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 OL
TEXAS UTILITIES ELECTRIC COMPANY, et al.	)	
(Comanche Peak Steam Electric Station, Units 1 and 2)	)	(Application for an Operating License)

STATEMENT OF MATERIAL FACTS AS TO WHICH
THERE IS NO GENUINE ISSUE

REGARDING

CASE'S 4TH MOTION FOR SUMMARY DISPOSITION:
TO DISQUALIFY THE USE OF SA307 AND SA36 THREADED PARTS

1. Applicants most recently have stated that they use only A36 steel (with one exception) both in U-bolts and bolts used in Richmond inserts (see page 8 of Affidavit of Robert C. Iotti and John C. Finneran, Jr., Regarding the Licensing Board's December 18, 1984 Memorandum, attached to Applicants' 1/7/85 Motion for Reconsideration of Board's Memorandum (Reopening Discovery; Misleading Statement), copy attached).

2. Applicants have admitted that to the extent that previous statements made by Applicants imply that A36 and A307 steels are identical materials, they are inconsistent with Applicants' statements in their 12/5/84 Response (Affidavit of Robert C. Iotti and John C. Finneran, Jr. accompanying 'Applicants' Response to Board Memorandum (Information on Composition of A36 and A307 Steel),' December 5, 1984) (Id., at pages 4 and 5, copy attached). /1/

/1/ It should be noted that CASE does not agree that this is the extent of Applicants' inconsistencies, and we will be addressing these and other inconsistencies regarding these and other statements by Applicants in other pleadings.

3. Regardless of what name Applicants place on it, the material which Applicants have used for cinched-down U-bolts and for bolts in Richmond inserts is the same in that the material referred to as SA-307 must conform to the requirements of Specification A 36 and is therefore equivalent to, or (for all intents and purposes) is made from, SA-36 material (see attached Affidavit of CASE Witness Jack Doyle, Footnote 2, pages 3 and 4; see also Attachments B and D from Applicants' 1/7/85 Motion for Reconsideration of Licensing Board's Memorandum (Reopening Discovery; Misleading Statement) /2/).

4. (a) The ASME Code to which Applicants are committed states, at ASME Section III, Appendix XVII, Table XVII 2461.1-1, Note 1:

> "Friction type connections loaded in shear are not permitted. The amount of clamping force developed by SA-307 bolts is unpredictable and generally insufficient to prevent complete slippage."

(See CASE Exhibit 752, copy attached; see also attached Affidavit of CASE Witness Jack Doyle, at pages 2 and 6.)

(b) SA307 material and SA36 material is unpredictable for dynamic (including seismic) loads. (See attached Doyle Affidavit, page 2.)

/2/ It should be noted that Mr. Doyle's Affidavit was prepared prior to receipt of Applicants' 1/7/85 Motion for Reconsideration. He has not yet had time to thoroughly review it (although he did notice one particular portion which he addressed), and it may be necessary for him to supplement his Affidavit. CASE prepared this Statement of Material Facts after receipt of Applicants' 1/7/85 Motion, and we have incorporated some of the information contained in it. From our rather brief review of it, CASE believes that the Board a ready has sufficient information contained in the codes and NRC regulations upon which to make a decision regarding the issues under discussion.

5. The ASTM Code, to which Applicants are committed, states regarding ASTM A 307, Standard Specification for Carbon Steel Externally Threaded Standard Fasteners, page 251, at section 1.3:

> "Nonheaded anchor bolts, either straight or bent, to be used for structural anchorage purposes, shall conform to the requirements of Specification A 36 with tension tests to be made on the bolt body or on the bar stock used for making the anchor bolts."

(See Attachment B attached; see also attached Doyle Affidavit at pages 3 and 4, Footnote 2.)

6. ASME specification SA-307 states at section 1.3:

> "Nonheaded anchor bolts, either straight or bent, to be used for structural anchorage purposes, shall conform to the requirements of ASTM Specification A 36, for Structural Steel, with tension tests to be made on the bolt body or on the bar stock used for making the anchor bolts."

(See Attachment D attached; see also attached Doyle Affidavit at pages 3 and 4, Footnote 2.)

7. Applicants' testing of the bolt material was performed to show that the bolts could take a certain static load. The test did not show whether the joints could sustain seismic loads nor what effect the non-friction joint would have on the dynamics of the system itself. (See attached Doyle Affidavit at page 5.)

8. (a) Applicants are using a bolting material which is only qualified for static loads.

   (b) These joints present an unknown quantity as relates to the dynamics of the total system.

   (See Doyle Affidavit at pages 4 and 5.)

9. In order to cope with dynamic loads, the joints must be predictable; that is, slippage must be a controlled criteria. (See attached Doyle Affidavit at page 5.)

10. The AISC Code, 8th Edition, page 5-24, Table 1.5.2.1, Threaded Parts, prohibits the use of bolts and threaded materials made of SA307 and A36 steels subjected to other than static loads. (See attached copy of AISC Code, 8th Edition, page 5-24; see also attached Doyle Affidavit at pages 5 and 6.)

11. (a) Although in the past Applicants were only committed to the 7th Edition of the ASTM Manual of Steel Construction, the logic for the change made in the 8th Edition existed even prior to the 7th Edition if one were doing dynamic analysis.

    (b) During the time prior to the 8th Edition, the AISC Code addressed loading in terms of static application; that is, even for structures which included earthquake considerations, the earthquake load was assumed to be an equivalent static horizontal load based on KCZW (dynamic derivation of loads was not utilized). In the case of nuclear power plants, the earthquake loads are based on the response spectra and damping factors, in which case the predictability of the joint is required. Otherwise, both the response spectra and the damping factors are also unpredictable.

    (See attached Doyle Affidavit at page 6.)

12. Applicants have amended their FSAR to include both the 7th and 8th Editions of the AISC Code. (See Transcript of meeting between Cygna Energy Services and the NRC Technical Review Team, 12/20/84, page 80, lines 6 through 9, copy attached; see also attached Doyle Affidavit at page 6).

13. If one built the perfect nuclear structure, perfect piping systems, and perfect pipe supports, and then connected these items with unpredictable bolting materials, one would have a total system which is no longer perfect as independent components and which is now, as a system, unpredictable. (See attached Doyle Affidavit at page 7.)

14. (a) The use of A307 and/or A36 threaded parts in the manner in which Applicants utilize them at Comanche Peak is a unique design feature.

(b) Applicants did not identify such unique design feature in their PSAR as required by the provisions of 10 CFR 50.34(a)(2) and (8), which state:

> "(a) Preliminary safety analysis report. Each application for a construction permit shall include a preliminary safety analysis report. The minimum information to be included shall consist of the following:
>
> > "(2) A summary description and discussion of the facility, with special attention to design and operating characteristics, unusual or novel design features, and principal safety considerations."
> >
> > "(8) An identification of those structures, systems, or components of the facility, if any, which require research and development to confirm the adequacy of their design; and identification and description of the research and development program which will be conducted to resolve any safety questions associated with such structures, systems or components; and a schedule of the research and development program showing that such safety questions will be resolved at or before the latest date stated in the application for completion of construction of the facility."

(See 10 CFR 50.34(a)(2) and (8); see also attached Doyle Affidavit at pages 1 and 2.)

15. (a) Applicants recognize Messrs. Paul F. Rice and Edward S. Hoffman as authorities and attached one page (268) from a document by Messrs. Rice and Hoffman to Applicants' 5/18/84 Motion for Summary Disposition Regarding the Effects of Gaps on Structural Behavior Under Seismic Loading Conditions.

(b) On pages 264 through 271 of the same article, Messrs. Rice and Hoffman indicate that A307 bolts are not permitted in connections subject to vibration or stress reversal.

(c) Connections at Comanche Peak are subject to vibration.

(d) Connections at Comanche Peak are subject to stress reversal.

(e) The ASME Code requires the Applicants to minimize vibration where it states:

> "**NF-3112.2 Design Mechanical Loads.** . . . The requirements of (a), (b), and (c) below shall apply.
>
> > ". . . (c) Component supports shall be designed to minimize vibration."

(See attached pages 2 through 5 of Affidavit of CASE Witness Mark Walsh and Attachment A thereto, which was attached to CASE's 8/13/84 Answer to Applicants' Motion for Summary Disposition Regarding the Effects of Gaps on Structural Behavior Under Seismic Loading Conditions.)

16. ANSI N45.2.11, to which Applicants are committed, at 3. DESIGN INPUT REQUIREMENTS, 3.2 Requirements, states, in part:

> "The design input requirements should include the following where applicable:
>
> > "(9) Mechanical requirements such as vibration, stress, shock and reaction forces." (Emphases added.)

17. Supports at Comanche Peak will experience dynamic (seismic loads). (See attached Doyle Affidavit at pages 2, and 5 through 7.)

18. Applicants cannot properly use A307 material and/or A36 material for cinched-down U-bolts and for bolts in Richmond connections in the manner currently being utilized at Comanche Peak. (See Material Facts 1 through 17 preceding.)

19. For the reasons discussed herein, Applicants are in violation of:

    (a) NRC regulations, including 10 CFR 50.34(a)(2) and (8); 10 CFR Part 50, Criteria I and II;

    (b) ASME Code, Section III, Appendix XVII, Table XVII 2461.1-1, Note 1; ASME specification SA-307, section 1.3; ASME NF-3112.2;

    (c) ASTM A307 specification, Note 1.3;

    (d) AISC 8th Edition, Table 1.5.2.1.;

    (e) ANSI N45.2.11, section 3.2; and

    (f) standard industry practice (as indicated by Messrs. Rice and Hoffman).

strength bolts in a line parallel to the direction of stress, the distance from the center of the end bolt to that end of the connected part toward which the stress is directed shall be not less than A_bC/t for single shear or $2A_bC/t$ for double shear, where A_b is the nominal cross-sectional area of the bolt, t is the thickness of the connected part and C is the ratio of specified minimum tensile strength of the bolt to the specified minimum tensile strength of the connected part.

XVII-2462.3 When Fastener Stress Is Lower Than Permitted. The end distance prescribed in XVII-2462.1 and XVII-2462.2 may be decreased in such proportion as the fastener stress is less than that permitted in Table XVII-2461.1-1 but it shall not be less than the distance specified in XVII-2463.5 and need not exceed 1½ times the transverse spacing of fasteners.

XVII-2462.4 When Two or More Fasteners in Line of Stress Are Provided. When more than two fasteners are provided in the line of stress, the provisions of XVII-2462.5 shall govern.

XVII-2462.5 Minimum Distance to Any Edge. The minimum distance from the center of a bolt hole to any edge used in design or in preparation of shop drawings shall be that given in Table XVII-2462.5-1.

TABLE XVII-2461.1-1
ALLOWABLE BOLT TENSION AND SHEAR STRESSES

Description of Fastener	Bolt Specification			Nominal Bolt Size In.	Tension (F_t) % of Y.S.	Shear (F_v)	
	Spec.	Type or Grade	Class			Friction Type Connections % of Y.S.	Bearing Type Connections % of Y.S.
Threading Not Excluded From Shear Planes	SA-325	1		1/2 to 1 incl.	44	16	18
				1-1/8 to 1-1/2 incl.	50	19	19
	SA-307	B		All	61	See note 1	20
	SA-540	B21 (Cr-Mo-V)	5	To 2 incl.	43	16	18
				Over 2 to 8 incl.	42	16	17
			4	To 3 incl.	46	17	19
				Over 3 to 6 incl.	46	17	19
			3	To 3 incl.	42	15	18
				Over 3 to 6 incl.	42	15	18
			2	To 4 incl.	41	15	17
			1	To 4 incl.	41	15	17
		B22 (4142-H)	5	To 2 incl.	43	16	18
				Over 2 to 4 incl.	42	16	17
			4	To 4 incl.	46	17	19
			3	To 4 incl.	42	15	18
			2	To 3 incl.	41	15	17
			1	To 1-1/2 incl.	41	15	17
		B23 (E-4340-H)	5	To 6 incl.	43	16	18
			4	To 9-1/2 incl.	46	17	19
			3	To 9-1/2	42	15	18
			2	To 9-1/2	41	15	17
			1	To 8 incl.	41	15	17
		B24 (4340 Mod.)	5	To 6 incl.	43	16	18
			4	To 9-1/2	46	17	19
			3	To 9-1/2 incl.	42	15	18
			2	To 9-1/2 incl.	41	15	17
			1	To 8 incl.	41	15	17

TABLE XVII-2461.1-1 (Cont'd)

Description of Fastener	Bolt Specification			Nominal Bolt Size In.	Tension (Ft) % of Y.S.	Shear (F_v)	
	Spec.	Type or Grade	Class			Friction Type Connections % of Y.S.	Bearing Type Connections % of Y.S.
Threading Excluded From Shear Planes	SA-325	1		1/2 to 1 incl.	44	16	24
				1-1/8 to 1-1/2 incl.	50	19	27
	SA-307	B		All	61	See note 1	30
	SA-540	B21 (Cr-Mo-V)	5	To 8 incl.	43	16	26
			4	To 6 incl.	46	17	27
			3	To 6 incl.	42	15	25
			2	To 4 incl.	33	15	25
			1	To 4 incl.	23	15	25
		B22 (4132-H)	5	To 4 incl.	43	16	26
			4	To 4 incl.	46	17	27
			3	To 4 incl.	42	15	25
			2	To 3 incl.	33	15	25
			1	To 1-1/2 incl.	23	15	25
		B23 (E-4340-H)	5	To 6 incl.	43	16	26
			4	To 3-1/2 incl.	46	17	27
			3	To 3-1/2 incl.	42	15	25
			2	To 9-1/2 incl.	33	15	25
			1	To 8 incl.	23	15	25
		B24 (4340 Mod.)	5	To 6 incl.	46	16	26
			4	To 9-1/2 incl.	46	17	27
			3	To 9-1/2 incl.	42	15	25
			2	To 3-1/2 incl.	33	15	25
			1	To 8 incl.	23	15	25

NOTE:

1. Friction type connections loaded in shear are not permitted. The amount of clamping force developed by SA-307 bolts is unpredictable and generally insufficient to prevent complete slippage.

XVII-2463 Maximum Edge Distance

The maximum distance from the center of any bolt to the nearest edge of parts in contact with one another shall be 12 times the thickness of the plate, but shall not exceed 6 in.

XVII-2464 Minimum Pitch

The minimum distance between centers of bolt holes shall preferably be not less than 3 times the nominal diameter of the bolt. Less distance may be used only if adequate installation wrenching clearance is available.

