable with respect to Euler Buckling.

CYGNA

The only support for which beam-column effects are applicable is CASE Exhibit 891. Since the critical buckling load for member BC is so large the amplification factor is 1.00. Therefore, beam-column effects have no influence.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #6
Exhibit No.: 891

1.0 CASE Question

16-inch pipe with about 20 kips load along 3-1/2 inches of length induces high bearing stresses which require pads. This is not addressed.

- ASME Code against flattening.

2.0 Cygna Interpretation

How did Cygna evaluate the stresses induced into the piping by the following, as related to the ASME Code caution against inducing excessive flattening into the pipe wall:

a. U-bolt?

b. 5" x 5" x 1/2" tube steel frame?

3.0 Response

Cygna originally evaluated the general code requirements for attachments to piping and Texas Utilities' application of the code.

In Section III, the ASME B&PV Code provides the following caution:

Subsection NB-3645 (Class 1 Components)

"Lugs, brackets, stiffeners, and other attachments may be welded, bolted, or studded to the outside or inside of piping. The effects of attachments in producing thermal stresses, stress concentrations, and restraints on pressure retaining members shall be taken into account in checking for compliance with stress criteria."

Subsections NC-3645 (Class 2) and ND-3645 (Class 3)

"External and internal attachments to piping shall be designed so as not to cause flattening of the pipe, excessive localized bending stresses, or harmful thermal gradients in the pipe wall. It is important that such attachments be designed to minimize stress concentrations in applications where the number of stress cycles, due either to pressure or thermal effect, is relatively large for the expected life of the equipment."

The Code statement for Class 1 components specifies that local effects due to attachments shall be taken into account for compliance with the stress criteria. The Code statement for Class 2 and 3 components, such as those associated with RHR systems, specifies that attachments shall be designed to minimize localized stresses of the pipe. It does not define the term "flattening." A reasonable interpretation would be that the designer of Class 2 and 3 piping should consider the significance of any additional stress induced in the pipe due to attachments. Such a consideration does not imply a requirement for calculations in all instances depending upon the method of attachment.

The Comanche Peak project did use CYLNOZ, a local stress analysis program, when welded attachments were made to the RHR system. It is not common practice to analyze the effects of bearing or clamping except where judgement indicates the need for such an evaluation based on the specifics of a particular design.

In its original review of the adequacy of the loads introduced into the pipe wall by support SI-1-325-002-S32R (CASE Exhibit 891), Cygna considered the following:

a. U-bolt. Cygna judged that the loads introduced into the piping due to design loads would not prevent the piping from performing its intended function. U-bolts are frequently used in the industry for similar applications. Further discussions on U-bolt applications are provided in response to Doyle Question #1.

b. 5" x 5" x 1/2" Tube. Although an unlikely achievement, the drawing detail specifies a 0" gap at all four bearing points. Cygna reviewers concluded significant stresses would not develop in the pipe. It should be noted that radial thermal growth for such a 16" pipe would be 1/50", about the width of two business cards. An analysis of these effects on the pipe was performed to substantiate our judgement on the worst case effects and is contained in Attachment D6-1. The results show that the stresses are acceptable. It is important to note that this is not a typical detail.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #7
Exhibit No.: 891

1.0 CASE Question

Clip angle 4" x 4" x 1/2" which supports U-bolt not addressed (critical to maintaining stability):

- o Section modulus .04 in cube
- o Moment arm at least 2 inches
- o 1100# load exceeds Code allowables.
- o Pre-tensioning to obtain a clamping force required could exceed this (not including thermal constraint and design loads)
- o Clamping force with no margin of safety for single degree system (not point contact or line contact) is force/coefficient of friction or about 4 times what is required for clamping force.

2.0 Cygna Interpretation

Did Cygna check the clip angles (item 15 on support no. SI-1-325-002-S32R) for a potential overstress condition due to: U-bolt torquing, thermal loads, and mechanical loads?

3.0 Response

During the original Cygna review of this pipe support, a judgement was made that the friction forces necessary to resist sliding of the support along the length of the pipe were minimal. At first, these small resisting forces were assumed to be developed by the U-bolt while the mechanical loads, those resulting from static, thermal, and dynamic analyses would be resisted by the box frame. Cygna believes that this was a reasonable assumption to make, given that the support drawing calls for Ø" clearance between the pipe and the box frame. However, because the U-bolt was connected to the support through clip angles that were not considered substantial, a theoretical loss of U-bolt capability was assumed. The reviewer assessed that given this possible loss of U-bolt function and capabilities, sufficient friction forces to resist sliding would still be developed between the box frame and the pipe. These frictional forces were calculated as part of the response to Doyle Question #5 and found to be sufficient to resist the sliding effects required to maintain stability.

A review of the installation instructions (not within the scope of the Cygna audit) indicates that the torque placed on the U-bolt nut in the regular course of installation would theoretically overstress the clip angles. Although the installation procedures were not considered in the Cygna review, the correct conclusion was reached since the reviewer assumed a loss of U-bolt capability.

Cygna does consider this support to be a poor detail if significant cinching loads have been applied to the U-bolt. Installation practice is a new consideration which will be accounted for as part of the on-going Phase 3 review.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #8
Exhibit No.: 893

1.0 CASE Question

There is no documentation in calculations to support the conclusion that flare welds are stronger than fillet welds—no calculations; therefore, why did Cygna accept this statement?

- o Flare weld strength depends on radius of flare (depth).

2.0 Cygna Interpretation

Why did Cygna consider flare welds stronger than fillet welds when no calculations were made?

3.0 Response

As specified in Cygna Design Criteria DC-2, "Pipe Support Design Review Criteria," welds were reviewed for compliance with AWS D.1.1. This weld was not judged to be "unsatisfactory." As shown below, in the case of a welded beam attachment for SI-1-079-001-S325, flare welds are stronger than a 1/4" fillet weld for two reasons:

1) Minimum effective throat thickness (t_e) is greater

- o For flare weld:

$$t_e = 5/16\ R = 5/16\ (5/8") = 0.20"$$

where R = minimum weld groove radius
= inside radius + thickness
= 1/8" + 1/2" = 5/8"

- o For fillet weld:

$$t_e = 0.707\ (1/4") = 0.18"$$

since 0.20" > 0.18", a flare weld is 10% stronger than a 1/4" fillet weld.

2) More weld length

For the welded beam attachment considered, the weld length is 2" along the square side versus 3" along the beveled side. Consequently the installed flare weld along the bevel will give this support 50% more capacity for the same t_e.

Therefore, changing from a 1/4" fillet weld to a minimum flare bevel groove weld increases the capacity of the weld by 65%.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #9
Exhibit No.: 892

1.0 CASE Question

The reduction of weld capacity in the calculation is based on 135 degrees. Actual tangential angle is 150.3 degrees. Therefore, an error exists. Did Cygna take note of this?

- More stress in weld than stated.
- Wide/thin ratio induces cracking as well as the 1.4:1 ratio of width to depth.

2.0 Cygna Interpretation

What was the basis for concluding that the stanchion-to-pipe weld shown in CASE Exhibit 892 is adequate?

3.0 Response

ITT Grinnell design procedure, SA 3912, (Attachment D9-1) states that credit shall only be taken for the portion of the weld up to 135 degrees. Cygna concurred with this procedure and confirmed that it was properly employed on the subject support. Attachment D9-2 shows that the weld length included in the strength calculation was only that portion where the angle between the stanchion and the pipe was less than or equal to 135°.

WELD PROPERTIES FOR
PIPE/STANCHION AND ELBOW/STANCHION CONNECTIONS

FOR

COMANCHE PEAK PROJECT

PROCEDURE SA 3912

FOR INFORMATION ONLY

Prepared By Anwan [illegible] 2-8

Checked By Francis [illegible] 2/8/

Approved By [illegible] 2/8/83

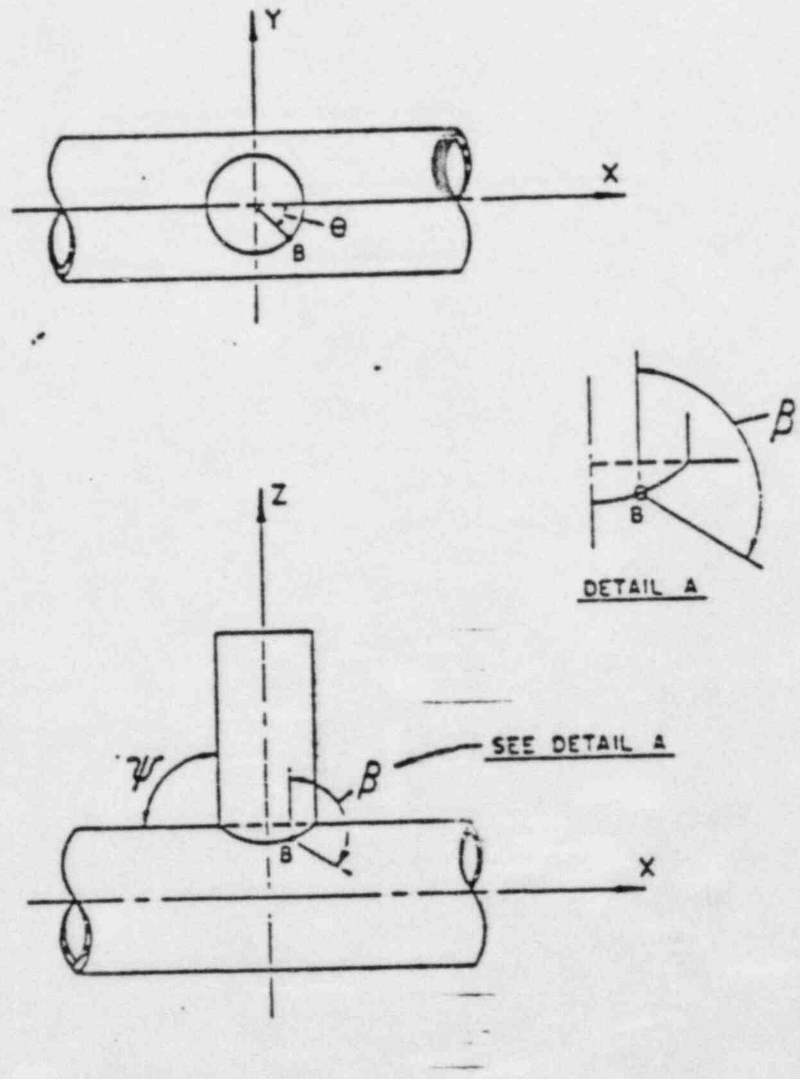
Revision A 02/08/83

CMP-6 Rev. 1

CC# ______

ATTACHMENT D9-1
(Cont.)
(Page 2 of 4)

WELD ANGLES FOR STRAIGHT PIPES WITH STANCHION ATTACHMENTS



THE β-VALUES OBTAINED FOR θ VARYING FROM 0° TO 90° ARE REPEATED EVERY 90° FOR STRAIGHT PIPE ATTACHMENTS

FIG. 1

ATTACHMENT D9-1 (Page 3 of 4)
(Cont.)

4.1 TABLE 1

WELD PROPERTIES OF STRAIGHT PIPES WITH STANCHION ATTACHMENTS (Ref. Fig. 1 & 4)

LIMITING WELD ANGLE = 135°

NOM. PIPE SIZE	NOM. STANCH. SIZE	OVERALL WELDED LENGTH	WELD PROPERTIES				
			L_W	S_Y	S_X	J_W	L_S
2 1/2	1	4.24	4.24	1.36	1.36	1.79	4.24
	1 1/2	6.24	6.24	2.84	2.84	5.39	6.24
	2	8.04	5.36	4.17	1.74	7.01	5.36
	2 1/2	11.08	6.16	5.64	1.57	10.37	6.16
3	1 1/2	6.16	6.16	2.84	2.84	5.39	6.16
	2	7.82	7.82	4.43	4.43	10.52	7.82
	2 1/2	9.72	6.48	6.12	2.54	12.44	6.48
	3	13.49	7.50	8.36	2.32	18.71	7.50
4	2	7.69	7.69	4.43	4.43	10.52	7.69
	2 1/2	9.42	9.42	6.49	6.49	18.66	9.42
	3	11.72	8.46	9.29	4.60	24.32	8.46
	4	17.35	9.64	13.81	3.85	39.76	9.64
6	3	11.34	11.34	9.62	9.62	33.67	11.34
	4	14.82	14.82	15.90	15.90	71.57	14.82
	6	25.54	14.19	29.96	8.34	126.87	14.19

ATTACHMENT D9-1 (Page 4 of 4)
(Cont.)

4.1 TABLE 1

WELD PROPERTIES OF STRAIGHT PIPES WITH STANCHION ATTACHMENTS (Ref. Fig. 1 & 4)

LIMITING WELD ANGLE = 135°

NOM. PIPE SIZE	NOM STANCH. SIZE	OVERALL WELDED LENGTH	WELD PROPERTIES				
			L_W	S_Y	S_X	J_W	L_S
8	4	14.57	14.57	15.90	15.90	71.57	14.57
	6	22.14	17.22	33.86	19.76	177.62	17.22
	8	33.25	18.47	50.77	14.14	279.96	18.47
10	4	14.46	14.46	15.90	15.90	71.57	14.46
	6	21.65	21.65	34.47	34.47	228.37	21.65
	8	29.05	20.97	56.44	27.95	363.95	20.97
	10	41.44	23.02	78.88	21[illegible]7	542.05	23.02
12	4	14.41	14.41	15.90	15[illegible]	71.57	14.41
	6	21.45	21.45	34.47	34.47	228.37	21.45
	8	28.40	28.40	58.43	58.43	503.93	28.40
	10	36.56	24.37	85.53	35.49	650.46	24.37
	12	49.15	27.31	110.95	30.91	904.37	27.31
14	6	21.37	21.37	34.47	34[illegible]	228.37	21.37
	8	28.18	28.18	58.43	58.43	503.93	28.18
	10	35.93	27.84	89.16	52.02	755.87	27.84

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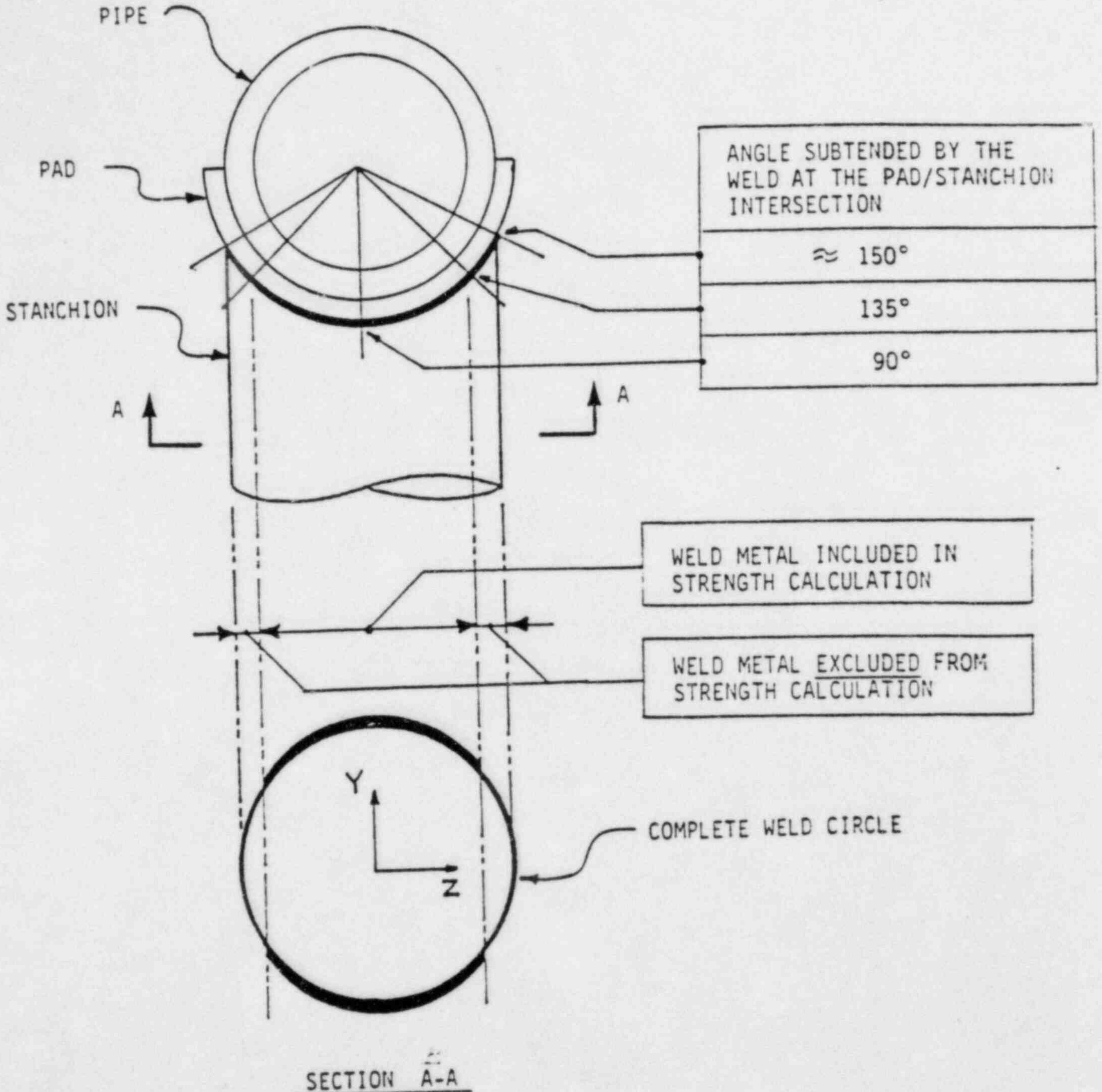


STANCHION - TO - PIPE WELD

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #10
Exhibit No.: 893

1.0 CASE Question

Changing from a flare weld to a fillet weld induces flange bending. Has this been addressed by Cygna?

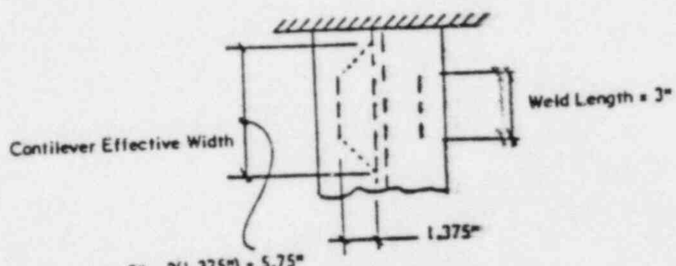
2.0 Cygna Interpretation

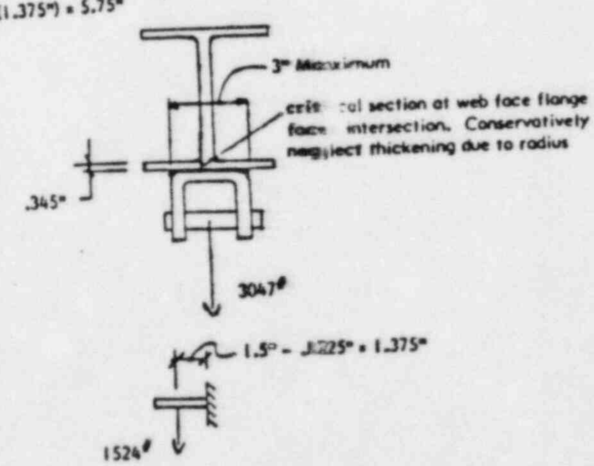
The calculation sheet attached to Exhibit 893 states that the weldment between the rear bracket and the beam flange was changed from a fillet to a flare/bevel weld. This fillet-to-flare change results in a 90 degree re-orientation of the weld lines, from perpendicular-to-parallel to the web of the wide flange. Did Cygna evaluate the additional loads on the flange?

3.0 Response

Cygna judged that this re-orientation would not cause an overstress in the flange. The following calculation verifies that judgement:

Check Flange Bending
Support No. SI-1-079-001-S325





$S = \frac{bh^2}{6} = \frac{5.75}{6}(.345)^2 = .114 \text{ in}^3$

$\sigma = \frac{M}{S} = \frac{1524(1.375)}{.114} = 18,382 \text{ psi}$

Allowable stress = $.75F_y = .75(32,350 \text{ psi}) = 24,262$ psi

$18,382 < 24,262$ psi

Cygna agrees that the maximum stress condition is due to flange bending.

CYGNA

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #11
Exhibit No.: 891, 894, 897

1.0 CASE Question

Effects of out-of-plane seismic excitation of support hardware not included in calculation. Did Cygna address this point?

- Additional loads on support?
- Additional loads on pipe?

2.0 Cygna Interpretation

Did Cygna evaluate the effects of support self-weight excitation in the off-direction, as related to:

a. support design?
b. pipe design?

3.0 Response

a. During Phase 2 of the Independent Assessment Program Cygna did identify this as a potential problem. In the IAP Report, Cygna noted that self-weight excitation was not included in the support design. Note 1 to Checklist PS-01 states:

> "Support Self-Weight Excitation
>
> In general, pipe support vendors have not included support loads due to self-weight excitation in their loading. Texas Utilities has done a generic study in response to Walsh/Doyle allegations which shows the effects are negligible. The NRC Site Inspection Team (SIT) has reviewed and accepted this evaluation in Item 3.h of Inspection Reports 50-445/82-26 and 50-446/82-14."

Since the IAP was performed for the NRC Staff, further evaluation of an issue already identified and reviewed by the Staff would have been redundant. Accordingly, Cygna noted the potential deficiency on the appropriate checklist and deferred to the Staff evaluation.

b. The effect of support masses on the piping analysis is discussed in the response to Doyle Question #4.

1.0 CASE Question

Restraint of rotation by the pipe because of coupling effect of hardware on both sides of a pipe:

- Load increase in 1 of 2 snubbers/struts
- Alteration of dynamics of pipe system during seismic event

2.0 Cygna Interpretation

Support RH-1-010-003-S22R consists of two struts attached to trunnions, which are welded to the pipe at diametrically opposing points. How was this considered in the piping and support evaluations?

3.0 Response

Cygna reviewed the pipe stress analysis to determine whether or not accepted modeling techniques were employed. Cygna determined that the RHR pipe stress model used by Gibbs & Hill was acceptable when compared to general practice. CASE has proposed the need to model certain pipe support configurations into the stress analysis which is different than the existing approach. Gibbs & Hill reran the analysis of piping segment AB-1-70 (see Walsh Question #11) using the CASE technique. The results, when compared to the original analysis, were different, however, there is no reason to believe the Gibbs & Hill model is inappropriate.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #13
Exhibit No.: 891

1.0 CASE Question

In Note 2 following pages PS-01-4 of 4 , Cygna decided to eliminate their stiffness criteria based on their knowledge that a report existed to address the problem (but without personal knowledge of what was contained in the document in detail). Why didn't Cygna consult with their experts -- for example Eric van Stijgeren (who was the editor on a paper by T.Y. Chow, C.H. Chen and O. Bilgen) -- in reference to deviations from generic stiffnesses in pipe supports and the effects on piping systems.

- Third paragraph introduction et. seq. (CASE Ex. 884).

2.0 Cygna Interpretation

Did Cygna evaluate the effects of support stiffnesses on the piping analyses?

3.0 Response

During Phase 2 of the Independent Assessment Program Cygna did identify this issue as a potential problem. As stated in the IAP Draft Report, Cygna questioned the pipe support stiffnesses utilized on Comanche Peak. Note 2 to Checklist PS-01 states:

> "<u>Pipe Support Stiffnesses</u>
>
> The NRC SIT raised the issue of support stiffness in item 3.j of the above referenced reports. Gibbs & Hill has performed a generic study for review by an NRC consultant. The study shows that using 1/16" deflection criteria on support design provides acceptable stiffnesses for the piping analysis (changes in support stiffness do not greatly affect piping results). The NRC review results were not available at the time of the Cygna review."

Since the IAP was performed for the NRC Staff, further evaluation of an issue already identified and reviewed by the Staff would have been redundant. Accordingly, Cygna recorded the potential deficiency on the appropriate checklist and deferred to the Staff evaluation.

1.0 CASE Question

In Note 1, the same source, did Cygna consider the additive effects of self-weight excitation if the stiffness is considered from node point to hard point as opposed to the stiffness of the frame independent of hardware, local effects, baseplate and anchor bolts?

- o Spring rate of baseplate/anchor bolts (particularly bearing-type joints) can be considerable (observation of baseplate II finite analysis).

2.0 Cyg Interpretation

Did Cygna consider the following:

a. The effect of support stiffness on the evaluation of self-weight excitation?

b. The flexibility of each element in the support load path?

3.0 Response

a. In order to evalute the influence of self-weight excitation on support design, one must apply the appropriate dynamic loads and then calculate the induced stresses and deformations. The applied load, in this case, is the support self-weight. Support stiffness is effectively considered twice in this process. First, it is included in calculating the applied dynamic load. This can be illustrated by the following elementary formulas:

1. Load = function (freq)

2. freq = (1/6.28) · SQRT (Kg/F)

where freq = support fundamental frequency
K = support stiffness
F = self-weight
g = gravity

Secondly, the determination of support stresses and deflections involves a structural evaluation which considers the support stiffness.

For a further description of Cygna's review process relative to support self-weight excitation, see the Cygna response to Doyle Question #11.

b. As stated in the response to Doyle question #13, Cygna recorded that support stiffness calculations on Comanche Peak were potentially deficient. When it was learned that the NRC Staff had evaluated this issue, Cygna deferred to the Staff evaluation rather than performing a redundant review.

Regarding the effects of component flexibilities on the overall support stiffness, current standard practice is not to include the baseplate connection. These effects are being studied by various industry groups. One such group is the Structural Engineers Association of California (SEAOC). An update on their activities is provided in Attachment W14-1. Until resolution is reached on the relative merits of considering the baseplate connection in the stiffness calculation, Cygna does not consider it reasonable to evaluate Comanche Peak against a requirement to include these effect.

Communications Report

Company: Texas Utilities	☒ Telecon	☐ Conference Report	
Project: Comanche Peak Steam Electric Station Independent Assessment Program - Phase 3		Job No: 84042	Date: 4/10/84
Subject: Column Base Plate Flexibility		Time: 3:00 pm	Place: SF
Participants: Helmut Krawinkler (415) 497-4124		of: SEAOC and Stanford University	
T. Wittig (415) 397-5600		Cygna	

Item	Comments	Required Action By

Reference: "Recommended Lateral Force Requirements and Commentary," SEAOC, 1980.

Mr. Krawinkler chairs a Structural Engineer Association of California (SEAOC) subcommittee on "Steel."

I asked for an update on activities related to the following excerpt from Commentary Section 4 of the referenced publication:

> "Column base connection performance is of particular concern where a fixed base is assumed in design. The effects of inelastic extension of anchor bolts on column moments, frame drift and stability need investigation;"

Mr. Krawinkler noted that this question is complex and that SEAOC has not established a position. Furthermore, there will be no position stated in the upcoming revision to the referenced document.

Regarding the application of this question to pipe supports, he emphasized that Section 4 is titled "Steel Ductile Moment Resisting Frames." The commentary note was added because hinge formation needed to develop ductile behavior in steel framed buildings could conceivably occur within the column base plate connection. Since information on the ductile behavior of such connections is insufficient, the issue was identified as

Signed: Ted T. Wittig /pm

Distribution: H. Krawinkler (Stanford Univ.), D. Wade, N. Williams, G. Grace, T. Wittig, Project File

Item	Comments	Required Action By
	requiring study. Applying this question to pipe supports is clearly inappropriate, because they are not designed as ductile moment resisting space frames. I told Mr. Krawinkler that our conversation would be reported during the hearings on Comanche Peak.	

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #15
Exhibit No.: 891, 898, 899, 900

1.0 CASE Question

Was thermal lockup considered for anchors which restrain pipe radial growth?

- o Induces frame moments.

2.0 Cygna Interpretation

- o How was the effect of thermal radial pipe growth considered in the review of CASE Exhibits 891, 898, 899, and 900?

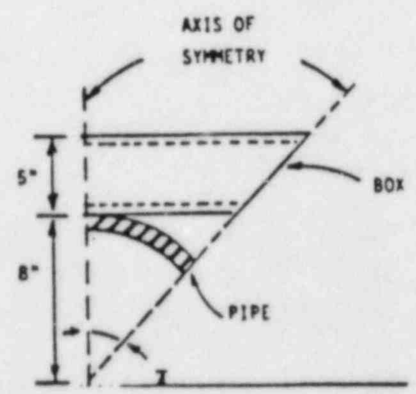
3.0 Response

Exhibit 891 (Support SI-1-325-002-S22R)

Exhibit 891 shows a box-frame enclosing a 16" diameter pipe. The design details specify a zero gap between the pipe and frame at the four points of contact. Cygna reviewers evaluated this configuration and judged that the thermal stresses would be acceptable.

To address some concerns raised during the ASLB hearing regarding this issue, Cygna performed a finite element analysis of the frame/pipe with zero gaps. Figure D15-1 shows the model. The pipe was heated to 350°F (T = 280°F) and the flexibility of both the pipe and frame were considered.

BOX-FRAME THERMAL MODEL



The results are summarized below:

Thermal Only

Element	Stress (psi)	Allowable (psi)	% Allowable
Pipe	37,700	64,800(1)	58%
Frame	38,300	56,400(2)	68%

Thermal + Mechanical

Element	Stress (psi)	Allowable (psi)	% Allowable
Pipe	39,300	64,800	61%
Frame	39,800	56,400	71%

Notes:

(1) 3 S_m per ASME B&PV Code, Section III, Figure NB-3222-1. S_m = 19,300 psi per Appendix I for SA376, Type 304, material at 350°F.

(2) 3S per ASME B&PV Code, Section III, Paragraphs NF3213.10 and NF3231.1a. S = 0.6 Sy, where Sy = 36,000 psi per Appendix I for A500, Grade B, tube steel at 70°F.

Note that the element stress allowables are based on membrane plus bending stresses defined in the ASME code. This is appropriate because the model employed discrete, shell elements.

CYGNA

Exhibits 898 and 900 (Supports SI-1-037-005-S22A and SI-1-030-003-S32A, respectively)

Exhibits 898 and 900 show two variations of framed supports where the pipe is welded to diametrically opposed trunions which form the horizontal member of the frame. Figure D15-2 illustrates this configuration.

