220



LAW OFFICES OF

BISHOP, LIBERMAN, COOK, PURCELL & REYNOLDS

1200 SEVENTEENTH STREET, N. W.
WASHINGTON, D. C. 20036
(202) 857-9800

TELEX 440574 INTLAW UI

IN NEW YORK
BISHOP, LIBERMAN & COOK
26 BROADWAY
NEW YORK NEW YORK 10004
(212) 248-6900
TELEX 222767

July 11, 1984



Geary Mizuno, Esq.
Staff Counsel
U.S. Nuclear Regulatory Commission
7735 Old Georgetown Road, Room 10107
Bethesda, Maryland 20814

Re: Texas Utilities Electric Company, et al.
(Comanche Peak Steam Electric Station,
Units 1 and 2), Docket Nos. 50-445 & 50-446 OL

Dear Geary:

Enclosed are Applicants' responses to questions posed by the NRC Staff regarding the motions for summary disposition submitted as part of Applicants' Plan to Respond to the Board's December 28, 1983 Memorandum and Order (Quality Assurance for Design). Additional information will be provided shortly.

Sincerely,

William A. Horin/Rcm
William A. Horin
Counsel for Applicants

rcm
Enclosures
Hand Delivery

cc: Juanita Ellis - Overnight Delivery (with encs.)
Remainder of Service List (w/o encs.)



DS03

Q Clarify margin of safety for Richmond inserts in a manner similar to that employed for the Hilti bolts in the Affidavit for Gaps.

---

Two sizes of Richmond inserts are used at CPSES: 1 inch and 1 1/2 inch inserts. In most instances the inserts employ SA-36 rods, but there are 47 inserts which utilize high strength bolts/rods. Of the 47 inserts which use high strength rods, 15 are 1 inch inserts. Of these 15, only 4 have bolt holes which are 1 1/8 inch in size. The remainder of the 1 inch inserts have bolt holes equal to 1 1/16 inch. All 1 1/2 inch inserts utilize bolt holes equal to 1 1/8" in size. This includes only those bolts with 1 1/8" holes loaded in shear. The remainder of the 1" inserts have bolt holes equal to 1 1/16" or are not loaded in shear. All 1 1/2" inserts utilize bolt holes equal to 1 1/8". The margin to failure for the various Richmond inserts, computed in a manner similar to that used for the Hilti bolts on pp. 8 and 11 of the Gap Affidavit is assessed in this answer. First we will address the inherent safety factor for the 1" inserts.

There appears to be some misinterpretation of the 4-19-84 Richmond Insert Test Report (Attachment B to the Affidavity) regarding the load-deflection chart for shear on 1 inch inserts (bolts were A-490). In the curves there are two load deflection lines for each specimen: one represents the deflection immediately after load, and the other is the deflection two minutes after load. In the chart, point 16A for example is the deflection of specimen 16 at failure. Point 16B is the two minute deflection of specimen16 one load level below the failure load. Thus the only points of interest when discussing ultimate deflections are the A points. In this light the ultimate deflection for the 1 inch inserts with A-490 bolts in shear are:

| Specimen | Ultimate Δ | Remarks |
|---|---|---|
| 16 | 0.22" | |
| 17 | 0.332" | This is a 2 min defl. before ult. load, since defl. at ult. was not recorded |
| 18 | 0.40 | |
| 19 | 0.270" | |
| 20 | 0.270" | |

Thus the minimum deflection for all tests was 0.22" and the average deflection was 0.2984"

The allowable shear for a 1-inch insert with high strength bolt is 11.5 kips. The test indicate that the corresponding deflections would range from 0.001" to 0.036" with the average being 0.0186".
A review of Unit 1 and Common supports that utilize 1-inch Richmonds with high strength bolts yielded only 4 such supports where the bolt holes are 1 1/8 inch, and the inserts are loaded in shear. None of these supports are in Containment. A worst case analysis of the above would indicate that the safety factor based on deflection would be 1.37. A more reasonable approach would be to use the averages. Based on average deflections the safety factor is 2.08. This would apply to 4 support only. The following is a summary of the situation as it applies to each of thses supports.