XVII-2465 Effective Bearing Area

The effective bearing area of bolts shall be the diameter multiplied by the length in bearing, except that for countersunk bolts ½ the depth of the countersink shall be deducted.

XVII-2466 Long Grips

SA-307 bolts, which carry calculated stress and the grip of which exceeds 5 diameters, shall have their number increased 1% for each additional 1/16 in. in the grip.

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

AFFIDAVIT OF ROBERT C. IOTTI AND
JOHN C. FINNERAN, JR. REGARDING
BOARD REQUEST FOR INFORMATION
CONCERNING A36 AND A307 STEEL

We, Robert C. Iotti and John C. Finneran, Jr., having first been duly sworn hereby depose and state, as follows:

(Iotti) I am Vice President of Advanced Technology for Ebasco Services, Inc. A statement of my educational and professional qualifications was transmitted with Applicants' letter of May 16, 1984, to the Licensing Board in this proceeding.

(Finneran) I am employed by Texas Utilities Generating Company as Project Pipe Support Engineer for Comanche Peak Steam Electric Station. A statement of my educational and professional qualifications is in evidence as Applicants' Exhibit 142B.

We previously submitted affidavits regarding cinching-down of U-bolts, U-bolts acting as two-way restraints, and Richmond Inserts, which were filed in support of Applicants' motions for summary disposition of these issues on June 29, May 23, and June 2, 1984, respectively.

Q. What is the purpose of your affidavit?

A. The purpose of this affidavit is to provide information in response to the Board's Memorandum (Information on Composition of A36 and A307 Steel), dated October 25, 1984.

Interchangeability of SA36 and SA307 Materials

Q. Are SA36 and SA307 steels the same material?

A. No. CASE incorrectly asserts that these materials are the same.[1] Although it is true that SA36 and SA307 materials are similar, there is a major difference in the specified mechanical requirements for SA36 and SA307 steels. As Applicants explained in our affidavit in support of Applicants' reply to CASE's answer to Applicants' motion for summary disposition regarding the effects of gaps (October 26, 1984) at 8-9, the material specification for SA36 requires both a test for ultimate tensile strength and a

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1 CASE's Answer to Applicants' Statement of Material Facts Relating to Richmond Inserts, Walsh Affidavit (September 10, 1984) at 10. The memorandum cited by CASE (CASE Exhibit 834) is not correct in referring to the tested materials as SA307. SA36 and SA307 rods are both used in some structural applications not involving pipe supports. The individual who prepared that memorandum (who commonly works with those other applications) apparently did not focus on the distinction when he prepared the memorandum.

test for minimum yield point, whereas the SA307 specification requires only a tensile test (ultimate tensile strength). Thus, unlike SA36, there is no established basis for determining certain characteristics, including relaxation, of components using SA307 material. In summary, it is not appropriate, therefore, to interchange the two steels as CASE has done. (See Memorandum at 1 ". . . variability in A36 (A307) steel . . .".)

Materials Employed in Applicants' Tests
in Support of Motions for Summary Disposition

Q. Which tests performed in support of Applicants' motions for summary disposition employed SA36 or SA307 steels?

A. None of the tests employed SA307 material in the test specimens. Three tests utilized in Applicants' motions employed SA36 steel specimens, as follows:

1) The tests performed by Westinghouse on cinched-down U-bolt assemblies were undertaken to assess the adequacy of cinched-down U-bolts to function as clamps. In these tests the U-bolts were SA36 material (see Attachment 1 to Applicants' Affidavit on Cinched-Down U-Bolts).

2) The tests conducted by ITT-Grinnell to determine the ultimate capacity of U-bolts under different loading conditions, in support of Applicants' May 23, 1984, motion regarding U-bolts acting as two-way restraints, utilized SA36 test specimens. (see Attachment 1 to Applicants' Affidavit on U-bolts acting as two-way restraints.)

3) Tests conducted by TUGCO on Richmond inserts in March 1983 and May 1984 were used by Applicants in support of their motion for summary disposition on Richmond Inserts. (See Attachments A and F to the Affidavit supporting Applicants' motion on Richmond Inserts.)

RESPONSE TO NRC QUESTIONS OF
MEETING OF AUGUST 8-9 and August 23, 1984

A. U-bolt Cinching

a) Provide additional justification for the assessment that strain relaxation of the U-bolt ceases as the U-bolt stress reduces to approximately 1/2 of the yield strength. Justification should be provided as additional data and also provide actual properties of the U-bolt material employed.

There is scant, if any, data available on strain relaxation properties of SA-36 material. Some relevant data is reported in ASTM DS60 "Compilation of Stress-Relaxation Data for Engineering Alloys," for material having the same composition as SA-36 steel (note that this reference does not mention the material designation). The ASTM material specification for A-36 is presented as Exhibit A1. Also included as Exhibit A2 are the pertinent portions of ASTM DS60 which provide data for ferritic steels having chemical composition and physical properties similar to but varying to different degrees from those of A-36. Also provided are the definitions given in DS60, which are relevant to the question of what causes relaxation and whether creep is important. Unfortunately not much data is available directly at the temperatures of interest, i.e., less than 500°F although considerable information may be inferred from the data at the higher temperatures as will be discussed later. In fact, only materials 2 and 25 have data at room temperature. Material 2 has the proper chemical composition but its physical properties are significantly different from those of A-36. Material 25 has physical properties similar to A-36 but does not quite meet all

of the chemical specifications. Figure A1 shows the stress strain curve of material 25 at various temperatures within our range of interest, i.e. less than 500°F. This curve is used to illustrate the meaning of material relaxation (as opposed to overall mechanical relaxation which will be discussed later) for monotonic loading, i.e. noncyclic. For the material to relax, plastic strain is required. Ferritic steels like A-36 exhibit a well defined proportional limit at which plastic strain begins. The yield strengths of these materials are given at the 0.1% or 0.2% elastic strain offset (in general it is the latter, although for material 25 the former is used). In figure A1 the details of the stress strain curve between the proportional limit and the yield point are not shown. From that figure, if the material is strained below the proportional limit no material relaxation will occur. Strains in excess of the proportional limit will result in relaxation, the amount of relaxation being proportional to the amount of plastic strain (or volume of material that has yielded). At room temperature the strain corresponding to the proportional limit is about 0.075 percent. At that level of initial strain, therefore, little or no relaxation should be expected. Figure A4, developed using the information on Material 25 of ASTM DS60, shows that the relaxation is negligible. At 532°F, the strain corresponding to the proportional limit point is 0.065 percent. Since the material 25 has been strained to .075% relaxation should be expected. Moreover, the heating of the material from room temperature to 532°F and the return to

room temperature contributes to relaxation. How this happens is explained by Figure A2, obtained via private communication with M.J. Manjoine, one of the authors of ASTM DS60 and a recognized authority in materials behavior. This figure is an expanded view of a portion of Figure A3, also provided by M.J. Manjoine. Figure A3 deduces the behavior of ferritic steels like A-36 at the lower temperatures from the fact that the behavior exhibited at the higher temperatures (above 700°F) for which the data is available is the same as that exhibited for mild austenitic steels which have data available at all temperatures. The behavior of austenitic steels is shown in figure A7 which is taken directly from reference 4 (see p. 27). As figure A2 shows a material which is strained to or above the proportional limit will lose load at constant strain simply as a result of the lower yield strength at temperature and the higher modulus of elasticity at room temperature than at temperature. Thus, if material 25 had been strained to yield at 532°F, upon its return to room temperature it could exhibit 35 percent of its initial stress. This would occur upon return to room temperature regardless of whether "material" relaxation occurs. If the material is maintained at temperature, loaded for sufficient time, material relaxation would also occur. This can lead to an additional 15-20 percent loss of load. However, for the latter time is needed to redistribute the load. Although we do not know for a fact, it is fairly obvious that the material relaxation characteristics of material 25 at 532°F must have been determined

at temperature, since as figure A4 indicates, there is some twenty percent relaxation. Similar significant strain relaxation should be expected at all temperatures for initial strains of 0.225 percent, and this is indeed the case.

If the applied load results in a stress below 1/2 of the yield strength at temperature, the corresponding strains would be well below those corresponding to the proportional limits, and thus no relaxation should be expected.

So far only monotonic loads have been discussed. To complete the discussion of material relaxation, it must be pointed out that the stress strain curve for steels are different between the cases of monotonic and cyclic loads. For the monotonic loads discussed so far, the point at which mild ferritic steel materials begins to yield is higher (by approximately 15 percent - private communication with M.J. Manjoine) than the point at which yielding will occur under cyclic loads.

The difference is shown in Figure A5.

It is important that a distinction be made between "cyclic" loads such as are experienced by the U-bolts, whereas the load can be cycled from a low to a high level without stress reversal, and "stress reversal" loads which are cyclic but for which the load causes the stresses to be alternatively tensile and compressive. The relaxation behavior for the two cases can be vastly different. Figure A8 (reference 5) shows that stress strain curve for ferritic steel under reversing constant

amplitude loads (reversing strain). Figure A9 (reference 6) shows an idealized curve for the kind of mild steel which is characteristic of both ferritic steels like A-36 and austenitic steels like A-304. Figure A10 (reference 6) shows the static (monotonic) stress strain curve and the cyclic (strain reversal) curve for a material like A-36. The cyclic curve is the envelope of the stress-strain curves exhibited during the cycling as shown by the dashed line of figure A9. It is important to compare the type of relaxation which one can experience under cyclic loadings with no strain reversal to those which can be experienced for the latter. To do so we will utilize Figure A11, (provided by M.J. Manjoine), which combines both types of loadings. In the case of cyclic loading with no strain reversal, the second cycle will have a proportional limit PL1 which is about 15 percent lower than the monotonic proportional limit. However, if the cyclic is one of relatively large strain reversal (i.e., strains near yield here defined as .2% offset), then the proportional limit will be much lower as indicated by point PL2 in the figure.

For strain reversal conditions, according to Mr. Manjoine there is little difference between the stress strain curve of ferritic steels like SA-36 and austenitic steels like SA-304. Thus, the material relaxation properties of SA-36 can be inferred for cyclic loads from those of SA-304 for which considerably more data is available.

Figure A6, reproduced from ASTM-DS60 (reference 4) shows the relaxation behavior of SA-304. It can be seen that for cyclic loading with strain reversal there can be always some material relaxation, but that for stresses below $1/2\,\sigma y$, the amount of relaxation is minor.

Material relaxation, however, is only one of the parameters of interest in the overall relaxation of the U-bolt assembly. Relaxation of the assembly preload can be due to a combination of material relaxation and other mechanical relaxation phenomena that may manifest themselves during the various loading cycles, such as wear, local yielding with load redistribution, etc.

It is difficult to predict the amount of relaxation that might occur as a result of wear or yielding of surface irregularities. It is for that reason that the long term, accelerated vibration test was conducted, i.e., to simulate the number of cycles that the assembly would see during its entire lifetime of operation. It is possible, however, to estimate the amount of mechanical relaxation that takes place due to local yielding, although it is impossible to tell how quickly it will occur since the time required for load redistribution depends on too many factors. Such overall estimates can proceed from a knowledge of the stress state at each location of the assembly, which permits an estimate of the volume of material that might be at yield. This volume of material will relax over time, redistributing load, and giving the appearance that the overall assembly relaxes. It is germane to estimate what amount of

relaxation could occur when the shank of the U-bolt is stressed to a maximum stress of 1/2 yield strength. At such loads there are portions, however small, of the assembly which experience higher stresses and can in fact be at yield. These regions are shown in Figure A12 as points A, B, C, D and E. Points A, B and C yield at the outer fibers when the U-bolt is cinched up and preloaded to relatively low value of loads as a result of straightening the U-bolt legs. Yielding is, however, limited to the outer fibers near and opposite the pipe, and the material which yields occupies negligible volume.

For consistency with future discussion of Westinghouse test data, we will use a yield strength of the material of the U-bolt equal to 36,000 psi, even though actual material yield is about 45,000 psi. Test results obtained by strain gauges have all been referred to the 36,000 nominal yield strength. When the stress in the shank is equal to 1/2 the yield strength in the U-bolt shank area, for instance for the 10-inch assembly (refer to Attachment 1 to the Affidavit) with the 3/4 inch U-bolt, the corresponding load is 7,956 lbs., which gives a threaded area stress in excess of 1/2 of yield, i.e., 23,820 psi. However, as figure A13 indicates, the nut engagement results in stress concentration within the threaded area. Stress concentration can raise the average stress above yield. Since we have two nuts, a similar stress concentration profile will exist in the bolt within the other nut because of the nut engagement to the first one. For the 3/4-inch bolt, the nuts are 5/8 inch thick with six

threads. Approximately half of the bolt volume within both nuts will have stress concentration in excess of 1.5. Thus, a total length of 5/8 inches will have stresses at or close to yield.

The same is true in the other leg of the U-bolt. Thus, about 1.25 inches of material out of a total of 31 inches will experience relaxation of the order 15 percent (relaxation from yield stress - see figure A2) if at room temperature. The remaining threaded area (approximately 5 inches) will experience less relaxation since it is more lightly stressed. The amount of relaxation that it can experience can be estimated using figure 2, suggested by M.J. Manjoine. This additional threaded material would relax approximately 7.5 percent. Thus, one can approximate the overall mechanical relaxation that would occur for loads resulting in stresses in the shank of one-half yield as

$$\frac{5\ (.075) + 1.25\ (.15)}{3.25} = 1.7\%,$$ or very low relaxation.

Perhaps more relevant than theoretical calculations to the question of when overall (material and mechanical) relaxation ceases for the U-bolts, is the actual data taken during the various tests conducted by Applicants (see reference 1). One such test is the thermal cycling test.

Results of the thermal cycling test on the 4" Sch 160 stainless steel specimen indicated that the stress in the U-bolt was approximately 31,100 psi (or approximately 86.4% of the assumed yield strength of 36,000 psi and essentially equal to the cyclic yield strength). The total material would thus relax.

After nine cycles the residual stress was measured to be approximately 19,900 psi or 55 percent of the assumed yield strength. (Ambient temperature for pipe and U-bolt was essentially the same before cycling (105°F) and just before the 10th cycle (107.5°F). The U-bolt was heated to an average temperature of about 400°F (see page 16 of Attachment 3 to the Affidavit). From Figure A2 one can deduce that the temperature cycling would result in a relaxation of approximately 36 percent, of which the initial 25 percent would be due to the temperature cycling alone. The result of the thermal cycling test does in fact confirm that the room temperature stress before the thermal cycling, i.e., a nominal 31,100 psi, was reduced to 19,900 or a 36 percent reduction.

