UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

DOCKETED
USNRC

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

A11:11

AFFIDAVIT OF CASE WITNESS
JACK DOYLE

in Response to Applicants' Changes in Affidavits
Attached to Their 1984 Motions for Summary Disposition

A. HISTORY OF ALLEGATIONS

1. General Positions in the Hearings

The first point to be made is critical to the understanding of the problems involved in these hearings, and that point is: Unit 1 was complete and ready for fuel loading in the Spring of 1983 in the opinion of Applicants.

This point was driven home on the last day of the September 1982 hearings by the Applicants, with strong support from the NRC Staff. See: Tr. 5409, where NRC Staff counsel Mr. Mizuno made the point that the testimony was sufficient to address the concerns of Messrs. Doyle and Walsh; Tr. 5410-5411, where Applicants' counsel, Mr. Reynolds, commented that an initial decision could be made on the current record, with Mr. Mizuno concurring at Tr. 5411/11-15; and at Tr. 5412/4-7, Mr. Mizuno added that the Staff planned no further direct testimony on what have come to be called the

Walsh/Doyle allegations; finally, at Tr. 5416/11-25, Mr. Reynolds stated again that the record was sufficient. Applicants concluded this because they assumed that they had developed a strong position based on two factors: (1) the allegations submitted were without merit in their opinion out-of-hand; and (2) Applicants, among other factors, had relied on engineering judgement and what they alleged to be standard industry practice.

2. The NRC Staff's Position

The report of the NRC Staff's Special Inspection Team (SIT) position (Inspection Report 82-26/82-14, NRC Staff Exhibit 207, bound in preceding Tr. 6290), signed in February 1983, concurred in the Applicants' assessment of the plant and particularly CASE's allegations when, in the first paragraph on page 7 of the SIT Report, it states in general that of 19 areas of concern raised by CASE, 12 had no merit, 6 others were at least partially known to Applicants due to the Applicants' design review process and had been or were (at that time) being rectified. (As we all know, the NRC Staff reversed themselves at a later point on the fact that the six others mentioned above were at least partially known to Applicants due to their design review process.) One allegation relating to the bending of the bolt for Richmond inserts was in part confirmed, but the NRC added the caveat in the last sentence of paragraph 1 ". . . the Special Inspection Team considers the stresses involved are unlikely to lead to bolt failure."

The NRC Staff at this point (February 1983) and later during the May 1983 hearings argued vigorously in support of Applicants' position, and in fact committed themselves to the premise that CPSES was designed properly:

". . . the Special Inspection Team did not find any violations of any NRC regulations" (see SIT Report, page 2, at Results, item 1). SIT went one step further and stated that they had performed an inspection of 100 vendor certified supports for 15 design attributes which would be indicative of the problems alleged by Messrs. Walsh and Doyle. The purpose of this review was to determine whether design deficiencies had survived the Applicants' iterative design review process. The review did not disclose any discrepancies which would indicate a failure of Applicants' design verification program to identify and correct supports to assure compliance wih Applicants' design criteria (see SIT Report, page 58).

3. Overall Positions

From the above, there is no doubt regarding the positions of the various parties in these hearings. At that point in time, we had:

(a) The Applicants' position was that the plant was essentially completed with only the review process to be completed, which would result in only minor corrections, most of which were assumed to be paper corrections.

(b) The Staff's position was that the allegations were insignificant or without merit and the Applicants' design of the plant was in compliance with the intent of the Atomic Energy Act, the Codes, the law, and good engineering practice.

(c) CASE's position was that the facility had many design errors, calculational errors, and code noncompliances. In fact, the design was effectively indeterminate.

4. The status of the licensing hearings

At that point in time a controversy existed and that controversy was: CASE believed the plant was indeterminate vs. Applicants and NRC Staff assuming the plant was well designed and proceeding effectively. The hearing process would now progress to determine which position was correct.

