UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

AFFIDAVIT OF JOHN C. FINNERAN, JR. IN SUPPORT OF APPLICANTS' REPLY TO CASE'S ANSWER TO APPLICANTS' MOTION FOR SUMMARY DISPOSITION REGARDING CONSIDERATION OF FRICTION FORCES

I, John C. Finneran, Jr., being first duly sworn hereby depose and state, as follows: I am the Project Pipe Support Engineer for the Comanche Peak Steam Electric Station. In this position I oversee the design work of all pipe support design organizations for Comanche Peak. A statement of my educational and professional qualifications was received into evidence as Applicants' Exhibit 142B.

Q. What is the purpose of this affidavit?

A. This affidavit provides information in support of Applicants' Reply to CASE's Answer to Applicants' Motion for Summary Disposition Regarding Consideration of Friction Forces.

Q. What is your reply to CASE's assertions with respect to Applicants' first statement of material facts?

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A. CASE claims in its response to Applicants' first statement of material facts that Gibbs & Hill designed the upper lateral restraint and, thus, have designed other "supports" than the moment restraints, and that the STRUDL group "was under Gibbs & Hill's supervision," implying that the practice of that group, viz., to include friction effects only when instructed, was somehow inconsistent with Applicants' position in their motion (Affidavit at 2). In the first instance, the upper lateral restraint is not a pipe support, it is a restraint for movement of the steam generator. Further, although the STRUDL group was under the supervision of a Gibbs & Hill employee, it was not a Gibbs & Hill organization. When the STRUDL group performed analyses for each pipe support design organization, friction was included in those analyses consistent with each organizations' instruction. As we noted in our motion (see my affidavit at 1-2), some design organizations did not consider friction for pipe movements less than 1/16". Thus, the STRUDL group would not always be required to include friction. This is fully consistent with Applicants' position throughout the proceeding.

Q. What is the next aspect of CASE's answer to which you wish to provide a reply?

A. CASE does not specify any particular disagreement with Applicants' second through fifth statements of material fact although claiming that it disagrees to a certain extent with

the fourth statement, without specifying any particular point with which it disagrees. Thus, the next aspect of CASE's answer to which I wish to reply concerns their comments regarding our sixth statement of material fact.

Q. What is your reply to CASE's answer to Applicants' sixth statement of material fact?

A. CASE presents several arguments regarding this statement of fact, none of which disprove Applicants' position. Specifically, CASE contends (Affidavit at 7), that Applicants have misconstrued Section NF-3231.1 of the ASME Code. CASE directs its argument to the statement in my affidavit accompanying Applicants' motion (Finneran Affidavit at 4-5) that if the effects of friction are included in the design of supports, the allowables applicable to such loading combinations could be increased pursuant to NF-3231.1. CASE incorrectly construes Applicants' position to be that friction effects may be included in all analyses in order to obtain a general increase in allowables for all loading combinations. To the contrary, mechanical loading combinations (not including friction) must satisfy applicable allowables without any increase. If friction effects are included, those loading combinations may utilize the increased allowable.[1] CASE simply misinterpreted Section NF-3231.1.

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1 As I previously noted, Applicants' standard practice is not to take advantage of this increase in allowables, even when friction is included.

CASE next asserts that when the effects of friction alone are considered, the stress ratios (load divided by allowable) are of such magnitude that inclusion of mechanical loads would create an overstressed condition (Affidavit at 7). CASE illustrates its position using a calculation Applicants prepared in response to an NRC Staff question which showed that when the effects of friction alone are calculated for a particular support the stress ratio was determined to be .775. CASE contends that this leaves only .225 of the ratio for mechanical loads. CASE does not acknowledge, however, that (as I indicated in my statement to Mr. Fair referenced by CASE) friction forces do not act alone, and that when the normal load (which gives rise to the friction force) is included in the calculation, it tends to offset the effe t of the friction force. In this instance, when friction and the normal load are combined the stress ratio actually drops from the .775 calculated for friction alone to .46. Specifically, as noted in Table 1 of my original affidavit, the loading combination of friction plus Level A loads for this support (SI-1-029-055-S32R) produces a .96 stress ratio (3042 divided by 3181), using a conservative calculation technique (treating the compression force in the support member as a tension load on the weld (see p. 4 of 7, bottom of calculation)). To compare accurately the friction plus normal load combination with the friction load alone it is

necessary to evaluate the weld stresses as they actually occur. Doing this for the level A loads plus friction load combination for the weld yields:

$$f_t \text{ (weld tension stress)} = \frac{4972\ (15.5)}{34.47} - \frac{16573}{20.8} = 1439 \text{ lb/in}$$

Then the resultant tension stress on the weld is:

$$f_R = (1439^2 + 239^2)^{1/2} = 1459 \text{ lb/in}$$

The resulting stress ratio is then .46 (allowable stress = 3181 lb/in). Performing a similar evaluation for Level B loads yields a stress ratio of .42. In sum, it is simply incorrect to assert, as CASE does, that the stress ratios calculated considering friction alone are at all indicative of the stress ratios for combined mechanical and friction loading.