Another test which provides insight on the stress relaxation is the creep test which was performed immediately after completion of the thermal cycling test, without retorquing the bolts.

For the 4-inch specimen the microstrain measured in the two U-bolt legs at the ambient temperature before the creep test (77°F) were 856 and 775 microstrain for legs 1 and 2 respectively. (These microstrains correspond to a load of 4,870 and 4,409 lbs.) After the creep test with the ambient temperature being 91.4°F, the strains were measured to be 853 and 773 microstrain, respectively. When one accounts for the fact that at 91.4° there is a preload induced by the difference in thermal expansion between the stainless steel pipe and the carbon

steel U-bolt, and that had the ambient temperature returned to 77°F the preload would have been reduced by approximately 45 lbs., the final load at the completion of the creep test would be approximately 4,580 lbs. compared to 4,639 (or 1.2 percent decrease).

Since 4,580 lbs. corresponds to a stress of 23,367 psi (shank area), which is above 1/2 of the assumed yield strength of 36,000, this decrease, if real and not due to instrument uncertainty, would be due to the strain relaxation. The question of whether it may be due to creep is addressed in the answer to the next question.

For the 10" Sch 40 line, where the temperature is low (pipe 250°F and U-bolt 150°F) creep is clearly not a concern. The strains measure prior to the creep test (after the thermal cycling test) were 283 and 280 microstrains respectively in legs 1 and 2 of the U-bolt (at an ambient temperature of 75.8°F). The initial microstrains correspond to a load of 3,625 and 3,578 lbs. respectively. These loads correspond to a stress equal to 8,200 psi in the shank or 10,800 psi in the thread area of the U-bolt. In either case the stresses are well below the 1/2 yield strength, with the exception of highly local area in the thread within the nut, and hence little, if any, relaxation should be exhibited.

The strains after the creep test were measured to be 281 and 276 microstrains respectively corresponding to an average load of 3,567 lbs.

The drop in load of approximately 39 lbs. is partly due to the lower environment temperatures after the test which was 66.9°F instead of 75.8°F.

The drop in load corresponding to the 9 degrees difference is calculated to be approximately 11 lbs. Thus, relaxation (if any) was less than 0.8 percent.

The seismic test provides further evidence of the relaxation phenomenon. Initial information provided from the test, which is attached as Exhibit A3, indicated a reduction in load from 4,484 lbs. in both U-bolt legs to about 4,291 lbs. and 4,355 lbs. in legs 1 and 2 respectively, when the assembly was vibrated at 9 Hz with a constant amplitude of 7,000 lbs. This relaxation of approximately 12 percent could not be justified on the basis of the applied load which would result, coupled with the initial preload of 4,484 lbs. (50 ft. lb. torque) in maximum load experienced by the U-bolt of approximately 6,100 lbs., and a corresponding stress of 18,200 psi in the threaded area and 13,800 psi in the shank area. This led to questioning the validity of the 7,000 lb. load, and to the realization that the actual applied vibratory load had been higher, and to the results published in the Affidavit, which are included here as Exhibit A4. As seen in the Exhibit, the actual load applied to the U-bolt was in excess of 10,000 lbs during the peak portion of the cycle and initially in excess of 8,600 lbs. during the pull portion of the cycle. On the average the force seen by the U-bolt during the cycling was in excess of 6,600 lbs. (peak load of

more than 8,600 lbs. plus preload of 4,484 lbs.) which would have resulted in a stress in the thread area of about 19,800 lbs. which is 11 percent higher than the nominal 1/2 yield strength, hence justifying the relaxation seen.

Finally, the data obtained during the long term accelerated vibration test merits some attention.

As stated in our Affidavit, the initial preload stress was equal to about 9,020 psi. After the initial reposition of the assembly which occurred approximately 5.15 minutes into the test (see attached raw data - Exhibit A5), and which resulted in an average loss of preload equal to 640 lbs, the preload was seen to decrease slightly, then increase again then decrease with a final preload being about 450 less than the preload existing after the initial adjustment. During the period of time between the 4th sweep (21 minutes) and the 36th sweep (189 minutes) there was essentially no change in the preload. At the latter time is when the sudden cocking mentioned in the Affidavit on p. 30 took place, which resulted in some further preload decrease.

Relaxation of the material discussed within the context of this reply does not change the total strain of the material. (See definition in 2 of Exhibit A2.) The preload at the end of the test is still sufficient to prevent loss of contact between the pipe and backing plate (see figures 17 and 18 of Attachment 1 to the Affidavit with an applied load of 1,500 lbs. and a preload of approximately 3,200 lbs.), thus the motion which resulted in further relaxation is most likely due to accumulated strain over

the more than 10^6 cycles experienced at an applied load of 1,500 lbs. These cycles represent the number that the support may experience during its lifetime, and hence the test results confirm that in spite of some relaxation, adequate preload would be retained throughout life.

Cyclic plastic strain accumulation may occur at these loads, which are abnormally high for the period of time tested. An elasto plastic finite element analyses of a similar U-bolt, backing plate, pipe arrangement, conducted per an 8-inch pipe (same size U-bolt as the 10" pipe, indicates that for sufficiently high preload, the U-bolt can experience some plasticity in the transition region between the straight shank and the curved portion and at the inner surface of the U-bolt apex. This occurs from the bending moment place on the U-bolt from the straightening action of the preload or full external load. This small amount of plasticity occurs even though the average stresses through the U-bolt cross section is low, and in fact, for the particular case examined are only 2,000 psi. Under the large number of cycles seen by the specimen the accumulated plastic strain can result in sufficient permanent deformation to permit relaxation. Also, wear and yielding of surface imperfections can accomplish the same thing.

b) Provide more information as to why creep of the U-bolt should not be a consideration, considering the result for the 4-inch pipe. Provide material of U-bolt nut. Include explanation on effect of different ambient temperatures on loss of preload shown by this test.

Relative to the possibility of creep phenomena existing in the U-bolt, the maximum temperatures measured for each of the three test specimens, during the Creep Test are listed below. It is to be noted that the temperature in the U-bolt varied along its length.

4-inch specimen

Pipe temperature:	560°F
U-bolt temperature:	445°F
Nut temperature:	340°F

10-inch specimen

Pipe temperature:	250°F
U-bolt temperature:	150°F
Nut temperature:	140°F

32-inch specimen

Pipe temperature:	560°F
U-bolt temperature:	350°F
Nut temperature:	170°F

Also note that all three U-bolts are SA-36 Carbon Steel. Reference 2 suggests a temperature of 752°F (400°C, 673°K) as the minimum used for creep tests performed for carbon steels. Finite creep is not discernable in carbon steels at temperatures lower than this. Figure A7 (from reference 4) further confirms this. Reference 3 defines the temperature below which self-diffusion is too slow to influence creep as approximately one-half of a metal's absolute melting temperature. The absolute melting temperature for SA-36 carbon steel is in excess of 1366°K (1093°C, 2000°F). Similarly, reference 4 defines the temperature below which creep is not discernable as 0.4 Tm (Tm metal absolute melting temperature) which would correspond to 524°F.

Based on the fact that none of the U-bolt temperatures exceeded 500°F it can be concluded that no finite creep occurred in the U-bolts. Since the nut material is ASTM-A563GrA and none of the temperatures exceeded 340°F, no finite creep occurred in the nuts. The curve shown in Firgure A3 for ferritic steels like SA-36, and Figure A7, taken from reference 4 for austenitic steel, confirms that relaxation is not due to creep until temperatures of approximately 800°F are available.

The small decrease in U-bolt preload experienced during the test, of the 4 inch sch 160 pipe is believed to be a result of relaxation as explained in the answer above.

Based on the above, and test results obtained, it is concluded that none of the U-bolt test specimens were subject to creep phenomena during the Creep Test.

The explanation of the effect on the loss of preload from the different ambient temperatures is given in the answer to the preceding question, namely the higher ambient temperature at the end of the test would have the effect of underestimating the loss of preload by about 45 lbs.

c) What is the thickness of the backing plate for the 4" pipe - U-bolt configuration?

The thickness of the backing plate is 3/4 of an inch. The drawing provided was a poor copy where the copying has resulted in a 3 looking like a 1. Enclosed (Enclosure A1) is a better copy of the drawing reflecting the 3/4 inch thickness.

d) Clarify the statement made in the opening remarks regarding the 32" pipe on page 42 of Attachment 3 to the Affidavit.

The statement as written: "The stresses measured in the test and calculated for the 32" pipe, cross piece and U-bolt are comparable." was not meant to state that the magnitude of the stresses calculated or measured were comparable numerically. It is quite obvious from the 32 inch pipe test data that the data scatter would make such comparison questionable. It simply meant that the very low stresses calculated by finite element analysis were confirmed to be low by test.

e) Verify that stresses in the pipe would still be acceptable if one had used the C indices rather than the B indices of the Code on p. 54 and following of the Affidavit.

This question refers to the effect on the pipe stress intensities that would be computed, had the piping moment stresses been computed utilizing the C indices (Class 2 and 3) rather than B indices (Class I).

The effect of ASME Class 2 and 3 rules on the piping stresses has been discussed in the affidavit on pages 63 to 66. On page 65 of the affidavit, a comparison is made in Tables L and M of the deadweight and seismic (Equation 9 - Class 1 rules) and the thermal (Equation 12 - Class 1 rules) piping moment stresses developed using Class 1 and Class 2/3 stress indices.

The changes in stress indicated by the results reported in these tables are given below (Table A[1]). Note that a positive value implies an increase in stress, and a negative sign a decrease in stress if Class 2/3 rules are used.

TABLE A[1]

CHANGE IN PIPE STRESS

		CHANGE IN STRESS (KSI)	
PIPE SIZE	MATERIAL	DEADWEIGHT+ SEISMIC	THERMAL
4" SCH 160	Stainless	2.15	4.01
10" SCH 40	Stainless	2.55	0.07
10" SCH 80	Carbon	1.97	-2.86
32" MS	Carbon	2.49	-0.32

The results of this change on the stress intensities calculated using Class 2/3 rules is given below (Tables B[1] and C[1]). These tables can be compared to Tables H and I given on page 60 of the Affidavit.

TABLE B[1]

TOTAL PIPE STRESS INTENSITY

PIPE SIZE	PRELOAD TORQUE	APPLIED STRUT LOAD	TOTAL STRESS INTENSITY
4" SCH 160	60 ft/lbs	2,000 lbs	70.3 ksi
10" SCH 40S	100 ft/lbs	10,000 lbs	76.83 ksi
10" SCH 80	100 ft/lbs	10,000 lbs	53.80 ksi
32" MS	240 ft/lbs	100,000 lbs	49.34 ksi

TABLE C[1]

MAXIMUM PRIMARY AND SECONDARY STRESS INTENSITIES

PIPE SIZE	EQ. 9 PRIME STRESS INTENSITY (KSI)	EQ. 9 ALLOWABLE (KSI)	EQ. 12 SECONDARY (KSI)	EQ. 12 ALLOWABLE (KSI)
4" SCH 160	33.75	50.52	36.55	50.52
10" SCH 40S	63.16	60.00	13.67	60.00
10" SCH 80	40.12	60.00	13.68	60.00
32" MS	33.06	58.26	16.28	58.26

With the exception of Equation 9 for the 10" SCH 40S pipe size, all of the pipes evaluated meet the Equation 9 and Equation 12 allowables. The Equation 9 stresses reported for the 10" SCH 40S pipe are conservative since:

1. The pipe stress includes the secondary stress due to pressure pipe growth restriction.

2. A higher stress push load is used than seen by the Comanche Peak 10" U-bolt supports.

3. A higher mechanical primary pipe moment stress is used than seen by the 10" Comanche Peak pipes.

The significance of each of these items is given below:

1. The total circumferential pressure stress from the computer analysis is 10.51 ksi. The circumferential pipe stress due to pressure is 8.84 ksi. The secondary pressure stress is 10.51 - 8.84 = 1.67 ksi, which is presently included as primary stress.

2. The largest U-bolt strut load as determined from ITT Grinnell U-bolts loads is 8,585 pounds. In the evaluation, a 10,000 load was used. This is equivalent to a 2 ksi reduction in pipe stress.

$$= (72.71) - 58.59 \left[1 - \left(\frac{8585}{10,000}\right)\right] = 2.0 \text{ ksi}$$

3. On pages 61 and 62 of the affidavit, a comparison is made between the primary piping moment stresses used in the U-bolt evaluation to actual randomly selected computer piping analysis stresses. From Table J of the affidavit, it can be seen that the mechanical primary pipe moment stress used in the U-bolt cinching evaluation is 3.3 ksi higher, (10.45 - 7.063 = 3.3 ksi).

Adjusting the 10" SCH 40S stress intensities given in Table C[1] to remove the conservatisms discussed above results in a primary stress intensity value of 56.19 ksi. Note that without consideration of item 3, 3.3 ksi, the primary stress intensity value is 59.49 ksi and is still below the allowable stress. The secondary stress intensity if 15.34 ksi. Thus, the 10" SCH 40S pipe is within the acceptable limits of 60.0 ksi for primary and secondary stress.

f) Provide an example of how the total value of stress intensity can be obtained from the finite element results and how the value can be divided into equation (9) and equation (12) stress intensities.

The easiest way to show how the stress intensity is obtained is to refer to the figure VII-2 of Attachment 3 of the Affidavit which defines it as the maximum of either the absolute difference between the major principal stress or minor principal stress and zero or the algebraic difference of the two principal stresses, and to apply this figure to an actual example. The example chosen is the 4" sch 160 pipe. For the elements having the largest circumferential and longitudinal stresses, the finite element analyses determined that the principal stresses are virt lly identical to the circumferential and longitudinal stresses (see Attachment 3 of Affidavit at page 57). The longitudinal, circumferential, major and minor principal stresses for the highest stressed piping element of the 4" sch 160 pipe

are given for both the inside and outside surfaces and for the maximum load case in the table of p. 58 of Attachment 3 to the Affidavit. These values are reproduced below:

	Long. Stress (ksi)	Circum. Stress (ksi)	Princ. Stress (ksi) Major	Minor
4" sch 160 inside	10.49	44.79	44.78	10.50
outside	-26.65	-34.07	-26.63	-34.08

where the negative sign denotes compressive stresses.