1. Bolt pattern is 4 Richmond inserts in a square pattern. Total shear load of the joint is low enough that two bolts can take all the shear and remain below allowables
2. Bolt pattern resisting shear is 8 Richmond inserts (5-A36 and 3-A193). It is not reasonable to assume that any one bolt will have to deflect 1/8" before another bolt begins to share the load.
3. Bolt pattern resisting shear consists of 14 Hiltis plus 4 Richmonds. Same conclusion as in 2 above.
4. Bolt pattern resisting shear is 24 Richmond inserts (22 A36 and 2 A193). Same conclusion as above.

It must be stated that the Richmond insert tests were conducted is shear with A-490 bolts. The high strength bolts actually used in the 1-inch inserts in the field are A-193 Grade B. This material is 14-25 % more ductile than A-490.
Having defined the inherent safety factor for the limiting case of the Richmond inserts, i.e the 1-inch inserts with high strenth bolts and 1 1/8 inch bolt holes, it is appropriate for completeness to define the safety factors for the other Richmonds.
There is no test data for the 1 inch inserts which utilize A-36 rods. However there is test data comparing the 1 1/2 inch insert behavior in shear when employing high strenth and A-36 rods. This data (see Att. A of Richmond Affidavit) shows that deflections of inserts using A-36 rods are approximately twice those experienced by the inserts with the high strenth

bolts at the same load (this is true whether one compares the stiffest connections or the averages). Assuming that a similar behavior would hold true for the 1-inch inserts, the safety factor computed on a deflection basis for the 1-inch insert having a 1 1/8 inch bolt hole and an A-36 rod would be a mimimum of 2.23 or an average of 3.68. For those 1-inch inserts utilizing 1 1/16 inch bolt holes, the safety factors based on deflection would be 2.33 minimum and 3.68 average for high strength bolts and 3.27 minimum or 5.98 average for A-36 rods.

In the same manner we can derive the deflection-based safety factors for the 1 1/2-inch inserts. For the 1 1/2-inch inserts, the ultimate deflections in shear were the following (for high strenth bolts):

| Specimen | Ultimate Defl. | Remarks |
|---|---|---|
| 1 | 0.510" | Not Failed |
| 2 | 0.770" | |
| 3 | 0.540" | |
| 4 | 0.550" | |
| 5 | 0.585" | |

The minimum deflection for all tests was 0.510" and the average was 0.591". The allowable shear for the 1 1/2-inch insert is 27 kips (when a low strength rod is used the allowable shear load is determined by the rod and is equal to 17 kips).

At this load (27 kips), the smallest deflection is 0.012", and the average deflection is 0.046". For the 1 1/2-inch insert utilizing high strength rods, the minimum safety factor on the basis of deflection would therefore be 2.32 and the average safety factor would be 3.46.

For the Richmond inserts using A-36 rods, shear deflections at failure are not available since the tests were not run to failure. However, if we use the deflection at the highest load utilized, and only the stiffest connection of those tested, we find that the factors of safety on a deflection basis is 0.821"/(0.125"+0.130") = 3.22.

Q Verify tolerances for Richmond bolt holes. Verify material employed for the Richmond bolts. Verify whether Richmonds are used in a pattern either by themselves or jointly with Hiltis.

---

The answers to these questions have been provided as part of the previous answer.

Q Address the questtion of importance of rebar in regard to the safety factors of Richmonds via the method of ACI349.

---

ACI 349 utilizes a factor, Ø, equal to 0.85 in the formula which computes ultimate tensile capacity of the insert (see affidavit p.8), for inserts embedded beyond the rebar layer, and a factor Ø = 0.65 for plain concrete (ie no reinforcement). If one makes the conservative assumption that the influence of the reinforcement on Richmond insert performance is equal to that predicted by ACI 349, then the minimum factor of safety in tension produced by the Applicants' tests is 3.26 x 0.65/0.85 = 2.50.

Although this answers the question posed by Mr. John Fair we do not see the direct relevance of the question to the issue of Richmonds' safety factors. The factor of safety recommended by the manufacturer, ie F.S = 3.0 was developed from tests which employed reinforced concrete. The placement of inserts at CPSES utilizes concrete which is reinforced. The rebar utilized in the latest series of tests is the minimum type of surface reinforcement encountered in the field (#7 grade 60 bars at 10 inches on center in each direction near the surface) (see Affidavit at p 14).