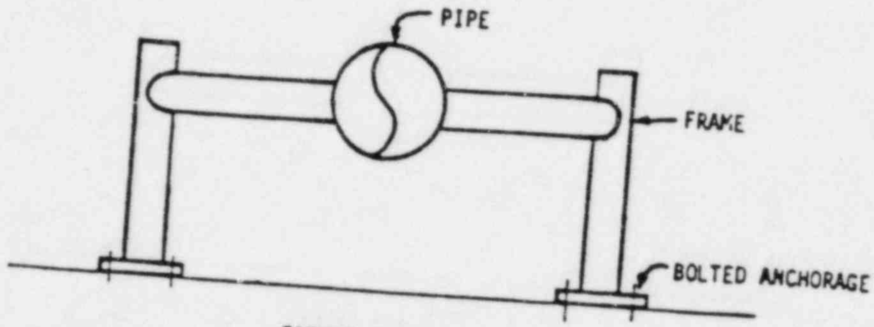


FIGURE D15-2

Cygna reviewers considered the effect of pipe thermal expansion to have negligible impact on the support design since the stresses induced in the support result from constraint of free end displacements. Under these conditions the Code allows a 200% increase in allowable stress when the mechanical levels are combined with the effects of the pipe thermal expansion.

To demonstrate this conclusion, Cygna performed a hand calculation for CASE Exhibit 898 (See Attachment D15-1) which incorporate the effects of pipe thermal expansion with mechanical loads. The results of this calculation show that all stresses in the frame and baseplate are below the allowables.

Exhibit 899

Figure D15-3 illustrates this configuration.

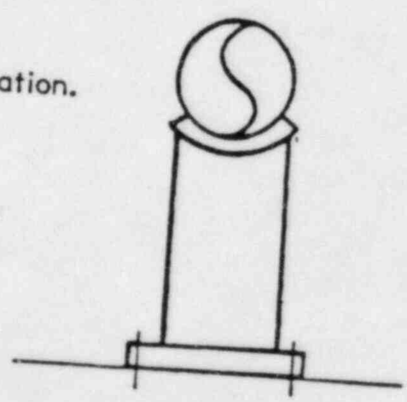


FIGURE D15-3

By inspection, thermal radial growth of the pipe is primarily unrestrained. A secondary restraint will develop at the bimetallic weld due to thermal gradients and the material differences. Cygna's reviewers judged the effect of this secondary restraint to be negligible.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #16
Exhibit No.: 891, 897, 898, 899, 900, 906

1.0 CASE Question

The baseplate analysis is based on distribution of shear relative to load path/stiffness for all bolts in the pattern. Did Cygna address this problem?

- With oversized holes and the inability to eliminate construction tolerances (location of the bolts combined with location of the bolt holes), it is not possible for all of the bolts in the system to be active (see CASE Exhibit 906).

- The stiffness of the bolts is such that deflection cannot be counted on as a means to achieve full pattern participation.

- Even if deflection could result in full activity, the first bolts deflecting would receive the larger portion of the load in an ideal symmetrical and . ystems.

- For non-symmetrical system and systems of variable stiffness, the inactivity of a number of the bolts will alter the accuracy of the computerized analysis.

2.0 Cygna Interpretation

N/A

3.0 Response

The determination of the distribution of shear forces to the anchor bolts of a baseplate is based upon the same methodology which has for decades been sucessfully used for the design of bolted connections of both bearing and friction type. In this "conventional method" of bolted connection design it is assumed that all bolts in the pattern are active to one degree or another depending upon the location of the pattern center of twist relative to each bolt. Should the center of twist lie within the bolt pattern, some bolts may be completely inactive compared to others in the pattern. Where the pattern center of twist is far exterior to the bolt pattern it is more likely that all bolts will be equally active in resisting shear forces. Using this method the forces on the most highly stressed bolt within the pattern then determines the bolt size to be used for the <u>entire</u> pattern.

Cygna finds no problem with this standard design methodology which is referenced in all standard textbooks which deal with the design of bolted connections.

In responding to the question, it will be assumed (conservatively) that no friction whatsoever can be developed between either the baseplate and the concrete or the anchor bolt nut/washer and the baseplate. For this extreme case it will be explained how full baseplate functionality to resist the ultimate design shear forces is maintained.

Construction tolerances associated with either locating the bolt hole in the baseplate or the bolt hole in the concrete have absolutely no influence on the distance that a baseplate must move before it bears directly on an anchor bolt. The only thing that affects the maximum distance that a baseplate must move until it bears directly against the bolt is the difference between the diameters of the bolt hole and the bolt. At Comanche Peak this maximum distance is 1/16" for bolts less than 1" and 1/8" for bolts 1" and greater, although most baseplates with 1" holes which have the lesser oversize of 1/16" specified. Oversized holes is a fact of life in connection design. Codes specify the allowable oversize for various types of connections.

With oversized holes (and again conservatively neglecting friction) it is not possible for all bolts to be initially active. Even after all bolts become active some bolts will be resisting much higher forces than others. This is a well recognized fact in any bearing connection. What is essential for a bearing connection is that it be able to reach its design ultimate capacity. It is not important that all bolts be stressed to the same level.

In the design of a connection oversized holes would never be specified in a connection constructed from brittle material or from materials which exhibit non-ductile behavior. Connections must be made of materials which exhibit relatively ductile behavior so that shear force redistribution can occur among the bolts in the pattern.

For a bearing connection a relationship exists among the size of the hole oversize, the ultimate shear displacement of the bolts, the stiffness characteristics of the bolts, the percentage of bolts not initially in bearing and the desired baseplate safety factor. This relationship is derived below.

Consider a baseplate with N total bolts of the same diameter and embedment. X of the N bolts are in immediate bearing with the baseplate. Therefore, N-X bolts are not in immediate bearing and are all (conservatively) assumed to have a maximum gap of Δ_0(the hole oversize). Thus (N-X) bolts will lag the response of the X bolts by a displacement of

Let

P_o = Total Design Shear Load on Baseplate

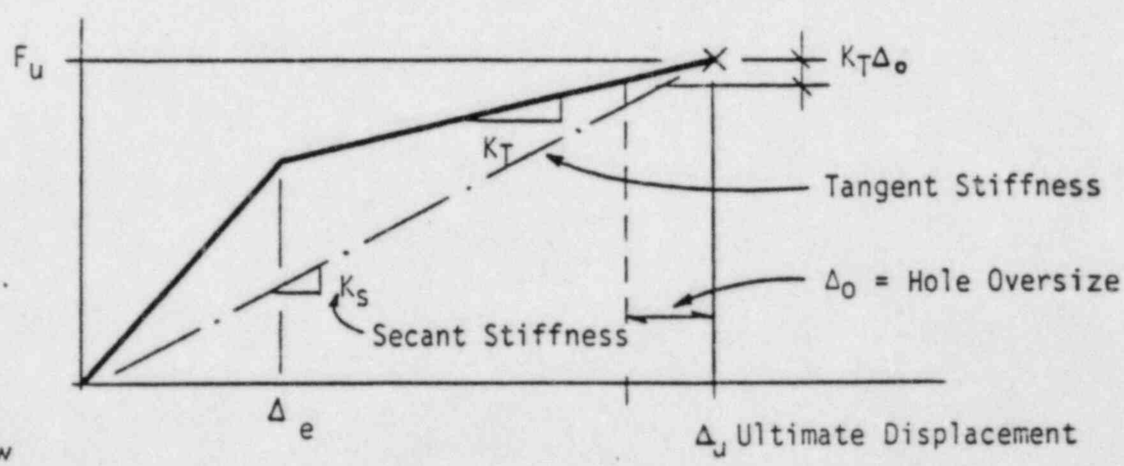
SF = Baseplate Safety Factor Desired

P_u = Ultimate Baseplate Load = $(SF)P_o$

F_u = Ultimate Bolt Shear Force

F_D = Allowable Bolt Shear Force = $\frac{F_u}{5}$ per Design Criteria

The actual bolt shear force-displacement curves can be closely approximated by a bilinear force - displacement curve such as the one shown below.



Now

$$P_u = XF_u + (N-X)(F_u - K_T\Delta_o) \qquad (1)$$

$$P_o = NF_D = NF_u/5 \qquad (2)$$

$$P_u = (SF)P_D = (SF)NF_u/5 \qquad (3)$$

This is a sufficiently high safety factor for a baseplate. It can be seen that the 1/8" oversize hole only reduced the overall baseplate factor of safety below the bolt factor of safety by 4%.

The "conventional method" is the basis for both hand analysis and computerized analysis of baseplates to determine the relative distribution of shear forces within a bolt group. The "conventional method" is a design tool, it is not a rigorous nonlinear analytical technique. Where used for connection design with sufficiently ductile materials it guarantees that the required ultimate shear capacity of the baseplate will be reached.

ABBOT A. HANKS

ESTABLISHED 1866

1116 INDIANA STREET, P. O. BOX 77263
SAN FRANCISCO, CA 94107
(415) 282-8600

File No. 112189-61
Report No. 8785

January 30, 1974

HILTI FASTENING SYSTEMS, INC.
360 Fairfield Avenue
Stamford, Connecticut 06904

SUBJECT: KWIK-BOLT TESTING PROGRAM - LOAD VS. DISPLACEMENT GRAPHS

At your request, we have conducted a comprehensive program of testing of the seven different diameters of Kwik-Bolts (1/4" through 1 1/4") to determine their performance characteristics in 2,000, 4,000 and 6,000 psi concrete. The results obtained from this program are as noted on the attached graphs.

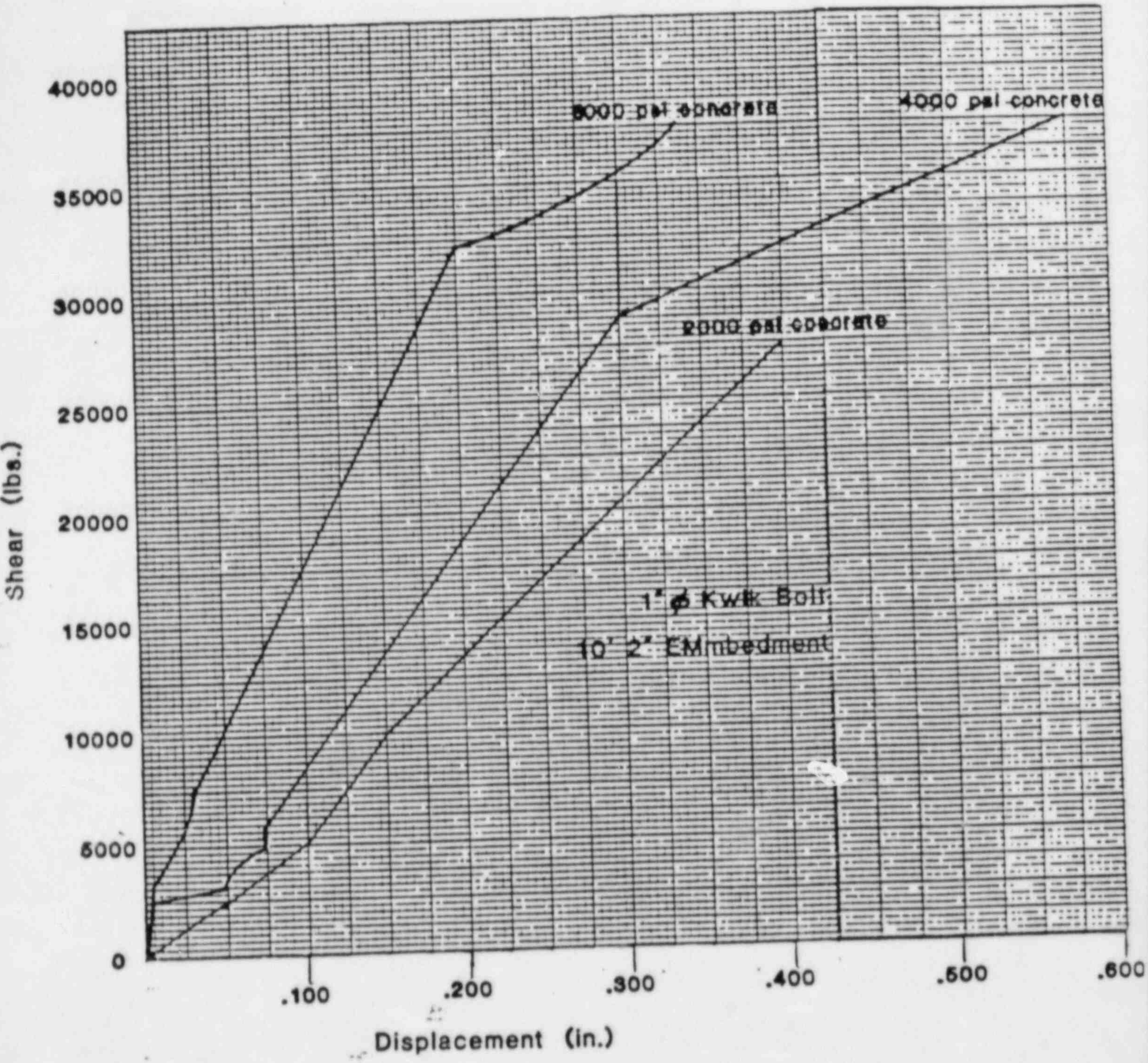
Anchors, drills and drill bits were furnished by HILTI from regular production runs and are considered to be indicative of that material normally used for installations of this type.

Concrete was supplied by a local batch plant and placed under Abbot A. Hanks supervision by a general contractor. Non reinforced slabs were used for testing. The concrete mix for the test slabs used limestone aggregate in accordance with ASTM C-33 (3/4" maximum) and Type II cement. The concrete was placed in typical construction manner and finished with a bull-float. Test slabs were designed for 28 day strengths of 2,000, 4,000 and 6,000 psi. Compressive strengths were verified from standard 6 x 12 inch cylinders from each slab prepared in accordance with ASTM C-31 and tested in accordance with ASTM C-39.

Tensile and shear testing was performed using a hollow-core hydraulic jack equipped with a calibrated pressure gauge. For tensile testing the testing equipment was supported by a three legged reaction tripod which distributed the loading outside a 24" diameter circle for anchors 3/4" diameter or less and outside a 30" diameter circle for 1" and 1 1/4" diameter anchors.

For shear testing, all anchors were at least 30" away from the reaction point of the hydraulic testing equipment. The load was applied as close to the surface as possible to minimize the effects of bending. In addition, several washers were placed between the shear plate and concrete surface to minimize friction between the two surfaces.

Enclosure D16-1



CYGNA

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #17
Exhibit No.: None

1.0 CASE Question

Has Cygna verified the statement: "No 2-inch topping"?

- This affects the calculation for Hiltis relative to embedment, since a non-monolithic shear plane has been established.

2.0 Cygna Interpretation

Three support drawings within Cygna's scope of review contain a note regarding the 2" topping. These are:

- RH-1-010-002-S22S, Rev. 5
- RH-1-024-011-S22A, Rev. 1
- SI-1-038-013-S22A, Rev. 2

On the first two drawings, the note states "No 2-inch topping". On the other drawing, 2 inches of topping is specified.

What credit was taken for this topping in the calculation of minimum expansion anchor embedment?

3.0 Response

To verify the adequacy of expansion anchor embedment leng Comanche Peak began with the full length of the anchor and then subtracted items as the plate thickness, thread length, grouting and topping. Therefore, in calculating minimum embedment length, no credit was taken for the strength of the topping.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Doyle #18
Exhibit No.: 898, 899

1.0 CASE Question

The baseplate analysis was performed without including stiffeners alters the stiffness matrix of the baseplate and consequently the distribution of moments and tension to the bolts. Beyond this point, stiffeners remain unqualified. Has Cygna addressed this?

2.0 Cygna Interpretation

Did Cygna consider the bolt loading for the baseplate in the stiffened condition? Also, did Cygna qualify the stiffeners?

3.0 Response

It is a conservative approach to ignore the effects of stiffeners on plate or bolt design.

When stiffeners are added a redistribution of forces to the bolts does occur in the presence of stiffeners. More importantly, this redistribution of forces is favorable since it will produce higher bolt forces and therefore a more conservative design for the stiffened baseplate.

It is important to recognize that the most critical elements in the design of the baseplate are the anchor bolts. The high degree of indeterminancy of the plate portion of the baseplate combined with the significant membrane resistance (in addition to bending) which must develop prior to failure of the plate material, makes the overall failure of the baseplate by failure of the plate material very unlikely. The more likely failure mode for a baseplate results from bolt failure since the bolt system is generally less indeterminate and does not possess the alternate load carrying mechanism that membrane action provides for the plate. Recognizing this, baseplate analysts tend to make assumptions which maximize the tensile forces in the most highly stressed bolt(s). One such assumption is to neglect the presence of stiffeners.

Stiffeners make a flexible baseplate behave more like a rigid plate. By making the plate more rigid, the internal moment arm, created in the plate by the compressive force in the concrete and the tensile force in the bolts, becomes a maximum. Therefore, to resist a given applied external moment, the maximum bolt tension will be smaller in a rigid (stiffened) plate than in a flexible (unstiffened) plate.

On the other hand, stiffeners have no effect on bolt shear forces. This is because the in-plane stiffness of a baseplate is already very large and the addition of stiffeners do little to increase this already high stiffness. Well proportioned stiffeners (relatively thick and deep with length to depth ratio < 3) are generally not a problem in baseplate design. A simple and conservative stiffener analysis shows stresses well below allowables.

Detailed baseplate calculations for SI-1-037-005-S32A and RH-1-024-011-S22A (Attachments D18-1 and D18-2) for the stiffened and unstiffened cases support the above observations in a general way. The tables on the next page show that the maximum bolt tensile forces and plate stresses are greater for the cases without stiffeners than they are with stiffeners.

From these tables it can also be observed that for bolts with a larger provision ratio, the bolt loading for the unstiffened condition is greater. Bolt provision ratio is defined as follows:

$$\text{B P ratio} = \frac{T}{T_A} + \frac{V}{V_A}$$

where:

T = actual tension
T_A = allowable tension
V = actual shear
V_A = allowable shear

Table 1 - Support SI-1-037-005-S32A

Bolt #	Bolt Force (lbs) Without Stiffeners	Bolt Force (lbs) With Stiffeners	Provision Ratio Without Stiffeners	Provision Ratio With Stiffeners
1	1,900	1,700	0.27	0.25
2	0	240	0.13	0.15
3	60	460	0.11	0.14
4	2,260	2,000	0.27	0.25

Table 2 - Support RH-1-024-011-S22A
(Case 1)

Bolt #	Bolt Force (lbs) Without Stiffeners	Bolt Force (lbs) With Stiffeners	Provision Ratio Without Stiffeners	Provision Ratio With Stiffeners
1	1,170	1,580	0.40	0.43
2	1,260	800	0.35	0.31
3	0	0	0.45	0.46
4	240	770	0.41	0.45
5	3,660	2,100	0.35	0.23
6	2,510	2,710	0.40	0.42

Table 3 - Support RH-1-024-011-S22A
(Case 2)

Bolt #	Bolt Force (lbs) Without Stiffeners	Bolt Force (lbs) With Stiffeners	Provision Ratio Without Stiffeners	Provision Ratio With Stiffeners
1	0	0	0.31	0.31
2	560	610	0.29	0.29
3	1,610	1,670	0.56	0.59
4	3,140	2,930	0.63	0.62
5	3,050	2,070	0.31	0.23
6	250	870	0.23	0.28

Table 4

Support Number	Maximum Plate Stress (psi) Without Stiffeners	Maximum Plate Stress (psi) With Stiffeners
SI-1-037-005-S32A	9300	6600
RH-1-024-011-S22A (Case 1)	8500	3600
RH-1-024-011-S22A (Case 2)	9800	3800

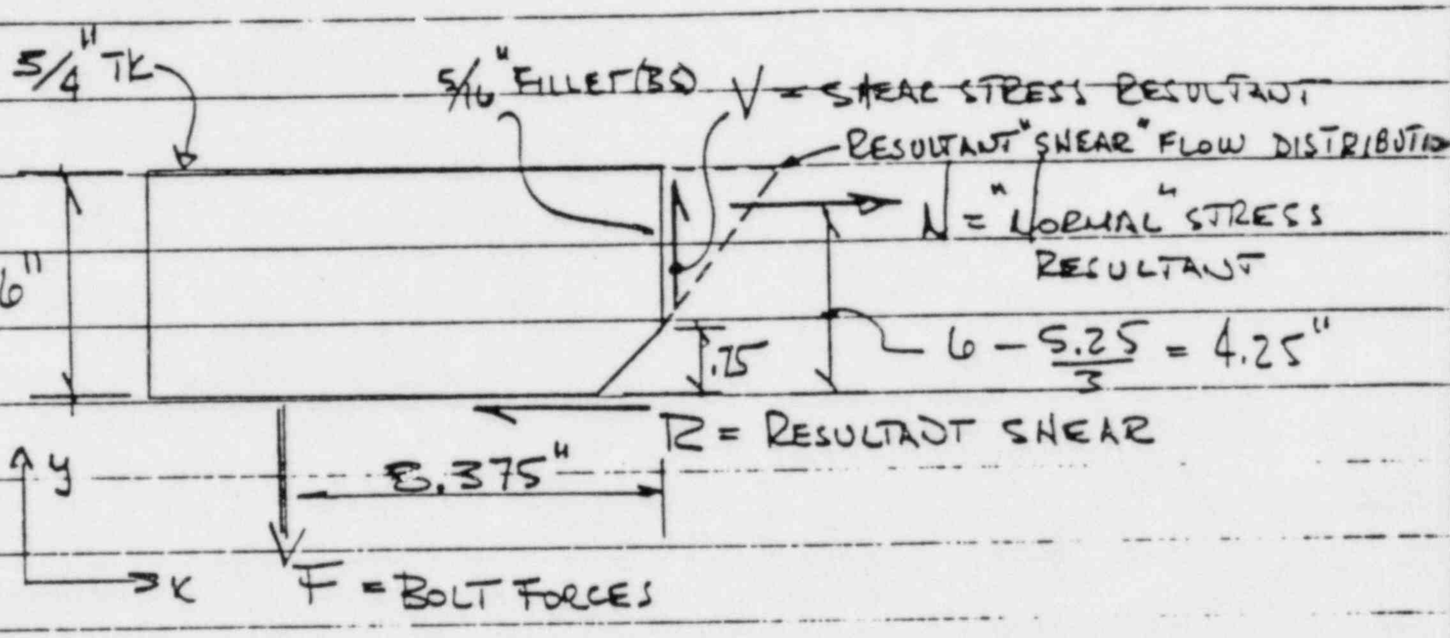
G. BJORKMAN

Enclosure D18-1

1 of 2

STIFFENER ANALYSIS

SUPPORT No. SI-1-037-005-S32A



From Cygna computer out put
"TUSI PLATE No. 2 FOR AB-1-70"

FOR BOLTS 1 & 4

$F = 1706^{\#} + 2003^{\#} = \underline{3709}^{\#}$

$\Sigma F_x = 0: \quad R = N$

$\Sigma F_y = 0: \quad V = F$

$\Sigma M = 0: \quad 4.25\,N = 8.375\,F$

$N = \underline{7,309}^{\#}$

CYGNA	
PROJECT	TUSI
TITLE	BASEPLATE ANAL
PREPARED BY	GSB
DATE	3-7-84
CHECKED BY	BGJ
DATE	3-15-84
JOB NO.	84042
FILE NO.	
SHEET NO.	

Max. Weld shear flow due to ↓

$(2)(7309)/\underbrace{(2)(5.25")}_{\text{TOTAL WELD LENGTH}} = 1392\ \#/in$

Weld shear flow for V = F = 3709#

$3709/2(5.25) = 353\ \#/in$

Resultant max. shear flow

$\sqrt{1392^2 + 353^2} = 1436\ \#/in$

WELD SIZE STIFFENER TO TUBE

E60

Allowable weld shear flow $= .707(5/16)(18,000)$

$= 3977\ \#/in > 1436\ \#/in$

OK!

Tube wall thickness = 3/8" > 5/16 weld size

∴ Punching shear ok!

But check anyway

Max punching shear stress = $1392\ \#/in / 3/8"$

$= 3712$ psi

This is less than $.4F_y = (.4)(32,350) = 12,940$ psi

ok

CYGNA

PROJECT TUSI

TITLE BASEPLATE ANAL.

PREPARED BY GSB

DATE 3-7-84

CHECKED BY RGJ

DATE 3-15-84

JOB NO. 84042

FILE NO.

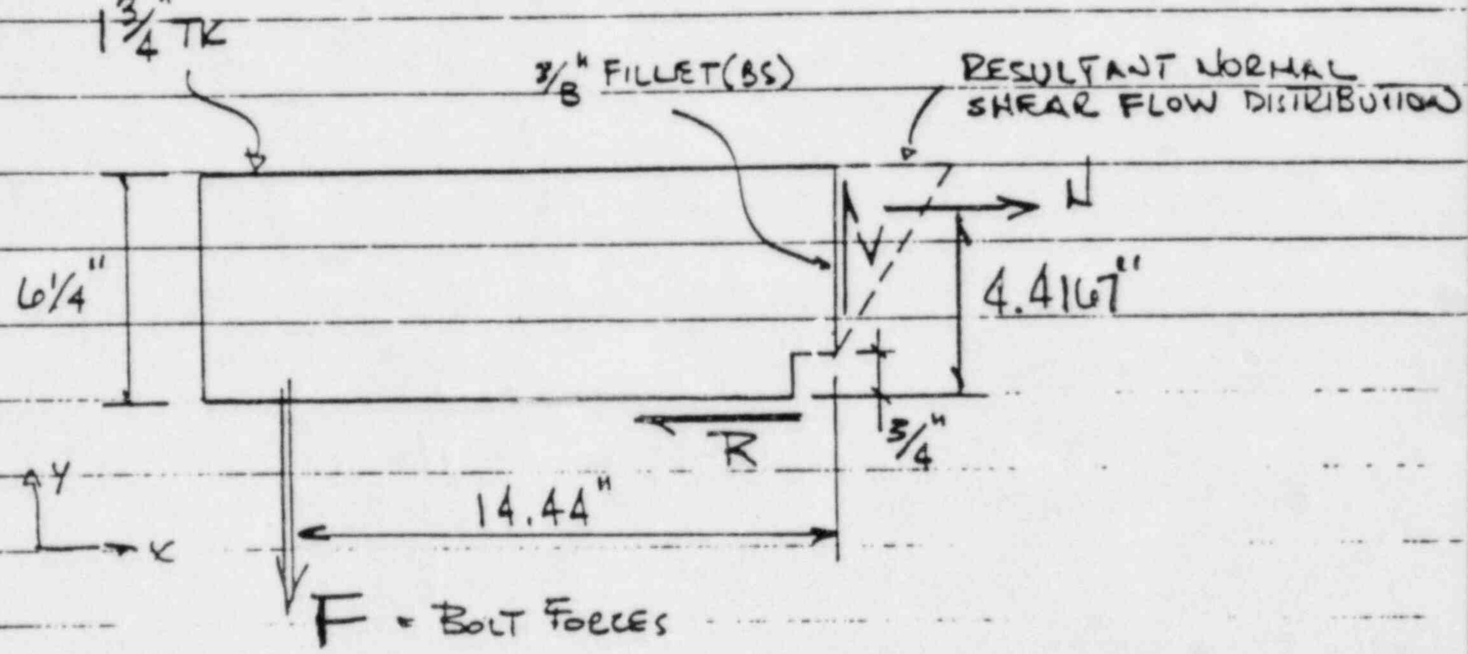
SHEET NO.

G. BJORKMAN

10/2

STIFFENER ANALYSIS

SUPPORT NO. RH-1-024-011-S22A



From Cygna computer output
"TUSI PLATE NO. 4 FOR AB-1-71A"

FOR BOLTS 3 & 4

$F = 1665^{\#} + 2930^{\#} = 4595^{\#}$

$\Sigma F_x = 0:$ $R = N$

$\Sigma F_y = 0:$ $V = F$

$\Sigma M = 0:$ $4.4167 N = 14.44 F$

$N = 15{,}023^{\#}$

CYGNA	
PROJECT	TUSI
TITLE	BASEPLATE AN
PREPARED BY	GSB
DATE	3-7-84
CHECKED BY	RGJ
DATE	3-15-84
JOB NO.	84042
FILE NO.	
SHEET NO.	

2 of 2

CYGNA	
PROJECT	TUSI
TITLE	BASEPLATE [illegible]
PREPARED BY	GSB
DATE	3-7-84
CHECKED BY	RGJ
DATE	3-15-84
JOB NO.	84042
FILE NO.	
SHEET NO.	