A confirmation of the max. circumferential stress can be found in the table of page 71 of Attachment 3 of the Affidavit for element 627. Note that on that table, there is no distinction regarding the surface at which the maximum stresses occur. For instance, the 44.79 ksi tensile circumferential stress occurs on the inside surface, while the -26.65 ksi compressive longitudinal stress occurs on the outside surface of element 627. To the local stresses computed by the finite element analysis one must add the longitudinal equation 9 pressure and piping moment stresses. These are available from the table on page 56 of attachment 3 of the Affidavit. They are:

Longitudinal Pressure Stress	4.8 ksi
EQ. 9 Piping Moment Stress	± 12.146 ksi
EQ. 12 Piping Moment Stress	± 22.49 ksi

Adding the longitudinal pressure to the stresses previously tabulated we obtain:

	Principal Stresses	
	Major (Circumferential)	Minor (Longitudinal)
4" sch 160 Inside	44.79	15.29
Outside	-34.07	-21.85

To add the piping moment stresses to the longitudinal (minor principal) stresses, we choose the sign which will produce the largest stress intensity.

This is seen in a Mohr circule depicted in Figure A14, where inside surface stresses are used.

Thus, the total stress intensity is given by 44.79- (-19.346) = 64.136 ksi, which is the total stress intensity given on page 59 of Attachment 3 of the Affidavit or in table H of page 60 of the Affidavit.

For comparison purposes, the stress intensity derived for the outside surface is:

Maj. Princ. (Circumferential) stress = -34.07

Minor Princ. (Longitudinal) stress = -26.63 + 4.8 ± 12.146 ∓ 22.49 = -56.466

The max. stress intensity is thus 56.47 ksi.
Using the alternative signs would have produced a stress intensity of 34.07 + 12.8 = 47.5 ksi which is lower.

As shown above, the highest stress intensity occurs on the inside surface.

To determine the primary and secondary stress intensities, several alternatives are available. The most straightforward determines the primary stress intensity from the principal primary stresses and derives the secondary stress intensity by subtraction of the primary from the total. For the example chosen, we proceed as follows:

(i) The secondary portion of the circumferential stress is obtained as the stress due to thermal expansion by subtracting the circumferential stress due to preload + thermal given on page 59 of Attachment 3 of the Affidavit as -39305 psi, from the circumferential stress due to preload alone, which is given in the preceding page as -26091 psi. These occur on the outside surface. The primary circumferential stress becomes -34.07 +13.21 = -20.86 ksi.

(ii) The primary longitudinal stress is similarly derived by considering only the equation 9 piping moment stress, i.e., neglecting the equation 12 stress and subtracting the difference between the longitudinal

stress due to preload + thermal and that due to preload only, which equals 6.5 ksi. The longitudinal stress thus becomes -21.85 -12.146 + 6.5 = -27.5 ksi.

(iii) Thus, the primary stress intensity is -27.5 ksi and the secondary stress intensity becomes 56.47 - 27.5 = 28.97 ksi.

Similarly, we obtain the primary and secondary stress intensities for the inside surface.

(i) Primary circumferential 44.79 - 10.81 = 33.98 (10.81 is the difference between preload + thermal and preload only circumferential stresses for the inside surface and these do not appear in any table, but are available from the computer output).

(ii) Primary longitudinal = 15.29 $\pm$ 12.146 - 4.24 = -1.1 where again 4.24 is the difference between the longitudinal stress due to preload + thermal and that due to preload only.

Please note that the primary stress intensity is thus 35.1 ksi instead of the value of 31.6 reported on page 59 of the Attachment 3 to the Affidavit.

(iii) The secondary stress intensity then becomes 64.14 - 35.1 = 29.04 ksi instead of the 32.54 ksi reported.

The difference between the numbers here and in the Affidavit occurred when inadvertently the outside secondary circumferential stress was subtracted from the inside total circumferential stress.

g) Define what is meant by partial preload in the tables Attachment 3 to the Affidavit.

Partial preload refers to a loading condition in which the torque of the U-bolt is a fraction of the maximum torque that is assumed to be applied to the U-bolt. For instance, for the 4" sch 160 pipe U-bolt assembly full preload corresponds to a torque of 60 ft.-lbs., and partial preload corresponds to a torque of 9 ft.-lbs.

h) Confirm the location of strain gauges S5 and S10 in Figure 21 of the Test Report SQ&T-EQT-860 (Attachment 1 of the Affidavit).

Sketch 5 on p. 58 of Attachment 1 of the Affidavit is in error. It inadvertently suggests that the same U-bolt strain gauge identification scheme used for the Torque vs. Preload, Friction and Load Distribution Tests was used for the Thermal Cycling and Creep Tests. This was not the case. Since high temperature strain gauges were required for the Thermal Cycling and Creep Tests, low temperature gauges that may have been used for previous tests were removed. The high temperature strain gauges were not instrumented to be consistent with the low temperature gauges. Also, the low temperature gauges were identified by BLH channel number. When test data for Thermal Cycling and Creep Tests was first received from the lab, the strain gauges were identified by serial number. Thus, in EQ&T-EQT-860, the strain gauges used for the Thermal Cycling and Creep Tests are not identified by the sample S1 through S5 sequence as

in the other tests. Since channel numbers are directly traceable to serial numbers, any results contained in the test report are easily traceable to the appropriate test data.

The high temperature strain gauges as installed for the Thermal Cycling and Creep Tests are identified on the attached Figure A15 for each of the U-bolt sizes. The strain gauge on the three-gauge U-bolt leg that is located 90° from the two other gauges is not required to monitor U-bolt preload and, therefore, is not referenced in any of the test results.

(i) Correct typo on p. 66 of Test Report

Leg 2 (gauges S4, S11) should read 3516 instead of 5316 pounds.

(j) Provide material properties of the U-bolts and nuts used.

The mechanical properties of the U-bolts are as follows:

1/2" U-bolt	Sy = 45130, 45290 psi;	Su = 63080, 63590 psi
3/4" U-bolt	Sy = 44350;	Su = 65120 psi
2 3/4" U-bolt	Properties not provided by Vendor	
Nuts	ASTM - A563 GrA.	

References:

1. Report No. EQ&T-EQT-860, "Comanche Peak Steam Electric Station U-Bolt Support/Pipe Test Report".
2. I.A. Oding, "Creep and Stress Relaxation in Metals", Oliver and Boyd, 1965.
3. F. Garofolo, "Fundamentals of Creep and Creep-Rupture in Metals", Macmillan, 1966.
4. Compilation of Stress Relaxation Data for Engineering Alloys, "ASTM Data Service Publication DS-60."
5. ASTM Journal of T&E, Vol. 1, # 4, p. 275, 1973.
6. H.R. Jhansale and T.H. Topper "Engineering Analysis of the Inelastic Stress Response of a Structural Metal Under Variable Cyclic Strains," ASTM STP 519, 1973, pp. 246-270.

MR. VIVIRITO: One moment, please.

If you look at the test result, and you are concerned with the question of shakedown and twice yield, when you look at the test that tests the entire assembly, the bolt, the Richmond and the concrete, there really is no pronounced yield before you go to ultimate. It is rather gradual.

When you look at the typical test strain curve for steel, you have a very sharp yield for a long point, and then it goes up. The assembly of the entire bolt, the bolt, the Richmond and the concrete, there is no real pronounced yield. So it really is not relevant in terms of shakedown.

JUDGE BLOCH: What you are saying is that the yield for these bolts at the time that Stardyne was run was lower than it should have been?

MR. VIVIRITO: No, sir. What I'm saying is that the entire assembly must work together. You have the bolt and the Richmond and the concrete. And the only way you can evaluate the entire assembly is by a test, and indeed the observed phenomena in running these tests is that there is no real -- there is no real pronounced yield.

JUDGE BLOCH: There is no yield on this bolt?

1 MR. VIVIRITO: No, not on the bolt. On the
2 entire assembly.
3 What we are looking at is the way the entire
4 assembly works. And you can't really analyze the
5 interaction between the bolt and the Richmond and the
6 concrete. That is why a test is the only valid way
7 to arrive at an allowable.
8 And when you look at that entire assembly,
9 indeed when you look at this curve as compared to
10 a stress strain curve for typical steel, stress strain
11 curve for typical steel has a very pronounced curve
12 for a long time, and if indeed you go too far you
13 get a considerable elongation because you are in the
14 flat part of the curve.
15 These curves do not have any pronounced
16 yield. And that is the phenomena of a steel concrete
17 interaction.
18 JUDGE BLOCH: So at the very least this
19 unit acting as a whole has a yield that must far
20 exceed normal stresses?
21 That is based on this test?
22 MR. VIVIRITO: There is no pronounced yield.
23 It goes finally to an ultimate.
24 JUDGE BLOCH: No pronounced yield also means
25 that the yield is far above the normal stresses.

SPECIFICATION FOR CARBON STEEL EXTERNALLY AND INTERNALLY THREADED STANDARD FASTENERS

SA-307

(Identical with ASTM Specification A 307-76a except for Pars. 2.6 and 4.1)

1. Scope

1.1 This specification covers the chemical and mechanical requirements of two grades of carbon steel externally and internally threaded standard fasteners, in sizes ¼ in. (6.35 mm) through 4 in. (104 mm). This specification does not cover requirements for externally threaded fasteners having heads with slotted or recessed drives. The fasteners covered by this specification are frequently used for the following applications:

1.1.1 *Grade A Bolts*, for general applications, and

1.1.2 *Grade B Bolts*, for flanged joints in piping systems where one or both flanges are cast iron.

1.2 If no grade is specified in the inquiry, contract, or order, Grade A bolts shall be furnished.

1.3 Nonheaded anchor bolts, either straight or bent, to be used for structural anchorage purposes, shall conform to the requirements of ASTM Specification A 36, for Structural Steel, with tension tests to be made on the bolt body or on the bar stock used for making the anchor bolts.

NOTE—The values stated in U.S. customary units are to be regarded as the standard.

2. Materials and Manufacture

2.1 Steel for bolts shall be made by the open-hearth, basic-oxygen, or electric-furnace process.

2.2 Steel for nuts shall be made by the open-hearth, basic-oxygen, electric-furnace, or bessemer process.

2.3 Bolts may be produced by hot or cold forging of the heads or machining from bar stock.

2.4 Bolt threads may be rolled or cut.

2.5 Nuts may be produced by hot pressing, cold punching, cold forging, or machining from bar stock.

2.6 Galvanized bolts, nuts and washers shall be hot-dip galvanized in accordance with the requirements of ASTM Specification A 153, Class C, for zinc coating (Hot Dip) on Iron and Steel Hardware. Nuts shall be tapped oversize, after galvanizing, by the diametral amounts listed in ASTM Specification A 563 for Carbon Steel Nuts unless otherwise specified. When specified by the purchaser, fasteners may be mechanically galvanized provided that the coating and coated product meet the coating thickness, adherence and quality requirements of ASTM Specification A 153, Class C. Mechanically galvanized nuts for assembly with mechanically galvanized bolts shall be tapped oversize prior to mechanically galvanizing but need not be retapped after mechanically galvanizing. Nuts shall be provided with a suitable lubricant.

3. Chemical Requirements

3.1 Steel for bolts and nuts shall conform to the following chemical requirements:

	Grade A		Grade B	
	Bolts	Nuts	Bolts	Nuts
Phosphorus, max, %	0.06	0.13	0.04	0.12
Sulfur, max, %	0.15	0.23	0.05	0.15

3.2 Resulfurized material is not subject to rejection based on product analysis for sulfur.

3.3 Bolts and nuts are customarily furnished from stock, in which case individual heats of steel cannot be identified.

3.4 Application of heats of steel to which bismuth, selenium, tellurium, or lead has been intentionally added shall not be permitted for Grade B bolts.

4. Mechanical Requirements

4.1 Bolts shall not exceed the maximum hardness required in Table 1. Bolts less than

ATTACHMENT B

AMERICAN NATIONAL STANDARD

ASTM A 307 – 80

Standard Specification for CARBON STEEL EXTERNALLY THREADED STANDARD FASTENERS[1]

This standard is issued under the fixed designation A 307; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

1. Scope

1.1 This specification[2] covers the chemical and mechanical requirements of two grades of carbon steel externally threaded standard fasteners, in sizes ¼ in. (6.35 mm) through 4 in. (104 mm). This specification does not cover requirements for externally threaded fasteners having heads with slotted or recessed drives or for mechanical expansion anchors. The fasteners covered by this specification are frequently used for the following applications:

1.1.1 *Grade A Bolts*, for general applications, and

1.1.2 *Grade B Bolts*, for flanged joints in piping systems where one or both flanges are cast iron.

1.2 If no grade is specified in the inquiry, contract, or order, Grade A bolts shall be furnished.

1.3 Nonheaded anchor bolts, either straight or bent, to be used for structural anchorage purposes, shall conform to the requirements of Specification A 36 with tension tests to be made on the bolt body or on the bar stock used for making the anchor bolts.

1.4 Suitable nuts are covered in Specification A 563. Unless otherwise specified, the grade and style of nut for each grade of fastener, of all surface finishes, shall be as follows:

Fastener Grade and Size	Nut Grade and Style[4]
A, ¼ to 1½ in.	A, hex
A, over 1½ to 4 in.	A, heavy hex
B, ¼ to 4 in.	A, heavy hex

[4] Nuts of other grades and styles having specified proof load stresses (Specification A 563, Table 3) greater than the specified grade and style of nut are also suitable.

1.5 The values stated in inch-pound units are to be regarded as the standard.

2. Applicable Documents

2.1 *ASTM Standards*:

A 36 Specification for Structural Steel

A 153 Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware[5]

A 370 Methods and Definitions for Mechanical Testing of Steel Products[6]

A 563 Specification for Carbon and Alloy Steel Nuts[6]

B 454 Specification for Mechanically Deposited Coatings of Cadmium and Zinc on Ferrous Metals[8]

2.2 *American National Standards*:[9]

ANSI B1.1 Unified Screw Threads

ANSI B18.2.1 Square and Hex Bolts and Screws

3. Materials and Manufacture

3.1 Steel for bolts shall be made by the open-hearth, basic-oxygen, or electric-furnace process.

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[1] This specification is under the jurisdiction of ASTM Committee F-16 on Fasteners, and is the direct responsibility of Subcommittee F 16.02 on Steel Bolting.

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[2] For ASME Boiler and Pressure Vessel Code applications see related Specification SA-307 in Section II of that Code.

[3] *Annual Book of ASTM Standards*, Part 4.

[4] *Annual Book of ASTM Standards*, Part 3.

[5] *Annual Book of ASTM Standards*, Parts 1 and 4.

[6] *Annual Book of ASTM Standards*, Parts 4 and 9.

[7] *Annual Book of ASTM Standards*, Parts 1, 2, 3, 4, 5, and 10.

[8] May be obtained from American National Standards Institute, Inc., 1430 Broadway, New York, N. Y. 10018.