5. My personal position

By the way of reiterating my position on the controversy as it stood at that point in time as well as as it stands today, the key words are "compliance" and "indeterminate." It has always been my position that the design calculations have been grossly deficient, which results in hardware that is indeterminate. That is, we don't know whether or not it will fail and, if so, which ones will fail or not. By failure, I have always meant fail to meet criteria or good engineering practice. For example, see Tr. pages 154 through 156 of the 3/23/85 meeting with the NRC Staff which was a feedback discussion where I stated that I know a structure can be overstressed without falling down; see also page 169 of same meeting; see also my affidavit attached to CASE's 10/6/84 First Motion for Summary Disposition, top of page 62. Beyond that, regardless of what methodology Applicants offer to justify such supports, whether it be exaggerated allowables, industry standards, or what other reduction factors are used, the fact remains that the codes and provisions of 10 CFR have been and are being violated.

6. Board Order

The Board on December 28, 1983, issued a Memorandum and Order (Quality Assurance for Design) which effectively indicated, in part, that the Applicants had failed to support their position that the supports in question were in compliance with 10 CFR and offered the Applicants a second bite at the apple to prove that the supports were designed properly.

7. The purpose of the Board Order

While the Board did make several suggestions, there was no purpose stated in the Board Order that would suggest that Applicants could rewrite standard engineering procedures, the codes, or the law to bend the designs into qualifying to criteria which did not previously exist, nor will these new-found avenues of justification apply to future facilities, nuclear or otherwise.

8. Applicants' choice for complying with Board Order

In answer to the Board's 12/28/83 Memorandum, the Applicants chose three approaches: First, they retained Cygna Energy Services to perform an independent assessment; second, the filing of a series of Motions for Summary Disposition; and third, Applicants committed to retain the services of an independent expert from the academic community to review the basic engineering principles to be addressed in the Plan and to provide testimony to the Board. /1/.

/1/ See: Applicants' 2/3/84 Plan to Respond to Memorandum and Order (Quality Assurance for Design); 2/24/84 Tr. pages 10,337/14-10,340/9 (with clarification by Judge Bloch regarding 10,340/3); 3/13/84 Supplement to Applicants' Plan to Respond to Memorandum and Order (Quality Assurance for Design); Licensing Board's 6/29/84 Memorandum and Order (Written-Filing Decisions, #1. . .); etc.

9. The purpose of Applicants' summary disposition motions

The purpose of Applicants' Motions for Summary Disposition was two-fold, and was stated by Applicants; for example, at page 10 of the 9/28/84 Applicants' Reply to CASE's Answer to Applicants' Motion for Summary Disposition Regarding Local Displacements and Stresses, the Applicants state: "The Board should find . . . that Applicants' practice is appropriate and based on sound engineering principles." The general concluding statement more often used by Applicants is as follows: "For the foregoing reasons, Applicants' Motion for Summary Disposition should be granted." The inference being again that the foregoing reasons indicate that the practice in the discussion is appropriate and based on sound engineering principles.

It must be pointed out that since Applicants originally based qualification on assumptions without calculations, no sound basis existed. They are, by use of tests and extravagant mathematical procedures, trying to qualify after the fact while stating that this after-the-fact material was common knowledge all along (by inference), because anyone in their engineering department could write off the problem based on his engineering judgement, which would have required precise knowledge of the results for all of the tests and finite element programs which were forthcoming, including the highly theoretical exercise through academia utilized to qualify the upper lateral restraint beam that is designed so close to the razor's edge that the biological shield is jeopardized.

B. GENERAL DISCUSSION OF CONTENTS OF APPLICANTS' SUMMARY DISPOSITIONS

10. For this Board to find for Applicants, the evidence by Applicants would have to prove conclusively that under the most adverse loading conditions regardless of geometry of the support, no probability of overstress would exist within the guidance of applicable codes and laws, and indeed, this is what Applicants claim (see Applicants' Motion for Summary Disposition, for instance, Regarding Consideration of Local Displacements and Stresses where, at page 4, they state: "Applicants' pipe support designs fully satisfy applicable stress allowables even when these effects are included in the design.").

As discussed in detail later herein, in their Summary Dispositions, and most importantly, in their latest affidavit to correct those Summary Dispositions /2/, Applicants constantly refer to the fact that while an item does not conform to the allowable loads, it is of no significance since the item will still perform its intended function. On this point, Applicants are not only wrong, they are in violation of 10 CFR Part 50, Appendix B, III. Design Control, the first paragraph of which states:

> "Measures shall be established to assure that applicable regulatory requirements and the design basis, as defined in paragraph 50.2 and as specified in the license application, for those structures, systems, and components to which this appendix applies are correctly translated into specifications, drawings, procedures, and instructions."