Max. weld shear flow due to J

$2(15{,}023)/\underbrace{2(5.5)}_{\text{weld length}} = 2732\ \#/in$

Weld shear flow due to $V = F = 4595\#$

$4595/2(5.5) = 418\#$

Resultant max. shear flow

$\sqrt{2732^2 + 418^2} = 2764\ \#/in$

Allowable weld shear Flow $= (.707)(3/8)(18{,}000)$
$= 4772\ \#/in$

Check Punching shear

Tube wall thickness $= 1/2'' > 3/8$ weld size

$\therefore$ OK

But check anyway

Max. punching shear stress $= 2732\ \#/in / 1/2''$
$= 5464$ psi

This is less than $.4F_y = .4(32350) = 12{,}940$ psi

OK

S. F. PRIME RUN DATE : WED, 07 MAR 1984 14:00:17

Enclosure D18-2

```
========================================
 PROGRAM NAME               : RPG
 VERSION                    : 2.0
 VERIFICATION STATUS        : Verified
 PROGRAM RELEASE DATE       : Oct. 12,1983
 PROGRAM CONSULTANT         :

 CYGNA CORPORATION
 600 MONTGOMERY STREET
 SAN FRANCISCO, CALIF. 94111
 (415) 397-5600
========================================
```

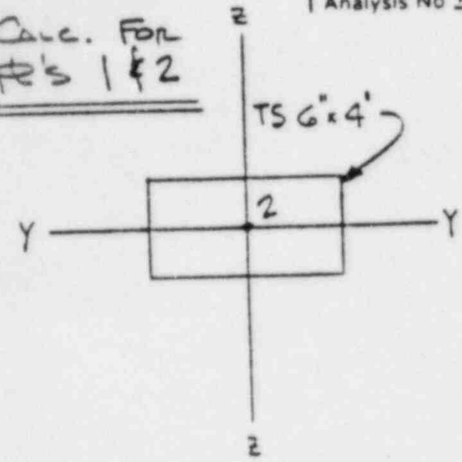
```
*************************************************************
*         *                          *            *          *
* CYGNA   *   COMPUTER OUTPUT SUMMARY * JOB NO.:   * FILE NO.:*
*         *                          *  84042     *XXXXXXXXXX*
*************************************************************
* CLIENT:                  * PROJECT:                        *
* TUSI                     * CPSES                           *
*************************************************************
* MODEL IDENTIFICATION:                                      *
*           AP-1-70(PL.'S 1&2) & AB-1-71A(PL.'S 1-4)         *
*************************************************************
* RUN IDENTIFICATION:                                        *
*            "TUSI"  BASEPLATE ANALYSIS                      *
*************************************************************
* PROGRAM: RPG        * BY: R.G.JOHNSON       * CHECKED:GSB  *
* VERSION: 2.0        *   WED, 07 MAR 1984    * DATE: 3-12-84*
*************************************************************
*                                             * APPROVED:    *
* COMMENTS:                                   * DATE:        *
*                                                            *
*************************************************************
```

TABLE OF CONTENTS

Calculation Sheet		Prepared By R.G. [signature]	Date 3-6-84
Project C. P. S. E. Station		Checked By [signature]	Date 5-12-84
Subject Baseplate Analysis		Job No 84042	File No
System S1-1-037-005-S32A		Sheet No	1
Analysis No AB-1-70	Rev No 0		

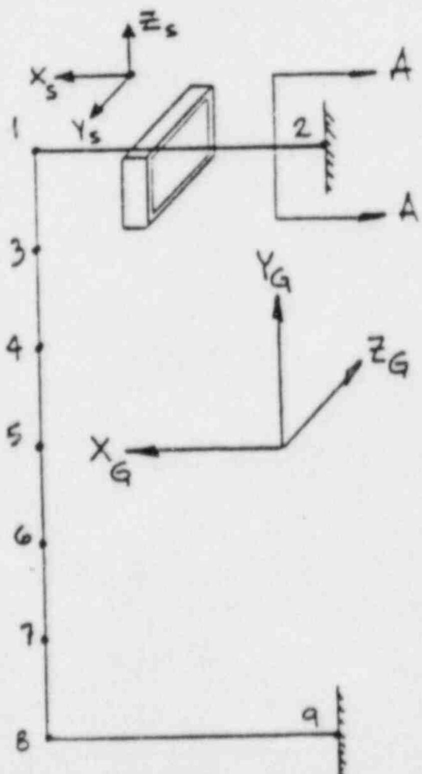
CYGNA

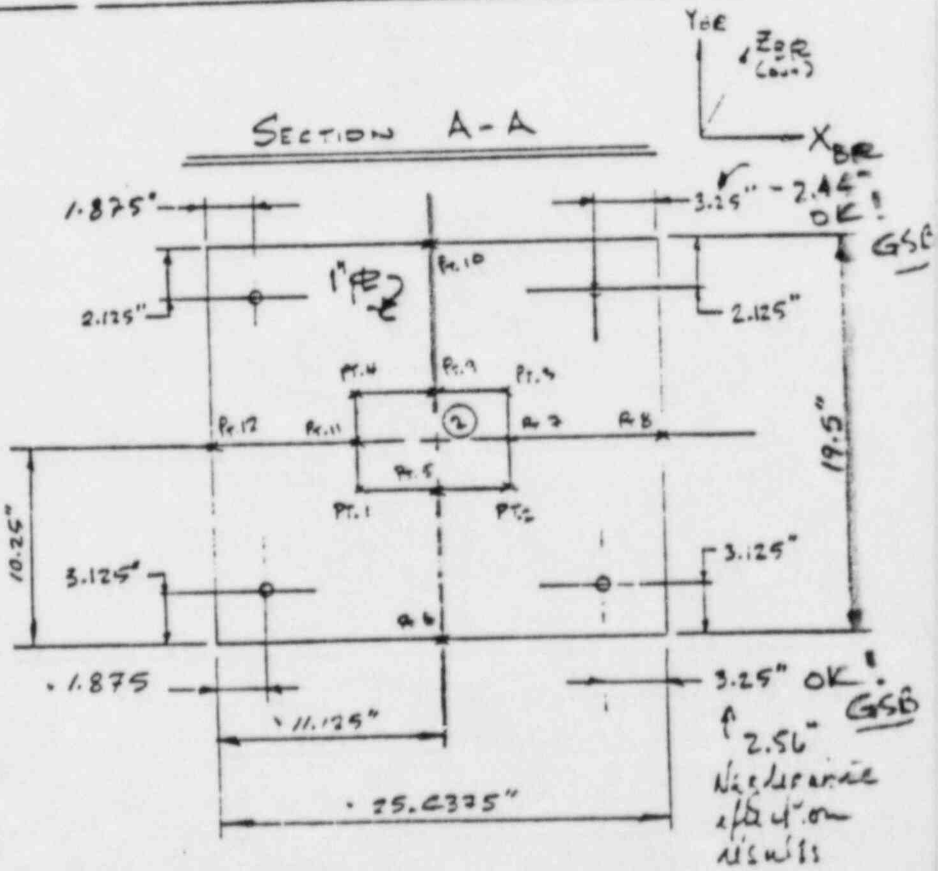
Calc. For
BP's 1 & 2



Support Loadings @ 2

Force (lbs)	Moments (in-lbs)
$F_{x_s} = 1982$	$M_{x_s} = 10302$
$F_{y_s} = 3641$	$M_{y_s} = 6822$
$F_{z_s} = 816$	$M_{z_s} = 61240$





Locations From Left hand Bottom Corner

Pt. 1	X = 8.375"	Y = 8.5"	Pt. 9	X = 11.125"
Pt. 2	X = 13.875"	Y = 8.5"	Pt. 10	X = 11.125"
Pt. 3	X = 13.875"	Y = 12"	Pt. 11	X = 8.375"
Pt. 4	X = 8.375"	Y = 12"	Pt. 12	X = 0.0"
Pt. 5	X = 11.125"	Y = 8.5"	Pt. 9	Y = 12.0"
Pt. 6	X = 11.125"	Y = 0"	Pt. 10	Y = 19.5"
Pt. 7	X = 13.875"	Y = 10.25"	Pt. 11	Y = 10.25"
Pt 8	X = 25.4375"	Y = 10.25"	Pt. 12	Y = 10.25"

Baseplate Loadings @ 2

Force (lbs)	Moments (in-lbs)
$F_{Y_{BR}} = 816^{\#}$	$M_{Y_{BR}} = 61240$ in-lb
$F_{X_{BR}} = 3641^{\#}$	$M_{X_{BR}} = 6822$ in-lb
$F_{Z_{BR}} = 1982^{\#}$	$M_{Z_{BR}} = 10302$ in-lb

Location of 2:
X = 11.125" Y = 10.25"

*. TUSI PLATE NO. 1 FOR AB-1-70

PROGRAM BPLATE
VERSION 2.0

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : TUE, 06 MAR 1984
11:24:19

* SI-1-037-005-S32A NO STIFFENERS

INPUT VALUES

INPUT BY: R.G. Johnson DATE: 3-7-84
CHECKED BY: [illegible] DATE: 3-12-84

PLATE : X-DIM = 25.4400 IN. Y-DIM = 19.5000 IN. THICKNESS = 1.0000 IN.
E = 0.2900 08 PSI NAU = 0.3000 ALLOWABLE STRESS= 0.2700E 05 (NOTE)
F.E. MESH : NEL-X = 8 NEL-Y = 8 REGULAR MESH? NO ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = 0.6250 06 POISSON*S RATIO =-0.30 (IKONK =0

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	1.875
2	3.250
3	3.250
4	2.755
5	2.745
6	4.156
7	4.156
8	3.252

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.125
2	2.688
3	2.688
4	1.748
5	1.752
6	2.687
7	2.687
8	2.125

BOLTS : SET = 1 DIAM = 0.0000 IN. EFF Y.P. = 0.235E 06 PSI
TOISE= 0.0000 IN. BLEN= 1.0000 IN. EFF G= 0.500E 06 PSI PLOAD= 0.000
ALLOWABLE NORMAL STRESS/LOAD= 0.130E 05 ALLOWABLE SHEAR STRESS /LOAD= 0.830E
NUMBER OF BOLTS = 4

BOLT NUMBER	GENERATED VALUES X-COORDINATE	GENERATED VALUES Y-COORDINATE	GRID POINTS X-GRID	GRID POINTS Y-GRID	NODE NUMBER	MATER TY
1	1.875	3.125	2	2	11	
2	22.19 ok	3.125	[illegible]	[illegible]	17	

3	22.19 ok	17.38	.	.	71	1
4	1.875	17.38	7	)	65	1

NUMBER OF SECTIONS = 1

SECTION REFERENCE POINT

SECTION NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	RIGID	TYPE
1	11.13	10.25	5	5	1	99

LINE SEGMENTS OF THIS SECTION ARE LOCATED AT -

------------- END I X-COORDINATE	------------- Y-COORDINATE	------------- END J X-COORDINATE	------------- Y-COORDINATE
8.3753	8.5010	13.876	8.5010
13.876	8.5010	13.876	12.001
13.876	12.001	8.3753	12.001
8.3753	12.001	8.3753	8.5010

LOAD INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

SEC	X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
1	0.364E 04	0.816E 03	0.198E 04	0.652E 04	0.612E 05	0.103E 05	0.000F

PROGRAM EFLATE
VERSICN 2.C

CYGNA ENERGY SERVICES
1C1 CALIFORNIA STREET
SAN FRANCISCC, CALIF. 94111

DATE : TUE, C6 MAR 198
11:24:19

* *
* SI-1-037-C05-S37A
* *

PLATE STRESS SUMMARY

MINIMUM Z-DISF=-C.933E-C4 (NCDE= 34) MAXIMUM Z-DISF= C.166E-C1 (NCDE= 39

MAX EQUIVALENT STRESS FOR BASEFLATE = C.930E 04 FSI (AT ELEMENT = 35)
ALLOWABLE EQUIVALENT STRESS FOR EASEFLATE= C.270E C5

BASEPLATE FRCVISION RATIC = $\frac{\text{C.9296E C4}}{\text{C.2700E C5}}$ = C.3443E CC

BOLT SUMMARY

BOLT FRCVISION RATIC = $\frac{T}{TA} + \frac{V}{VA}$

WHERE T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND V AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

BOLT FORCES AND STRESSES

NUMBER	T	TA	V	VA	PROVISION RATIO
1	C.19C1E C4	C.13CCE C5	C.1C25E C4	C.83CCE C4	C.2698E CC
2	C.0CCCE CC	C.13CCE C5	C.1C78E C4	C.83CCE C4	C.1299E CC
3	C.624CE C2	C.13CCE C5	C.8852E C3	C.83CCE C4	C.1115E CC
4	C.2259E C4	C.13CCE C5	C.7942E C3	C.83CCE C4	C.2694E CC

CYGNA

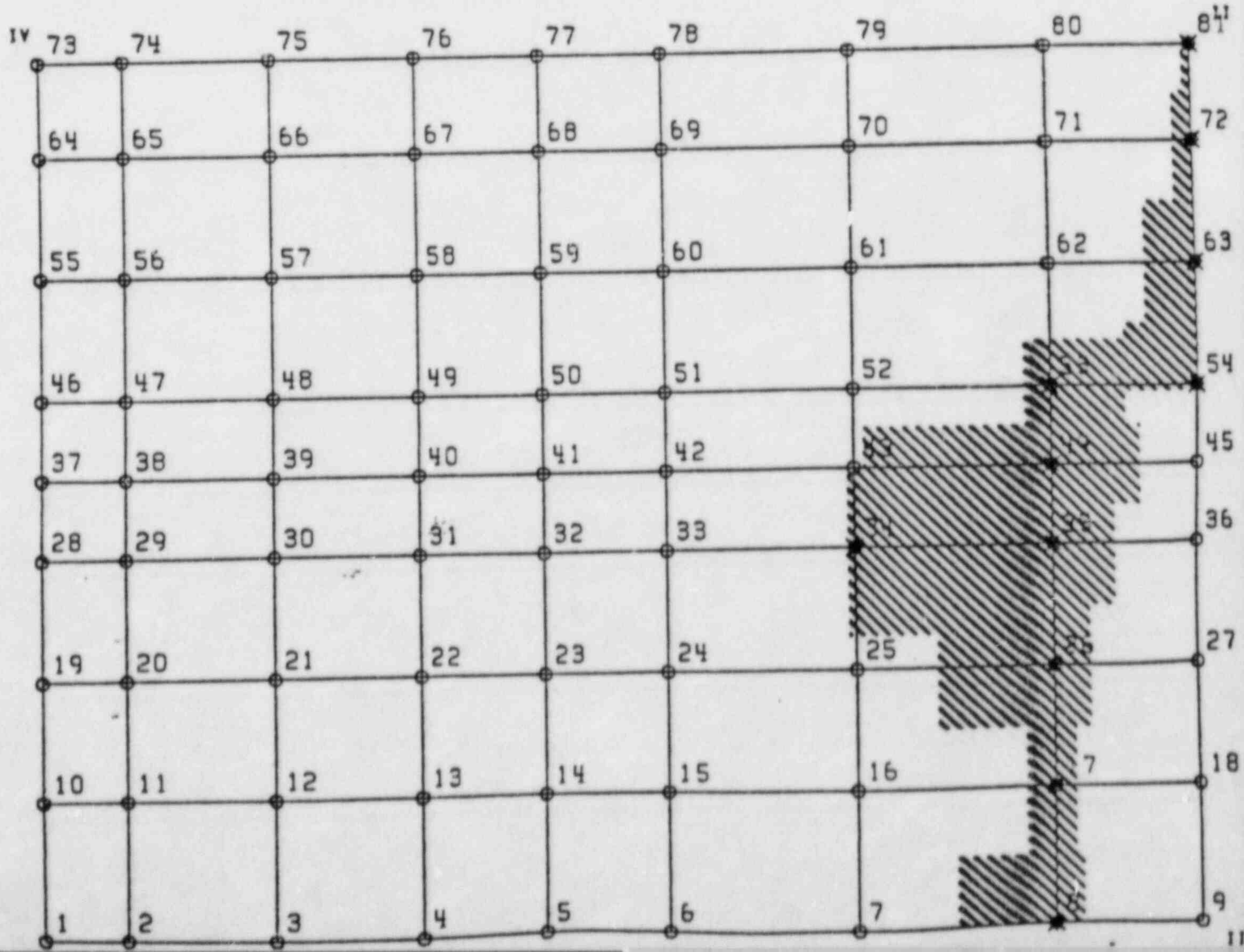
PROJECT TUSI
TITLE BASEPLATE ANAL.
PREPARED BY RGJ
DATE 3-7-84
CHECKED BY GSB
DATE 3-12-84
JOB NO. 84042

TUSI PL No. 1 FOR AB-1-70
SUPPORT No. SI-1-037-005-S32A
(NO STIFFENERS)

57	58	59	60	61	62	63	64
49	50	51	52	53	54	55	56
41	42	43	44	45	46	47	48
33	34	35	36	37	38	39	40
25	26	27	28	29	30	31	32
17	18	19	20	21	22	23	24
9	10	11	12	13	14	15	16
1	2	3	4	5	6	7	8

CYGNA
PROJECT TUSI
TITLE BASEPLATE ANAL.
PREPARED BY RGS
DATE 3-7-84
CHECKED BY GSB
DATE 3-12-84
84042

TUSI PL No. 1 FOR AB-1-70
SUPPORT No. SI-1-037-005-S32A
(NO STIFFENERS)



CYGNA

PROJECT TUSI

BASEPLATE ANAL

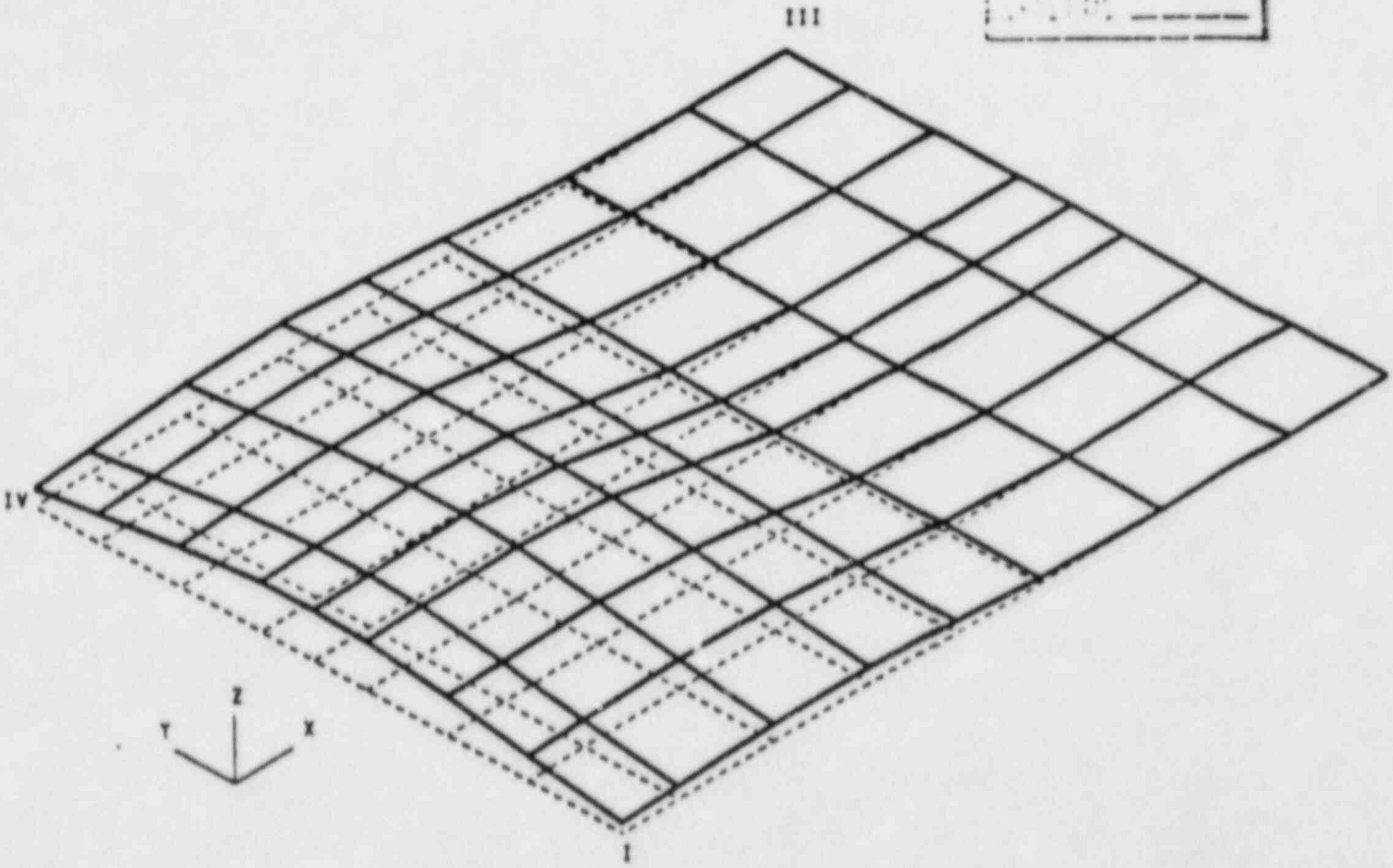
RGJ

3-7-84

GSB

3-12-84

84042



TUSI PL No. 1 FOR AB-1-70

SUPPORT No. SI-1-037-005-S32A

(NO STIFFENERS)

RFG 2.C (PRIME) WED, C7 MAR 1984 14:00:17 USER: R.G.JOHNSON PAGE
CLIENT: TUSI TITLE: "TUSI" BASEPLATE ANALYSIS

2. TUSI PLATE NO. 2 FOR AB-1-70

PROGRAM BPLATE CYGNA ENERGY SERVICES DATE : TUE, C6 MAR 1984
VERSION 2.C 101 CALIFORNIA STREET 11:55:30
SAN FRANCISCO, CALIF. 94111

* SI-1-C37-005-S32A WITH STIFFENERS *

INPUT VALUES INPUT BY: RGJohnson DATE: 3-7-84
CHECKED BY: [illegible] DATE: 3-12-84

PLATE : X-DIM = 25.4400 IN. Y-DIM = 19.5000 IN. THICKNESS = 1.0000 IN.
E = 0.290E 08 PSI VNU = 0.3000 ALLOWABLE STRESS= 0.2700E 05
F.E. MESH: NEL-X = 8 NEL-Y = 8 REGULAR MESH? NO ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = 0.625E 06 POISSON'S RATIO = 0.30 (IFONK =0)

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	1.875
2	3.255
3	3.255
4	2.782
5	2.747
6	4.156
7	4.156
8	3.252

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.125
2	2.688
3	2.688
4	1.750
5	1.750
6	2.687
7	2.687
8	2.125

BOLTS : SET = 1 DIAM = 0.0000 IN. EFF Y.P. = 0.235E 06 PSI
TOISF= 0.0000 IN. BLEN= 1.0000 IN. EFF G= 0.500E 06 PSI PLOAD= 0.000
ALLOWABLE NORMAL STRESS/LOAD= 0.130E 05 ALLOWABLE SHEAR STRESS /LOAD= 0.830E
NUMBER OF BOLTS = 4

BOLT NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	NODE NUMBER	MATERIAL TYPE
1	1.875	3.125	2	2	11	1
2	22.19 OK	3.125	8	2	17	1

3	22.19	17.38	6	6	71	1
4	1.875	17.38	7	6	65	1

NUMBER OF SECTIONS = 1

SECTION REFERENCE POINT

SECTION NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	RIGID	TYPE
1	11.13	10.25	5	5	1	99

LINE SEGMENTS OF THIS SECTION ARE LOCATED AT -

------------ END I ------------ X-COORDINATE	Y-COORDINATE	------------ END J ------------ X-COORDINATE	Y-COORDINATE
8.3757	8.5010	13.875	8.5010
13.875	8.5010	13.875	12.001
13.875	12.001	8.3757	12.001
8.3757	12.001	8.3757	8.5010
11.128	8.5010	11.128	0.00000
13.875	10.251	25.440	10.251
11.128	12.001	11.128	19.500
8.3757	10.251	0.00000	10.251

LOAD INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

SEC	X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
1	0.364E 04	0.816E 03	0.198E 04	0.682E 04	0.612E 05	0.103E 05	0.000E

PROGRAM BPLATE
VERSION 2.C

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : TUE, C6 MAR 1984
11:55:30

* *
* SI-1-037-005-S32A
* *

PLATE STRESS SUMMARY

MINIMUM Z-DISP=-C.434E-C3 (NODE= 45) MAXIMUM Z-DISP= C.151E-C1 (NODE= 37)

MAX EQUIVALENT STRESS FOR BASEPLATE = C.661E C4 PSI (AT ELEMENT = 33)
ALLOWABLE EQUIVALENT STRESS FOR BASEPLATE= C.27CCE C5

BASEPLATE PROVISION RATIO = $\frac{C.6612E\ 04}{C.27CCE\ C5}$ = C.2449E CC

BOLT SUMMARY

BOLT PROVISION RATIO = $\frac{T}{TA} + \frac{V}{VA}$

WHERE T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND V AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

BOLT FORCES AND STRESSES

NUMBER	T	TA	V	VA	PROVISION RATIO
1	C.17C6E C4	C.13CCE C5	C.1C22E C4	C.83CCE C4	C.2544E CC
2	C.2446E C3	C.13CCE C5	C.1C82E C4	C.83CCE C4	C.1492E CC
3	C.4572E C3	C.13CCE C5	C.8794E C3	C.83CCE C4	C.1411E CC
4	C.2CC3E C4	C.13CCE C5	C.7989E C3	C.83CCE C4	C.25C3E CC

CYGNA
TUSI
BASEPLAT ANAL.
RGJ
3-7-84
GSE
3-12-84
B4042
FILE NO. ______
SHEET NO. ______

TUSI TR No. 2 FOR AB-1-70
SUPPORT No. SI-1-037-005-S32A
(WITH STIFFENERS)

57	58	59	60	61	62	63	64
49	50	51	52	53	54	55	56
41	42	43	44	45	46	47	48
33	34	35	36	37	38	39	40
25	26	27	28	29	30	31	32
17	18	19	20	21	22	23	24
9	10	11	12	13	14	15	16
1	2	3	4	5	6	7	8

CYGNA

TOS I

BASEPLATE ANAL

RGT

3-7-84

GSB

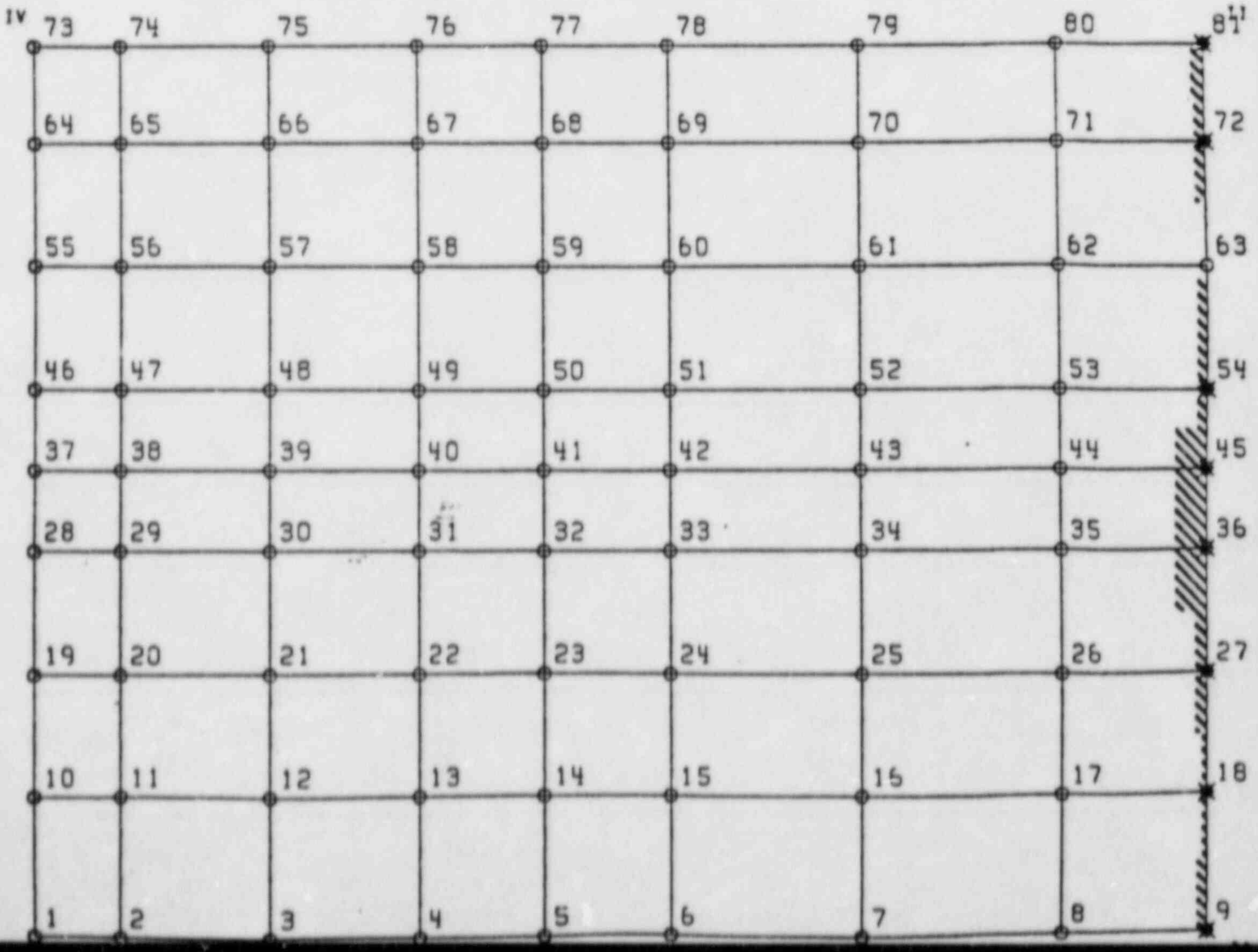
3-12-84

B4042

TOSI PL No. 2 FOR AB-1-70

SUPPORT No. SI-1-037-005-S32A

(WITH STIFFENERS)



CYGNA

TUSI

BASEPLATE ANAL.