JUDGE BLOCH: The tests were not done under normal operating conditions?

WITNESS CHEN: Yes, but I think they are over and above the normal operating conditions. So our concerns about the bolts would have been resolved by the fact that they were subjected to a much higher load and did not fail, and furthermore, that the allowables for the bolts for normal operating conditions ensures that there will be a low yield. In fact, I think they are .75 times the yield stress.

MR. MIZUNO: Chairman Bloch, if I could just ask you a few questions.

JUDGE BLOCH: Please.

BY MR. MIZUNO:

Q. Dr. Chen, are the allowables under the normal conditions lower than the allowables under the LOCA conditions?

A. (WITNESS CHEN) Yes.

Q. If you tested the entire insert to failure and you showed that it met the required factor of safety, then of necessity, it had to go and show that the allowables for the normal and upset conditions were also met?

A. (WITNESS CHEN) Yes.

JUDGE BLOCH: Of course, somewhat leading

redirect. But as I understand it, that is not necessarily true. Because we are talking in the one case about a one-time stress and in the other about a cyclical stress; isn't that correct?

WITNESS CHEN: Yes.

MR. SCINTO: Mr. Chairman, I thought the problem and the confusion was the point that at the test, with the LOCA condition, with the one-time stress there was deformation. The argument was that deformation was nonelastic behavior; therefore, the test did not demonstrate elastic behavior under normal usage.

JUDGE BLOCH: That is not what we are discussing now.

MR. SCINTO: I want to add that the witness testified that under normal loads those bolts act elastically, because of the nature of the load.

JUDGE BLOCH: Providing you stay below yield right? You can go above yield --

MR. SCINTO: I don't want to testify. I think the witness has indicated that.

JUDGE BLOCH: Under normal loads you can't go above yield; is that right?

WITNESS CHEN: That's right.

JUDGE BLOCH: Under a one-time load you can.

WITNESS CHEN: Yes.

JUDGE BLOCH: We did a test which assured ourselves that we meet the accident conditions, one_ time loads?

WITNESS CHEN: Yes.

JUDGE BLOCH: How did you extrapolate down from those tests to assure yourself that the normal loads were met?

WITNESS CHEN: That is not how I did that.

JUDGE BLOCH: How did you do that?

WITNESS CHEN: By looking at the allowables for the normal and upset conditions.

Manual of

STEEL CONSTRUCTION

EIGHTH EDITION

American Institute of Steel Construction, Inc.
400 North Michigan Avenue
Chicago, Illinois 60611

1.3.5 Wind

Proper provision shall be made for stresses caused by wind, both during erection and after completion of the building.

1.3.6 Other Forces

Structures in localities subject to earthquakes, hurricanes and other extraordinary conditions shall be designed with due regard for such conditions.

1.3.7 Minimum Loads

In the absence of any applicable building code requirements, the loads referred to in Sects. 1.3.1, 1.3.2, 1.3.5, and 1.3.6 shall be not less than those recommended in the American National Standards Institute *Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, ANSI A58.1, latest edition.

SECTION 1.4 MATERIAL

1.4.1 Structural Steel

1.4.1.1 Material conforming to one of the following standard specifications (latest date of issue) is approved for use under this Specification:

Structural Steel, ASTM A36
Welded and Seamless Steel Pipe, ASTM A53, Grade B
High-Strength Low-Alloy Structural Steel, ASTM A242
High-Strength Low-Alloy Structural Manganese Vanadium Steel, ASTM A441
Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500
Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501
High-Yield Strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514
Structural Steel with 42,000 psi Minimum Yield Point, ASTM A529
Hot-Rolled Carbon Steel Sheets and Strip, Structural Quality, ASTM A570, Grades D and E
High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy, with Improved Corrosion Resistance, ASTM A606
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy, Columbium and/or Vanadium, ASTM A607
Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM standards. Additionally, the fabricator shall, if requested, provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

1.4.1.2 Unidentified s
for parts of minor importa
physical properties of the st
of the structure.

1.4.2 Other Metals

Cast steel shall conform
edition:

Mild-to-Medium-
Applications.
High-Strength St
Grade 80-50

Steel forgings shall con
edition:

Steel Forgings Ca
ASTM A668

Certified test reports
with the standards.

1.4.3 Rivets

Steel rivets shall conf
edition:

Steel S

Manufacturer's certifica
with the standard.

1.4.4 Bolts

Steel bolts shall conform
edition:

Low-Carbon Steel
Fasteners, AS
High Strength Bo
Nuts and Plai
Quenched and Ter
Quenched and Ten
ASTM A490

In connections, A449 bolt
quiring bolt diameters greate
for high-strength anchor bol
Manufacturer's certifica
with the standards.

1.4.5 Filler Metal and Fl

Welding electrodes and
cations of the American We

* Approval of these welding ele
toughness requirements, wh
Commentary Sect. 1.4.

sses caused by wind, both during

quakes, hurricanes and other ex-
due regard for such conditions.

code requirements, the loads re-
hall be not less than those recom-
institute *Building Code Require-*
ngs and Other Structures, ANSI

following standard specifications
this Specification:

TM A53, Grade B
Steel, ASTM A242
l Manganese Vanadium Steel,

Carbon Steel Structural Tubing
500
Carbon Steel Structural Tubing,

Tempered Alloy Steel Plate,
14
mum Yield Point, ASTM A529
Strip, Structural Quality, ASTM

-Vanadium Steels of Structural

Steel with 50,000 psi Minimum
A588
d Cold-Rolled, High-Strength,
sion Resistance, ASTM A606
d Cold-Rolled, High-Strength,
anadium, ASTM A607
igh-Strength Low-Alloy Struc-

ts of tests made by the fabrica-
STM A6 or A568, as applicable,
ufficient evidence of conformity
tionally, the fabricator shall, if
structural steel furnished meets

1.4.1.2 Unidentified steel, if free from surface imperfections, may be used for parts of minor importance, or for unimportant details, where the precise physical properties of the steel and its weldability would not affect the strength of the structure.

1.4.2 Other Metals

Cast steel shall conform to one of the following standard specifications, latest edition:

Mild-to-Medium-Strength Carbon-Steel Castings for General Applications, ASTM A27, Grade 65-35
High-Strength Steel Castings for Structural Purposes, ASTM A148, Grade 80-50

Steel forgings shall conform to the following standard specification, latest edition:

Steel Forgings Carbon and Alloy for General Industrial Use, ASTM A668

Certified test reports shall constitute sufficient evidence of conformity with the standards.

1.4.3 Rivets

Steel rivets shall conform to the following standard specification, latest edition:

Steel Structural Rivets, ASTM A502

Manufacturer's certification shall constitute sufficient evidence of conformity with the standard.

1.4.4 Bolts

Steel bolts shall conform to one of the following standard specifications, latest edition:

Low-Carbon Steel Externally and Internally Threaded Standard Fasteners, ASTM A307
High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers, ASTM A325
Quenched and Tempered Steel Bolts and Studs, ASTM A449
Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints, ASTM A490

In connections, A449 bolts may be used only in bearing-type connections requiring bolt diameters greater than 1½ inches. A449 bolt material is also acceptable for high-strength anchor bolts and threaded rods of any diameter.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

1.4.5 Filler Metal and Flux for Welding

Welding electrodes and fluxes shall conform to one of the following specifications of the American Welding Society, latest adoption, as appropriate:*

* Approval of these welding electrode specifications is given without regard to weld metal notch toughness requirements, which are generally not necessary for building construction. See Commentary Sect. 1.4.

1.5.2.2 Design for rivets, bolts, and threaded parts subject to fatigue loading shall be in accordance with Appendix B, Sect. B3.

TABLE 1.5.2.1

ALLOWABLE STRESS ON FASTENERS, KSI

Description of Fasteners	Allowable Tension[g] (F_t)	Allowable Shear[g] (F_v)			
		Friction-type Connections[e,i]			Bearing-type Connections[i]
		Standard size Holes	Oversized and Short-slotted Holes	Long-slotted Holes	
A502, Grade 1, hot-driven rivets	23.0[a]				17.5[f]
A502, Grades 2 and 3, hot-driven rivets	29.0[a]				22.0[f]
A307 bolts	20.0[a]				10.0[b,f]
Threaded parts meeting the requirements of Sects. 1.4.1 and 1.4.4, and A449 bolts meeting the requirements of Sect. 1.4.4, when threads are not excluded from shear planes	$0.33F_u$[a,c,h]				$0.17F_u$[h]
Threaded parts meeting the requirements of Sects. 1.4.1 and 1.4.4, and A449 bolts meeting the requirements of Sect. 1.4.4, when threads are excluded from shear planes	$0.33F_u$[a,h]				$0.22F_u$[h]
A325 bolts, when threads are not excluded from shear planes	44.0[d]	17.5	15.0	12.5	21.0[f]
A325 bolts, when threads are excluded from shear planes	44.0[d]	17.5	15.0	12.5	30.0[f]
A490 bolts, when threads are not excluded from shear planes	54.0[d]	22.0	19.0	16.0	28.0[f]
A490 bolts, when threads are excluded from shear planes	54.0[d]	22.0	19.0	16.0	40.0[f]

[a] Static loading only.

[b] Threads permitted in shear planes.

[c] The tensile capacity of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_b, shall be larger than the nominal body area of the rod before upsetting times $0.60F_y$.

[d] For A325 and A490 bolts subject to tensile fatigue loading, see Appendix B, Sect. B3.

[e] When specified by the designer, the allowable shear stress, F_v, for friction-type connections having special faying surface conditions may be increased to the applicable value given in Appendix E.

[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 inches, tabulated values shall be reduced by 20 percent.

[g] See Sect. 1.5.6.

[h] See Appendix A, Table 2, for values for specific ASTM steel specifications.

[i] For limitations on use of oversized and slotted holes, see Sect. 1.23.4.

1.5.3 Welds

Except as modified by the pro
to meet the stress requirements g

ALLOWAI

Type of Weld and Stress[a]	
	Complete-I
Tension normal to effective area	Same a
Compression normal to effective area	Same a
Tension or compression parallel to axis of weld	Same a
Shear on effective area	0.30 strengt except metal s yield st
	Partial-Pe
Compression normal to effective area	Same a
Tension or compression parallel to axis of weld[e]	Same a
Shear parallel to axis of weld	0.30 strengt except metal s yield st
Tension normal to effective area	0.30 strengt except metal s yield st
Shear on effective area	0.30 strengt except metal s yield st
Tension or compression parallel to axis of weld[e]	Same a
	Pl
Shear parallel to faying surfaces (on effective area)	0.30 strengt except metal s yield st

[a] For definition of effective area, see S
[b] for "matching" weld metal, see Tabl
[c] Weld metal one strength level strong
[d] See Sect. 1.10.8 for a limitation on us
[e] Fillet welds and partial-penetration gr members, such as flange-to-web conn or compressive stress in these elemen

ted parts subject to fatigue loading
B3.

STENERS, KSI

Allowable Shear[a] (F_v)			
ction-type Connections[a,d]			Bearing-type Connections[c]
tard e es	Oversized and Short-slotted Holes	Long-slotted Holes	
			17.5[f]
			22.0[f]
			10.0[b,f]
			$0.17F_u$[b]
			$0.22F_u$[b]
	15.0	12.5	21.0[f]
	15.0	12.5	30.0[f]
	19.0	16.0	28.0[f]
	19.0	16.0	40.0[f]

et rod, based upon the cross-sectional
r than the nominal body area of the

ading, see Appendix B, Sect. B3.
ress, F_v, for friction-type connections
reased to the applicable value given

on members have a fastener pattern
exceeds 50 inches, tabulated values

M steel specifications.
see Sect. 1.23.4.

1.5.3 Welds

Except as modified by the provisions of Sect. 1.7, welds shall be proportioned to meet the stress requirements given in Table 1.5.3.

TABLE 1.5.3

ALLOWABLE STRESS ON WELDS

Type of Weld and Stress[a]	Allowable Stress	Required Weld Strength Level[b,c]
Complete-Penetration Groove Welds		
Tension normal to effective area	Same as base metal	"Matching" weld metal must be used.
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Tension or compression parallel to axis of weld	Same as base metal	
Shear on effective area	0.30 × nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed 0.40 × yield stress of base metal	
Partial-Penetration Groove Welds[d]		
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Tension or compression parallel to axis of weld[e]	Same as base metal	
Shear parallel to axis of weld	0.30 × nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed 0.40 × yield stress of base metal	
Tension normal to effective area	0.30 × nominal tensile strength of weld metal (ksi), except tensile stress on base metal shall not exceed 0.60 × yield stress of base metal	
Fillet Welds		
Shear on effective area	0.30 × nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed 0.40 × yield stress of base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Tension or compression parallel to axis of weld[e]	Same as base metal	
Plug and Slot Welds		
Shear parallel to faying surfaces (on effective area)	0.30 × nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed 0.40 × yield stress of base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used.

[a] For definition of effective area, see Sect. 1.14.6.
[b] for "matching" weld metal, see Table 4.1.1, AWS D1.1-77.
[c] Weld metal one strength level stronger than "matching" weld metal will be permitted.
[d] See Sect. 1.10.8 for a limitation on use of partial-penetration groove welded joints.
[e] Fillet welds and partial-penetration groove welds joining the component elements of built-up members, such as flange-to-web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds.

~~ORIGINAL~~

UNITED STATES
NUCLEAR REGULATORY COMMISSION

IN THE MATTER OF: DOCKET NO:

INDEPENDENT ASSESSMENT PROGRAM -

COMANCHE PEAK STEAM ELECTRIC STATION

LOCATION: BETHESDA, MARYLAND PAGES: 1 - 119

DATE: THURSDAY, DECEMBER 20, 1984

ACE-FEDERAL REPORTERS, INC.