CASE's next claim concerns Applicants' use of a 5:1 safety factor for Hilti bolts (Affidavit at 8). CASE notes that a list of Hilti allowables using a factor of safety of 4:1 is included in the PSE design manual. CASE fails to point out that the list CASE refers to is simply a letter from Hilti, Inc., furnished by NPSI, containing load capacity data for use by organizations which order Hilti bolts from NPSI, which Applicants do not.[2] The sheet is included in Section XII of the manual with other load

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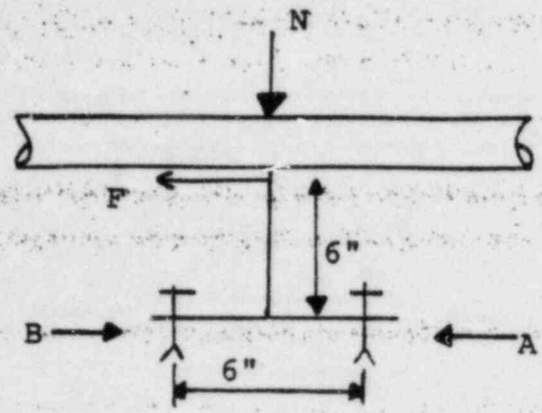
2 Hilti, Inc. recommends a 4:1 safety factor for Hilti bolts. Applicants have adopted, however, a 5:1 safety factor for Hiltis. This practice is documented in Applicants' design specifications. A copy of the page from these specifications which calls for the use of the 5:1 safety factor was provided to CASE with Applicants' motion.

capacity data, for general information. The actual design requirements for Hilti bolts at Comanche Peak, using a 5:1 safety factor, are reflected in Section V of the PSE manual. In fact, CASE used data from Section V only two pages later (Affidavit at 10) in its own illustration of these effects.

CASE's final claim with respect to this portion of Applicants' motion is that failure to consider friction effects could result in failure of anchor bolts (Affidavit at 9-10). To illustrate its point, CASE uses a hypothetical loading condition which is premised on the same fundamental misconception regarding friction loads which I discussed above with respect to stress ratios. CASE apparently does not understand that a friction load does not act alone. A friction load must act with the normal load from which it results. In CASE's hypothetical the normal load has simply been neglected. Consideration of the normal load for CASE's hypothetical shows that the bolt which CASE claims would pull out cannot even be put in tension. Using CASE's assumed friction force (F_f of 4900 lbs.), the normal load (N) producing that friction load is easily calculated as 4900 lbs divided by an assumed coefficient of friction (.3), which is 16,333 lbs. Simply summing all moments about



bolt B demonstrates that bolt A cannot even be put in tension, although CASE represented that there will be a 4900 lb. tension load on the bolt. CASE's assertion is, therefore, false.

Q. What is your response to CASE's arguments regarding Applicants' seventh statement of material fact?

A. CASE contends that Applicants employed an incorrect moment arm in the evaluation of this support. CASE argues that use of the correct moment arm would result in "a 37% increase of all tabulated values." (Affidavit at 11.) CASE is correct in its calculation of the actual moment arm. However, CASE incorrectly asserts that the increase would be applied to "all" values. That increase actually applies only to one moment term (My). Correctly including this revised moment arm in the calculation demonstrates that the stress remains below the applicable allowable.

I have recalculated the stresses in the weld, as follows. First applying the increased moment arm to the My term (multiplied by 1.37) yields

$$\frac{13.835}{5.17} \times 1.37 + \frac{1.010}{16.88} - \frac{4.768^3}{12.63} = 3.3485 \text{ kips/in}$$

The resultant force is then

$$F_R = [3.3485^2 + .062^2 + .167^2]^{1/2} = 3.353 \text{ kips/in}$$

This remains below the 3.431 allowable used in the original calculation. Thus, contrary to CASE's assertion, the stress ratio for the weld remains below 1.