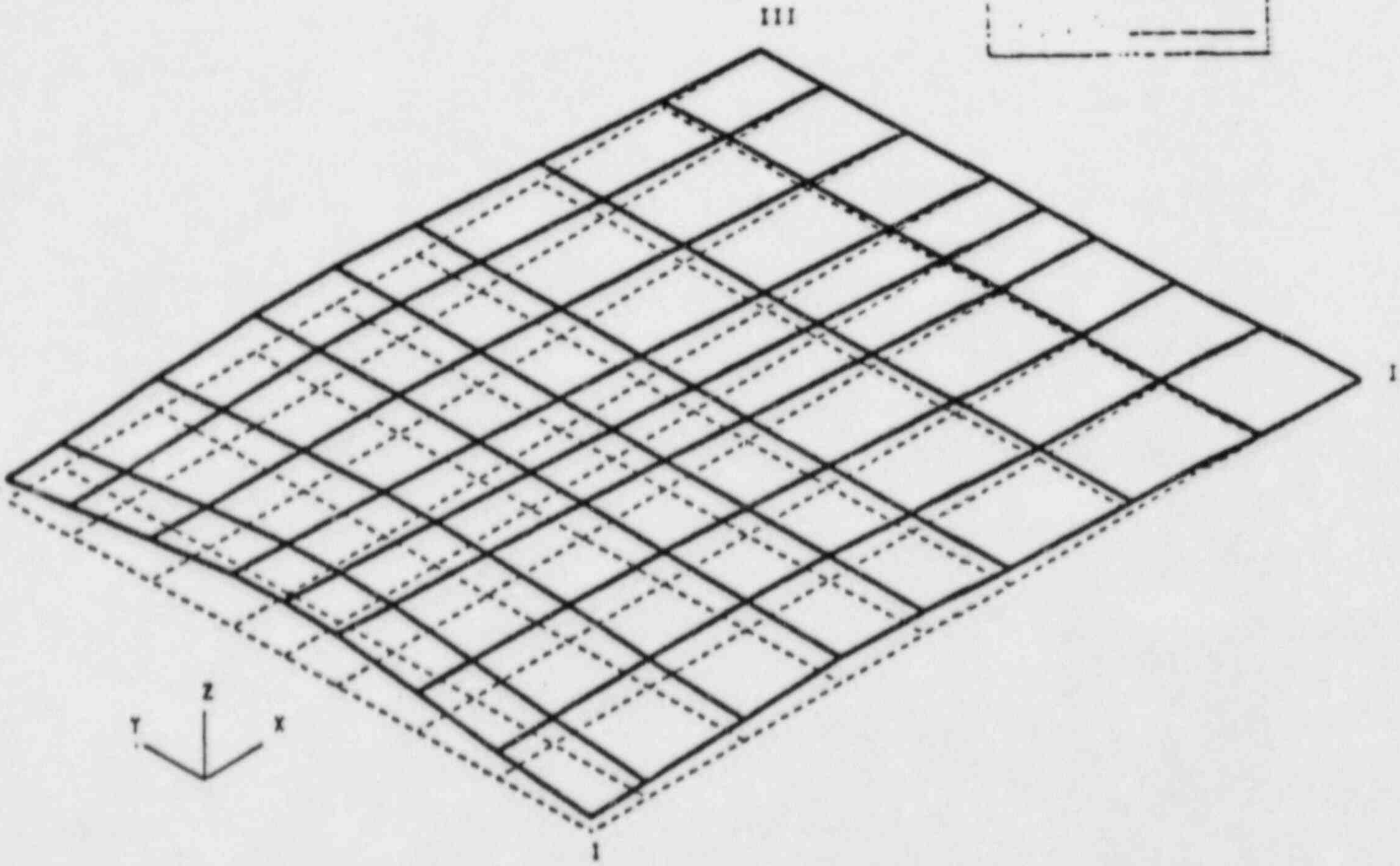
RGJ

3-7-84

GSB

3-12-84

840A2



TUSI PL NO. 2 FOR AB-1-70

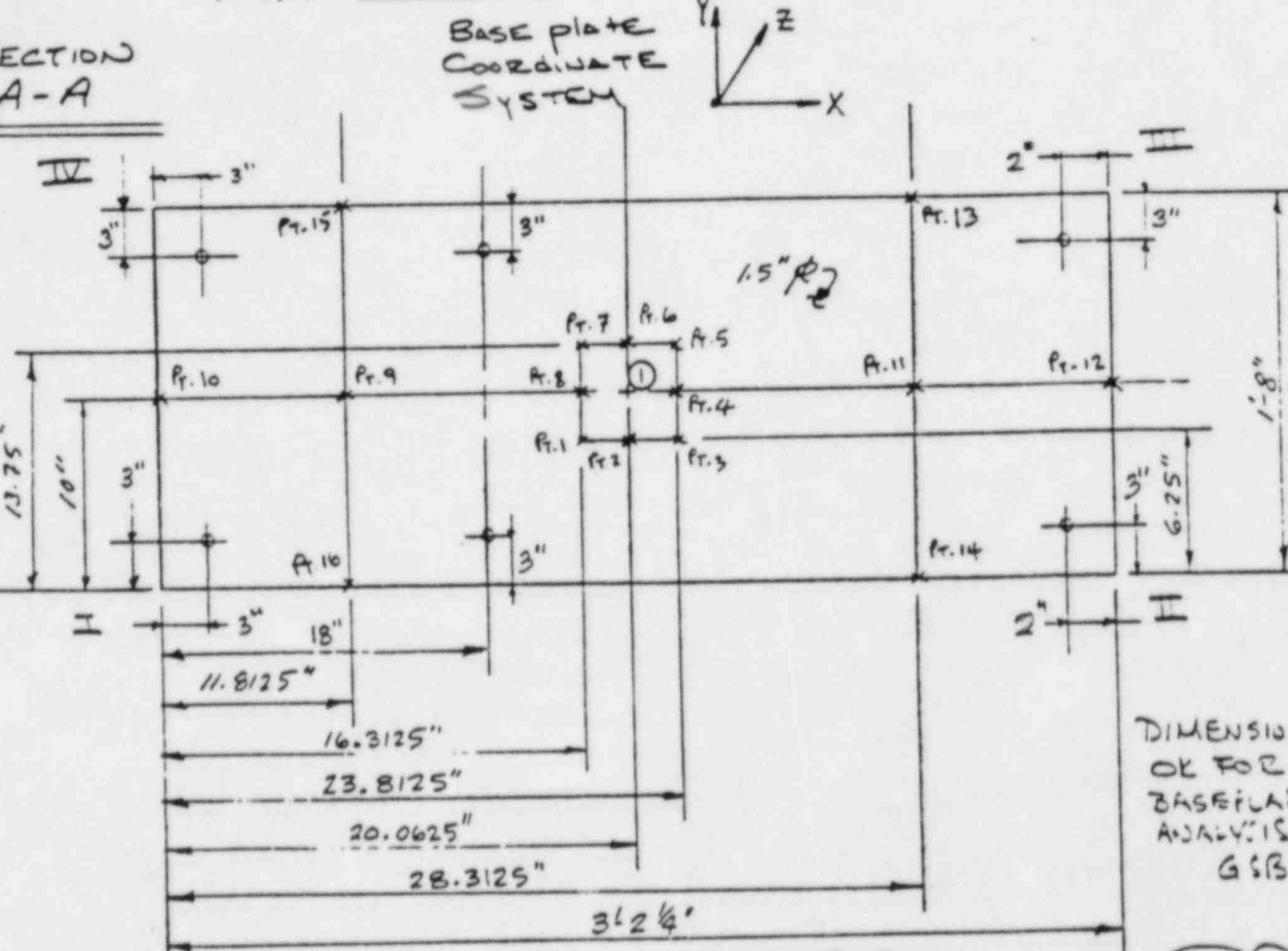
SUPPORT NO. SI-1-037-005-S32A

(WITH STIFFENERS)

EPLATE: DEFORMED SHAPE PLOTS

CYGNA	Calculation Sheet	Prepared By: RG [signature]	Date: 3-6-84
		Checked By: [signature]	Date: 3-12-84
	Project: C. P. S. E. STATION	Job No: 84042	File No:
	Subject: BASEPLATE ANALYSIS		
	System: RH-1-024-011-S22A	Sheet No: 2 of 2	
	Analysis No: AB-1-71A Rev No: 0		

SECTION A-A



DIMENSIONS OK FOR BASEPLATE ANALYSIS GSB

BOLT LOCATIONS FROM CORNER

BOLT #1	X = 3"	Y = 3"	FROM 1
" #2	X = 18"	Y = 3"	FROM 1
" #3	X = 2"	Y = 3"	FROM 2
" #4	X = 2"	Y = 3"	FROM 3
" #5	X = 18"	Y = 3"	FROM 4
" #6	X = 3"	Y = 3"	FROM 4

PT. 14	X = 28.3125"	Y = 0.0"
PT. 15	X = 11.8125"	Y = 20"
PT. 16	X = 11.8125"	Y = 0.0"

POINT LOCATIONS FROM CORNER I

PT. 1	X = 16.3125"	Y = 6.25"
PT. 2	X = 20.0625"	Y = 6.25"
PT. 3	X = 23.8125"	Y = 6.25"
PT. 4	X = 23.8125"	Y = 10"
PT. 5	X = 23.8125"	Y = 13.75"
PT. 6	X = 20.0625"	Y = 13.75"
PT. 7	X = 16.3125"	Y = 13.75"
PT. 8	X = 16.3125"	Y = 10"
PT. 9	X = 11.8125"	Y = 10"
PT. 10	X = 0.0"	Y = 10"
PT. 11	X = 28.3125"	Y = 10"
PT. 12	X = 38.25"	Y = 10"
PT. 13	X = 28.3125"	Y = 20"

Calculation Sheet	Prepared By: RS J...	Date: 3-6-84
Project: C.P.S.E. Station	Checked By: [illegible signature]	Date:
Subject: Baseplate Analysis	Job No: 84042	File No:
System: RH-1-024-011-S22A	Sheet No: 1 of 2	
Analysis No: AB-1-71A Rev No: 0		

Support Loadings at Attachment Location :—

Force (lbs)   Moments (in-lbs)

$F_{X_S} = 2620$ ok $M_{X_S} = 207336$

$F_{Y_S} = 5326$ $M_{Y_S} = 72239$

$F_{Z_S} = 4143$ $M_{Z_S} = 129712$

Baseplate Loadings at Attachment Location :—

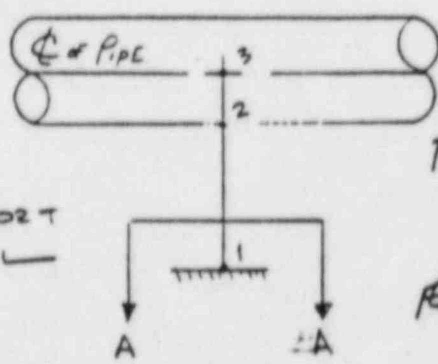
Force (lbs)   Moments (in-lbs)

$F_{X_{BP}} = 5326$ $M_{X_{BP}} = 72239$

$F_{Y_{BP}} = 4143$ $M_{Y_{BP}} = 129712$ $\left\{ M_{Y_{BP}} = -129712 \right.$

$F_{Z_{BP}} = 2620$ $M_{Z_{BP}} = 207336$

Support Model



Four Baseplate Models were Run :-

℄ (1) AB-1-71A.BP1.EPL (without Stiff's and Positive $M_{Y_{BP}}$ Loading)

℄ (2) AB-1-71A.BP2.EPL (with Stiff.'s and Positive $M_{Y_{BP}}$ Loading)

℄ (3) AB-1-71A.BP3.EPL (without Stiff's and Negative $M_{Y_{BP}}$ Loading)

℄ (4) AB-1-71A.BP4.EPL (with Stiff.'s and Negative $M_{Y_{BP}}$ Loading)

3. TUSI PLATE NO. 1 FOR AB-1-71A

PROGRAM EPLATE
VERSION 2.C

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : WED, C7 MAR 1984
11:21:35

* *
* RH-1-024-C11-S22A No STIFFENERS *
* *

INPUT VALUES

INPUT BY: RCJ... DATE: 3-7-84
CHECKED BY: [signature] DATE: 3-12-84

FLATE : X-DIM = 38.2500 IN. Y-DIM = 20.0000 IN. THICKNESS = 1.5000 IN.
E = C.2900E 08 PSI NU = C.3000 ALLOWABLE STRESS= C.2700E 05
F.E. MESH: NEL-X = 15 NEL-Y = 10 REGULAR MESH? NC ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = C.625E 06 POISSON'S RATIO = C.30 (IKONK =0)

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	3.000
2	2.938
3	2.938
4	2.938
5	2.250
6	2.250
7	1.650
8	2.061
9	3.752
10	2.249
11	2.249
12	2.646
13	2.646
14	2.650
15	2.000

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.000
2	1.625
3	1.625
4	1.750
5	2.000
6	2.000
7	1.750
8	1.625
9	1.625
10	3.000

: SET = 1 DIAM = C.CCCC IN. EFF Y.P. = 0.235E 06 PSI
C.CCCC IN. EFLEN= 1.CCCC IN. EFF G= C.5CCE C6 PSI PLCAD= C.CCC LF
BLE NORMAL STRESS/LCAD= 0.130E C5 ALLOWABLE SHEAR STRESS /LCAD= C.830E C
OF BOLTS = 6

----- GENERATED VALUES -----		--GRID POINTS--		NODE	MATERIAL
X-COORDINATE	Y-COORDINATE	X-GRID	Y-GRID	NUMBER	TYPE
3.CCO	3.CCO	2	2	18	1
18.CC	3.CCO	8	2	24	1
36.25	3.CCO	15	2	31	1
36.25	17.CC	15	10	159	1
18.CC	17.CC	8	10	152	1
3.CCO	17.CC	2	10	146	1

OF SECTIONS = 1

N REFERENCE POINT

----- GENERATED VALUES -----		--GRID POINTS--		RIGID	TYPE
X-COORDINATE	Y-COORDINATE	X-GRID	Y-GRID		
20.06	10.CC	4	6	1	99

SEGMENTS OF THIS SECTION ARE LOCATED AT -

-------- END I -------------		------------- END J -------------	
X-COORDINATE	Y-COORDINATE	X-COORDINATE	Y-COORDINATE
12	6.25CC	20.060	6.25CC
60	6.25CC	23.812	6.25CC
12	6.25CC	23.812	10.CCC
12	10.CCC	23.812	13.750
12	13.750	20.060	13.750
60	13.750	16.312	13.750
12	13.750	16.312	10.CCC
12	10.CCC	16.312	6.25CC

INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
.532E C4	0.414E C4	-C.262E C4	C.722E C5	C.130E C6	-C.207E C6	-C.CCOE

PROGRAM BPLATE
VERSION 2.C

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : WED, 07 MAR 1984
11:21:35

* *
* RH-1-024-011-S22A
* *

PLATE STRESS SUMMARY

MINIMUM Z-DISP=-0.376E-03 (NODE= 11) MAXIMUM Z-DISP= 0.195E-01 (NODE= 166)

MAX EQUIVALENT STRESS FOR BASEPLATE = 0.846E 04 PSI (AT ELEMENT = 96)
ALLOWABLE EQUIVALENT STRESS FOR BASEPLATE= 0.2700E 05

$$\text{BASEPLATE PROVISION RATIO} = \frac{0.8461E\ 04}{0.2700E\ 05} = 0.3134E\ 00$$

BOLT SUMMARY

$$\text{BOLT PROVISION RATIO} = \frac{T}{TA} + \frac{V}{VA}$$

WHERE T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND V AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

BOLT FORCES AND STRESSES

NUMBER	T	TA	V	VA	PROVISION RATIO
1	0.1172E 04	0.1300E 05	0.2598E 04	0.8300E 04	0.4032E 00
2	0.1264E 04	0.1300E 05	0.2088E 04	0.8300E 04	0.3487E 00
3	0.0000E 00	0.1300E 05	0.3773E 04	0.8300E 04	0.4546E 00
4	0.2382E 03	0.1300E 05	0.3261E 04	0.8300E 04	0.4113E 00
5	0.3663E 04	0.1300E 05	0.5856E 03	0.8300E 04	0.3523E 00
6	0.2511E 04	0.1300E 05	0.1738E 04	0.8300E 04	0.4025E 00

CYGNA

TUSI

BASEPLATE ANAL

RGJ

3-7-84

GSB

3-12-84

84042

TUSI PL No. 1 FOR AB-1-71A

SUPPORT No. RH-1-024-011-S22A

(NO STIFFENERS)

136	137	138	139	140	141	142	143	144	145	146	147	148	149	150
121	122	123	124	125	126	127	128	129	130	131	132	133	134	135
106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
91	92	93	94	95	96	97	98	99	100	101	102	103	104	105
76	77	78	79	80	81	82	83	84	85	86	87	88	89	90
61	62	63	64	65	66	67	68	69	70	71	72	73	74	75
46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

CYGNA

TUSI

BASEPLATE ANLL.

RGJ

3-7-84

GSB

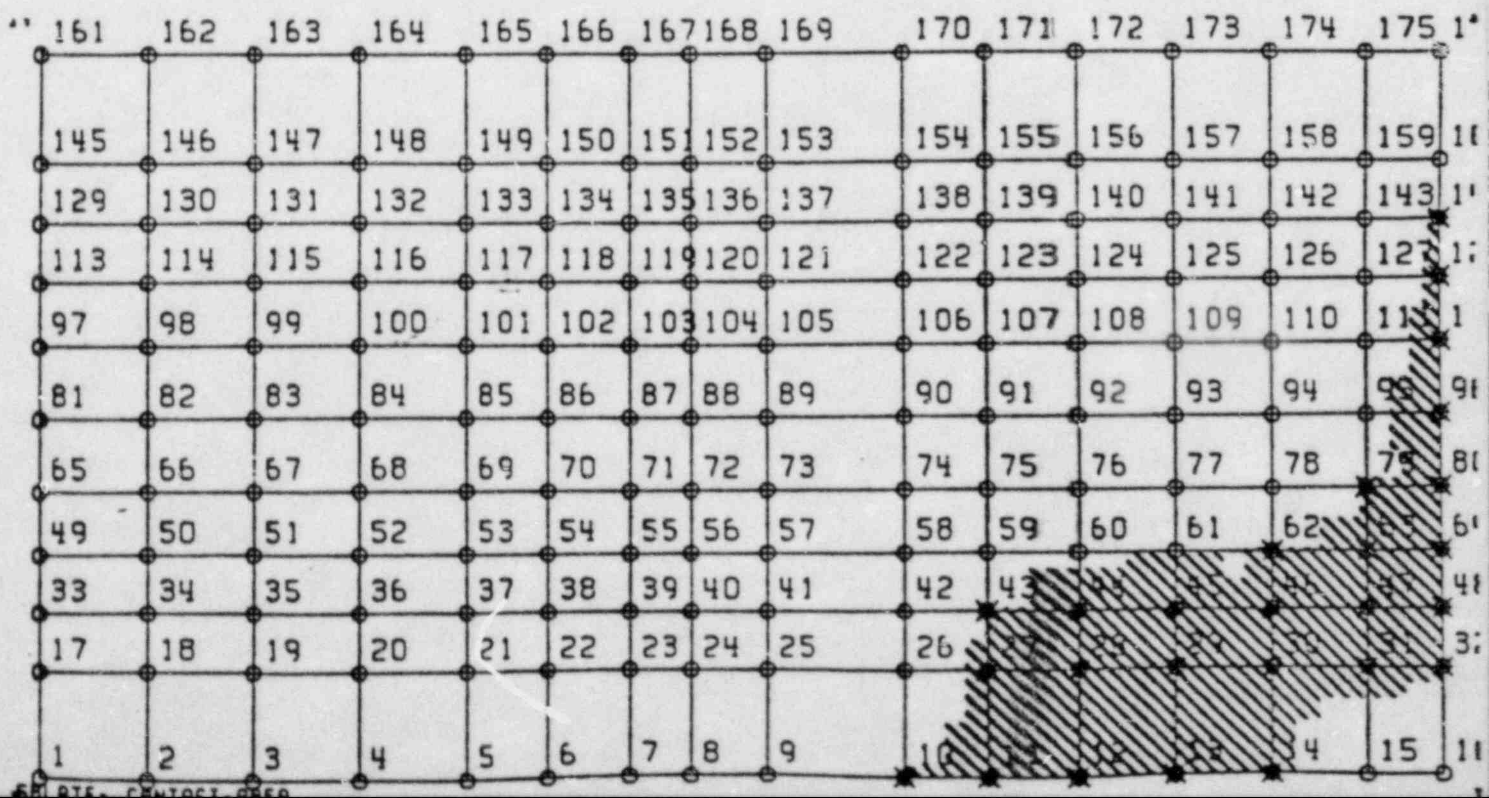
3-12-84

84042

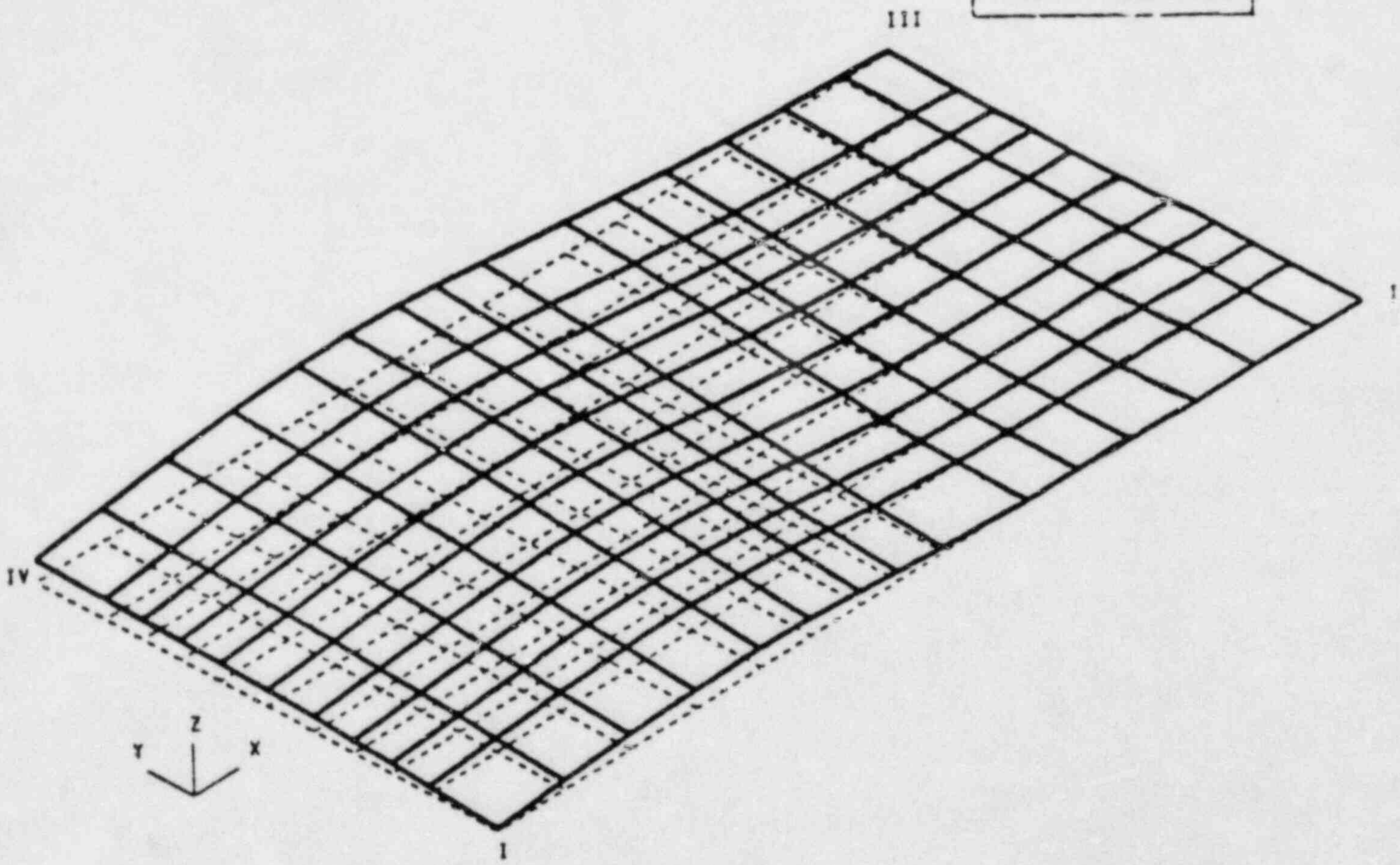
TUSI PL No. 1 FOR AB-1-71A

SUPPORT No. RH-1-024-011-S22A

(NO STIFFENERS)



CYGNA
TUSI
BASEPLATE ANAL
RGI
3-7-84
GSE
3-12-84
84042



TUSI TE No. 1 FOR AB-1-71A
SUPPORT No. RH-1-024-011-S22A
(NO STIFFENERS)

EPLATE: DEFORMED SHAPE PLOTS

4. TUSI PLATE NO. 2 FOR AB-1-71A

PROGRAM BPLATE
VERSION 2.0

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : WED, 07 MAR 1984
10:43:21

* *
* RH-1-024-011-S22A WITH STIFFENERS *
* *

INPUT VALUES

INPUT BY: RG .... DATE: 3-7-84
CHECKED BY: R Bornemann DATE: 3-12-84

PLATE : X-DIM = 38.2500 IN. Y-DIM = 20.0000 IN. THICKNESS = 1.5000 IN.
E = 0.290E 08 PSI NU = 0.3000 ALLOWABLE STRESS= 0.2700E 05
F.E. MESH: NEL-X = 15 NEL-Y = 10 REGULAR MESH? NO ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = 0.425E 06 POISSON*S RATIO = 0.30 (IWONK =0)

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	3.000
2	2.936
3	2.938
4	2.935
5	2.250
6	2.250
7	1.655
8	2.061
9	3.752
10	2.749
11	2.749
12	2.649
13	2.646
14	2.645
15	2.000

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.000
2	1.625
3	1.625
4	1.750
5	2.000
6	2.000
7	1.750
8	1.625
9	1.625
10	3.000

BOLTS : SET = 1 DIAM = 0.0000 IN. EFF Y.M. = 0.235E 06 PSI
TDISP= 0.0000 IN. EFLEN= 1.0000 IN. EFF G= 0.500E 06 PSI PLOAD= 0.000 LE
ALLOWABLE NORMAL STRESS/LOAD= 0.130E 05 ALLOWABLE SHEAR STRESS /LOAD= 0.830E 0
NUMBER OF BOLTS = 6

BOLT NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	NODE NUMBER	MATERIA TYPE
1	3.000	3.000	2	2	18	1
2	18.00	3.000	[illegible]	2	24	1
3	36.25	3.000	15	2	31	1
4	36.25	17.00	15	10	159	1
5	18.00	17.00	8	10	152	1
6	3.000	17.00	2	10	146	1

NUMBER OF SECTIONS = 1

SECTION REFERENCE POINT

SECTION NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	RIGID	TYP
1	20.06	10.00	8	6	1	99

LINE SEGMENTS OF THIS SECTION ARE LOCATED AT -

------------- END I ------------- X-COORDINATE	Y-COORDINATE	------------- END J ------------- X-COORDINATE	Y-COORDINATE
16.311	6.2500	20.060	6.2500
20.060	6.2500	23.812	6.2500
23.812	6.2500	23.812	10.000
23.812	10.000	23.812	13.750
23.812	13.750	20.060	13.750
20.060	13.750	16.311	13.750
16.311	13.750	16.311	10.000
16.311	10.000	16.311	6.2500
16.311	10.000	11.811	10.000
11.811	10.000	0.00000	10.000
23.812	10.000	28.310	10.000
28.310	10.000	36.250	10.000
28.310	10.000	28.310	20.000
28.310	10.000	28.310	0.00000
11.811	10.000	11.811	20.000
11.811	10.000	11.811	0.00000

LOAD INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

SEC	X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
1	0.533E 04	0.414E 04	0.262E 04	0.722E 05	0.120E 06	0.207E 06	0.000E

PROGRAM BPLATE CYGNA ENERGY SERVICES DATE : WED, 07 MAR 1984
VERSION 2.0 101 CALIFORNIA STREET 10:43:21
SAN FRANCISCO, CALIF. 94111

* RH-1-024-011-S22A

PLATE STRESS SUMMARY

MINIMUM Z-DISP=-0.706E-03 (NODE= 12) MAXIMUM Z-DISP= 0.125E-01 (NODE= 164)

MAX EQUIVALENT STRESS FOR BASEPLATE = 0.362E 04 PSI (AT ELEMENT = 12)
ALLOWABLE EQUIVALENT STRESS FOR BASEPLATE= 0.2700E 05

$$\text{BASEPLATE PROVISION RATIO} = \frac{0.3616E\ 04}{0.2700E\ 05} = 0.1339E\ 00$$

BOLT SUMMARY

$$\text{BOLT PROVISION RATIO} = \frac{T}{TA} + \frac{V}{VA}$$

WHERE T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND V AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

BOLT FORCES AND STRESSES

NUMBER	T	TA	V	VA	PROVISION RATIO
1	0.1575E 04	0.1300E 05	0.2597E 04	0.8300E 04	0.4340E 00
2	0.8044E 03	0.1300E 05	0.2024E 04	0.8300E 04	0.3058E 00
3	0.0000E 00	0.1300E 05	0.3797E 04	0.8300E 04	0.4574E 00
4	0.7734E 03	0.1300E 05	0.3277E 04	0.8300E 04	0.4543E 00
5	0.2106E 04	0.1300E 05	0.5570E 03	0.8300E 04	0.2291E 00
6	0.2711E 04	0.1300E 05	0.1741E 04	0.8300E 04	0.4183E 00

CYGNA

TUSI

BASE PLATE ANAL

RGJ

3-7-84

GSB

3-12-84

84042

TUSI TR No. 2 FOR AB-1-71A

SUPPORT No. RH-1-024-011-S22A

(WITH STIFFENERS)

136	137	138	139	140	141	142	143	144	145	146	147	148	149	150
121	122	123	124	125	126	127	128	129	130	131	132	133	134	135
106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
91	92	93	94	95	96	97	98	99	100	101	102	103	104	105
76	77	78	79	80	81	82	83	84	85	86	87	88	89	90
61	62	63	64	65	66	67	68	69	70	71	72	73	74	75
46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

SALOIS ELEMENT NUMBERS
RH1-024-011-S22A

CYGNA

PROJECT TUSI

TITLE BASEPLATE ANAL

RGJ

3-7-84

GSB

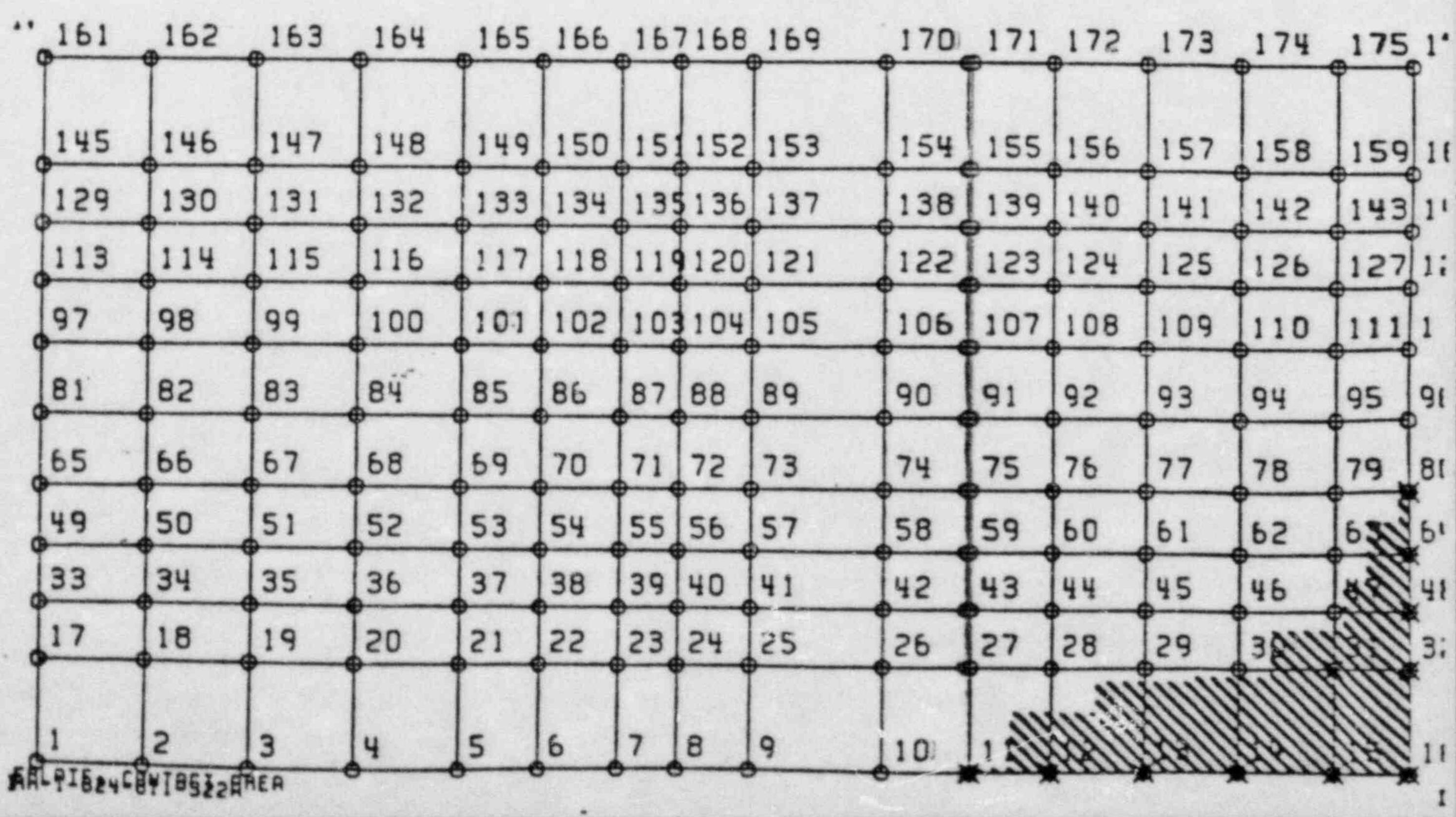
3-12-84

84042

TUSI PR No. 2 FOR AB-1-71A

SUPPORT No. RH-1-024-011-S22A

(WITH STIFFENERS)