Official Reporters
444 North Capitol Street
Washington, D.C. 20001
(202) 347-3700

NATIONWIDE COVERAGE

1 separate editions of the AISC manual were employed by
2 different design organizations at Comanche Peak. General
3 Note 9 to our pipe support checklist on Phase 3 summarized
4 the examples which CYGNA found during our review of use of
5 the 7th and 8th editions of the AISC manual.
6 We found no design impact and in fact TUGCO later
7 issued a DCA, a design change authorization, which changed
8 their pipe support design specification MS46A to adopt both
9 of those editions.
10 Item 10, cable tray damping values. This was
11 particularly born out of the hearings which we participated
12 in. It may or may not be one of the issues on the
13 Walsh/Doyle list. The discussion at that point in time
14 centered around the use of welded structure damping values
15 from Reg. Guide 161 versus bolted.
16 CYGNA still stands behind its position that we
17 provided in response to Walsh Question No. 5 in our prefiled
18 testimony where we feel that the use of damping values for
19 bolted structures for cable trays as a system was perfectly
20 appropriate.
21 I have also down here some reference to the Phase
22 4 review. That's because cable tray supports and conduit
23 supports are specifically part of the Phase 4 review so
24 there will be some further documentation on their position
25 in that report.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

CASE'S ANSWER TO APPLICANTS' STATEMENT OF MATERIAL FACTS AS TO WHICH THERE IS NO GENUINE ISSUE REGARDING THE EFFECTS OF GAPS ON STRUCTURAL BEHAVIOR UNDER SEISMIC LOADING CONDITIONS

in the form of

AFFIDAVIT OF CASE WITNESS MARK WALSH

1. Applicants state:

"All bolts in multiple bolt, bearing-type connections will react imposed shear loads within at most the distance of the bolt hole tolerances. (Iotti, Finneran Affidavit at 8 /1/.)"

I disagree with this statement (and with some of the statements in the Affidavit). At the point all bolts begin to react the shear loads (that is, when the last bolt will have received a 1 lb. shear load), the first bolt that has reacted the shear load may have the shear load of 1,000 lbs., and this may have exceeded the allowable shear capacity of the bolt. This is assuming that the first bolt that reacted the shear load has not failed when the last bolt begins to resist the shear load.

/1/ I believe that the actual citation should be at 4-5.

One of Applicants' primary arguments which is contained in the back-up Iotti/Finneran Affidavit is regarding the definition of "oversized" bolt holes. This is addressed in detail in answer 2. following.

One example of statements with which I disagree which is made in the Affidavit of Dr. Iotti and Mr. Finneran is found on page 5 of the Affidavit (last paragraph, continuing on page 6), wherein they cite "Structural Design Guide to AISC Specifications for Buildings," by Paul F. Rice and Edward S. Hoffman (Attachment B to Affidavit).

Although the one page (268) from that document which Applicants have attached appears to be accurate /2/, their discussion and the portion cited (which is out of context) are very misleading. (See Attachment A, pages 264 through 271 of the Rice/Hoffman text.) In the portion attached by Applicants, Messrs. Rice and Hoffman are only talking about connections that receive static loads (i.e., loads that do not change direction) because the yielding stress criteria is not applicable in friction-type connections, as will be discussed later. The connections referenced by Messrs. Rice and Hoffman specifically exclude the supports that have dynamic loads (as will be shown below) such as most of those supports at CPSES. In addition, the inelastic deformation in bearing-type connections is recognized by the AISC Code at 1.5.2.2, where the allowable bearing stress is 1.35 Fy.

Therefore, the Applicants' statements are lacking reference to the

/2/ It should be noted that I have not reviewed the entire text of the other reference cited by Applicants, "Plastic Design of Steel Frames," and cannot state whether or not it is taken out of context.

specific amount of inelastic deformation allowed by the AISC Code for a non-dynamically loaded structure.

On pages 265-266 of the Rice/Hoffman text, following a discussion of AISC and ASTM specifications, it is stated (the numbers Messrs. Rice and Hoffman have placed in parentheses refer to sections from AISC or ASTM):

> "The use of ordinary (A307) bolts is limited by a number of Specification requirements. The allowable stresses are low: tension Ft = 20 ksi on the threaded area; and shear Fv - 10 ksi (1.5.2.1). The slip before full bearing is achieved on a group of ordinary bolts effectively rules out the sharing of stress in a mixed connection. Holes are to be taken as 1/16 in. larger than the nominal diameter (1.23.4), and the ordinary bolt does not expand to fill out the hole like a driven rivet nor can it be used for dependable friction. Stress sharing may not be assumed between ordinary bolts and rivets or welds (1.15.10; 1.15.11). In addition, low-strength bolts are not permitted in important field connections including . . . connections subject to **vibration**, impact, or **stress reversal** (1.15.12) . . . " (Emphases added.)

As discussed above, according to Messrs. Rice and Hoffman (Applicants' own chosen authority), A307 bolts are not permitted in connections subject to vibration, such as those at Comanche Peak.

Applicants have admitted that the connections at Comanche Peak are subject to vibration. Applicants' witness Finneran stated, in regard to the support which Jack Doyle and I noticed that had failed during hydrotesting (Tr. 4793/15-4794/4):

> "Q: (By Mr. Reynolds) Would you render an opinion on why the paint that Mr. Doyle talked about may have flaked during the flow of fluid through the pipe?
>
> "BY WITNESS FINNERAN:
> "A. I would say that possibly vibration may have been a cause for paint coming off of the deformed area during flow of fluid during the pipe; one possible cause.
>
> "Q. Yes. So what you are saying is that it could have been that when the deformation was caused during construction, the paint cracked but remained on, and then when vibration occurred due to hydrostatic flow, the paint chipped off?

"BY WITNESS FINNERAN:
"A. It's a possibility. I couldn't say if that's exactly what happened." (Emphases added.)

And at Tr. 5002/24-5003/9, Mr. Finneran further testified:

"Q: (By Mr. Walsh) Is vibration a common occurrence at Comanche Peak?

"BY WITNESS FINNERAN:
"A. I think all piping systems that have fluids in them or flowing to them are possibly subject to some vibration.

"Q: How about other pipes, main steam? Will they have vibrating effects?

"BY WITNESS FINNERAN:
"A. Quite possibly there will be vibration in the main steam piping." (Emphases added.)

The ASME Code requires the Applicants to minimize vibration where it states:

"**NF-3112.2 Design Mechanical Loads.** . . . The requirements of (a), (b), and (c) below shall apply.

". . . (c) Component supports shall be designed to minimize vibration."

In addition, according to Messrs. Rice and Hoffman (Applicants' own chosen authority), Applicants are also barred from using A307 bolts because of stress reversal.

As indicated above, Messrs. Rice and Hoffman cited the AISC code (to which the Applicants are committed through Specification MS-46A), Section 1.15.12, which states, in part:

"**Field Connections**

"Rivets, high strength bolts or welds shall be used for the following connections:

. . . Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

"In all other cases field connections may be made with A307 bolts." (Emphases added.)

CASE requested, through discovery on the issue of Applicants' Motion for Summary Disposition regarding generic stiffnesses, the drawings which the Applicants used in their Motion. Of the 60 supports which the Applicants provided (I count 59, but this is immaterial to this point), 52 had reversible loads which is a reversal of stress on the supporting connection. (See Attachment B, the referenced 59 drawings.) On the drawings, the reversal of loads is shown in the block listing the loads and the direction of the load is indicated as + or - . Of the 7 supports which do not contain reversible loads (CT-1-013-006-S22S, CT-1-013-004-S32S, MS-1-001-002-C72S, MS-1-01-001-C72S, CT-1-013-002-C42S, CC-2-011-719-A53R, CT-1-013-011-S22R), 5 are spring cans and 2 are rigid-type supports. Based on this random sample, 88% of these supports require high strength bolts due to the requirement of a reversal of stress (load) /3/, according to the AISC Code (to which the Applicants are committed in design specification MS-46A).

It is also obvious from the preceding discussions that Applicants are in violation of ANSI N45.2.11, 3. DESIGN INPUT REQUIREMENTS, 3.2 Requirements, which states, in part:

> "The design input requirements should include the following where applicable:
>
> "(9) Mechanical requirements such as vibration, stress, shock and reaction forces." (Emphases added.)

/3/ Stress is equal to the load divided by the cross-sectional area of the item under consideration.

ATTACHMENT A

Structural Design Guide to AISC Specifications for Buildings

Paul F. Rice
Edward S. Hoffman

VNR
VAN NOSTRAND REINHOLD COMPANY
NEW YORK CINCINNATI ATLANTA DALLAS SAN FRANCISCO
LONDON TORONTO MELBOURNE

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ISBN 0-442-26904-8

This book is intended
tine designs with the s
steel and joist construc

Each new AISC Spe
language for safe stru
plastic design, each n
economy of material
design calculations.

The increasing comp
for design. Compute
however, and compute

This Guide does not
the Manual of Steel
mentary, shows how t
Manual. It converts s
to direct design. It
design.

Tables 3-1 and 3-2
AISC equations for a
beam section, or solu
about three minutes.

Specification require
tions have been assem
frames, composite co
based on structural s
cussed to aid the s
specifications.

It is assumed that u
as well as the terms a
requirements in the

5

CONNECTIONS

General

The latest AISC Specifications permit a wide variety of connections. The basic requirement, appropriate with the sophisticated combinations of different types of steel to be connected, different design requirements of connections, and different means of connections, is a performance requirement consistent with the overall development of the Specifications. This requirement states simply "... that the design of connections be consistent with the assumptions as to the type of construction...." (1.2). Each of the detailed requirements for the design of connections simply builds upon this basic requirement. By implicitly or explicitly requiring that the design of a particular type of connection be consistent with the design assumptions as to the type and amount of force to be transmitted, and rotation capacity (or rigidity) consistent with the rotation assumed necessary to develop the connection forces, the basic performance requirement is completed (1.2).

The Specifications explicitly recognize inelastic behavior in connections of members designed as elastic: "virtually unchanged" angles at the joints in rigid frames, "nonelastic" deformation of parts of connections in Type 2 and 3 construction, and "inelastic rotation" for wind connections with Type 2 construction (1.2). Elastic behavior in the connections of members under plastic design is implicitly recognized (2.1).

Scope

For the purposes of this Chapter, connections are most conveniently considered as classified on two bases; (1) materials used (rivets, bolts, pins, or welds), and (2) the assumed behavior of the connection (design requirements: rigid, semi-rigid, or plastic for moment; shear transmission only; tensile or compressive force only; or combinations). In addition to forming joints between two or more steel members or parts of members, connections are required to elements composed of other structural materials. For composite action with concrete elements not bonded by encasement, shear connections are required (1.11.1). Shear connections may utilize specially designed shear connectors or standard welded stud connectors (1.11.4; 1.4.6). For connection of steel column bases to transmit any direct tension or shear, anchor bolts are required (1.22).

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It is not intended in this chapter to duplicate the design aids, detail data, and examples in the *AISC Handbook*, Part 4, Connections. Equally, space limitations do not permit presentation of a wide range of examples to illustrate even the recently published research in this area. (See "Selected References".) Rather, the purpose here is limited to explanations and illustrations of all applicable Specification requirements that might be overlooked or troublesome in routine work. This aim will include indication of reasonable interpretations to resolve apparent conflicts or ambiguities in the Specifications, and extension of such interpretations where the Specifications seem to have omissions. It has been desirable to extend this aim somewhat in that design aids for bearing plate and base plate connections were included as well as an extension of concrete bearing connection design to cover an apparent gap between the AISC Specifications and the ACI Building Code. During the interim between preparation and publication, AISC specifications were revised to agree with the latest ACI Building Code.

Rivets, Pins, and Bolts

Rivets and Pins. The requirements for the use of rivets and pins were established many years ago and were in many AISC Specifications. Since most of the late research has been directed toward welded, and more recently high-strength bolted connections, there has been little change in the Specifications for the use of rivets. Familiarity with the requirements for, and a sharply reduced use of, rivets in building construction results in little need for interpretations of these Specifications. Rivets of Grades 1 and 2 are available under ASTM A502 (1.4.3). Allowable stresses (for tension and bearing only) are given in Table 1.5.2.1 (1.5.2.1). Net sections for tension members must be used (1.14.2); computed as prescribed (1.14.3); and allowance of $\frac{1}{16}$ in. made plus the nominal diameter of the rivet holes (1.14.5).

The use of pin connections, originally popular in truss construction, has declined in modern steel building, and is usually encountered only for very special situations requiring special design. The general requirements for the use of pins are brief and essentially unchanged from previous Specifications (1.14.6).

Perhaps the most used application of these Specification requirements today will be in alterations or additions to existing buildings in which rivets or pins were used. For new construction, bearing-type connections can not be assumed to share stress with welds (1.15.10). If used in combination, the welds must be designed for the entire stress. In strengthening existing construction, bearing connections can be assumed to carry the in-place loads, and the new welds designed only for the additional stress (1.15.10).

Bolts. Bolts may be classified by strength as (1) low, A307, for $F_t = 20$ ksi; and (2) high, A325, for $F_t = 40$ ksi, and A490, for $F_t = 54$ ksi . . . (1.5.2.1). The ordinary low strength (A307) bolts are usable only in bearing connections (1.5.2.2). The high strength bolts may be designed for either bearing or friction connections (1.5.2.1).

The use of ordinary (A307) bolts is limited by a number of Specification requirements. The allowable stresses are low: tension $F_t = 20$ ksi on the threaded area; and shear $F_v = 10$ ksi (1.5.2.1). The slip before full bearing is achieved on a group of ordinary bolts effectively rules out the sharing of stress in a mixed connection. Holes are to be taken as $\frac{1}{16}$ in. larger than the nominal diameter (1.23.4), and the ordinary bolt does not expand to fill out the hole like a driven rivet nor can it be used for dependable friction. Stress sharing may not be assumed between ordinary bolts and rivets or welds (1.15.10; 1.15.11). In addition, low-strength bolts are not permitted in important field connections including column splices in all buildings with $H \geqslant 200$ ft., and where width/height < 0.25; also where width/height < 0.40, for $H \geqslant 100$ ft.; beam-column or column-bracing connections

where $H > 125$ ft.; frames carrying cranes with more than five-ton capacity; and connections subject to vibration, impact, or stress reversal (1.15.12); nor for flange-to-web nor cover plate-to-flange connections of built-up girders (1.10.4).

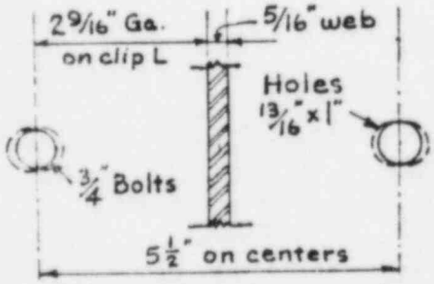
High strength bolts (1.16.1) and welds are considered essentially equivalent as connections, and, for friction-type joints assembled prior to the welding, the high-strength bolts may be assumed to share stress with welds in a mixed connection (1.15.10) or with rivets (1.15.11). Gross sections may be used for the design of compression members (1.14.2), and for the flanges of both built-up and rolled-shape girders provided the area of holes is equal to or less than fifteen percent of gross flange area (1.10.1). For tension members net section area is the basis of design (1.14.2). In friction-type joints resisting direct tension, the shear stress permitted with high-strength bolts must be reduced (1.6.3).