CASE also contends that the increased moment arm would cause the stress ratio for the anchor bolts to exceed one (Affidavit at 11). However, as already noted, the increase in moment arm only affects the moment about one axis. Thus, the stress ratio for the anchor bolts only changes from .81 to .89, contrary to CASE's claim. This can be seen by increasing the My moment in the original Hilti bolt calculation (Attachment A to my original affidavit, second p. 3 of 6) from 15.438 in-kips to 21.150 in-kips. I have attached a new FUB II analysis printout reflecting this change in My (Attachment A). The Hilti interaction now becomes

$$\frac{3509}{4688} + \frac{756}{5375} = .89$$

CASE also questions other dimensions employed in Applicants' calculation (Affidavit at 11-12). The 1'6" dimension questioned by CASE is from the center of the pipe to the centerline of the vertical struts and not to the tube

3 This load is evaluated here as a compression load (which it is) rather than the tensile load conservatively assumed in the original calculation.

steel as CASE apparently believes. The tube steel <u>is</u> installed with a 1/16" clearance as indicated on the drawing.

CASE next contends that Applicants utilized an incorrect allowable in assessing the shear yield stress (Affidavit at 12). CASE apparently does not understand the nature of the shear yield stress check employed in the subject calculation. That check utilized a shear yield stress value from the AISC Code.[4] (<u>See</u> AISC Manual, 7th Edition, p. 5-123 (Attachment B)). This check was performed simply to provide added assurance of the adequacy of the weld. That AISC formula, in fact, yields an allowable ($Sy/\sqrt{3} = .577\ Sy$) which is equivalent to that which CASE apparently argues should be employed ($1.5 \times .4\ Sy = .6\ Sy$; <u>see</u> Affidavit at 12, 13). It should be noted in any event that CASE's position is premised on Regulatory Guide 1.124, which applies to class 1 supports. The subject support is a class 3 support.

Q. What is your response to CASE's arguments with respect to Applicants' eighth statement of material fact?

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4 Contrary to CASE's claim (Affidavit at 13) Applicants are not "committed" to any edition of the AISC Code for weld design. Applicants' requirements for weld design are set forth in subsection NF of ASME Code Section III. Applicants do not reference the AISC Manual for the purpose of establishing weld design criteria for ASME supports.

A. In challenging Applicants' eighth statement of material facts, CASE first claims that it does not know "how random Applicants' sample was or the criteria used for their selection" (Affidavit at 14). As I stated in my original affidavit (at 5-6), Applicants' sample of supports was selected by applying two criteria: (1) the support is of the type which (as both parties agree) would be most significantly affected by inclusion of friction effects, viz., short, stiff support members with relatively large pipes, and (2) the pipe thermal movement is less than 1/16" (so friction would likely have been neglected in the original design). To identify supports which satisfied these criteria, I requested that my engineers review support drawings at random to identify these supports. The resulting supports were, therefore, randomly selected in accordance with the established selection criteria.

CASE next attempts to discredit Applicants' calculations regarding the sample supports by examining the calculations for one such support, contending that it "will illustrate the shortsightedness of neglecting assumed minor effects" (Affidavit at 14-15). In this regard CASE first asserts (Affidavit at 15, paragraph (2)) that the stress ratio calculated with the effects of friction included was "almost four times as high" as that without friction. Applicants fail to see the significance of this argument. In the first instance, the initial stress ratio was so low

that even with the increase due to friction the stress ratio remains well within that which is acceptable. In addition, CASE fails to point out that the allowable when including friction would be higher than the normal allowable and, thus, the stress ratio would be even lower.

CASE further claims (Affidavit at 15, paragraph (3)) that inclusion of the effects of friction in that calculation would increase the level A stress ratio in the weld from .25 to .96. CASE believes that this demonstrates the weld would not be able to take much increase in load before it would exceed allowable. (CASE asserts that such a load increase "could" be caused by several effects, but does not quantify its argument.) Contrary to CASE's assertion, as I previously demonstrated (see pp. 4-5) the weld stress ratio is actually .46 when the effects of friction are realistically calculated. Further, both the .96 and .46 stress ratios are based on the normal allowables and do not take advantage of the increase in allowable that would be permitted. Thus, ample margin to the allowable remains even when friction effects are included.