CYGNA

TUSI

BASEPLATE ANAL

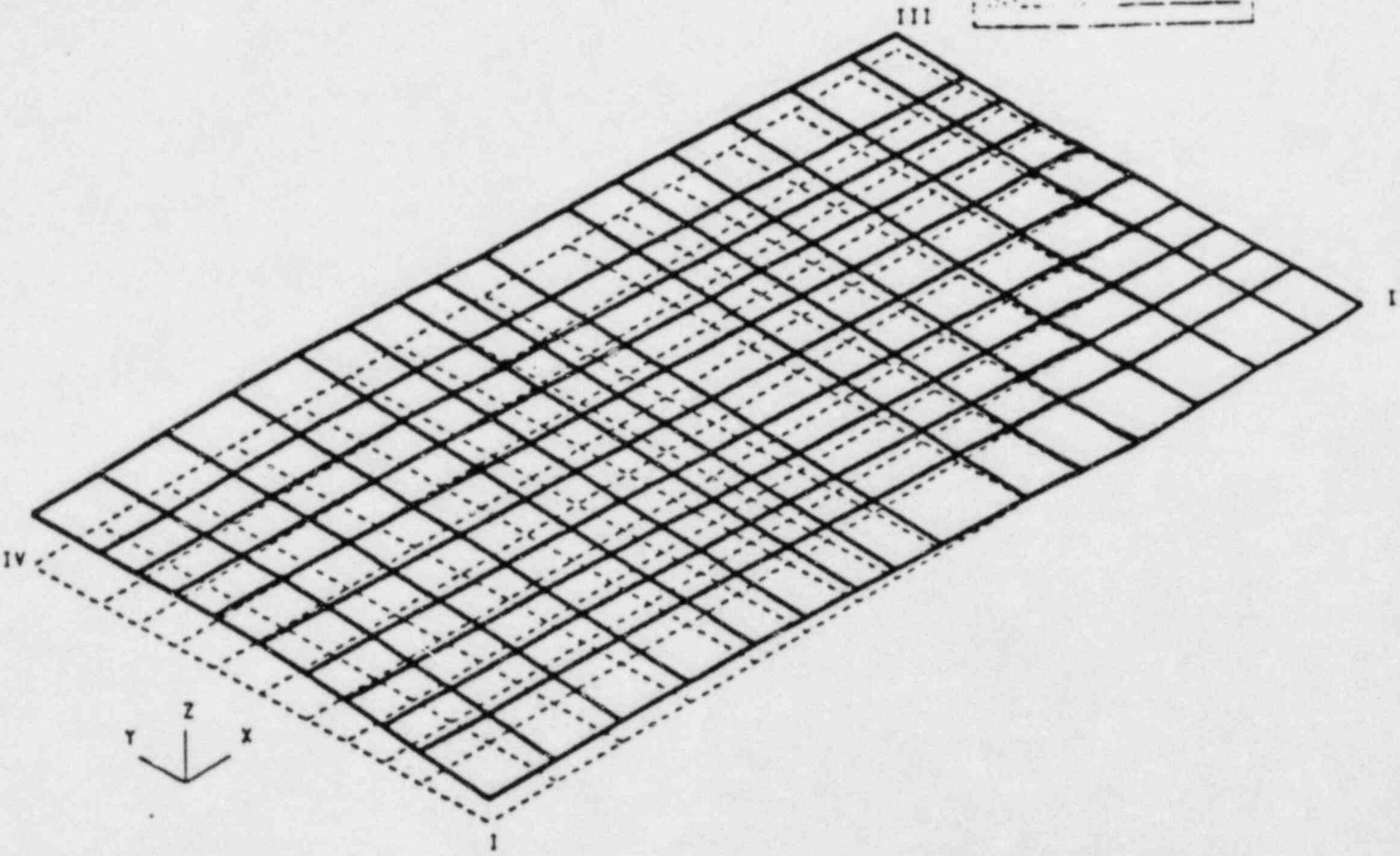
RGJ

3-7-84

GSB

3-12-84

84092



TUSI PL NO. 2 FOR AB-1-71A

SUPPORT NO. RH-1-024-011-S22A

(WITH STIFFENERS)

EPLATE: DEFORMED SHAPE PLOTS
RH-1-024-011-S22A

5. TUSI PLATE NO. 3 FOR AB-1-71A

PROGRAM BPLATE
VERSION 2.C

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : WED, 07 MAR 1984
12:03:11

* *
* PH-1-024-011-S22A NO STIFFENERS *
* *

INPUT VALUES

INPUT BY: RG [signature] DATE: 3-7-84
CHECKED BY: [signature] DATE: 3-12-84

PLATE : X-DIM = 38.2500 IN. Y-DIM = 20.0000 IN. THICKNESS = 1.5000 IN.
E = 0.290E 08 PSI NU = 0.3000 ALLOWABLE STRESS= 0.2700E 05
F.E. MESH: NEL-X = 15 NEL-Y = 10 REGULAR MESH? NO ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = 0.625E 06 POISSON'S RATIO = 0.30 (IKONK =0)

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	3.000
2	2.938
3	2.938
4	2.938
5	2.250
6	2.250
7	1.688
8	2.061
9	3.752
10	2.749
11	2.749
12	2.646
13	2.646
14	2.650
15	2.000

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.000
2	1.625
3	1.625
4	1.750
5	2.000
6	2.000
7	1.750
8	1.625
9	1.625
10	3.000

BOLTS : SET = 1 DIAM = 0.0000 IN. EFF Y.M. = 0.235E 06 PSI
THIST= 0.0000 IN. EFLEN= 1.0000 IN. EFF G= 0.500E 06 PSI PLOAD= 0.000 LE
ALLOWABLE NORMAL STRESS/LOAD= 0.130E 05 ALLOWABLE SHEAR STRESS /LOAD= 0.830E 0
NUMBER OF BOLTS = 6

BOLT NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	NODE NUMBER	MATERIA TYPE
1	3.000	3.000	2	2	18	1
2	18.00	3.000	8	2	24	1
3	36.25	3.000	15	2	31	1
4	36.25	17.00	15	10	159	1
5	18.00	17.00	8	10	152	1
6	3.000	17.00	2	10	146	1

NUMBER OF SECTIONS = 1

SECTION REFERENCE POINT

SECTION NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	RIGID	TYPE
1	20.06	10.00	6	6	1	99

LINE SEGMENTS OF THIS SECTION ARE LOCATED AT -

------------- END I ------------- X-COORDINATE	Y-COORDINATE	------------- END J ------------- X-COORDINATE	Y-COORDINATE
16.312	6.2500	20.060	6.2500
20.060	6.2500	23.812	6.2500
23.812	6.2500	23.812	10.000
23.812	10.000	23.812	13.750
23.812	13.750	20.060	13.750
20.060	13.750	16.312	13.750
16.312	13.750	16.312	10.000
16.312	10.000	16.312	6.2500

LOAD INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

SEC	X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
1	0.533E 04	0.414E 04	0.262E 04	0.722E 05	0.130E 06	0.207E 06	0.000E 0

PROGRAM BPLATE
VERSION 2.0

CYGNA ENERGY SERVICES
101 CALIFORNIA STREET
SAN FRANCISCO, CALIF. 94111

DATE : WED, 07 MAR 1984
12:03:11

* *
* BH-1-024-011-S22A
* *

PLATE STRESS SUMMARY

MINIMUM Z-DISP=-0.366E-03 (NODE= 5) MAXIMUM Z-DISP= 0.225E-01 (NODE= 171)

MAX EQUIVALENT STRESS FOR BASEPLATE = 0.960E 04 PSI (AT ELEMENT = 100)
ALLOWABLE EQUIVALENT STRESS FOR BASEPLATE= 0.2700E 05

$$\text{BASEPLATE PROVISION RATIO} = \frac{0.9797E\ 04}{0.2700E\ 05} = 0.3629E\ 00$$

BOLT SUMMARY

$$\text{BOLT PROVISION RATIO} = \frac{T}{TA} + \frac{V}{VA}$$

WHERE T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND V AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

BOLT FORCES AND STRESSES

NUMBER	T	TA	V	VA	PROVISION RATIO
1	0.0000E 00	0.1300E 05	0.2598E 04	0.8300E 04	0.3131E 00
2	0.5590E 03	0.1300E 05	0.2088E 04	0.8300E 04	0.2945E 00
3	0.1614E 04	0.1300E 05	0.3773E 04	0.8300E 04	0.5788E 00
4	0.3138E 04	0.1300E 05	0.3261E 04	0.8300E 04	0.6343E 00
5	0.3048E 04	0.1300E 05	0.5856E 03	0.8300E 04	0.3050E 00
6	0.2485E 03	0.1300E 05	0.1738E 04	0.8300E 04	0.2285E 00

CYGNA

PROJECT TUSI
SUBJECT BASEPLATE ANAL
BY RGJ
DATE 3-7-84
CHECKED GSB
DATE 3-12-84
JOB 84042

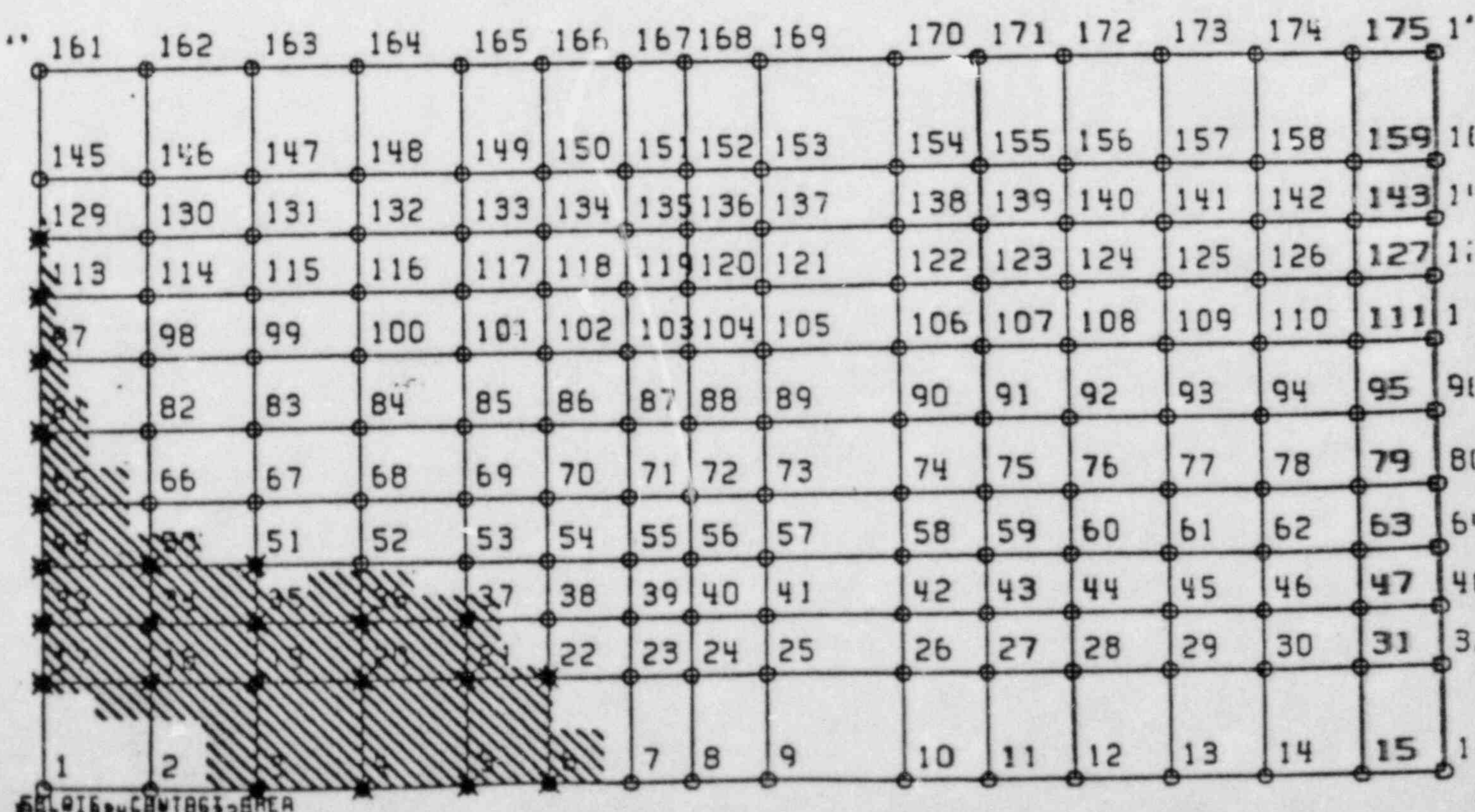
TUSI ℞ No. 3 FOR AB-1-71A
SUPPORT No. RH-1-024-011-S22A
(No STIFFENERS)

136	137	138	139	140	141	142	143	144	145	146	147	148	149	15
121	122	123	124	125	126	127	128	129	130	131	132	133	134	13
106	107	108	109	110	111	112	113	114	115	116	117	118	119	12
91	92	93	94	95	96	97	98	99	100	101	102	103	104	10
76	77	78	79	80	81	82	83	84	85	86	87	88	89	90
61	62	63	64	65	66	67	68	69	70	71	72	73	74	75
46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

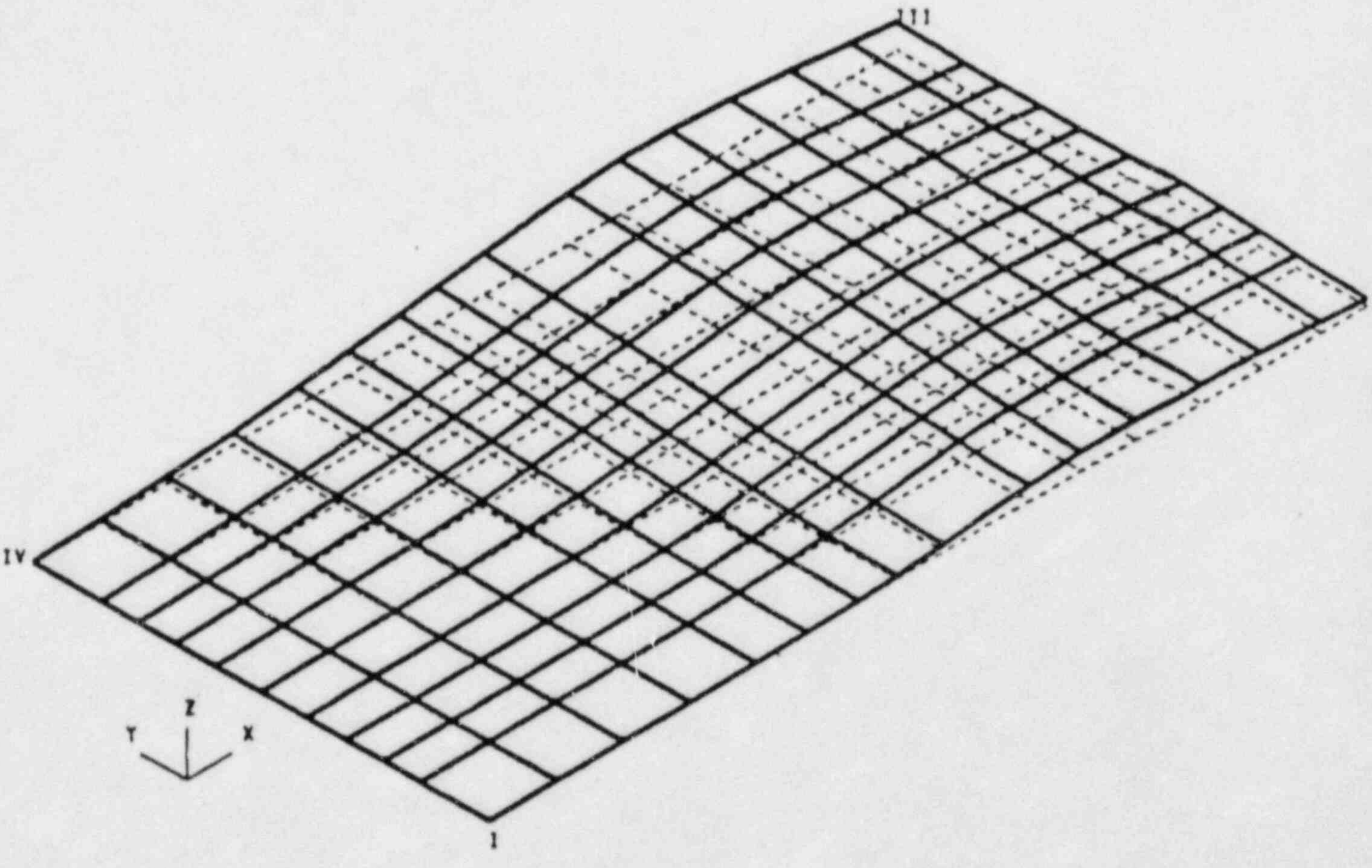
RH1-024-011-S22A ELEMENT NUMBERS

CYGNA
PROJECT TUSI
TITLE BASEPLATE ANAL
PREPARED BY RGJ
DATE 3-7-84
CHECKED BY GSB
DATE 3-12-84
JOB NO. 84042
FILE NO. ______
SHEET NO. ______

TUSI TR No. 3 FOR AB-1-71A
SUPPORT No. RH-1-024-011-S22A
(NO STIFFENERS)



CYGNA
TUSI
BASEPLATE KWL
RGJ
3-7-84
GSB
3-12-84
84042



TUSI PL NO. 3 FOR AB-1-71A
SUPPORT NO. RH-1-024-011-S22A
(NO STIFFENERS)

EPLATE: DEFORMED SHAPE PLOTS
RH-1-024-011-S22A

(. TUSI PLATE NO. 4 FOR AB-1-71A

PROGRAM BPLATE CYGNA ENERGY SERVICES DATE : WED, C7 MAR 1984
VERSION 2.C 1C1 CALIFORNIA STREET 12:3C:29
SAN FRANCISCO, CALIF. 94111

* *
* RB-1-C24-C11-S22A WITH STIFFENERS *
* *

INPUT VALUES INPUT BY: ..RGJ........ DATE: 3-7-84
------------ CHECKED BY: [illegible] DATE: 3-12-84

FLATE : X-DIM = 38.25CC IN. Y-DIM = 20.CCOC IN. THICKNESS = 1.5COC IN.
E = C.29CE C8 PSI NU = C.3CCC ALLOWABLE STRESS= C.27CCE C5
F.E. MESH: NEL-X = 15 NEL-Y = 1C REGULAR MESH? NC ELEMENT TYPE =PLATE
FLOOR : STIFF/AREA/UNIT DISP = C.625E C6 POISSON'S RATIO = C.3C (IKONK =C)

GENERATED ELEMENT DIMENSIONS (X-DIRECTION)

ELEMENT NUMBER	X-DIMENSION
1	3.CCC
2	2.935
3	2.935
4	2.935
5	2.25C
6	2.25C
7	1.6[illegible]
8	2.C61
9	3.752
1C	2.249
11	2.249
12	2.649
13	2.646
14	2.645
15	2.CCC

GENERATED ELEMENT DIMENSIONS (Y-DIRECTION)

ELEMENT NUMBER	Y-DIMENSION
1	3.CCC
2	1.625
3	1.625
4	1.75C
5	2.CCC
6	2.CCC
7	1.75C
8	1.625
9	1.625
1C	3.CCC

BOLTS : SET = 1 DIAM = 0.0000 IN. EFF Y.M. = 0.235E 06 PSI
TDISP= 0.0000 IN. EFLEN= 1.0000 IN. EFF G= 0.500E 06 PSI PLOAD= 0.000 LF
ALLOWABLE NORMAL STRESS/LOAD= 0.130E 05 ALLOWABLE SHEAR STRESS /LOAD= 0.830E (
NUMBER OF BOLTS = 6

BOLT NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	NODE NUMBER	MATERIA TYPE
1	3.000	3.000	2	2	18	1
2	18.00	3.000	8	2	24	1
3	36.25	3.000	15	2	31	1
4	36.25	17.00	15	10	159	1
5	18.00	17.00	8	10	152	1
6	3.000	17.00	2	10	146	1

NUMBER OF SECTIONS = 1

SECTION REFERENCE POINT

SECTION NUMBER	----- GENERATED VALUES ----- X-COORDINATE	Y-COORDINATE	--GRID POINTS-- X-GRID	Y-GRID	RIGID	TYP
1	20.06	10.00	6	6	1	59

LINE SEGMENTS OF THIS SECTION ARE LOCATED AT -

------------- END I ------------- X-COORDINATE	Y-COORDINATE	------------- END J ------------- X-COORDINATE	Y-COORDINATE
16.311	6.2500	20.060	6.2500
20.060	6.2500	23.812	6.2500
23.812	6.2500	23.812	10.000
23.812	10.000	23.812	13.750
23.812	13.750	20.060	13.750
20.060	13.750	16.311	13.750
16.311	13.750	16.311	10.000
16.311	10.000	16.311	6.2500
16.311	10.000	11.811	10.000
11.811	10.000	0.00000	10.000
23.812	10.000	28.310	10.000
28.310	10.000	36.250	10.000
28.310	10.000	28.310	20.000
28.310	10.000	28.310	0.00000
11.811	10.000	11.811	20.000
11.811	10.000	11.811	0.00000

LOAD INFORMATION (FORCES -LBS) (MOMENT -IN-LBS)

SEC	X-SHEAR	Y-SHEAR	Z-FORCE	X-MOMENT	Y-MOMENT	Z-TORQUE	ZDIST
1	0.533E 04	0.414E 04	0.262E 04	0.722E 05	-0.130E 06	0.207E 06	0.000E

AM BPLATE CYGNA ENERGY SERVICES DATE : WED, 07 MAR 1984
ON 2.0 101 CALIFORNIA STREET 12:30:29
SAN FRANCISCO, CALIF. 94111

* *
-1-024-011-S27A
* *

```
****************************
*                          *
*   PLATE STRESS SUMMARY   *
*                          *
****************************
```

UM Z-DISP=-0.4078E-02 (NODE= 5) MAXIMUM Z-DISP= 0.1386E-01 (NODE= 173

QUIVALENT STRESS FOR BASEPLATE = 0.3785E 04 PSI (AT ELEMENT = 90)
ABLE EQUIVALENT STRESS FOR BASEPLATE= 0.2700E 05

```
                          0.3785E 04
LATE PROVISION RATIO = ------------ = 0.1402E 00
                          0.2700E 05
```

```
****************************
*                          *
*       BOLT SUMMARY       *
*                          *
****************************
```

```
                       T        V
PROVISION RATIO = -----  +  -----
                      TA       VA
```

T AND TA ARE ACTUAL AND ALLOWABLE TENSION FORCES/STRESSES
AND VA ARE ACTUAL AND ALLOWABLE SHEAR FORCES/STRESSES

FORCES AND STRESSES

F	T	TA	V	VA	PROVISION RATIO
	0.0000E 00	0.1300E 05	0.2597E 04	0.8300E 04	0.3129E 00
	0.6126E 03	0.1300E 05	0.2024E 04	0.8300E 04	0.2910E 00
	0.1665E 04	0.1300E 05	0.3797E 04	0.8300E 04	0.5855E 00
	0.2930E 04	0.1300E 05	0.3277E 04	0.8300E 04	0.6202E 00
	0.2068E 04	0.1300E 05	0.5570E 03	0.8300E 04	0.2262E 00
	0.8716E 03	0.1300E 05	0.1741E 04	0.8300E 04	0.2768E 00

CYGNA

PROJECT TUSI
TITLE BASEPLATE ANAL
PREPARED BY RGS
DATE 3-7-84
CHECKED BY GSB
DATE 3-12-84
84042

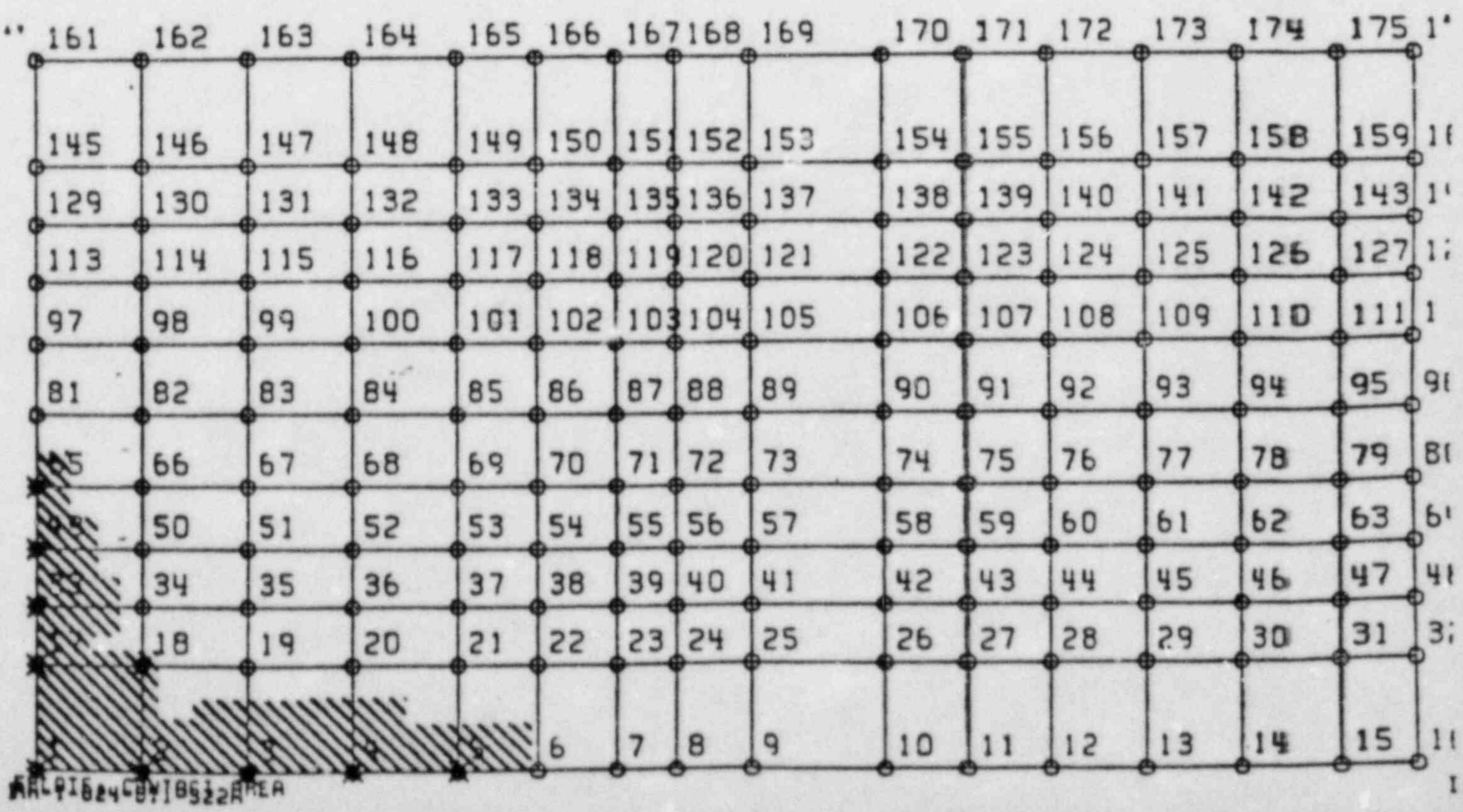
TUSI PL No. 4 FOR AB-1-71A
SUPPORT No. 2H-1-024-011-S22A
(WITH STIFFENERS)

136	137	138	139	140	141	142	143	144	145	146	147	148	149	150
121	122	123	124	125	126	127	128	129	130	131	132	133	134	135
106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
91	92	93	94	95	96	97	98	99	100	101	102	103	104	105
76	77	78	79	80	81	82	83	84	85	86	87	88	89	90
61	62	63	64	65	66	67	68	69	70	71	72	73	74	75
46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

CYGNA

PROJECT TUSI
TITLE BASEPLAT ANAL
PREPARED BY RGJ
DATE 3-7-84
CHECKED BY GSB
DATE 3-12-84
JOB NO. 84042
FILE NO. ____
SHEET NO. ____

TUSI TR No. 4 FOR AB-1-71A
SUPPORT No. RH-1-024-011-S22A
(WITH STIFFENERS)



CYGNA

PROJECT TUSI

TITLE BASEPLATE ANAL

PREPARED BY RGJ

DATE 3-7-84

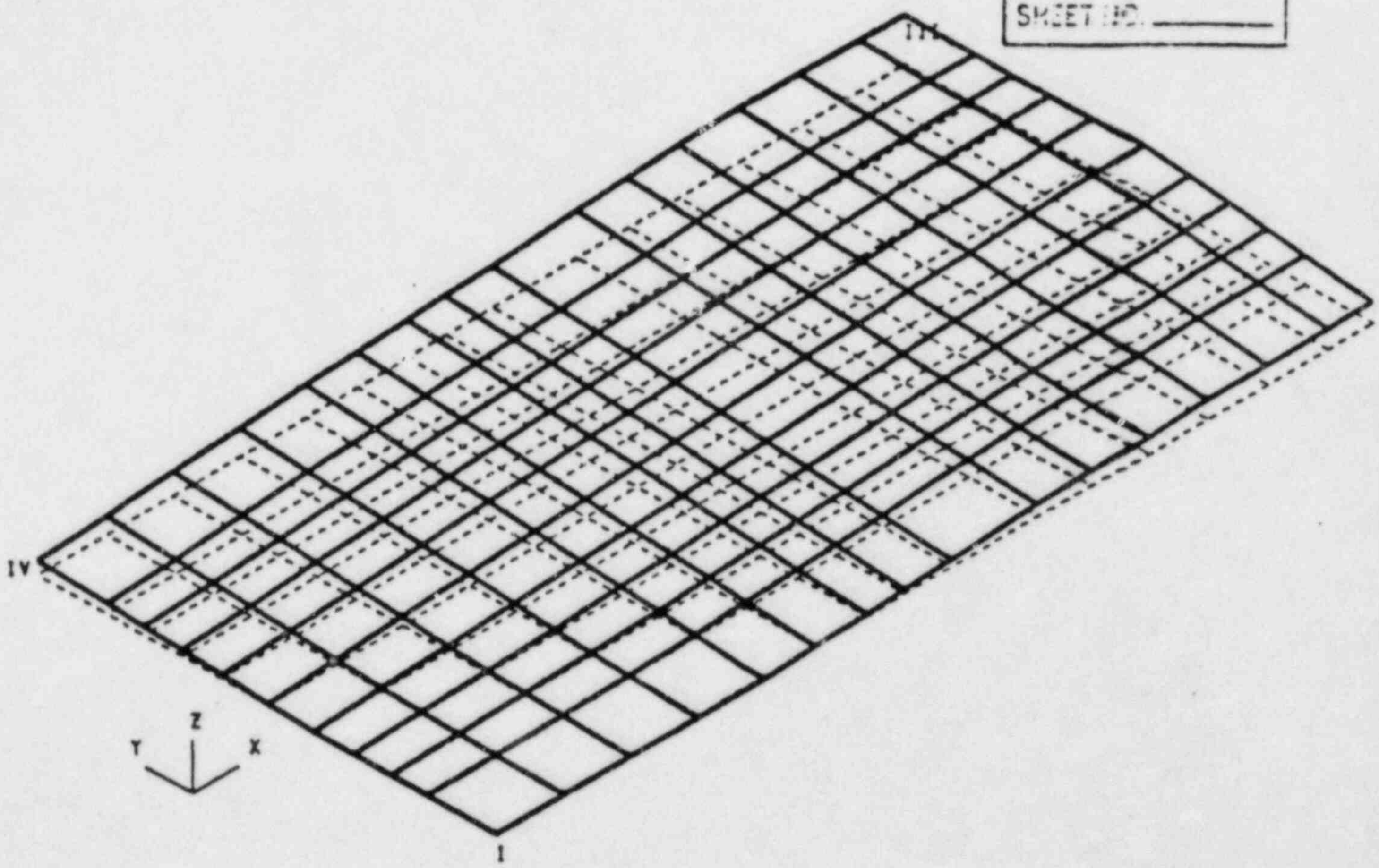
CHECKED BY GSB

DATE 3-12-84

JOB NO. 84042

FILE NO.