Slotted Holes for Bolted Shear Connections. The use of short-slotted holes is permitted under 1974 AISC Specification for "Structural Joints Using ASTM A-325 or A-490 Bolts," Section 3, subject to the approval of the designer. They can be used in either friction-type or bearing-type connections, provided a washer is installed over the hole.

The normal hole size for a $\frac{3}{4}''$ ϕ bolt is $\frac{13}{16}''$, whereas a short-slotted hole is $\frac{13}{16}''$ deep by $1''$ long (or $\frac{3}{16}''$ longer in the horizontal dimension). While the Specifications state that the hole can be either vertical or horizontal, the authors suggest only the horizontal slotted method be used. End clip holes only would be slotted, not the holes in the connection beam or column. See sketch, a full scale view of the end clip holes and bolt relationship for a typical $\frac{5}{16}''$ thick web.

The advantages to this system are many, several of which are:

1. Greater erection speed with less field burning of misaligned holes.
2. The use of one size clip angle with a set gauge will accommodate web thickness from $\frac{3}{16}''$ to $\frac{9}{16}''$.
3. The reduction in sizes of clips to fabricate and stock should help reduce costs.
4. The speed of erection (and elimination of mill web thickness tolerance problems) should help reduce cost.



Short-slotted holes layout for clip ∟s - shear connection

Welds

General. Full penetration groove-welds can be designed for full development, same stress as the base metal (1.5.2.1), by selection of the specified matching electrode and welding process (1.17.2). For all fillet, plug, and slot welds, and partial penetration groove-welds, reduced permissible stresses upon the effective throat area (1.14.7) are specified (1.5.2.1). In no case may the stresses exceed that for the base metal, or if different, the weaker base metal (1.5.2.1).

Special Considerations. A number of minor special considerations arise in the specification of welding. Generally, net sections are not a consideration except for plug and slot welds in which the gross area of the holes is deducted to check the fifteen percent maximum allowed (1.10.1; 1.14.3). The Specifications require preheating for various conditions, including all work when the temperatures are below 32°F (1.23.6). Except for single- and double-angle or similar minor members, welds are to be laid out to avoid eccentric axial force or such eccentricity must be considered in the design of the connection

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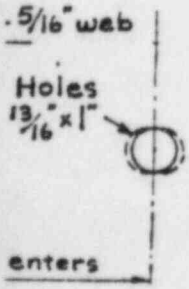

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and the member connected (1.15.3). For the usual shear connection requiring flexibility to accommodate the necessary simple-end rotations assumed, the locations of the welds and the selection of the connection elements must be coordinated (1.15.4).

As previously noted, where welding at high-strength bolted friction-type joints is required as a mixed connection with shared stress, the final tightening of the bolts is required prior to the welding. The *sequence* of completing purely welded connections is also important, though not explicitly covered by the Specifications (1.23.6). The heat generated in the operations of welding creates intense shrinkage strains which, if restrained, leave corresponding residual stresses (1.23.6). These stresses can be relieved by local inelastic yielding, but where local inelastic yielding is also restrained or limited, warping and lamellar tearing* may result. For many welded assemblies, the simple precaution of a specified *sequence* of welding may be employed to balance the strains and to avoid warping. Even after this precaution, certain complex assemblies may be expected to retain adverse residual stresses. For cases where this condition is anticipated or suspected, stress relief by heating must be specified by the Engineer (1.23.6). (Note: this service is not provided unless it has been specified and will normally be an added cost.)

The use of a proper *sequence* to avoid creation of shrinkage stresses or to minimize same can also be specified in many connections where lamellar tearing might occur. Particularly with thicker sections, where both the direction of the shrinkage is completely restrained and the resulting stress is normal to the surface of the section, consideration should be given to the welding sequence. If the condition can not be eliminated by a practicable sequence as a first choice for a solution, it may be possible to relieve the strains without developing large stresses by use of soft wire "cushions" or by revision of the entire connection detail. At least for simple cases it should, of course, be more economical to specify a particular welding sequence. (See Examples this chapter.)

Connection Design

Classification. In addition to the general requirements previously cited for the design of connections, certain arbitrary minimum design requirements for connections have been established. All connections for members carrying calculated stress must be "designed to support not less than six kips" (except lacing, sag bars, and girts), presumably six kips shear in flexural members, six kips tension in ties, and six kips bearing in compression members, all at allowable stress levels (1.15.1). Eccentric connections of axially loaded members are to be designed to transmit the resulting moments as well as the axial force (1.15.2). These minimum requirements naturally become most significant in the design of light members such as axially loaded members in trusses. Connections for such members are required to meet an additional requirement that they transmit the design load or the minimum six kips, whichever is larger, and develop at least half of the effective strength of the member (1.15.7). Note: joists are regarded as a special very limited-size truss in which the minimum connection capacity is simply specified as twice the design load for open web steel joists (4.5); or for the longspan and deep longspan joists, as the design stress or half the allowable strength of the member (103.5b). (See Chapter 4: Joists; Examples.)

As noted previously, connection types may be classified on the basis of the connection method or the design function. Broadly, connections may be described as *flexil* (for the transmission of shear only, 1.15.4), or *rigid* (maintaining the angle between th nembers connected and transmitting full moment capacity of the most flexible element at the

*"Commentary on Highly Restrained Welded Connections," *Engineering Journal*, AISC, 10, No. 3, 1973.

joint as well as the shear, 1.15.5), or *semi-rigid* (transmitting a pre-determined fraction of the full moment capacity as a rigid joint and further loads in shear as a flexible joint with corresponding angle change to supply rotation for the additional loads, 1.15.5; 1.2).

Flexible Connections. "Flexible" connections are designed to transmit shear without exceeding allowable unit stresses on the connectors as a group or the connection as a whole. The use of an average capacity for each of several connector elements sharing the total load is justified by allowing self-limiting localized stresses determined by an elastic joint analysis to exceed the yield point and create inelastic localized deformations of the connector materials, or by inelastic deformations of the connection elements (1.15.4). The simplest examples of localized deformation occur in the assembly of bearing-type bolted connections where the cumulative tolerances permitted exist on (1) out-of-round in the bolts, (2) oversize holes ($\frac{1}{16}''$), and (3) center-to-center location of the holes in the different elements connected. The extreme degree of such inelastic action occurs with a two-bolt bearing-type connection where one bolt is loosely fitted and one is very tight. Until the material of the connected element surrounding the loaded bolt or the bolt yields and deforms ($+\frac{1}{16}''$), the load is not shared and a 50 percent adjustment will be developed as the load increases. For larger (and thus more important) members, more bolts or rivets will be required and the degree of adjustment required on each will be less. Lesser adjustments are required for a long line of bolts or rivets intended to share stress equally. Even if perfectly fitted, yielding and inelastic deformations occur, maximum at and beginning at the first loaded bolt or rivet, and decreasing to a minimum at the last. (See Figs. 5-1

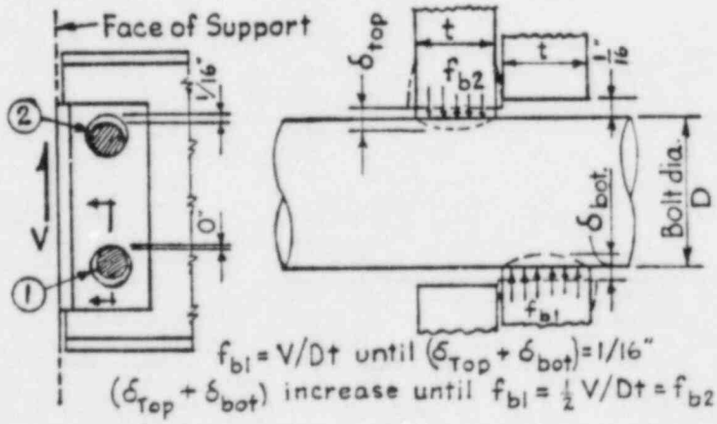
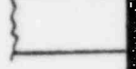


FIG. 5-1 Self-Limiting, Localized Deformations—Two Bolts.

and 5-2.) After this localized inelastic adjustment in the connectors for shear transmission, consider the inelastic adjustments that occur to reduce the "elastic theory" moments.

Inelastic deformation in the connection elements, typically angles, will occur and reduce the restraint which would transmit moment. The common double-angle shear bearing connection is extremely stiff longitudinally for the transmission of shear, and it depends upon the minor inelastic bearing deformations around each fastener to equalize the shear stresses in the fasteners. The same double-angle member is relatively flexible and will twist to permit a relatively large angular rotation reducing moment transmission. (See Fig. 5-3.)

Experience and tests confirm the practical assumptions of shear transfer only and the



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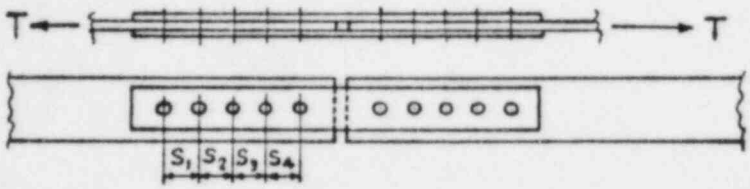

Inelastic deformations occur successively in the plates at each fastener and in the fasteners, beginning and largest at the first loaded, and continuing until the elastic strains in the spaces s_1, s_2, s_3, and s_4 become proportional to equal stress in all fasteners.

FIG. 5-2 Self-Limiting Deformations—Axial Stress on Line of Separate Fasteners.

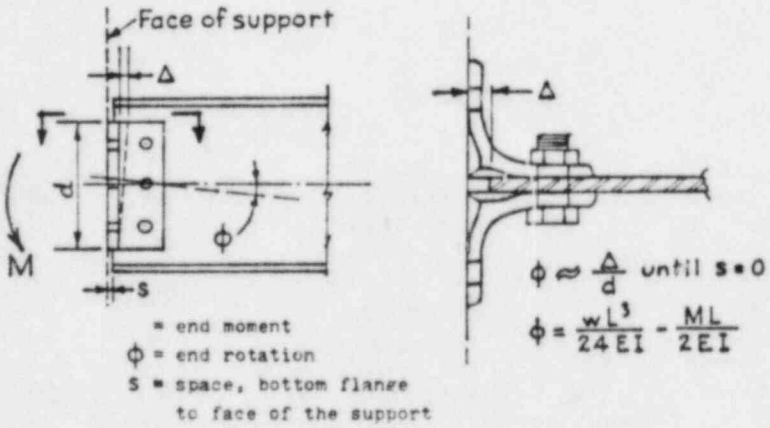

FIG. 5-3 Self-Limiting Deformation (Twist) in the Connection Elements (⊔⊔) Two Angles.

use of an average shear stress per unit weld or separate fastener. The actual rotations observed in one series range from 0.84 to 0.97 times Φ_0, the "simple beam rotation." These limits corresponding to moments ranging from three to sixteen percent were approximately the same for a single end plate connector or the common double-angle connector.[1] Figure 5-4 presents the usual device for an approximate analysis. An additional caution reported from these tests is that the moment stiffness increases abruptly when the space "*s*" (see Fig. 5-3) closes and the lower flange transmits compression to the face of the support. Coping the bottom flange where a quick analysis of the proportions of depth and required angle change show the usual clearance to be inadequate may be desirable for deep connections.

For Type 2 construction (flexible connections) all the reactions should be shown on the design drawings; alternatively, only those exceeding one half the tabulated uniform load capacity for the sections used, together with a general note that connections shall be designed for one half the capacity unless otherwise noted, should be shown.

[1] "Moment-Rotation Characteristics of Shear Connections," Kennedy, October, 1969, 6, No. 4, *Engineering Journal*, AISC.

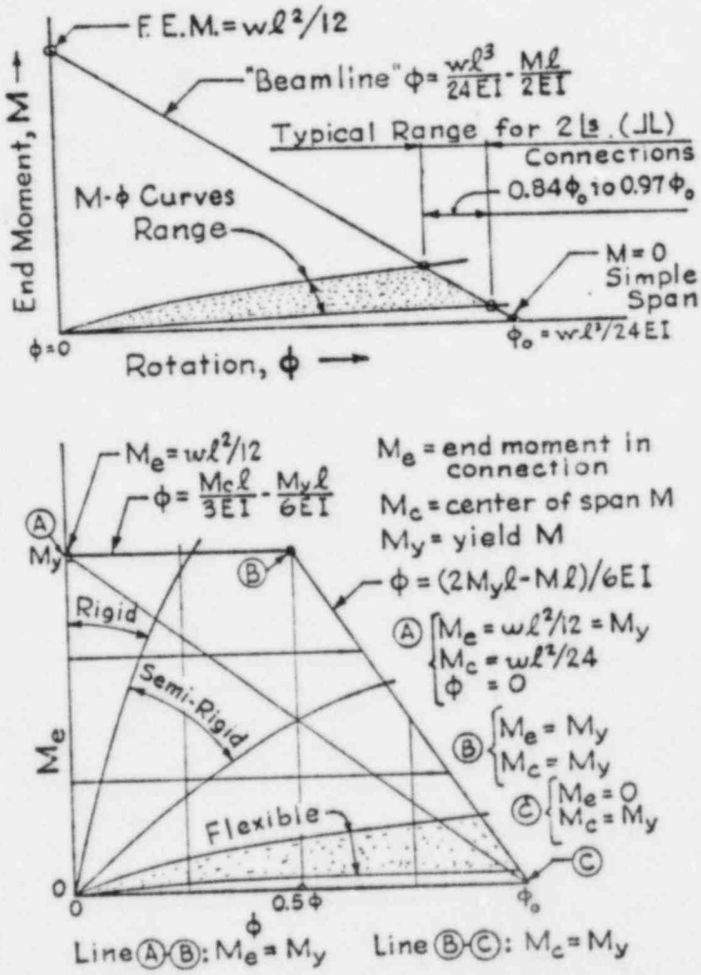

FIG 5-4 Uniformly Loaded Elastic Beam Line—Rotations at Connection.

Rigid Connections (Type 1 Construction). The AISC Specifications requirement is quite realistic: that rigid connections hold the original angles "virtually" unchanged (1.2). This requirement in elastic design is usually satisfied by connections designed to develop the full section of the flexural member or the full moment at yielding of the more flexible member connected. It will be noted from Fig. 5-4 that the rigid frame analysis ($\Phi = 0$ at the allowable stress) may be satisfied by such a connection which would have a very small rotation at 0.66 to $0.60F_y$, but would be capable of a significant rotation at yielding of the flexural member (line A-B).