Thus there is no instance at CPSES where the insert would be embedded in unreinforced concrete.

Applicants, derived the factor Ø = 0.84 from the manufacturer's test data , (see Affidavit at p. 8), rather than employing the Ø = .85 of ACI 349 for inserts embedded past reinforcement or Ø = .65 of ACI 349 for unreinforced concrete. Thus Applicants recognized that presence of reinforcement has a significant effect (or else we would have used Ø = .65).

We think that perhaps our statement (and the Staff's) on p. 17 of the Affidavit

" The amount of rebar is not a significant factor."

may have caused confusion and thus generated this question. What that statement is intended to address is the concern of CASE over differences in reinforcement.

It is not intended to imply that there is little difference between no reinforcement and reinforcement, but simply that there is little difference between types of reinforcement, ie 2 layer vs 4 layer, rebar size chosen .

Q Provide additional information on appropriateness of interaction formula and acceptance criterion for bending in the Richmond insert bolt, including consideration of fatigue.

---

To address this question Applicants have derived the interaction formula by modelling the bolt as a solid bar having a diameter (h) square equal to four tines the effective tensile area divided by , ie

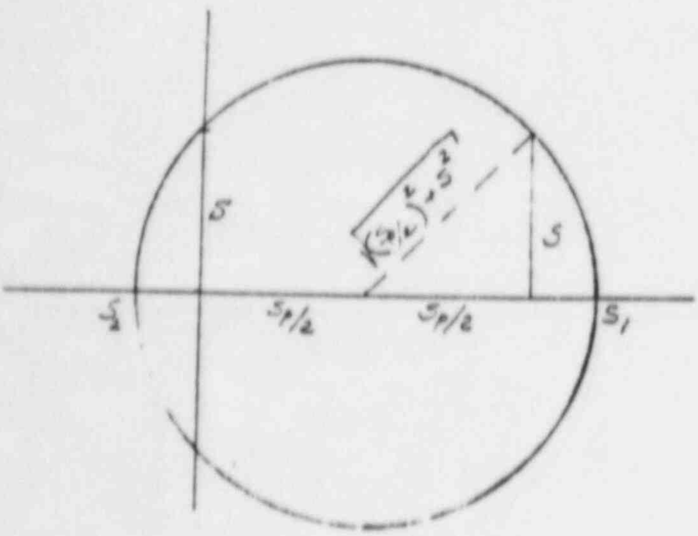
$$h = \sqrt{\frac{4A_{tens}}{\pi}}$$

The ASME Code, section NB 3232.2 permits the stress intensity of a bolt to equal the yield stress, $\sigma_y$ .

Under the combined tension and bending loading, the principal stress $S_p$ in the direction of the applied tension is given by

$$S_p = \frac{T}{\pi\left(\frac{h}{2}\right)^2} + \frac{32M}{\pi\left(\frac{h}{2}\right)^3}$$

The Mohr circle indicates that the stress intensity, when shear is the only other applied load is given by $S_1 - S_2$ where $S_1$ and $S_2$ are defined below




$$S_1 = \frac{S_p}{2} + \sqrt{\left(\frac{S_p}{2}\right)^2 + S_s^2}$$

$$S_2 = \frac{S_p}{2} - \sqrt{\left(\frac{S_p}{2}\right)^2 + S_s^2}$$

$$S.I. = 2\sqrt{\left(\frac{S_p}{2}\right)^2 + S_s^2}$$

Here $S_s$ = shear stress = $\pi\left(\frac{h}{2}\right)^2 S$ where S is the shear load.