SHEET NO.



TUSI TR No. 4 FOR AB-1-71A

SUPPORT No. RH-1-024-011-S22A

(WITH STIFFENER)

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #1
Exhibit No.: None

1.0 CASE Question

Appendix E of Cygna Report. Section DC-2.2.4. What was the yield point used for A500, Grade B tube steel?

2.0 Cygna Interpretation

During the course of the pipe support design effort at Comanche Peak, an ASME Code Case was issued (N-71-10) which reduced the allowable yield stress from that stated in the Code Case being employed in the design (N-71-9). What code case was used in the design?

3.0 Response

Comanche Peak typically used a yield strength equal to 42 ksi as required by ASME Code Case N-71-9. The value for yield strength based on ASME Code Case N-71-10 is 36 ksi. Cygna's original audit accepted calculations based on ASME Code Case N-71-9. Cygna later checked the calculations within the review scope to verify that the tube steel design stresses did not exceed 36 ksi set forth in Code Case N-71-10. In each case, the existing design met the 36 ksi allowable. (See Attachment WI-1 for list of supports checked.)

The ASME has since provided a response to Texas Utilities' inquiry into the need to adopt the lower yield strength values. A copy of this letter is provided in Attachment WI-2. Part of the response states that "...the provisions of later revisions to Code Cases are neither mandatory or retroactive." Further, based on the ASME review and notifications, and as stated in the letter, the change from 42 ksi to 36 ksi is not considered a safety concern.

ATTACHMENT WI-I (Page 1 of 1)

List of Supports Reviewed for Tube Steel Allowable

SI-1-075-002-S22K
RH-1-064-008-S22K
RH-1-010-004-S22S
RH-1-010-002-S22K
RH-1-064-010-S22R
SI-1-075-001-S22R
RH-1-064-007-S22R
SI-1-075-003-S22R
RH-1-064-011-S22R
SI-1-325-001-S32R
SI-1-042-002-S22K
SI-1-073-700-S32R
RH-1-008-007-S22R
RH-1-064-001-S22R
RH-1-010-001-S22R
RH-1-064-009-S22R
SI-1-325-002-S32R
SI-1-037-005-S32A
SI-1-070-007-S22A
RH-1-024-011-S22A

The American Society of Mechanical Engineers

United Engineering Center • 345 E. 47th St., New York, N.Y. 10017 • 212-644-7722 • TWX-710-581-5267

RECEIVED
NOV 23 1983
Texas Utilities Services, Inc.
CPSES Const. Office

ATTACHMENT W1-2 (Page 1 of 2)

George	Burgess
Merritt	Norman
Hall	Johnson
McBay	Popplewell
Calder	Creamer
Deem	Kissinger
Strange	Finneran
Stobaugh	Murray
Davis	
Hicks	
Gentry	R. Baker
	File

November 18, 1983

Texas Utilities Services Inc.
PO Box 1002
Glen Rose, TX 76043

Attn: M. R. McBay

Subject: Section III, Division 1
Code Case N-71-9 & N-71-10
ASTM A-500 Tubular Shapes

Reference: Your letter of October 25, 1983
ASME File # NI 83-101

Gentlemen:

Our understanding of the questions in your inquiry, and our replies are as follows:

Question 1: An Owner has contracted for construction of component supports under the provisions of Case N-71-9. Must component supports constructed from ASTM A-500 tubular shapes under the provisions of Case N-71-9 be redesigned or re-analyzed using the lower yield strength values published in a later revision of the Case (e.g., N-71-10) for the same material?

Reply 1: No, the provisions of later revisions to Code Cases are neither mandatory or retroactive.

Question 2: Why were the yield strength values for A-500 tubular shapes published in Case 1644-3 through N-71-9 reduced in N-71-10?

Reply 2: The Committee recognized that the yield strength of A-500 in the cold wrought condition may be slightly reduced in the heat affected zone of weldments. The revised values, given in N-71-10, for A-500 were those used for A-501 and A-36 material which were selected as conservative values for A-500 tubular shapes in the welded condition. The revised values may be changed at such time when material data for the welded condition, as required by the Code, is presented to the Committee for consideration. The higher

yield strength values published in N-71-9 are adequate because of the many safety factors and design constraints applied to the yield strength in the design of piping supports.

Question 3: If a component support is ordered under a Design Specification which required compliance with an Edition and Addenda of the Code which was issued prior to final approval of Case N-71-10, and the contract date for the support is after the date of Council approval of Case N-71-10, does the Code allow the construction of the support under the provisions of Case N-71-9?

Reply 3: Yes, in accordance with NA/NCA-1140.

* * * * * * *

We note that when, in the opinion of the Committee, a review of current Code provisions indicate a potential safety concern there are established means of notifying organizations and individuals who may be affected. These means include notification through Mechanical Engineering magazine and letters to holders of Certificates of Authorization and jurisdictional and regulatory authorities. These measures were determined not to be necessary in the case of the yield strength values for A-500 tubular shapes in Case 1644-3 through N-71-9.

Yours truly,

Kevin Ennis

Kevin Ennis
BPVC Assistant Secretary
(212) 705-7643

KE/dp

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #2
Exhibit No.: None

1.0 CASE Question

Observation Record PS-02-01. The applicant did not consider shear cone interaction of adjacent bolts.

2.0 Cygna Interpretation

Cygna Observation PS-02-01 was written to evaluate an apparent discrepancy between drawing information and calculations, as related to anchor bolt embedment lengths. Was shear cone interaction also addressed?

3.0 Response

Yes. Observation PS-02-01 identifies a concern with the calculation of bolt embedment lengths. Investigation revealed that the embedment was provided to the constructor as a function of total bolt length which is specified on the drawing. In addition, the greater of the two embedments derived from either the construction specification or the drawing governs.

Although not related to this concern, Cygna did check both the analyses and construction to ensure that bolt spacing requirements were met. Minimum bolt spacing criteria are necessary to assure full development of bolt capacity as specified by the manufacturer. Maximum bolt capacity is realized when the concrete shear cone is fully developed without interferences. Interaction or overlapping between adjacent bolt shear cones will reduce bolt capacity as a function of bolt diameters. The applicant properly considered these effects as stated in the Hilti Manufacturers catalog (see Attachment W2-1).

HILTI KWIK-BOLT TECHNICAL INFORMATION

1. **Anchor Spacing**

   The minimum anchor spacing and edge distance for 100% effective anchor performance according to EAMI (Expansion Anchor Manufacturers Institute) are as follows:

   Minimum Anchor Spacing = 10 hole diameters
   Minimum Edge Distance = 5 hole diameters

   According to EAMI, anchor efficiency is reduced on a straight-line basis down to 50% at 5 diameters center-to-center anchor spacing.

2. **Minimum Embedment**

   The minimum embedment for satisfactory anchor performance is 4½ bolt diameters (6½ bolt bolt diameters for the Super Kwik-Bolt). Deeper embedments will yield higher tension and shear capacity as indicated in the TR-111: "Kwik-Bolt Testing Program." Embedment depths indicated in all test reports are before setting (tightening).

3. **Maximum Working Loads**

   The maximum working loads should not exceed ¼ of the average ultimate values for a specific anchor size. Actual factor of safety to be used depends on the application and should be selected by the designer on this basis.

4. **Combined Loading**

   Combined loading should calculated on a straight line interaction diagram of pure shear (S) and pure tension (T).

$$\frac{S\ \text{applied}}{S\ \text{allowable}} + \frac{T\ \text{applied}}{T\ \text{allowable}} \leq 1$$

5. **Standard Kwik-Bolt Materials**

   a. Stud (bolt material is AISI 11L41 for bolt diameters ¼"-½" and AISI 1144 for diameters ⅝"-1¼", meeting the chemical requirements for ASTM specification A 108.

   b. The two independent expansion wedges are made from AISI 1050 spring steel.

   c. Nuts are of commercial manufacture, meeting ASTM A 307, Grade A (e.g., AISI series 10XX).

   d. Washers are fabricated from SAE standard material in accordance with ASA standard #B27.2-1949.

   e. Kwik-Bolts are plated in accordance with the requirements of Federal Specification QQ-Z-325C, Type II, Class 3, (clear chromate treatment).

   f. The Kwik-Bolt meets the dimensional requirements of Federal Specification FF-S-325, Group II, Type 4, Class 1.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #3
Exhibit No.: None

1.0 CASE Question

PI-01-10. There has been no detailed computer analysis performed to consider the concentrated loads (valves, etc.) and their effect on dead weight and seismic. Also, the seismic analysis will not be linearly proportional.

2.0 Cygna Interpretation

Observation PI-01-01 states:

> "The wall thickness used for the computer analysis piping segments 16"-SI-074-151R-2 and 16"-SI-073-151R-2 was 0.5 inches. The correct value is 0.375 inches."

To evaluate the impact of this error in wall thickness, Cygna increased the pipe stresses by the linear proportion 0.5/0.375. Please address the following:

a. The effect on thermal, pressure and deadweight stresses as the pipe wall thickness decreases.

b. The effect on seismic stresses, which are not linearly proportional to the change in wall thickness.

3.0 Response

a. Figure W3-1 can be used to illustrate the effect on thermal stresses due to a local decrease in pipe wall thickness.

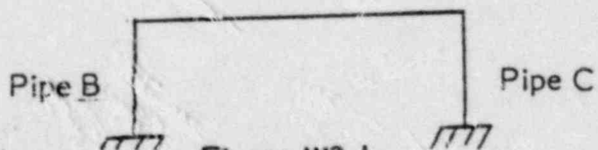


Figure W3-1

Assume that the thickness of Pipe A reduces from 0.5" (t_o) to 0.375" (t_1). As shown

below, the axial thermal stresses developed within Pipe A are unchanged as the wall thickness decreases:

$$\sigma_t = \text{thermal stress} = E \alpha T \qquad (1)$$

where

E = modulus of elasticity
α = coefficient of thermal expansion
ΔT = temperature change

The axial thermal force in Pipe A actually decreases as the wall thickness decreases, since

$$F_t = \text{thermal force} = \sigma_t A \qquad (2)$$

where

A = pipe area = πDt (3)
D = pipe diameter
t = wall thickness

Any reduction in the axial force within Pipe A will also reduce the moments induced at the connection to Pipes B and C. So, the thermal moment in Pipe A will decrease as the wall thickness decreases. Since the bending strength of Pipe A is also decreasing along with the wall thickness, the net effect on thermal bending stresses depends upon the piping configuration and is not predictable. However, the upper bound change in thermal bending stresses is t_o/t_1, the value used by Cygna.

Pressure stresses in the piping are also a linear function of wall thickness:

$$\sigma_1 = \text{circumferential (hoop) stress} = \pi pD/4t \qquad (4)$$
$$\sigma_2 = \text{longitudinal stress} = \pi pD/2t \qquad (5)$$

where p = internal pressure

Dead weight stresses due to the pipe itself are unaffected as its wall thickness decreases. This is shown below for a simple beam:

4

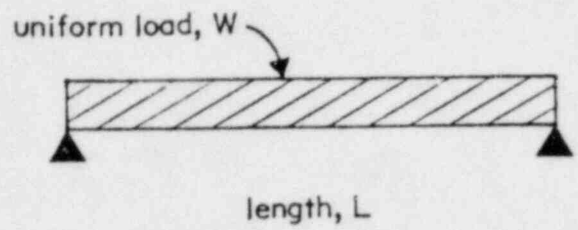


Figure W3-2

The maximum deadweight bending stress (σ_D) in Figure W3-2 is:

$$\sigma_D = \frac{W L^2 D}{8 I} \tag{7}$$

where

$$W = A\gamma = \pi D t \gamma \tag{8}$$

γ = volume weight of steel

$$I = \text{moment of inertia} = \pi D^3 t/64 \tag{9}$$

Inserting equations (3) and (9), shows that the wall thickness drops out of equation (7):

$$\sigma_D = \frac{(\pi D t \gamma)\, L^2 D}{8\, \pi D^3 t/64} = \frac{8\, \gamma L^2}{D} \tag{10}$$

Equation (7) shows that deadweight stresses due to other dead loads, such as insulation, would increase as the wall thickness decreases. This is because only the moment of inertia would changes.

In summary, pipe stresses induced by thermal, pressure and dead weight loadings are related as follows to a decrease in pipe wall thickness.

- Thermal

  Thermal stresses developed within the thinner pipe section are unchanged. Stresses induced by the thermal growth of attached piping will increase stresses by an amount linearly proportional to the pipe wall thickness:

  $$\sigma_{(temp)} \propto \frac{1}{t} \qquad (11)$$

- Pressure

  Pressure induced stresses will increase as the wall thickness decreases. The increase is linearly proportional:

  $$\sigma_{(pressure)} \propto \frac{1}{t} \qquad (12)$$

- Deadweight

  Pipe stresses induced by pipe deadweight are unchanged by a change in wall thickness.

  $$\sigma_{(pipe\ deadweight)} \text{ -- unchanged} \qquad (13)$$

  $$\sigma_{(other\ deadweight)} \propto \frac{1}{t} \qquad (14)$$

  Deadweight stresses due to loads other than the pipe itself will increase as the pipe wall thickness decreases.

Based on the above, the simplified procedure employed v Cygna to evaluate thermal, pressure and deadweight effects related to O: ervation PI-01-01 is reasonable and in fact conservative.

b. Figure W3-2 will also be used ot illustrate the effect of a decrease in pipe wall thickness on seismic induced stresses.

For a simply supported pipe loaded by a uniform weight, the fundamental pipe frequency is:

$$f = \frac{\pi}{2L^2}\sqrt{\frac{EIg}{W}} \tag{15}$$

where

f	=	fundamental frequency
L	=	span length
E	=	modulus of elasticity
I	=	moment of inertia (Equation (9))
g	=	gravity
W	=	weight per unit length

Only two terms, W and I, depend upon the wall thickness, therefore the frequency change due to a thickness change can be expressed as follows:

$$\Delta f = (f_o - f_1) = \frac{\pi\sqrt{Eg}}{2L^2}\left(\sqrt{\frac{I_o}{W_s + W_o}} - \sqrt{\frac{I_1}{W_s + W_1}}\right) \tag{16}$$

where

Δf	=	frequency change
f_o	=	frequency associated with thickness t_o
f_1	=	frequency associated with thickness t_1
W_s	=	total weight - unloaded weight
W_o	=	unloaded weight for t_o
W_1	=	unloaded weight for t_1
I_o	=	moment of inertia for t_o
I_1	=	moment of inertia for t_1

Substituting the equations for W and I, Equation (16) becomes:

$$\Delta f = \frac{\pi\sqrt{Eg}}{2L^2}\left(\sqrt{\frac{(\pi D^3\ t_o)/64}{W_s + (\pi D\ t_o \gamma)}} - \sqrt{\frac{(\pi D^3\ t_1)/64}{W_s + (\pi D\ t_1 \gamma)}}\right) \tag{17}$$

The following conclusions can be reached from Equation (17):

- When "other" loads (W_s) are zero, Equation (17) reduces to:

$$\Delta f = \text{constant} \times \left(\sqrt{\frac{t_o}{t_o}} - \sqrt{\frac{t_1}{t_1}} \right) = 0 \qquad (18)$$

  Therefore, the acceleration and stresses will be unchanged (see Equation (13)).

- When W_s is greater than zero, its influence is small. Per Brown & Root drawing, BRHL-SI-1-RB-061, Rev. 0, pipe segment SI-1-073 has the following properties:

  D = 16 in.
  L = 14.5 ft. = 147 in.(1)

  Using these properties, Equation (17) reduces to:

$$\Delta f = 109 \left(\sqrt{\frac{0.5}{W_s + 7}} - \sqrt{\frac{0.375}{W_s + 5}} \right) \qquad (19)$$

  where

E	=	29,000,000 psi
g	=	386 in/sec^2
γ	=	(490/1728) lbs/cu. in.
t_o	=	0.5 in.
t_1	=	0.375 in.

Note: (1) The distance from the containment flued head to support SI-1-073-700-S32R is 14'-6 3/8".

Table W3-I lists the results of a sensitivity analysis performed using Equation (19). It shows that the maximum frequency change inthe simple model of line SI-I-073 is one hertz for all values of W_s. For the sake of comparison, the weight of water in a 16-inch diameter pipe is 7.3 lbs/in.

Table W3-I

f, hertz	W_s, lbs/in
0.46	2
0.99	4
1.25	6
1.39	8
1.46	10
1.49	12
1.51	14
1.51	16
1.50	18
1.48	20
0.94	100
0.32	1,000
0.10	10,000

The small changes in frequency shown above have negligible effect on pipe stresses.

Therefore, the simplified procedure employed by Cygna to evaluate dynamic stress effects related to Observation PI-01-01 is conservative. The actual effect on pipe stresses will be less than the ratio t_o/t_1.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #4
Exhibit No.: None

1.0 CASE Question

PI-02. Is there an error in the table shown?

2.0 Cygna Interpretation

Referring to Observation PI-02-03, Attachment A, is there an error in the calculated table?

3.0 Response

There is a typographical error in the calculated table. The allowable for restraint RH-1-064-007-S22R should be "44000", rather than "4400".

As shown on the attached Table, Attachment W4-1, this correction puts the allowable for the aforementioned restraint into line with the other restraints tabulated.

CYGNA

ATTACHMENT W4-1
(Page 1 of 1)

Observation Record Review
Attachment A

Checklist No. PI-02		Revision No.	0
Observation No. PI-02-03		Sheet 1	of 1

	Yes	No
Valid Observation	X	
Closed	X	

Comments

1.0 Root Cause

Possible misunderstanding of the Gibbs and Hill procedure

2.0 Resolution

Using the range for the 3 rigid restraints, Cygna calculated the following:

Support	Load Range	CYLNCZ Stress	General Stress	Total	Allow
SI-1-032-003-S32R	2700	10362	6763	17125	45000
RH-1-064-007-S22R	1300	5172	5128	10300	4400 44000
RH-1-016-001-S32R	8615	11225	9328	20555	44000

The remaining 4 restraints are springs or snubbers and have no thermal load. Thus, there is no increase in stress above allowables.

Cygna also noted that the correct method was used for the welded attachments in anchors of Problem 1-70 and in all supports in Problem 1-69. Based on this, Cygna considers the error isolated. In addition, the RHR system will probably show the largest percentage difference (between maximum load and range), since it has many modes of operation. Thus, Cygna expects the error would have the most impact on this system. As the new calculations show, the impact on design is negligible and the observation is closed.

Approvals

Originator	M.K. Mani	Date	11/4/83
Project Engineer	John C. Finneran	Date	11/5/83
Project Manager	N.H. Williams	Date	11/5/83
Senior Review Team	[illegible]	Date	11/5/83

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #5
Exhibit No.: 889

1.0 CASE Question

CTS-00-03. F_{bx} = should be 21.2, not 23.2 or 22. The length is 6' not 5.5'.

- Why was only 1/2 SSE considered?
- Why was 4% damping used; not consistent with FSAR?
- Assumed cable tray was rigid when lumping the mass; this resulted in not combining the dynamic effects of the cable tray itself to the support; did not include effect on welds.
- The validity that the cable trays have the capacity to transfer a load around a corner when one run of cable tray has no axial restraint, as shown on drawing 2323 EI-0601-01.
- What documentation did Cygna see that justified the hangers' receiving a lateral load around corners that resist the axial load from the tray segment that contains no axial restraints.

2.0 Cygna Interpretation

In Observation CTS-00-03, Cygna discusses several apparent deficiencies in the modeling assumptions associated with the frame analyses for cable trays. As related to CASE Exhibits 889, 890 and 902, please address the following:

Exhibit 889

Why was an allowable bending stress (F_{bx}) = 22 ksi used?

Exhibit 890

a. Why was only 1/2 SSE considered?

b. Why was 4% damping used?

c. How were the dynamic effects of the trays included in the analysis?

d. On Drawing 2323-EI-0601-01, there appears to be no means for transferring load around the cable tray bend. Please discuss this.

e. What documentation formed the basis for accepting the condition mentioned in item (d)?

Exhibit 902

How was baseplate flexibility considered?

3.0 Response

Exhibit 889

The Gibbs & Hill calculation to determine the allowable bending stress for the channel section was performed in accordance with the guidelines set forth in the AISC Manual, Equation 1.5-7. This equation provides a method for calculating F_{bx} and also states that F_{bx} shall not exceed 0.6 F_y. The designer first calculated F_{bx} per Equation 1.5-7 to be 23.2 ksi, compared that value to 0.6 F_y, and then selected the lesser value. Section 1.5.1.3.4 of the AISC Manual specifies that F_{bx} for 36 ksi steel equals 22 ksi.

A direct calculation of 0.6 F_y for 36 ksi material would of course produce a value for F_{bx} equal to 21.6 ksi, rather than 22 ksi. As illustrated in AISC Section 1.5.1.3.4, this 1.8% difference is not considered significant. 22 ksi was used in the design.

If 6'-0" is used in equation 1.5-7 rather than 5'-6", as properly chosen by the designer, F_{bx} would equal 21.2 ksi. 5'-6" is correct based on the definitions provided in the AISC code where:

ℓ " = distance between cross-sections braced against twist or lateral displacements of the compression flange."

As shown on Attachment W5-1, this dimension is the clear span. Resistance to twist or lateral displacement is supplied by the welded connection to the vertical members.

Exhibit 890

a. Gibbs & Hill calculation SCS-101c, set 5, derives the applicable load combinations and shows that, for seismic loadings, the 1/2 SSE (OBE) condition controls. Attachment W5-2 summarizes how that conclusion was reached. Since the supports were designed to OBE loads, the members were checked against the normal allowables with no increase for seismic loads. Inherent in this normalization is the fact that normal strength allowables may be increased for SSE loadings. Since anchor bolt allowables remain constant (i.e., no increase) for SSE loadings, unlike structural members, Cygna questioned the acceptability of this design approach. The attached calculation (W5-3) was performed by Cygna to evaluate this situation. Gibbs & Hill had also evaluated this concern in 1979 and arrived at a similar conclusion.

b. USNRC Regulatory Guide 1.61 specifies that bolted structures, such as this, should be evaluated using 4% of critical damping. Although some connections in the cable tray support system are welded, Cygna concurs with the designer's selection of 4% damping, rather than the 2% damping value specified in R.G. 1.61 for welded structures. The designer's selection is appropriate for the following reasons:

   o The lower damping value for all welded structures recognizes that such a structure will dissipate less energy than structures with mechanical connections. In the case of the cable trays, there are many significant mechanisms for dissipating energy, e.g., the cables are loosely connected to the trays, the trays are connected mechanically to the structural frames, and the frames are bolted to the concrete.

   o Various papers on cable tray behavior illustrate that cable tray systems exhibit damping values greater than 4%. Attachment W5-4 is one such paper (see page 181).

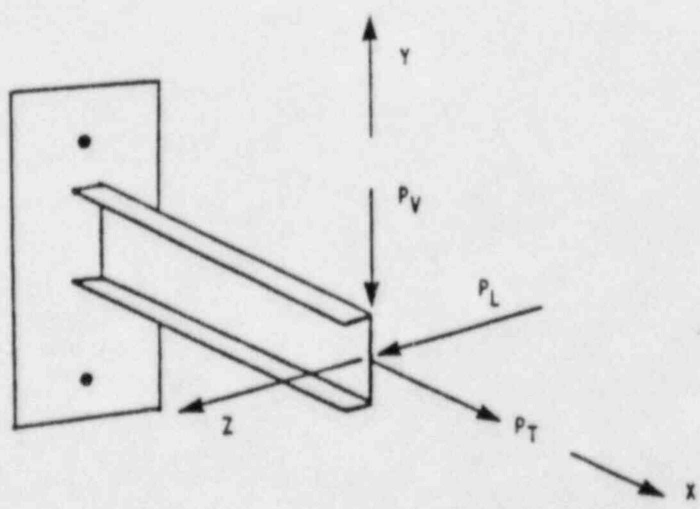
c. Gibbs & Hill designed the cable tray system for peak spectral accelerations. Since 100% of the tray weight was included and peak accelerations were employed, any influences due to tray flexibility have been conservatively incorporated.

d. As shown in Attachment W5-5, the tray system in question is adequately supported. An axial restraint is provided near each bend. The schematic in Attachment W5-5 is taken directly from Drawing 2323-E1-0601-1 (CASE Exhibit 957).

e. Gibbs & Hill calculation SCS-113c, set 3, addresses the longitudinal restraints discussed in item (d).

Exhibit 902

The calculation in question concerns the analysis of the two-bolt baseplate of a Detail "E" which was installed on a riser and is utilized as a three-way restraint.



Gibbs & Hill's analysis considered a rigid baseplate which was analyzed to resist rotations about the Y and Z axis. Gibbs & Hill's analysis showed that compression against the concrete provides sufficient resistance in conjunction with the tension in the anchor bolts.

A subsequent analyses by Cygna, using the baseplate II program of CDC, verified the Gibbs & Hill results.

Gibbs & Hill Reanalysis Calculation:

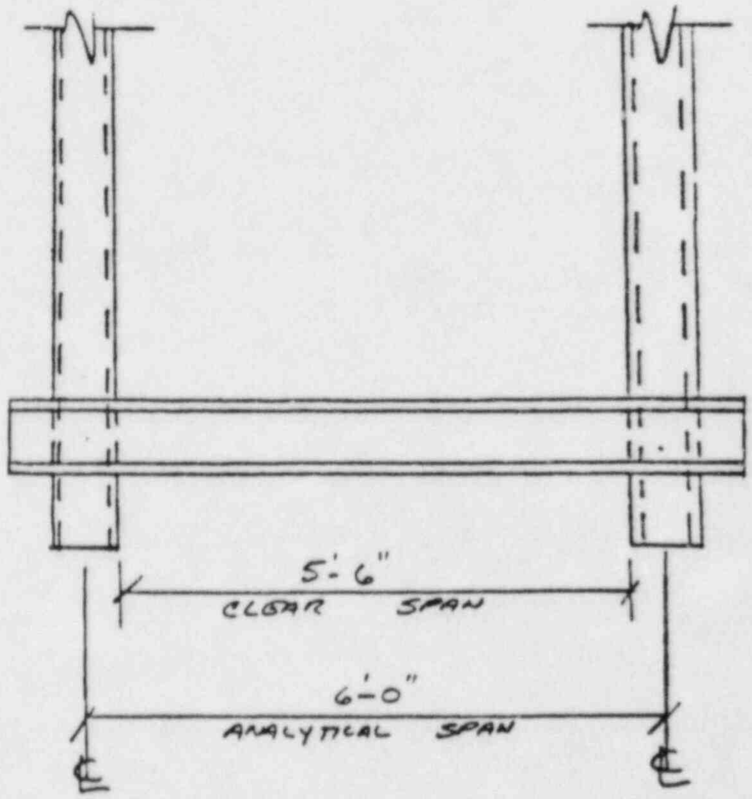
SCS-146C, Set 8, Sheets 65-69 (also Tech. File 11.2.1.50, Sheets 15/81-19/81)

Cygna Baseplate Analysis:

Calculation Binder 83090/1-F, Section A
Computer Binder 83090/1.1-F, Sections A, B and C

Interaction Ratios:

Gibbs & Hill Revised Calculation	=	.584
Cygna Baseplate Analysis	=	.464



5'-6"
CLEAR SPAN
6'-0"
ANALYTICAL SPAN
℄
℄

The loading combinations are:

(1) Operating Condition: $S = D + L + F_{EQO}$

(2) Safe Shutdown Condition: $1.6S = D + L + F_{EQS}$

The earthquake loads are:

EQUIVALENT STATIC LOADINGS (G's)

	Earthquake Intensity	
Seismic Direction	SSE	1/2 SSE*
Horz.	4.0 G	2.67 G
Vert.	2.5 G	1.67 G

*Numerically equal to 2/3 SSE values

And the sign convention for vertical loads is:

Positive: in the gravitational direction (down)

Negative: opposite gravitational direction (up)

Now, by substituting in equation (1) above the Operating Condition may be calculated as follows:

(Horz) (a) $S = 2.67\ (D + L)$

(Vert) (b) $S = (D + L) \pm 1.67\ (D + L) = \begin{array}{ll} 2.67\ (D + L) & \textbf{(down)} \\ -0.67\ (D + L) & \textbf{(up)} \end{array}$

And by substituting in equation (2) above, the Safe Shutdown Condition may be calculated as follows:

(Horz) (c) $S = \frac{4.0}{1.6}\ (D + L) = 2.5\ (D + L)$

(Vert) (d) $S = \frac{(D + L)}{1.6} \pm \frac{2.5}{1.6}\ (D + L) \begin{array}{lll} = & 2.19\ (D + L) & \textbf{(down)} \\ = & -0.94\ (D + L) & \textbf{(up)} \end{array}$

Then by comparison, the governing load cases are:

(Horz) Equation (a) $S = 2.67\ (D + L)$

(Vert) Equation (b) $S = 2.67\ (D + L)$ **(down)**

Equation (d) $S = -0.94\ (D + L) \approx 1.0\ (D + L)$ **(up)**

CYGNA

Gibbs & Hill considered two loading cases in the design of their cable tray systems:

Normal + Severe (called "OBE"): $S = D + L + OBE$
Normal + Extreme (called "SSE): $1.6\ S = D + L + SSE$

where,

D = Dead Load
L = Live Load
OBE, SSE = Loads due to that Earthquake

By normalizing the equations with regard to S, the governing load case was determined to be "OBE" for which the members were checked against the normal allowables with no increase for seismic loads. Pages 3 through 6 show clearly that the ratio of "SSE" to "OBE" is always less than 1.6, so all members and welds are acceptable.