A diagram similar to Fig. 5-4 but with point A representing an end moment, $M_e = M_p$, and point C, a center span moment, $M_c = M_p$, can be prepared for plastic design. Connections capable of achieving full collapse load (hinges at both ends and center of span) would be required to reach $\Phi = 0.5\Phi_0$, point B. The simpler concept of "plastic redesign," where only end hinges are required to form at the factored load, would require connections with a somewhat less rotation capacity, along line A-B. See Fig. 5-5.

Semi-Rigid Connections (Type 3 Construction). (See Fig. 5-4.) Ideally, the semi-rigid connections for Type 3 construction will behave elastically between the $\Phi = 0$ ordinate

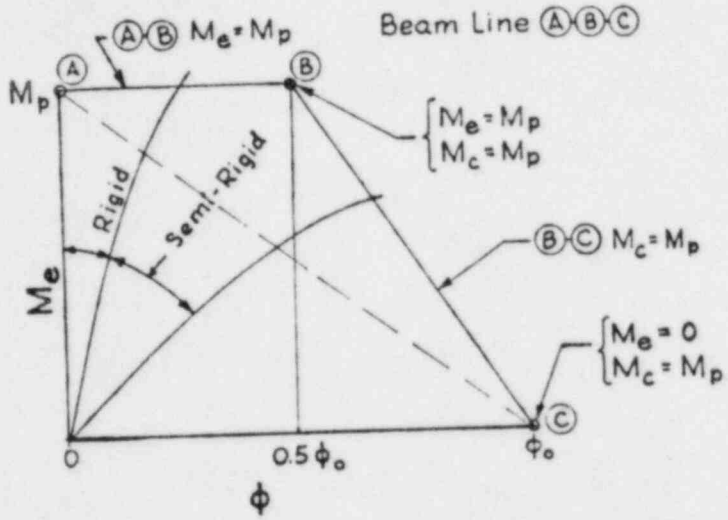

FIG. 5-5 Uniformly Loaded Plastic or Rotations at Connection.

and the "rigid" connection up to a predetermined end moment. Upon reaching this predetermined end moment (M_y for the connection), a rotation capacity sufficient to develop the yield moment at the center of the span, $M_c = M_y$, should be available. In practical cases where the nearest available rolled section will be above the design capacity required, the excess capacity will be provided at the midspan. As in plastic redesign, end hinges only will form at the full design load. Since these hinges are designed for $M_e = M_y$, more rotation capacity is required (to cross line B-C).

Masonry Bearing Connections

General. The AISC Specifications provide very conservative allowable stresses for bearing on masonry and concrete which apply in the absence of Code (statutory Building Code) regulations (1.5.5). For all masonry laid up in mortar, most statutory codes also provide low allowable stresses. Usually, codes distinguish among solid masonry units, bricks or block, and hollow units as well as among different classes of mortar and masonry materials. Values so prescribed range in general from 50 psi to 400 psi. The AISC Specification is therefore seldom applicable since it includes only stone masonry, $F_p = 0.400$ ksi, and brick laid in "cement" mortar,* $F_p = 0.250$ ksi (1.5.5). *The authors recommend use of the masonry bearing values prescribed in local Codes or those recommended by national associations dealing with masonry products.* For bearing stresses on concrete, the AISC Specifications, $F_p = 0.35f'_c$ on the full area and $F_p = 0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$ on fractions of the area, utilize recent ACI Building Code (ACI 318-71) refinements for economy.** The ACI Building Code is of course usually applicable under local statutory codes. *For beam-bearing plates and column base plates on concrete, the authors recommend the use of bearing values prescribed by the ACI Building Code.*

Beam Bearing Plates. The approved design (Chapter 2, pp. 82–83, *AISC Handbook*) for beam bearing plates is the formula:

$$t = \sqrt{3f_p(n)^2/F_b}$$

(*Continued on page 274*)

*All mortars utilize cementitious materials and the term "cement" can be quite properly applied to all.
**Supplement No. 3, 1974.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

AFFIDAVIT OF ROBERT C. IOTTI AND
JOHN C. FINNERAN, JR. REGARDING THE
LICENSING BOARD'S DECEMBER 18, 1984 MEMORANDUM

We, Robert C. Iotti and John C. Finneran, Jr., being first duly sworn, hereby depose and state as follows:

(Iotti) I am Vice President of Advanced Technology for Ebasco Services, Inc. A statement of my educational and professional qualifications was transmitted with Applicants' letter of May 16, 1984, to the Licensing Board in this proceeding.

(Finneran) I am the Pipe Support Engineer for the Pipe Support Engineering Group at 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 professional and educational qualifications was received into evidence as Applicants Exhibit 142B.

The purpose of this Affidavit is to provide information regarding various issues raised by the Licensing Board in its December 18, 1984, Memorandum, e.g., sampling of torques in cinched down U-bolts at CPSES and A36/A307 steel.

I. Sampling of Cinched Down U-bolts

While Applicants committed to retorque every cinched down U-bolt on single struts and snubbers to alleviate any safety questions about this issue, it was our view that the cinched down U-bolts in the field would not likely have excessive preload and would have been able to support the necessary loads. The basis for our position was not the sampling, but rather (1) the relaxation characteristic of the material used which would provide a reasonable upper bound on U-bolt preload values and (2) the characteristic of U-bolts at very low preload values to carry loads even if not stable in the truest sense.[1] In short, the torque sampling at issue here was not reported to demonstrate that work in the field was acceptable or that no additional work needed to be done. In this regard, given the decision to retorque, the results of the testing program and finite element analyses were ultimately used only to determine and provide assurance of the acceptability of the torque values to be used in retorquing. In retrospect, these results could have been obtained without taking a sample. In sum, for the ultimate use of the testing program and finite element analyses, the adequacy of the sampling is moot.

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1 While we did not attempt to define by test or finite element analysis an absolute minimum level of preload necessary to carry load, it was our judgment that even at a very low preload value a U-bolt would be capable of supporting the necessary load. Instead of attempting to confirm this by test, Applicants opted to retorque the affected U-bolts, as noted above.

We maintain that reporting and using the average torque values in the sample data was appropriate because of the following:

1) the forces on the pipe and the resulting local stresses would be highest at the cross piece to pipe interface and these are determined by the combined tension in the two legs of the U-bolt, i.e., the local pipe stresses are not affected by asymmetry in the leg torques;

2) the "stability" of the configurations depends on the overall frictional resistance at the pipe cross piece and pipe U-bolt interface and this overall resistance is not affected by asymmetry in the leg torques; and

3) the difference in torques between the two legs that can exist in the field is limited and there would be a tendency for any difference to be equalized when the assembly is exposed to vibratory motion (such as due to normal vibration or earthquakes).[2]

We have supervised the review of construction packages associated with the sampled U-bolts and have identified 43 of the appoximately 160 U-bolts sampled which were torqued prior to October 8, 1982 and not retorqued prior to the sample.[3] See

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2 The limitation in difference occurs because the difference in leg tension must be counterbalanced by the frictional resistance at the cross piece to pipe interface to satisfy moment equalibrium. External loads of a vibratory nature momentarily reduce at each cycle the normal force and hence the frictional resistance at the cross piece to pipe interface. This reduction, in turn, reduces the difference between the unequal tension in the two U-bolt legs.

3 The construction packages of 122 of the approximately 160 U-bolts sampled were reviewed. (We limited our review to construction packages which were readily retrievable.) The date checked in each package was the date of final QC acceptance of the associated support's initial installation. (In that QC acceptance occurs after a U-bolt is torqued, other U-bolts sampled may have been torqued prior to October 8, 1982 and not inspected until after this date.) Further, the packages associated with the U-bolts torqued prior to October 8, 1982 were checked to assure that subsequent modifications had not caused the U-bolts to be retorqued.

Table 1 which notes which construction packages were reviewed and which were torqued prior to October 8, 1982. (Installation of pipe supports in Unit 2 began in late 1977 and continued concurrent with pipe support installation in Unit 1 until late 1983 when virtually all of the Unit 1 pipe support installation was completed.) The torque values of such U-bolts are basically in the same range as those torqued after this date. Accordingly, there is no merit to the position that construction practices regarding torquing U-bolts were different for Unit 1 and Unit 2.[4]

II. Additional Information Regarding A36 and A307 Steels

In response to the Licensing Board's December 18, 1984, Memorandum (Reopening Discovery; Misleading Statement) we have reviewed previous testimony by Applicants in this proceeding to identify instances where Applicants discussed the relationship between A36 and A307 steels. We have identified three instances where Applicants discussed directly or indirectly the relationship between A36 and A307 steels, including the statement referenced by the Board in its Memorandum.[5] We agree that to the

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4 Based upon discussions with crew foremen, many of the same crews that torqued Unit 2 U-bolts also torqued Unit 1 U-bolts. In this regard, the vast majority of the construction packages of sampled U-bolts reviewed stated the construction foremen whose crews installed the support (and, by practice, torqued the U-bolts). Of the 45 construction foremen mentioned in the 122 construction packages reviewed, 28 still remained at CPSES. All but 3 of the 28 foremen stated that their installation crews worked in both Unit 1 and Unit 2.

5 See (1) Affidavit accompanying Applicants' motion for (footnote continued)

extent those previous statements imply that A36 and A307 steels are identical materials, they are inconsistent with our statements in our December 5, 1984, Response.[6] Accordingly, we clarify those comments below and address what we believe is the reason for that inconsistency.

First, however, we apologize for this inconsistency. At the time we prepared our Response we did not recall these previous statements. If we had, we would have addressed the matter in our Response. We trust that the additional information provided below will satisfactorily clarify the record. We think it is important to note, however, that correcting this apparent inconsistency does not indicate that CASE's assertions which we were addressing are correct or that any conclusions we have drawn in our motions are incorrect.

As we explained in our Response, the specifications for the mechanical properties of A36 and A307 steels are different (Response at 2-3). We provided this information for the Board in our original response because the important consideration for determining representativeness is the relative mechanical

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(footnote continued from previous page)
summary disposition regarding cinching of U-bolts (June 22, 1984), at 5 n.3, (2) Affidavit accompanying Applicants' response to CASE's answer to Applicants' motion for summary disposition regarding the effect of gaps (October 26, 1984) at 8-9, and (3) Affidavit accompanying Applicants' motion for summary disposition regarding Richmond Inserts (June 2, 1984), at 43-44 and Attachment A.

6 Affidavit of Robert C. Iotti and John C. Finneran, Jr. accompanying "Applicants' Response to Board Memorandum (Information on Composition of A36 and A307 Steel)," December 5, 1984.

properties. The actual chemical makeup of the steels is not, itself, relevant to the issues involved. However, to supplement our response, with respect to the actual chemical composition of the steels we note that their chemical specifications are not the same. As can be seen in the attached ASTM specifications, there are several chemicals in A36 steel which are controlled. The chemical composition of A307 steel is, however, controlled only with respect to two chemicals, phosphorous and sulfur. Thus, the chemical compositions of these steels are not necessarily similar. The limitations on their composition are not, however, mutually exclusive and, in fact, a steel may actually satisfy both specifications. Therefore, it is not accurate to state without qualification that these steels are the same. Consequently, to the extent Applicants' previous comments in this proceeding implied otherwise, they should be modified. Nevertheless, as discussed below, in certain applications the steels are, in effect, equivalent because of additional limitations imposed by specification and design requirements.

In our further review of this matter we have concluded that our earlier comments on this topic were premised on a presumption, in our case held by ourselves and those working for us whom we consulted, that the materials were, indeed, equivalent. As we demonstrated below, for a number of reasons this presumption is valid for certain application of the steels.

In certain circumstances, by specification and/or ASME design requirements, steels designated as A36 or A307 must have the same mechnical properties and chemical composition. In these situations the steels must satisfy the more extensive A36 chemical and mechanical property limitations, even if they are specified as an A307 steel.[7] Specifically, Section 1.3 of the ASTM Specification[8] for A307 steel (Attachment A) requires that for nonheaded anchor bolts, used for structural anchorage purposes, the material shall conform to the A36 specification. (Applicants use nonheaded bolts in their Richmond Inserts but order them as A36 in the first instance.) In addition, ASME provisions governing bolting material properties require that SA307 steel used for this purpose satisfy the chemical and mechanical requirements for SA36 steel. To illustrate, we have attached applicable portions of ASME Code Cases 1644-1, 1644-4 and N-249 (which superceded 1644) (see Attachments E, F and G). The tables establishing yield strength values expressly provide that SA307 steels shall meet SA36 chemical and mechanical

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7 An exception to this situation arises with respect to A36 headed bolts used for anchorage purposes which are also required to conform to the A307 Specification (see Section 3, ASTM Specification for A36 (Attachment A)). (See our affidavit regarding U-bolt cinching, at 5, n. 3 (cited by the Board in its Memorandum, at 4).)

8 The ASTM specification is applicable to bolts used in Richmond Inserts. The ASME specifications for A36 and A307 (under which these steels are designated SA36 and SA307) are essentially equivalent to the ASTM specifications (Attachments C and D). The "A" and "SA" prefixes have also been used interchangeably in this proceeding.

requirements.[9] Thus, for design purposes, SA36 and SA307 bolts which are governed by these requirements may be considered "equivalent". In summary, the presumption that A36 and A307 steels are "equivalent" is based on restrictions imposed by both specifications and design requirements. Again, however, the steels are not the same but only equivalent in certain contexts.

We wish to reemphasize that Applicants use only A36 steel both in U-bolts and bolts used in Richmond inserts.[10] CASE's repeated assertions to the contrary are incorrect. We hope the additional information we have provided regarding both A36 and A307 will assist in clarifying the subject for the Board.

The Board commented in its Memorandum (at 5) that:

> Applicants' tests related to friction, stiffness, relaxation and creep, characteristics of steel that are not readily assertained from data on yield and tensile strength.

We are not certain what the Board's precise concern is from this comment. We surmise that the Board also is interested in information to demonstrate the representativeness of steels in the field with respect to these properties. Accordingly, we provide information below to respond to that concern.

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9 We recognize that there are differences in the manner in which requirements for different grades of A307 steel are established under the Code Cases. We do not address those differences here because they are not relevant to Applicants' practices at issue.

10 A single exception to the use of A36 steel in bolts used in Richmond Inserts was noted in our affidavit regarding Richmond Inserts (at 9).

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

CERTIFICATE OF SERVICE

By my signature below, I hereby certify that true and correct copies of CASE'S 4th Motion for Summary Disposition: to Disqualify the Use of SA307 and SA36 Threaded Parts

have been sent to the names listed below this 14th day of January, 1985, by: Express Mail where indicated by * and First Class Mail elsewhere.

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