With respect to CASE's next two arguments (Affidavit at 16, paragraphs (4) and (5)) the following points should be noted. First, with respect to paragraph (4), there is a rigid support less than three feet from the subject support which prevents additional side load from being imparted. Thus, CASE's concern regarding the potential effect of

additional side load is unfounded. Further, with respect to paragraph (5), the note to which CASE refers is only a rough approximation of the friction load which would be imparted if the pipe were to move into the curvature of the U-bolt. This calculation was performed merely to confirm that the controlling friction load occurs when the pipe is against the backing plate. Further, it is not appropriate to characterize this rough approximation as indicating U-bolt stiffness. Applicants' have utilized actual test data for these values in our other motions for summary disposition.

Finally, CASE disputes a statement made in the Cygna Phase III Report, attributed to my original affidavit, regarding the consideration of friction in the upset loading condition (Affidavit at 16-17). The statement in the Cygna Report is not, however, derived from my affidavit. In fact, the statement does not relate at all to the issue of friction effects for small pipe movements. Thus, the point is not relevant to Applicants' motion.

John C. Finneran, Jr.

John C. Finneran, Jr.

STATE OF TEXAS
COUNTY OF SOMERVELL

Subscribed and sworn to before me this 19th day of September, 1984.

Bill J. Hodges

Notary Public
MY COMMISSION EXPIRES MARCH 28, 1988

0.	00
0.	01
0.	02
0.	03
6660.	04
1306.	05
1631.	06
756.	07
21150.	08
6377.	09
6.	10
6.	11
6.	12
6.	13
6.	14
6.	15
6.	16
6.	17
0.	18
0.	19
0.	20
1.	21
1.97	22
2.	23
0.	24
1.	25
1.	26
0.	27
0.	28
0.	29

F MAX
3508.91

P STR
17393.42

SHEAR
756.52
756.52
756.52
756.52

FUB II R3

1.5.1.2 Shear

While the shear yield stress of structural steel has been variously estimated as between one-half and five-eighths of the tension and compression yield stress and is frequently taken as $F_y/\sqrt{3}$, it will be noted that the permissible working value is given as two-thirds the recommended basic allowable tensile stress, substantially as it has been since the first edition of the AISC Specification, published in 1923. This apparent reduction in factor of safety is justified by the minor consequences of shear yielding, as compared with those associated with tension and compression yielding, and by the effect of strain hardening.

The webs of rolled shapes are all of such thickness that shear is seldom the criterion for design. However, the web shear stresses are generally high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the web thickness is less than

$$\frac{32M}{A_{bc}F_y}$$

where M is the algebraic sum of clockwise and counter-clockwise moments (in kip-feet) applied on opposite sides of the connection boundary and A_{bc} is the planar area of the connection web, expressed in square inches. This expression is based upon the assumption that the moment M is resisted by a couple having an arm equal to $0.95d_b$, where d_b is the depth of the member introducing the moment. Designating as d_c the depth of the member entering the joint more or less at right angles to it, and noting that A_{bc} is approximately equal to $d_b \times d_c$, the minimum thickness of the web not requiring reinforcement can be computed from the equation

$$\text{allowable shear stress} = 0.40F_y = \frac{12M}{0.95A_{bc}t_{min}}$$

1.5.1.3 Compression

1.5.1.3.1 Formulas (1.5-1) and (1.5-2) are founded upon the basic column strength estimate suggested by the Column Research Council.* This estimate assumes that the upper limit of elastic buckling failure is defined by an average column stress equal to one-half of yield stress. The slenderness ratio C_c, corresponding to this limit, can be expressed in terms of the yield stress of a given grade of structural steel as

$$C_c = \sqrt{\frac{2\pi^2E}{F_y}}$$

A variable factor of safety has been applied to the column strength estimate to obtain allowable working stresses. For very short columns this factor has been taken as equal to, or only slightly greater than, that required for members axially loaded in tension, and can be justified by the insensitivity of such members to accidental eccentricities. For longer columns, approaching the Euler slenderness range, the factor is increased 15 percent, to approximately the value provided in the AISC Specification since it was first published 46 years ago.

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* Column Research Council Guide to Design Criteria for Metal Compression Members, *Second Edition, Eqs. (2.11) and (2.12)*.

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	)	
(Comanche Peak Steam Electric Station, Units 1 and 2)	)	(Application for Operating Licenses)

CERTIFICATE OF SERVICE

I hereby certify that copies of the "Applicants' Reply to CASE's Answer to Applicants' Motion for Summary Disposition Regarding Consideration of Friction Forces," in the above-captioned matter was served upon the following persons by express delivery (*), or deposit in the United States mail, first class, postage prepaid, this 19th day of September, 1984, or by hand delivery (**) on the 20th day of September, 1984.

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