Thus we write

$$2\sqrt{\left(\frac{S_p}{2}\right)^2 + S_s^2} \le \sigma_y$$

$$\left(\frac{S_p}{2}\right)^2 + S_s^2 \le \sigma_y^2/4$$

Substituting for $S_p$ we have

$$\left(\frac{T}{\pi\left(\frac{h}{2}\right)^2} + \frac{32\,M}{\pi\left(\frac{h}{2}\right)^3}\right)^2 + S_s^2 \le \sigma_y^2/4$$

However, the allowable shear [illegible], tension [illegible] and moment [illegible] are given respectively by

$$S_A = 0.4\,\pi\left(\frac{h}{2}\right)^2\sigma_y \qquad T_A = 0.6\,\pi\left(\frac{h}{2}\right)^2\sigma_y \; ; \qquad M_A = \frac{.75\,\pi}{32}\left(\frac{h}{2}\right)^3\sigma_y$$

Therefore, the interaction formula can be written as

$$0.16\left(\frac{S}{S_A}\right)^2 + \frac{\left(0.6\,T/T_A + .75\,M/M_A\right)^2}{4} \le 0.25$$

For consistency with the approach chosen in theAffidavit, the coefficient of the moment term is reduced by a factor of 1.3 to account for the fact that the finite element analysis predicts lower bending stresses than the Mc/I formula by a factor of at least 1.3. With this change the interaction formula becomes:

$$1.12\left(\frac{S}{S_A}\right)^2 + \left(0.7937\,\frac{T}{T_A} + 0.754\,\frac{M}{M_A}\right)^2 \le 1.75 \qquad (1)$$

The interaction formula provided in the Affidavit (p. 27) can be compared with this formula.

$$\left(\frac{S}{S_A}\right)^2 + \left(\frac{T}{T_A}\right)^2 + \frac{M}{M_A} \le 1.75 \qquad (2)$$

Here the factor of 1.75 is used, since this is the factor above which we have indicated we would conduct further analysis of the Richmonds.
Two conclusions can be established by comparing formulas (1) and (2):

a) The interaction ratio of 1.75 does correspond to the situation whereby the maximum stress intensity will equal but not exceed the yield stress.

b) The second formula predicts higher interaction ratios, and is thus conservative for all combinations of tension, shear and bending, except for the case when the bending moment is very small, i.e. $M/M_A \leq 0.1$ and the shear is high, $S/S_A > 0.9$.

This case does not occur since high shear loads for the bolts and inserts is accompanied by high bending moments, and conversely, if the bending moment is small, it would not be considered explicitly, but would be coupled out as increased tension in the bolt, and this interaction formula would not be used.

Thus the interaction formul employed in the Affidavit is indeed conservative for all the cases in which it needs to be applied.

As a further note it is germane to state that the reason that the ASME Code does not consider the combination of shear, tension and bending in bolts, except when the shear occurs as torsion in the bolt, is due to the fact that at the point where the moment stress is maximum, the real shear stress is zero, and viceversa, as long as the entire section remains elastic. Inthe preceding Mohr circle analysis, we have assumed that the maximum shear stress occurs coincidentally with the maximum bending moment, and hence we have again been very conservative.

Since the Applicants have conservatively derived the interaction formula for shear, tension and bending in the insert bolt by allowing the outer fibers of the bolt to reach yield, a question was raised regarding low cycle fatigue capability of the bolt.

On rigid seismic supports and snubbers, the OBE and SSE loading conditions govern design, with normal operating conditions stresses being much, much lower. The alternating stress in the bolt is conservatively assumed to be equal in magnitude to the yield stress. (The worst conditions from a fatigue standpoint, would occur if all of the stress were alternating, i.e. the bolts were loaded in pure bending. This is not the case, since the connections have combined shear, tension, and bending). Utilizing the fatigue analysis methods of ASME NB-3222.4(e), with an alternating stress equal to the yield stress and a stress concentration

factor of 2.5 from reference 1, as permitted by the ASME NB-3232.3(c), the total number of cycles is computed to be in excess of 1000. This number is larger that the combined number of cycles of SSE and OBE, which is 720 per FSAR Section 3.7B.3.2.

Considering the conservatism of the assumptions made for the fatigue analysis Applicants conclude that employing the interaction formula given in the Affidavit provides the necessary protection against fatigue failure.

1. "Design of Machine Elements", V. Faires, McMillan & Co., New York, 1949.

Q Provide Additional information on modelling of connection, i.e. appropriateness of releasing $M_z$ and retaining $M_x$ moments, for larger size tube steel, i.e. larger than 4'.