For anchor bolts, Gibbs & Hill checked "OBE" loads against Hilti bolt allowable loads based on a minimum factor of safety of 4. As the loads increased to SSE levels, the bolt allowables, using IEB 79-02 as a guide, remain constant at a safety factor of 4. Therefore, the Hilti bolts may not meet a safety factor of 4 under "SSE" loading.

In response to Cygna's question, Gibbs & Hill stated that the factor of safety will not fall below 3 and quoted NRC Document MS 129-4 on the acceptability of a safety factor of 3.

Cygna's Approach

To accept this, Cygna must show that the increase in loads does not reduce the safety factor for "SSE" below 3.

Load Increases

The attached tables show the effective "OBE" and "SSE" G-levels for all buildings. The G-levels are determined from ARS peak values and combined in the fashion on Gibbs & Hill's position calculation.

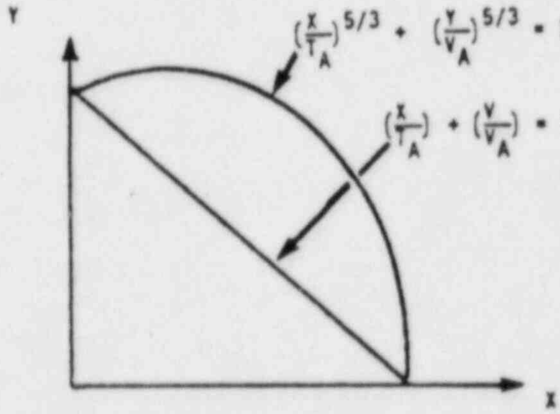
Effective G Values

Elevation	OBE (G)*	SSE (G)*	SSE/OBE
<u>Reactor Internal Structure</u>			
905.75	5.447	6.799	1.25
885.50	4.704	5.882	1.25
860.00	3.790	4.772	1.26
832.50	2.864	3.681	1.29
808.00	2.372	3.108	1.31
783.58	2.251	2.932	1.30
<u>Safeguards Building</u>			
896.5	4.560	5.948	1.30
873.5	4.365	5.790	1.33
852.5	3.698	4.956	1.34
831.5	3.072	4.158	1.35
810.5	2.603	3.698	1.42
790.5	2.212	3.056	1.38
785.5	2.158	2.967	1.37
773.5	2.056	2.790	1.36
<u>Electrical Building</u>			
873.33	3.855	4.944	1.28
854.33	3.578	4.606	1.29
380.00	2.988	3.893	1.30
807.00	2.620	3.638	1.39
778.00	2.452	3.385	1.38
<u>Auxiliary Building</u>			
899.50	5.132	6.446	1.26
886.50	4.664	5.948	1.28
873.50	4.255	5.501	1.29
852.50	3.864	5.003	1.29
831.50	3.339	4.451	1.33
810.50	2.788	3.731	1.34
790.50	2.535	3.560	1.40

Effective G Values (continued)

	Elevation	OBE (G)*	SSE (G)*	SSE/OBE
Fuel Building				
	918.00	4.397	6.058	1.38
	899.50	4.033	5.695	1.41
	860.00	2.866	3.980	1.39
	841.00	2.630	3.646	1.39
	825.00	2.455	3.367	1.37
	810.50	2.271	3.099	1.36

Interaction Diagram



Definitions:

X	=	Tensile Load
Y	=	Shear Load
T_A	=	T/FS, where T = Tensile Ultimate and FS = Factor of Safety
V_A	=	V/FS, where V = Shear Ultimate
X_o	=	"OBE" tensile load
Y_o	=	"OBE" shear load
Δ	=	Load Increase Factor $(\frac{\text{"SSE"}}{\text{"OBE"}})$

The exponential curve is based on the plots of shear tension loadings found in the Teledyne response to IEB 79-02. The document MS 129-4 is only a guide and does state that "OBE" safety factor should be 5. We can, however, argue that a safety factor of 4 for the bolts is adequate based on IEB 79-02.

I. Using Equation (1) - Linear Relationship:

To use this relationship to reach a factor of safety = 3, we must determine what the allowable load increase is above "OBE" loads.

a. Assume pure tension, with the "OBE" load just meeting the criteria

$$\frac{\Delta \cdot X_o}{\frac{T}{3}} = 1.0 \qquad X_o = \frac{T}{4}$$

$$\frac{\Delta \cdot \frac{T}{4}}{\frac{T}{4}} = 1.0$$

$$\therefore \Delta = 4/3 = 1.33$$

The same result will be true for pure shear.

b. For intermediate values of tension and shear ratios assume that the increase in the tensile and shear loads (in going from "OBE" to "SSE") are equal.

Assume

$$X_o = .75\ \frac{T}{4}$$

$$Y_o = .25\ \frac{T}{4}$$

$$\frac{(\Delta)\ .75\ \frac{T}{4}}{\frac{T}{3}} + \frac{(\Delta)\ .25\ \frac{X}{4}}{\frac{X}{3}} = 1$$

$$\Delta\ .75\ (3/4) + \Delta\ .25\ (3/4) = 1$$

$$\Delta = 4/3$$

$\therefore$ A load increase of 1.33 is allowed for the <u>linear</u> interaction equation over the range of values for X_o and Y_o. As can be seen from "Effective G Value Tables," some, but not all, areas of the plant would meet this criteria.

II. Equation (2) - Exponential Relationship

Using the relationship from the Teledyne paper, calculate the allowable load increase, Δ.

a. At the endpoints, the allowable load increase is 1.33 because the linear and exponential curves are coincident here.

b. At an intermediate value an

$$X_o = .75 \frac{T}{4}$$

$$Y_o = .25 \frac{V}{4}$$

$$\frac{\Delta\ .75 \frac{T}{4}}{\frac{T}{3}}^{5/3} + \frac{\Delta\ .25 \frac{V}{4}}{\frac{V}{3}}^{5/3} = 1$$

$$(\Delta\ (3/4)\ (\ .75)\ )^{5/3} + (\Delta\ (.25)\ (\ 3/4))^{5/3} = 1$$

$$.44\ \Delta^{5/3} = 1$$

$$\Delta^{5/3} = 2.25$$

$$\Delta = (2.25)^{.6} = 1.63 > 1.42,$$

so there are values of X_o and Y_o which will give a safety factor of 4 in "OBE" and 3 in "SSE".

$\therefore$ We must determine for what values of the tensile and shear ratio that $\Delta = 1.42$. For tensile and shear ratios between these values, the safety factor of 3 will be met for "SSE" loads.

c. Assume a linear "OBE" relationship such that

$$1 = R_T + R_V$$

where R_T = percent tension allowable
R_V = percent shear allowable
using a safety factor equal to 4

$$R_V = 1 - R_T$$

Substituting into the exponential relationship above:

$$\left(\frac{1.42\ R_T\ \frac{T}{4}}{\frac{T}{3}}\right)^{5/3} + \left(\frac{1.42\ (1-R_T)\ \frac{V}{4}}{\frac{V}{3}}\right)^{5/3} = 1.01$$

$$\left[(.75)(1.42)R_T\right]^{5/3} + \left[(.75)(1.42)(1-R_T)\right]^{5/3} = 1.0$$

$$R_T^{5/3} + (1-R_T)^{5/3} = 1/(1.07)^{5/3} = .900$$

$$R_T^{5/3} + (1-R_T)^{5/3} - .900 = 0$$

Solving numerically cn an HP-150:

$R_T = .93$ or $R_T = .07$

$R_V = .07$ $\quad$ $R_V = .93$

Therefore, for "OBE" loads within the above range of ratios the safety factor of 4 is m^t using a linear relationship and, for the maximum increase of 1.42 to "SSE" loads, the safety factor of 3 is met using the Teledyne interaction method.

Based on Cygna's review of 43% of the cable trays, all shear/tension ratios fall within the above range, so there is no safety impact.

REF: PROCEEDINGS, 48th ANNUAL CONVENTION, [illegible] SEAOC, CORONADO, CALIFORNIA, OCT. 4-6, 1979.

SEISMIC TESTING OF ELECTRIC CABLE SUPPORT SYSTEMS

by

Paul Koss
Bechtel Power Corporation
Los Angeles Power Division

INTRODUCTION

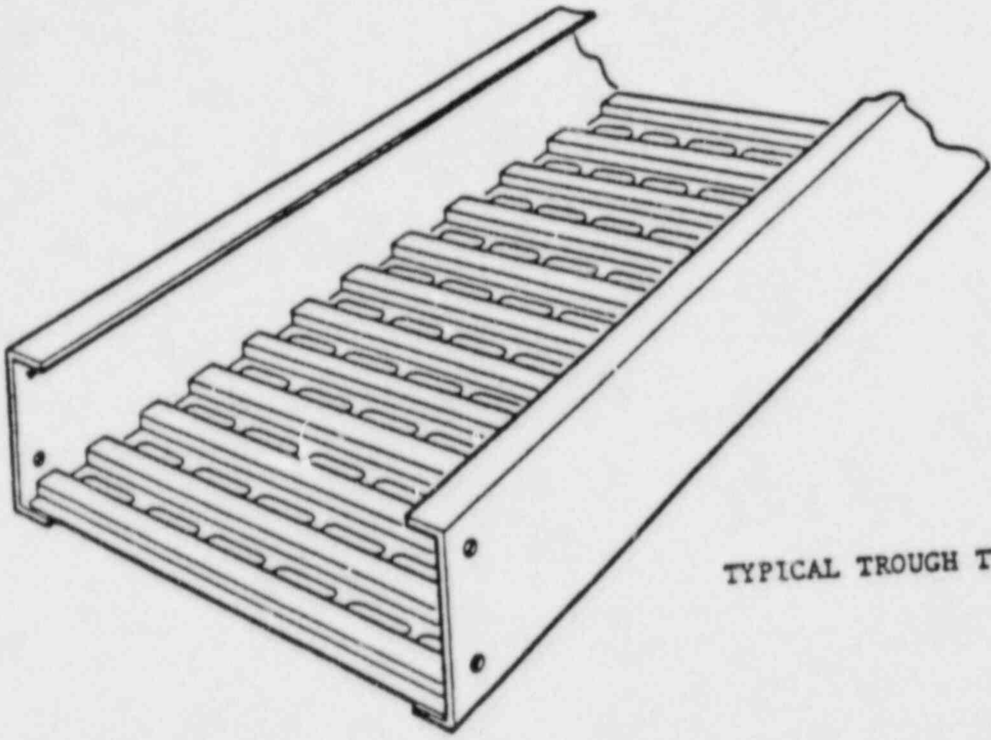
Over the past two decades, many earthquakes have occurred within the United States. Of these, several were of sufficient magnitude to cause structural damage to industrial facilities. Following such strong earthquakes, inspection of power generation and distribution facilities has offered valuable information as to the overall performance of engineered structures. The 1971 San Fernando earthquake has been of particular interest in this regard. It was one of the most severe earthquakes Southern California has experienced in recent history. A survey of structural damage to the Sylmar Converter Station, located within a few miles of the epicenter, provided data relative to the behavior of electrical distribution equipment and electrical raceway systems when excited by strong ground motion. Of special interest was the fact that simple unbraced raceway hanger systems were able to survive the earthquake without major structural failures. Another finding was that even at locations where a minor amount of structural distressing occurred, the cables within the tray systems did not lose their functional integrity. The fact that the converter station's unbraced support system survived the San Fernando earthquake generated interest regarding the practicality of using similar systems in nuclear power plants.

In the years following the San Fernando earthquake, an increasing emphasis has been put into the design of earthquake resistant structures. This has been particularly true of structural systems in nuclear power plants. As early as 1971, design standards were developed in the industry that outlined methodologies for the seismic design of raceway supports. In addition, USNRC regulatory guides and standard review plans were also being developed during the same period of time. Designs based upon these criteria have tended to require substantial amounts of bracing. By contrast, the Sylmar Station support systems were essentially unbraced. Consequently, it appeared that either the design methods or the design criteria, or possibly both, were unnecessarily conservative.

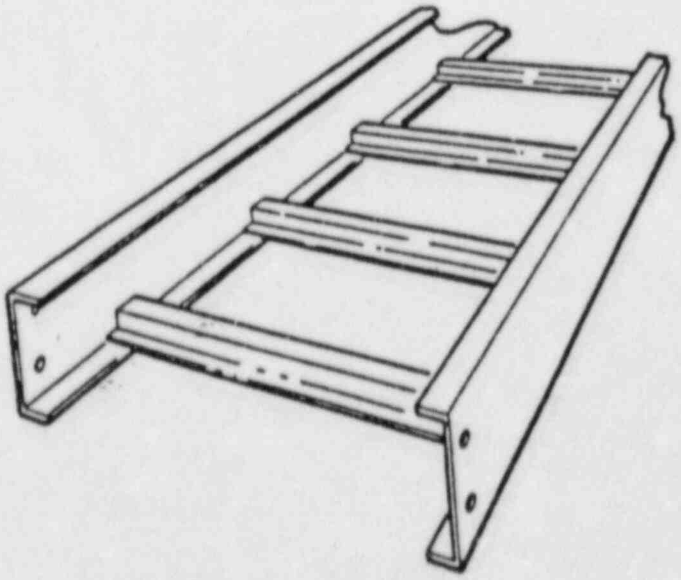
In order to bridge the gap between design procedures and observed behavior of these systems, a plan was initiated to test electrical raceway systems. The goal of the testing was to establish the best possible approach to create an economical, yet adequate, support system for electrical cabling within nuclear plants. By the first part of 1977, a clearly defined program that outlined the types and sizes of raceway systems that would be tested was established. This Cable Tray and Conduit Raceway Test Program was initiated and managed by the Los Angeles Power Division of Bechtel Power Corporation. The testing was conducted, and related consulting services were provided by ANCO Engineering, Inc., Santa Monica, California. In the last months of 1977 testing began. Full scale installations of both cable tray and conduit raceway systems were tested. By the end of 1978, over 2000 individual dynamic tests had been performed, generating over 50 volumes of raw data.

DEVELOPMENT OF THE TEST FACILITY

A typical raceway system consists of cable trays and conduits which are supported from overhead by threaded rod or strut. These supports take the form of a suspended trapeze and may support several trays in vertical tiers. The various commonly used cable trays can be classified as ladder or trough (see figure 1).



TYPICAL TROUGH TYPE TRAY



TYPICAL LADDER TYPE TRAY

Figure 1. Trough and Ladder Tray

Typically these suspended systems may extend vertically in excess of 10 feet, may be very long and may weigh up to 250 pounds per foot of length (multi-tier systems). In view of these unusual characteristics, it was decided to design and construct a special test table capable of input to long suspended systems. ANCO engineers undertook the design and construction of a shake table capable of random and steady state input to raceway systems.

The shake table was designed as an open steel frame, consisting of two parallel trusses interconnected by cross trusses and diagonal diaphragm bracing at the top (see figure 2). The bracing was sized to prevent resonance below 20 Hz. The frame is supported by five linkages which form an inverted pendulum. The angle of the linkage determines the relative amounts of vertical and horizontal table motion. The table can develop input either parallel or perpendicular to its length. The vertical component will act simultaneously and be a scalar of the horizontal, depending upon the angle of the linkages. In addition, the table can be rigidly fixed so that forces or displacements may be applied directly to the test specimen.

The table was designed to accommodate a test setup of 40'-0" in length with five vertical tiers. This required a clear height of 14'-0". The total estimated weight of the heaviest test setup was 10,000 lbs. This weight was used to design a servo actuator system capable of achieving a maximum input to the fully loaded five tier system of 1.1g.

The servo actuators are driven by high pressure hydraulic fluid stored in an accumulator and released through control valves whose setting can be varied in proportion to any arbitrary time varying electronic signal. Output from the test was recorded in the form of time histories on strip charts or tape, spectral plots from a real time analyzer and as response spectra from a digital computer.

TEST PROGRAM SCOPE

The first step in defining the scope of the testing program was the identification of possible significant variables in raceway system design. The following are the potentially significant variables that were identified in planning the test program scope:

- Tray and conduit types
- Tray and conduit loading
- Hanger types
- Hanger length
- Connection details
- Number of trays
- Number of conduits
- Conduit sizes
- Conduit clamps

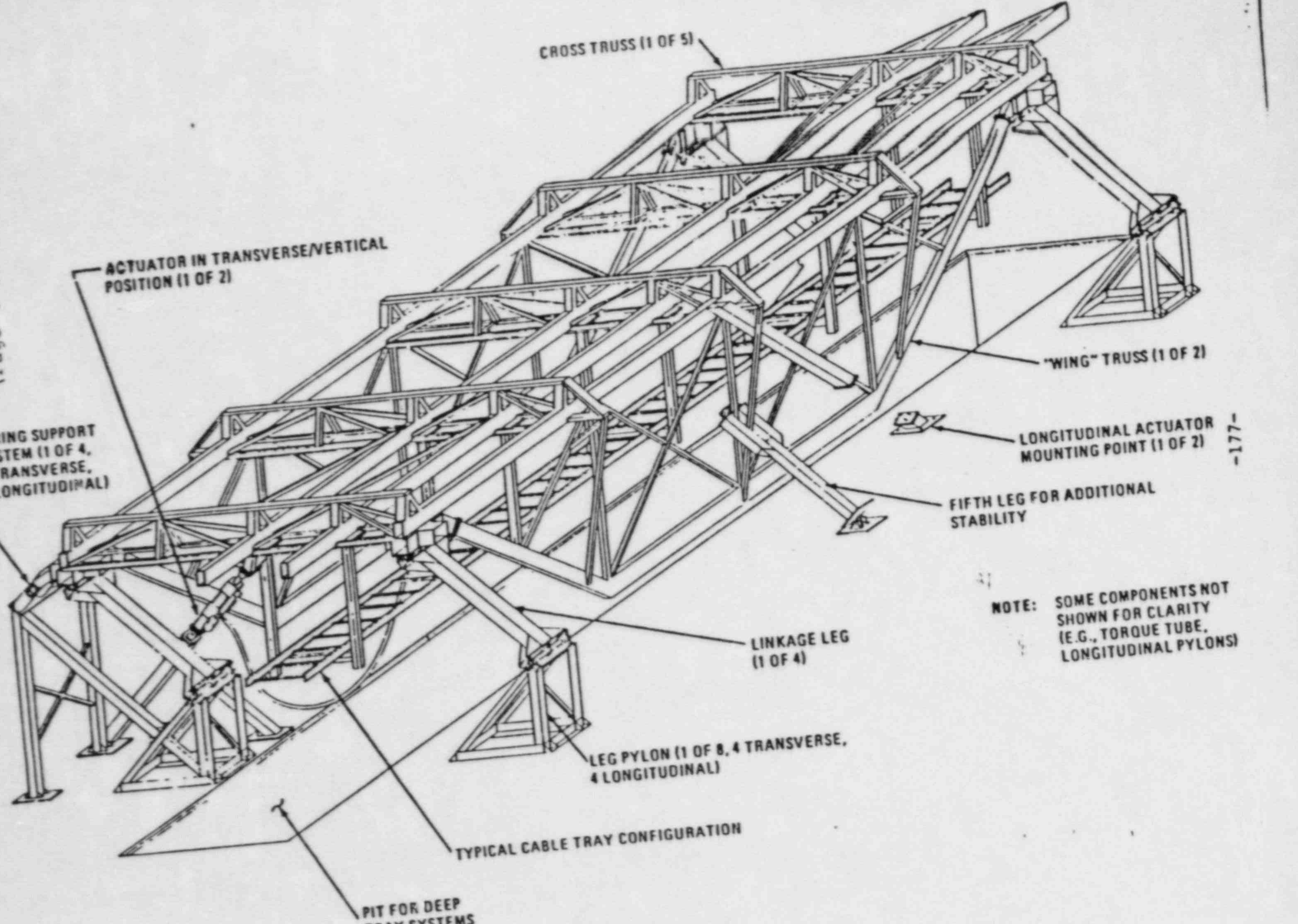


Figure 2 CABLE SHAKE TABLE

In order to evaluate the effects of these various parameters, more than 200 test setups were tested. These fell into three categories: (1) cable tray systems, (2) conduit systems, and (3) combined tray and conduit systems. Within each of these three categories of testing, test setups were developed to evaluate damping, frequency, and other significant characteristics for varying support types, connection details, and bracing.

In addition to dynamic testing with the shake table, a series of cyclic fatigue tests were performed on connection details. The purpose of these tests was to determine the resilience of these connections and establish a fatigue criterion for use in design.

TEST SEQUENCE AND SEISMIC INPUT

A typical test sequence consisted of up to ten individual tests. Initially a test setup would be subjected to snapback tests (with the table fixed rigidly). These tests were used to determine resonant frequencies and mode shapes. Next, a series of increasing amplitude sinusoidal tests were performed to establish a reference relationship between damping and amplification ratio at various output points. Finally, a series of simulated earthquake inputs were applied. These tests were used to determine how seismic input amplitude affects frequency and damping.

The earthquake time history used to formulate the majority of shake table input motions was a synthetic time history. This record was selected due to its conformance with USNRC Regulatory Guide 1.60. In addition, a group of four historical earthquake records was used during a limited group of tests. However, the actual input motion to the shake table was not the input motion corresponding to any one of the records mentioned. Rather, a modification to each record was made to account for effects of building amplification for the purpose of creating a "worst case" shake table input motion.

In addition to the synthetic time history, historical recordings of actual earthquakes were used to drive the systems. The following four earthquakes were used:

1. San Fernando 2/9/71, Hollywood Storage P.E. Lot, Comp N90E.

2. San Francisco 6/22/57, Golden Gate Park, Comp N10E.

3. Kern County 7/21/52, Taft Lincoln School Tunnel, Comp N21E.

4. El Centro 6/18/40, Imperial Valley Irrigation District, Comp S00E.

The process for selecting earthquakes was based upon the inspection of approximately ten historical recordings. Typically, each earthquake had three recorded components: two horizontal and one vertical. The goal of the selection process was to pick a nominal number of recordings that displayed different characteristics.

The synthetic earthquake was selected because of its conformance with Regulatory Guide 1.60. The response spectra shape was created to agree with USNRC guidelines.

The San Fernando earthquake is one of the best documented seismic events ever recorded and was selected primarily because of its significance.

The San Francisco earthquake, one of the shortest, was selected because of its duration characteristics and its frequency content. Most of the activity was over within the first two or three seconds of the shaking. The response spectra, depicting acceleration, shows two very distinct peaks at 4.0 and 7.0 hertz. Of the earthquakes available, none exhibited similar characteristics.

The Kern County (Taft) earthquake was selected based upon its frequency characteristics. There exists a broad band of energy between 1.5 and 8.0 hertz. In addition, the Kern County earthquake has a predominant spike around 3.0 hertz.

The El Centro earthquake was selected based upon its historical significance in the field of earthquake engineering.

TEST RESULTS

In general, rod supported raceway systems did not perform satisfactorily at input levels in excess of 0.5g. Overall collapse occurred at input levels in excess of 0.75g.

The strut supported systems that were tested survived all testing without loss of function. The type of damage that was observed in a few cases consisted mostly of fracturing of strut type angle fittings. This damage was due to low cycle fatigue resulting from significant ductile-plastic deformation that occurs at connections during large amplitude loading. Of the four angle fittings that were used to attach the hanger to the overhead steel (i.e., two fittings per vertical element, two vertical elements per hanger), never did more than one fitting of the four fracture during any one specific large amplitude test. Most of the systems were tested at input levels corresponding to 1.0 to 3.0g's maximum acceleration. These input levels were demonstrated to be equivalent to ground motion levels of 0.25 to 0.75g free-field acceleration. Never in the course of some 2000 dynamic tests did a total structural collapse of a strut-supported raceway occur. Nor was there any loss of function in the electrical circuits that were monitored. Specific results of the tests are described in the following paragraphs.

Damping

During the cable tray test program, two distinct nonlinearities associated with tray system dynamics were observed. These were: (1) inelasticity of joints and (2) amplitude dependent frictional losses due to cable vibration. Despite these nonlinearities, observed responses over a wide range of amplitudes indicated distinct vibrational modes whose frequencies degraded only with substantial changes in amplitude and a significant number of cycles of loading. Consequently, frictional losses due to cable vibration can be accounted for by selecting an appropriate amplitude dependent viscous damping. The damping of cable tray raceway systems is substantially greater than bolted steel structures due to the motion of cables within the trays. This phenomena was also observed to be amplitude dependent.

That is, the greater the input level the more pronounced were these losses. Equating these losses to an equivalent viscous damping by measurement techniques developed during the test program resulted in predicted equivalent viscous damping of up to 50% in some cases. A typical example of test results is shown in figure 3. After tabulating the results of the several hundred earthquake type vibration tests and cable tray systems (see figure 4), a conservative lower bound curve representing equivalent viscous damping as a function of input floor spectrum ZPA was plotted as shown in figure 5. This curve was plotted at two standard deviations below the mean value at each amplitude.