---

We are providing the answer in two parts:

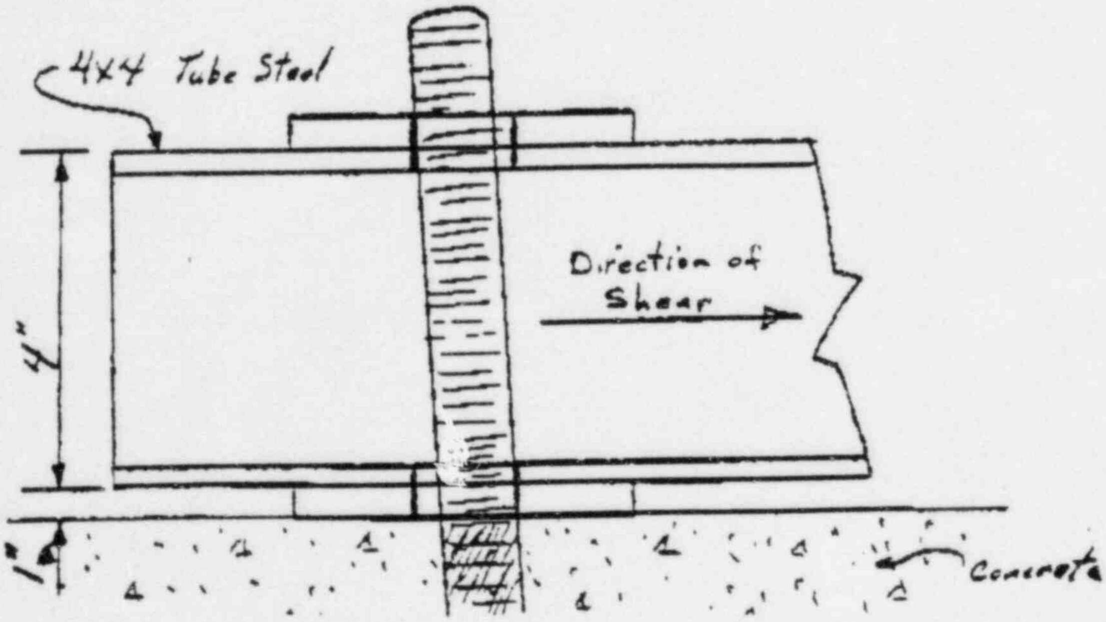
1. Larger tube steel size. The rotation of a simply supported beam at the point of support is inversely proportional to the moment of inertia. Therefore tube steel larger than 4x4x3/8 would have smaller end rotations. The potential of the tube steel connection developing a prying force is directly dependent on the product of the end rotation times the tube steel size. Since the moment of inertia increases exponentially with respect to the tube steel sizes, the product of the rotation times the tube steel size would decrease with increasing tube steel size. Therefore the 4x4x3/8 tube steel represents a worst case with respect to potential prying.
In addition, due to the increased bolt length and more flexible walls on larger tube steel, the joint would deflect more under load and therefor e be able to accomodate more end rotation before prying would develop.

2. Greater spans. The finite element analysis was based on a 20" span between inserts. In some cases the inserts are spaced up to 24" apart. To check this case Applicants calculated the increased rotation due to the larger span and determined that the clearance between the tube steeel and the washer is sufficient to accomodate the increased rotation.
A survey of approximately 3000 supports has revealed that a few cases where the tube steel spans between two non-adjacent inserts resulting in spans of 48". Utilizing 4x4x3/8 tube steel, the maximum load that the member can be designed for is 9.7 kips. If 9.7 kips are loaded at the point which produced the maximum end rotation, the end reaction would be 5.6 kips. If one considers only the flexibility of the tube steel, the bolt elongation and the compressibility of the concrete, and assumes that the rotation produces prying, the 5.6 kips reaction would increase to 8.9 kips due to prying action. Since the 1 inch insert is capable of 12.1 kips, the prying action would not overload the bolt (or insert).

Q Provide explanation on possible effects of angularity of Richmond inserts.

---

Figure 1 represents the most probable worst case condition of the misalignment of a Richmond insert bolt in the holes of the tube steel. Misalignment resulting in bolt inclination exceeding one in twenty is not permitted without bevelled washers, and even such misalignment is rare. For holes 1/8" larger than the bolt, it can be seen that before any shear can be transferred through the bottom flange, the bolt has to be deflected 1/8" at the top flange (0.2 inches for a one in twenty angularity). This situation is not unlike the situation involved in the distribution of shear to a multibolt connection. It is well recognized in that situation that some bolts in the connection, or some other part of the connection, may yield in order to bring all bolts into the shear reaction, so that ultimately all bolts share. The bolt shown in Figure 1 has a misalignment that is within the normally accepted construction tolerances. As the shear load on the connection is increased, a bending moment will be induced in the bolt. At some point, however, the bottom flange will be brought into contact with the bolt, and in the ultimate condition, the majority of the shear force will be transferred through the bottom flange, just as it would in a connection with no misalignment.




The question which must be answered is whether the bolt has the flexibility to close the 1/8" gap before failing. Looking at Attachment A to Applicants'

Richmond Insert Affidavit, one can see that these connections have typically quite ample flexibility. The smallest ultimate deflection of any bolt in that series of tests was 0.56". It should be noted that these tests were conducted with a test plate only. The bolt deflection of interest would have been higher. These tests were for 1 1/2 inch inserts. Similar results were obtained from the 4/84 tests on Richmond inserts (Attachment B to the same Affidavit). For the 1 inch bolt in these tests, the smallest shear test deflection was 0.22". However, assuming that this was the deflection at the top of the test plate (1" from the concrete), the deflection at the top flange of Figure 1 (about 5" from the concrete) would be considerably larger. These tests confirm that the Richmond inserts, even when used with high strength bolts (all prior deflections were obtained for high strength bolts) have the flexibility necessary to accomodate the misalignment of bolts in the field. An inspection of the results of Attachment F to Applicants' Richmond Inserts Affidavit indicates clearly that the use of an A-36 rod with the Richmond insert (the most common practice at CPSES) will also allow quite ample flexibility to account for these construction tolerances. These effects are well recognized in the AISC Code. Pages 5-14 and 5-15 of the AISC, 8th Edition describe Type 2 construction (i.e pinned joints), which is the type of "simple framing" assumed in the design of the Richmond insert-tube steel structures. Those pages state "Type 2 and 3 construction may necessitate some inelastic, but self-limiting, deformation of a structural steel part.

Q Address the Board's concern with adequacy of safety factors of Richmond inserts regarding cyclic loads.

---

The adequacy of the Richmond bolt regarding cyclic loads, i.e. fatigie, has been addressed conservatively in the answer to question 9(b). Here we provide the analogous answer with respect to the fatigue capacity of the insert proper.

Unlike the bolt, for which we could assume that at the maximum load, the maximum alternatin stresses could be no larger than the yield stress, thence analytically derive the number of cycles which the bolt can accept at such maximum load, the insert is not amenable to similar analyses, since the state of stress in the insert proper and the concrete is not precisely known. However, examination of the load deflection data collected by Applicants (see Affidavit, Attachment B) shows that generally at the rated loads of the inserts (11.5 kips in shear and tension for 1 inch inserts, and 27.kips and 31.3 kips in shear and tension respectively for 1 1/2 inch inserts) there is no significant departure from linear behavior. This indicates essentially elastic behavior of the insert below rated loads. This behavior is shown particularly vividly in the load-deflection curves of the combined shear-tension tests, and the tension tests. Shear load deflection curves are not as clear. In fact for the 1 inch inserts, some shear load-deflection curves seem to indicate departure from linear behavior at about 5 kips. However the combined shear-tension load deflection tests do not confirm this behavior.

One can than assume that if there was a proportional limit for the inserts, it would be above the rated load. Hence any cyclic load alternating between the rated loads, would elicit elastic behavior of the insert. Alternating loads at the full rated capacity of the inserts would only occur for SSE and OBE events. The number of cycles of these events is 720. By analogy to the bolt case, we would expect that elastic cycling of the inserts would also cause no fatigue failure at this number of cycles.