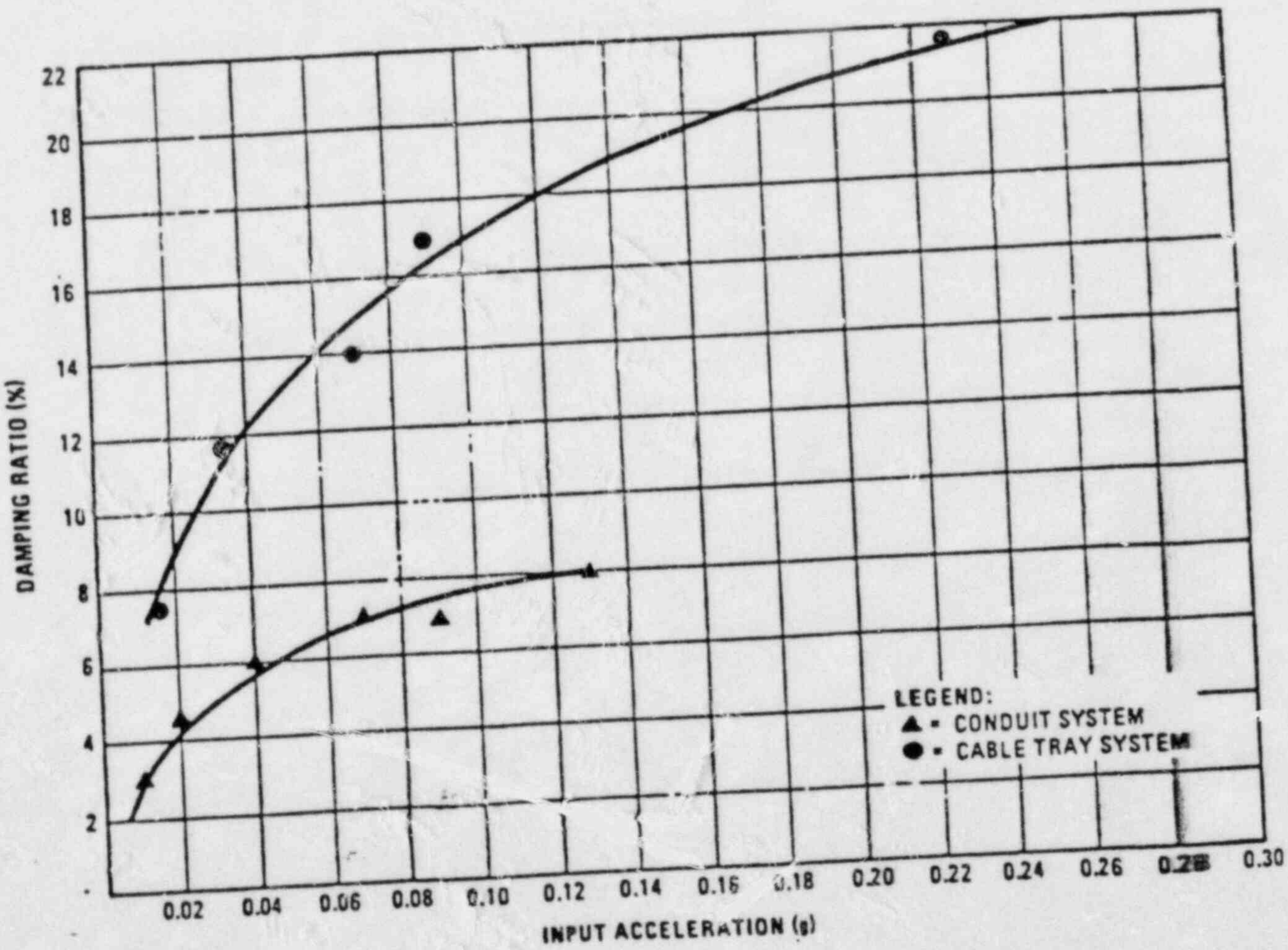


Figure 3. Typical Test Results from Conduit and Tray Tests

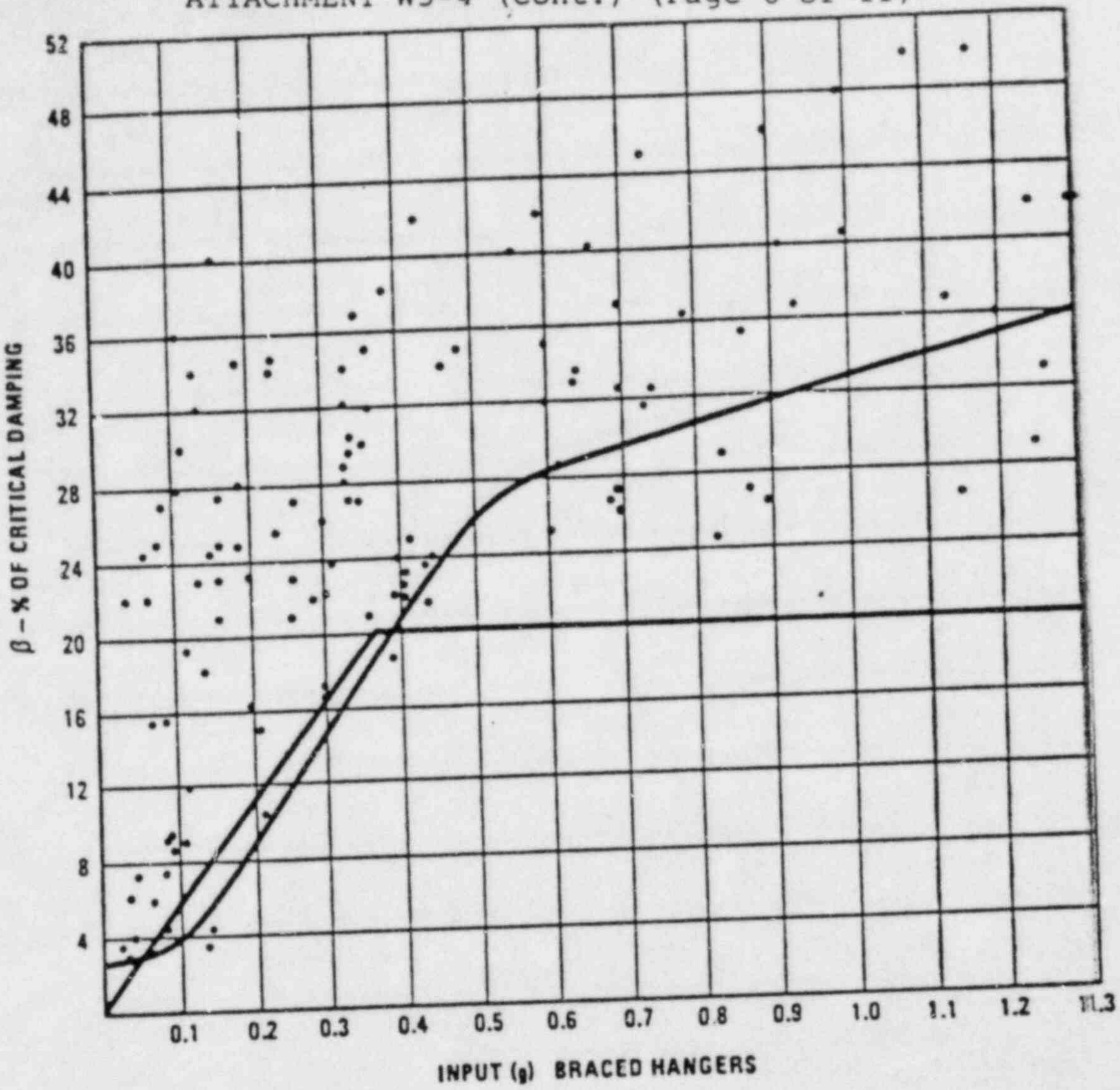


Figure 4. Damping vs. Input Level for Braced Hanger Systems

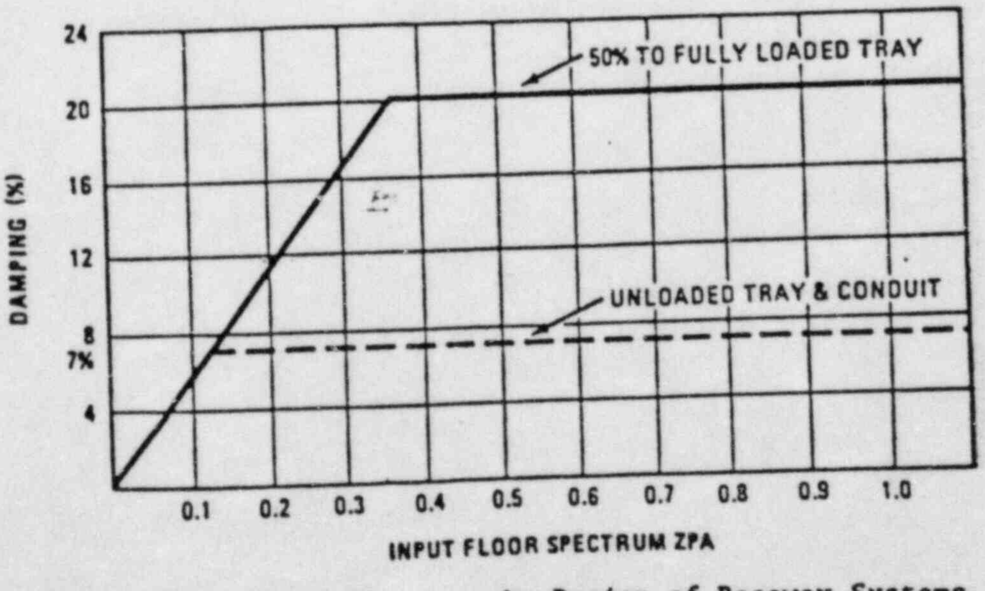


Figure 5. Recommended Damping for the Design of Raceway Systems

It should be noted that system damping varies from the above values when cable trays are lightly loaded. Specifically an unloaded tray will have an associated lower bound damping value of about 7%, which is more or less consistent with recommended values for bolted structures (see USNRC Reg. Guide 1.61). For a 24-inch tray, the damping will be in accordance with figure 5 (50% to fully loaded) as long as the tray has more than 20 lb/ft of cable.

The level of damping observed in supports that carry only conduit remains was generally about one half that observed in equivalent tray systems (see figure 3). Damping for such systems should be assumed at 7% of critical for input amplitudes in excess of 0.2g.

The overall behavior of combined systems does generally trace the behavior of its components; however, not all the specific characteristics of conduit carry through to the combination. The damping ratio of the segregated conduit system is on the order of 7% of critical. When this same conduit is added to the combined system, the overall system damping is equivalent to the damping of the cable tray system (i.e., 20% of critical).

Frequency

The testing of trapeze supports that are made from strut, and use predominantly strut type bolted fittings, demonstrated that these systems have fundamental frequencies falling between 2 and 5 hertz. The addition of heavy bracing, or the substitution of structural shapes in lieu of strut, or the attaching of supports directly to walls or columns, or the use of many welded connection details, will increase the frequencies somewhat. However, it is highly unlikely that the fundamental frequency of a raceway supported in any combination of the above methods would ever be above 10 hertz.

The dependency noted in the rate of increase of damping with respect to input has also been observed in cable tray system frequency characteristics. Generally speaking, resonant frequencies were found to be dependent upon the level of tray response. Typically, the frequency might be expected to decrease by 30 percent as input levels increase from 0.05g to 0.50g.

Connections stiffness is a major factor in determining the stiffness of a hanger system. The connections are either located where the hanger is attached to overhead supporting members or at the various joints within the hanger itself. Strut type connections do not act as a pure pin, nor do they maintain infinite rigidity. For partially braced or totally unbraced hanger systems, the moment-carrying capabilities of strut connections creates a mechanism through which initial loads may be distributed to flexible supports. The modeling of strut connections with rotational springs is a prerequisite to correct prediction of frequency characteristics, stress distribution, and deflection.

The quantity and size distribution of electrical cables that fill cable trays vary from tray to tray within a power plant. These variables were studied to assess their effects upon tray frequency. The testing demonstrated that type of size of cables do not influence overall system stiffness. The mass of the cable is the only factor that need be considered in computing cable tray system dynamic responses.

Fatigue Strength of Connections

In addition to the dynamic testing of support systems on the shake table, connection details were subjected to cyclic fatigue and strength tests. The purpose of these tests was to determine the extent to which nonlinear behavior of standard hot rolled clip angles could be utilized in the design of support systems. The primary interest was to establish low cycle fatigue information. In general, for less than 250 stress reversals, these connectors were capable of displacements of three to four times the elastic limit, which was defined by a static strength test. A typical cyclic test result is shown in figure 6. The correlation between the elastic limit of the static strength test and the horizontal limit of the fatigue curve was generally quite good. These results indicated that a reasonable ductility ratio for earthquake loadings was three to four.

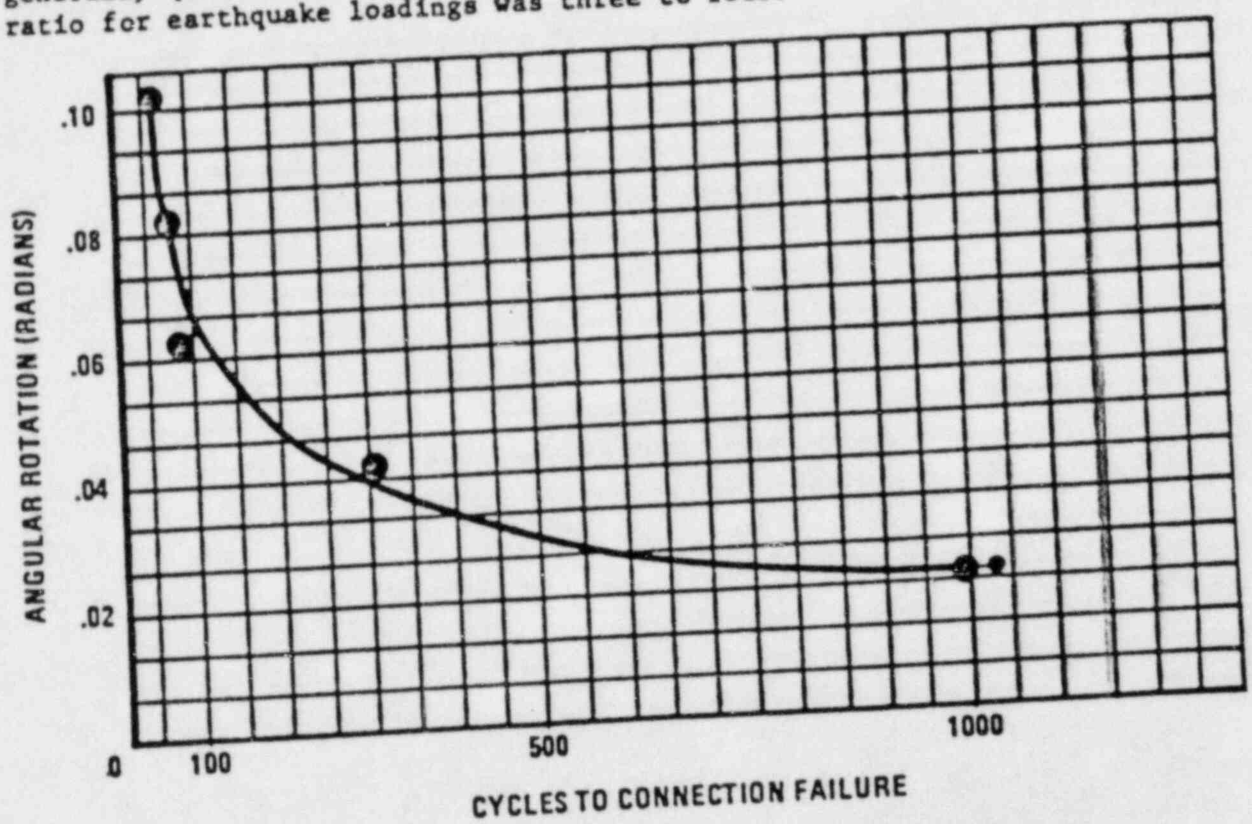


Figure 6. Typical Fatigue Curve for Hot Rolled A36 Clip Angle Connectors

CONCLUSION

The cable tray and raceway test program developed a substantial amount of data from over 2000 individual dynamic tests. This in turn resulted in some specific recommendations regarding design practice. Among these was the equivalent viscous damping in excess of 20% and the significant resilience of hot rolled clip angles under low cycle fatigue. Of particular significance is the general conclusion that lightly braced raceway systems can be expected to survive severe earthquakes (in excess of 0.5g) with no loss of function in the circuits they support.

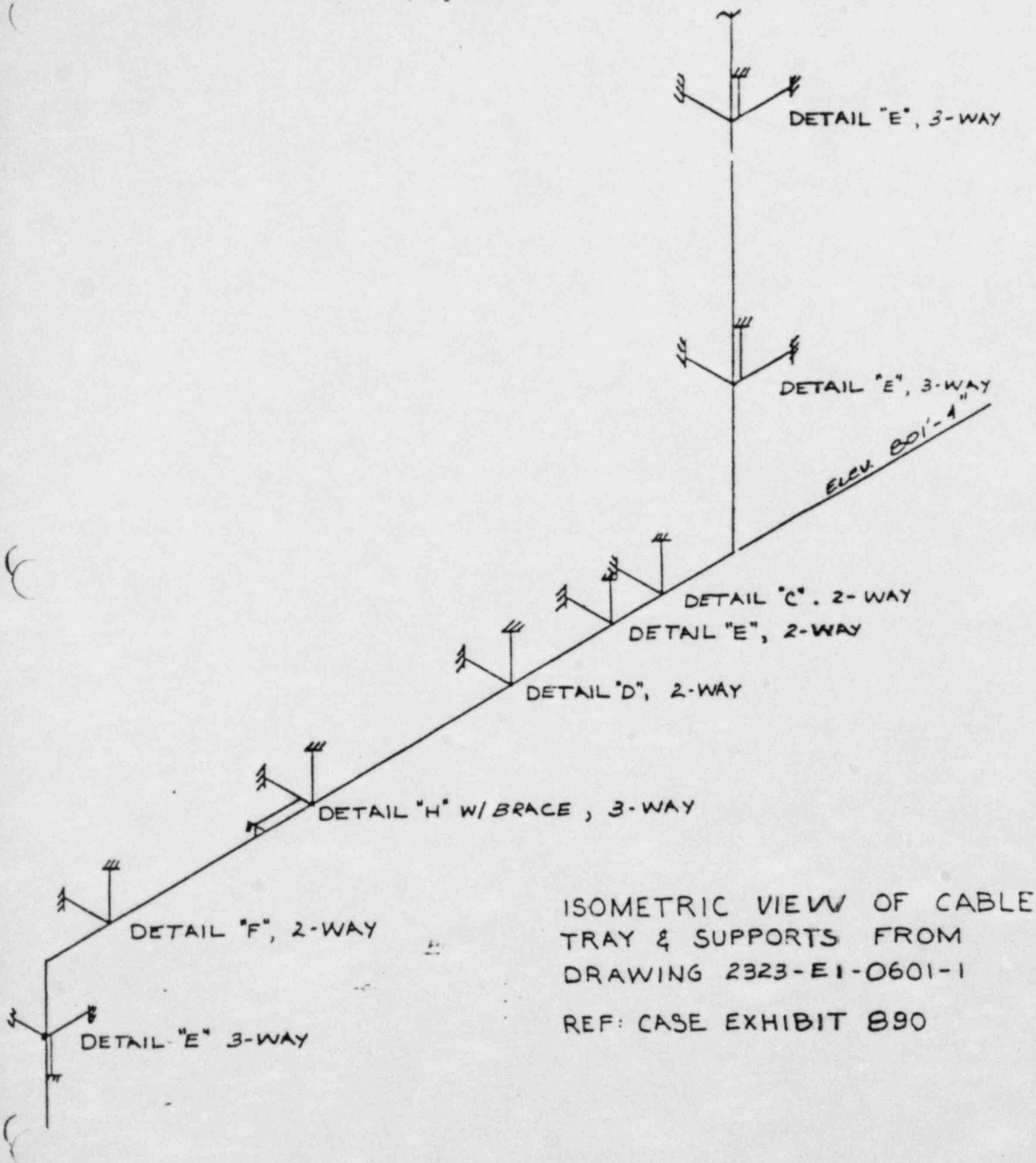
ACKNOWLEDGEMENTS

The Cable Tray and Conduit Raceway Test Program has been supported financially by contributions from the Arizona Nuclear Power Project participants, Bechtel Power Corporation, Boston Edison Co., Georgia Power Co., Mississippi Power and Light Co., Pennsylvania Power and Light Co., Philadelphia Electric Co., Public Service Electrica and Gas Co., Puget Sound Power and Light Company, Standardized Nuclear Unit Power Plant System (SNUPPS) participants, Southern California Edison Co., and the Tennessee Valley Authority.

REFERENCES

(1) Cable Tray and Conduit Raceway Test Program Test Report, ANCO Engineers and Bechtel Power Corporation, December 1978.

(2) USNRC Regulatory Guide No. 1.61, "Damping Values for Seismic Analysis for Nuclear Power Plants."



ISOMETRIC VIEW OF CABLE
TRAY & SUPPORTS FROM
DRAWING 2323-E1-0601-1

REF: CASE EXHIBIT 890

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #6
Exhibit No.: 907

1.0 CASE Question

CTS-00-05. In the description, it discusses a channel bent about its weak axis. The resolution does not consider this problem nor does the document CASE requested on discovery; see CASE Exhibit 907. On CMC 88306, are the originator and approver the same person?

2.0 Cygna Interpretation

Please discuss the following:

a. How did the resolution to Observation CTS-00-05 address the channel bent about its weak axis?

b. Are the signatures on CMC 88306 satisfactory?

3.0 Response

a. The purpose of Observation CTS-00-05 was to investigate the baseplate. This is illustrated by the following reprint from the Observation:

> "1.0 Description
>
> The anchor bolts, baseplate/angle and channel of cantilever support Detail "E" were originally designed as two-way restraints to resist axial loads on the channel and moments about its major axis. In order to use Detail "E" on a cable tray riser, where it must act as a three-way restraint, the channel section was modified to resist moments about its weak axis. The ability of this configuration to function as intended, i.e., to also resist moments about the weak axis, could not be guaranteed since the anchor bolts and the baseplate/angle were not evaluated for such a load."

The channel was correctly analyzed by Gibbs & Hill in Calculation SCS-146C, sets 4 and 8.

b. CMC-88306, Rev. 4, was originated and approved by the same person. This is acceptable for the following reasons:

- There is a controlled list of people authorized to approve CMC's for construction prior to design review. In the case of CMC 88306, the approver was on that authorized list.

- Project procedures do not prohibit someone on the authorized approval list from also being an originator.

- The subject CMC is an interim release for construction purposes. Each CMC receives a subsequent design review by the original design organization in accordance with Gibbs & Hill Procedure DC-7.

1.0 CASE Question

CTS-00-006 What is the "significant design margin" as shown in the resolution?

2.0 Cygna Interpretation

Observation CTS-00-06 states that "... further analyses by Gibbs & Hill (see Cygna Technical File 11.2.1.50, pp. 31-69), incorporating Cygna's comments, revealed that sufficient design margin existed to compensate for the increased stress levels." The "increased stress levels" refer to the potential increase in stress levels due to the items noted in the observation.

Please quantify the design margins.

3.0 Response

To demonstrate the adequacy of a judgement made in their qualification of standard details A, B, C, and D by similarity to standard detail D_i, Gibbs & Hill performed an analysis using the NASTRAN code. For the purposes of this analysis, the C6 x 8.2 section was oriented to match details A, B, C, and D. The results of this analysis are contained in calculation SCS-104C, Set #1, where it is shown that the member interaction ratio for the C6 x 8.2 section is 0.94 (maximum). This ratio is based on an analysis using tray weights of 35 lb/ft^2 and which included tray support self-weight excitation. The "significant margins" are due to the fact that the interaction alone was 6% below allowable and the tray loads were assumed to be 22% larger than the actual loads.

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Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #8
Exhibit No.: None

1.0 CASE Question

CTS-00-07. The analysis that included the beam element did not consider prying action and the flexibility of the baseplate to determine the center of compression.

2.0 Cygna Interpretation

N/A.

3.0 Response

Gibbs & Hill performed a refined analysis of the frame and baseplate to resolve Observation CTS-00-07. Cygna reviewed the results of this analysis and judged the frame, baseplate and anchor bolt design to be adequate.

In order to quantify the adequacy of that engineering judgement, relative to the anchor bolt design, Cygna performed an analysis of the frame/baseplate system using fixed boundaries at the hanger-to-baseplate connections. The fixed-end loads developed at these boundary points were then applied to a baseplate model. Cygna's program PSDS (Pipe Support Design System) was utilized for the analysis and design check. PSDS includes a standard baseplate/anchor bolt routine that considers mechanisms, such as prying action and baseplate flexibility.

The results of this analysis show the following design margins:

Bolt No.	Tensile Load (lbs)	Shear Load (lbs)	Design Interaction Ratio*
1	500	1540	.10
2	4[illegible]40	1830	.75
3	3040	1890	.45
4	2970	1820	.45
5	4210	1530	.65

*Design Interaction Ratio = $(\text{tensile load/allow.})^{5/3} + (\text{shear load/allow.})^{5/3} < 1.0$

The design interaction ratio equation, using an exponent of 5/3, was originally contained in Revision 0 to Cygna's review criteria for Comanche Peak. In Revision 1 of the Comanche Peak review criteria, the exponent was reduced to 1.0 to be consistent with the equation actually used by Gibbs & Hill.

Further justification for the 5/3 exponent is provided in the response to Walsh Question #5. It is also important to note that these results contain the following conservatisms: lumped tray masses, enveloped response spectra, higher than actual tray weights (35 psf vs. 28 psf).

AS-BUILT RE-VERIFICATION REPORT ...

ITEM DESCRIPTION SFX-033-007-F43R	ATTACHMENT W9-1 (Page 1 of 1)		STRESS PROBLEM 1-086A
SRP N/A	REV.	COMPL.	ADDITIONAL INFORMATION
1. All dimensions and elevations verified		NA	
2. Valve orientations verified		|	
3. Orientation verified for line mounted equip.		NA	
BRHL/GHH N/A	REV.		
1. Support mark numbers verified		NA	
2. Support locations verified		NA	
BRH SFX-033-007-F43R	REV. 2		
1. Direction of support verified		✓	
2. Type of support verified		✓	
3. General configuration verified		✓	
4. Clearances (where applicable)		✓	
5. Location verified and appropriate red marks entered on BRHL/GHH SF-X-FB-028	REV. 1	✓	Dwight E. Watts 1/31/83

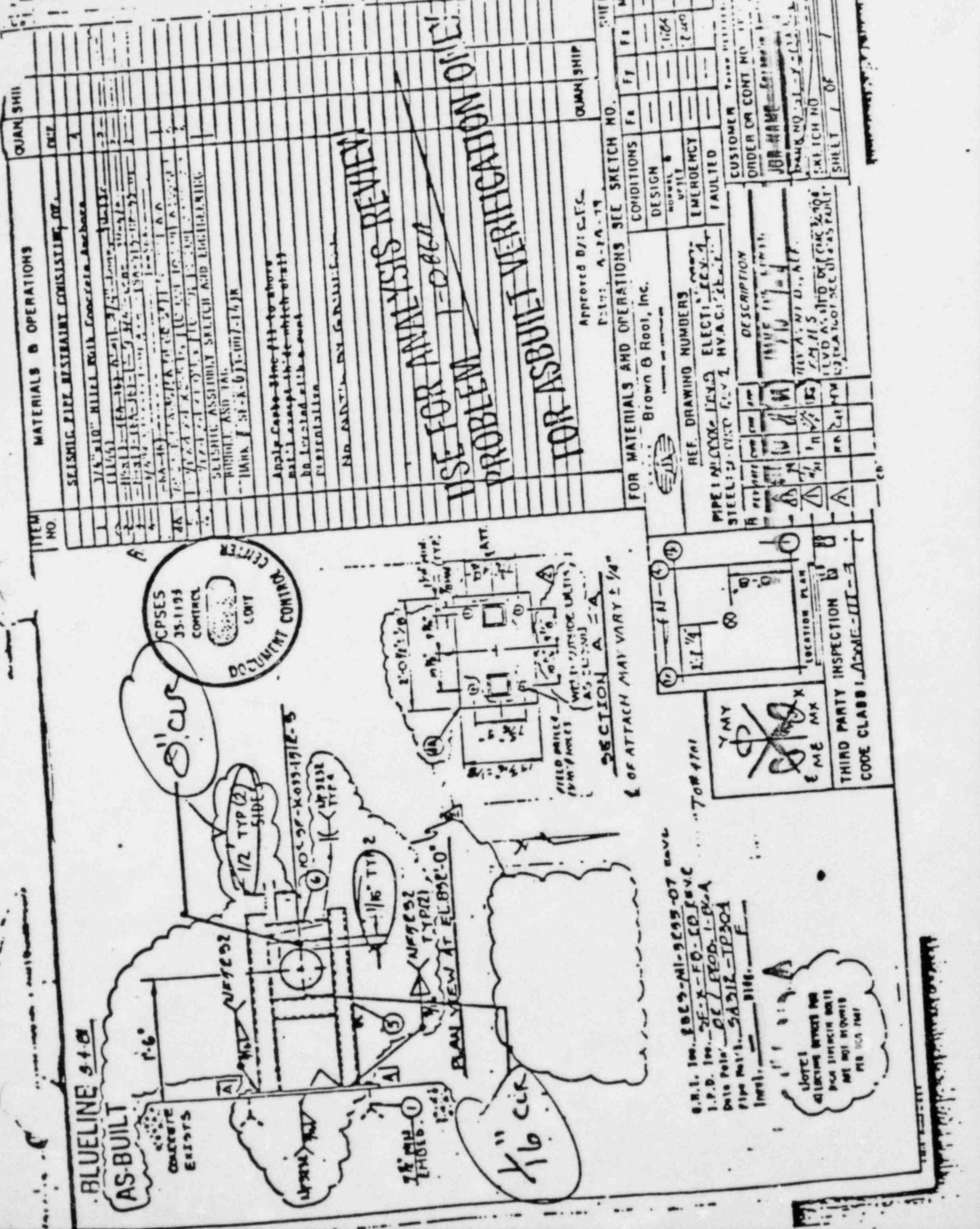
COMMENTS

TRANSMITTED TO DESIGN REVIEW SH 3-3-83

ABRVF # 1267 Reviewed and Filed. No Engineering Action Required.
By Amk Date 4-20-83

DISPOSITION	INSPECTION
The above listed documents were reviewed, no change to location or function has occurred therefore inspection is not required	Inspected by: Mike Osterman 2/1/83 (As-Built Inspector)
	Ron Michels 2/2/83
TSABC DATE	QAABC DATE

ATTACHMENT W9-2
(Page 1 Of 1)



Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #10
Exhibit No.: N/A

1.0 CASE Question

WD-07-02 What document did Cygna see that showed the temperature indicator would be installed at a later date?

2.0 Cygna Interpretation

What was the basis for closing Cygna Observation WD-07-02? What documentation was reviewed?

3.0 Response

Based on a conversation with Texas Utilities personnel, Cygna learned that temperature elements are normally installed after all other work in an area is completed. This is done in order to avoid damage to the instrument during construction. When Cygna performed the Spent Fuel Pool Cooling System walkdown, painting activities were still underway.

Further review also showed that local indicators, such as this one, are not safety-related devices.

The key documents reviewed by Cygna relevant to closing Observation WD-07-02 are discussed below:

1. Instrument Installation Checklist (Form No. 2-81)

   Form 2-81 is required to be completed by Comanche Peak procedure 35-1195-ICP4. In this case, it indicated that the device was not installed and that the "discrepancy" was "turned over to Brown & Root completion and TUGCO".

2. The Q-list was checked to ensure that the device was non-safety.

Comanche Peak ASLB Hearings
Response to CASE Questions
Question No.: Walsh #11
Exhibit No.: None

1.0 CASE Question

Pipe stress checklist, note 3, item a:

1) What is the basis for considering that the effects were negligible?

2) What pipe stress run did Cygna look at, since the inclined load was used in the design of support RH-1-010-003-S22R?

2.0 Cygna Interpretation

Pipe stress checklist (PI-02), note 3, states the following:

3. The following supports were modeled along the coordinate axis rather than inclined. The impact is negligible.

   a. RH-1-010-003-S22R at data point 1253 (8.6 degrees)

   b. SI-1-042-001-S22R at data point 793 (7.5 degrees)

a. What was the basis for concluding that support RH-1-010-003-S22R was adequate?

b. What pipe stress run was evaluated?

3.0 Response

a. Support RH-1-010-003-S22R is a simple restraint, inclined 8.6 degrees from a line drawn perpendicular to the pipe. Cygna judged that this small inclined angle would not significantly affect the support design or the piping analysis. An important element of this judgement is that the 8.6 degree, as-built alignment is only 3.6 degrees beyond the construction tolerance of 5.0 degrees.

   In order to verify the adequacy of this judgement, Cygna requested that Gibbs & Hill reanalyze piping segment AB-1-70. For this reanalysis, the piping model was revised to include the following:

- o Supports RH-1-010-003-S22R and SI-1-042-001-S22R were modeled with skew angles of 8.6 and 7.5 degrees, respectively.
- o Support RH-1-010-003-S22R was modeled as two trunnions with snubbers located 7 inches from the pipe centerline.
- o Support RH-1-064-010-S22R was modeled 1'-4" west of the elbow.

The results of this reanalysis are contained in Attachment D11-1 (Gibbs & Hill Calculation), D11-2 (computer output without modifications), and D11-3 (Computer output with modifications). These results are summarized below:

Maximum System Stress (psi)

ASME Equation	Old	New	Allowable
8	9,039	9,039	18,480(1)
9 (upset)	21,094	21,103	22,180(2)
9 (emergency)	24,451	24,463	33,260(3)
10	22,883	22,883	27,600(4)
11	27,881	27,881	46,080(5)

Notes:

(1) 1.0 S_h, per ASME B&PV Code, Section III, Paragraph NC-3652.1

(2) 1.2 S_h, per ASME B&PV Code, Section III, Paragraph NC-3652.2

(3) 1.8 S_h, per ASME B&PV Code, Section III, Paragraph NC-3611.3c

(4) $S_a = f(1.25\ S_c + 0.25\ S_h)$ where f = 1.0, for no more than 7,000 thermal cycles, per ASME B&PV Code, Section III, Paragraph 3652.3a.

(5) $S_a + S_h$, per ASME B&PV Code, Section III, Paragraph 3652.3b.

where S_h = 18480 for material SA-312, TP 304 at 280°F
S_c = 18800 psi
Per ASME B&PV Code, Section III, Appendix I

Support Loads (lbs)

	Normal		Upset		Emergency		
	Old	New	Old	New	Old	New	Allow.*
RH-1-010-003-S22R	1705	1459	3534	4519	3967	5189	15700
	105	164	-1724	-2894	-2756	-3565	-15700

*Per NPSI Load Data Capacity Sheet, dated 6/81, for an SRS No. 14 strut.

Regarding the following line excerpted from Attachments D11-2 and D11-3, the allowable Equation (9) stress for emergency conditions is 1.8 S_h per ASME B&PV Code, Section III, Paragraph NC-3611.3c. The comparison to 1.2 S_h in ADLPIPE is a built-in precaution, not a pass/fail test.

Stress Summary (Equation 9 Emergency and Faulted Conditions)

SEC	MEM	SEQ	POS	EQN 9	Additional Information
20	52	896	BEG	13016	
20	5	897	END	24451	Equation 9 exceeds 1.2 S_h

Nozzle Loads (lbs)

	Old	New
Load	3084	254!
Allowable	3120	3120
Ratio	0.98	0.81

In summary, the reanalysis showed no change in the pipe stresses, a decrease in nozzle loads, and support loads well below the allowable. This verifies the original engineering judgement.

CYGNA

b. Gibbs & Hill pipe stress run AB-1-70, Rev. 0, was evaluated by Cygna as noted on several Observations, including PI-00-01.