/2/ 11/12/85 Affidavit of John C. Finneran, Jr. and Robert C. Iotti Regarding Corrections and Clarifications to Affidavits Supporting Motions for Summary Disposition of Pipe Support Design Allegations

At 10 CFR 50.2 it states, in part, at (u):

"'Design bases' means that information which identifies the specific function to be performed by a structure, system, or component of a facility, and the specific values or ranges of values chosen for controlling parameters as reference bounds for design."

In addition, when one wishes to utilize load rating as a means of improving the posture as relates to allowables, it is permissible to resort to testing and use the load rating provisions of the ASME code; however, ASME, ASTM and other procedures must be utilized to determine the range of allowables relative to the loading conditions under consideration.

For example, see ASME Section III, Sub-section NF, paragraph 3260 Design by Load Rating, et seq., wherein the procedures are established for determining allowables based on test loads vs. the load service cases. It will be noted from the several paragraphs involved that neither load ratings nor indeed any other procedure which has been codified allows the use of ultimate results of testing as a caveat for negating the exceeding of allowables at the various service conditions.

The facts are that under load rating, the test load at termination of the test will be subject to factoring as follows:

(1) A reduction of the test load of 10% in the event that the number of tests is not statistically significant (see NF 3261).

(2) A further factor is required relative to the tolerance found in standard manufactured elements and that is the ratio of the nominal physical dimension of the parts as sold to the actual dimension as tested.

(3) The multiplication of these factors times the test load will result in an approved working ultimate load.

It must be noted that this value is an ultimate, just as was the guaranteed ultimate by the manufacturer or the mill prior to the test. To determine allowables for the various types of items under the various service loads, this test load (ultimate) must be factored as noted in NF 3262.2 for plates and shells, NF 3262.3 for linear type supports, and NF 3262.4 for component standard supports. From compliance with these three sections of ASME, one would now have allowable loads for the four cases involved in the design of a nuclear power plant; that is, Case A, normal; Case B, upset; Case C, emergency; and Case D, faulted.

In the case of the U-bolts, for example, if we assume for now that the dimensions of the tested bolts were nominal dimensions, the portion of the test load available for allowable load at Levels A and B is about 20% of the test load. This follows because NF 3262.4 (7) and (8) sets the limits for Levels A and B at test load times S or Fa/Su where S = 12.6 ksi per Table 1-7.1 of ASME Appendix 1 and Su = 58 ksi. Including the 10% reduction for statistically indeterminate samples, the code allowable would equal .9 (12.6/58) x test load or about 20% of what the test load reveals. The fact that a test specimen may still be accepting loads at a stress level of 75 ksi has no bearing on what the functional capability of subsequent or in fact prior specimens are capable of sustaining until the test load is properly factored to arrive at allowable loads. For ASME citations, see Attachments B and C hereto.

Having complied with the above and in possession of the new allowables rationally and correctly obtained, these and only these allowables may be used to qualify the component of concern. What the level of ultimate load

would have been is irrelevant. In short, to qualify a support, one may only compare the actual stresses at a particular service load to the acceptable allowable for that service load. The Applicants have utilized testing as a means of qualifying questionable supports which were designed without analysis of certain attributes. The Applicants have never developed acceptable allowables for the type of supports and service loads of concern. Therefore, Applicants cannot use the range of the test load to indicate that while a particular item does not meet the allowable load under normal analytical procedures, it will still perform its intended function.

This criterion, allowables based on load levels, would be required because the Applicants perform no calculations to determine the adequacy relative to the condition alleged, but rather accepted qualification on the basis of engineering judgement and alleged industry practice. In this regard, in reference to engineering judgement, Cygna Energy Services' John Ward (the former Chief Executive Officer of Cygna) stated it best at Tr. 9395, when he defined engineering judgement as being limited to "where the effects are minor."

C. CONTENTS OF APPLICANTS' NEW AFFIDAVIT

11. Preliminary Statements

As to the contents of Applicants' new affidavit, I note a statement on page 3 that I must admit I find it hard to conceive of Applicants making in view of the remainder of the contents of Applicants' affidavit. The statement I refer to is as follows: "In fact, a large portion of this affidavit consists of material or expansion of material previously

transmitted to CASE and the Staff." The material referred to was, for the main part, the result of the NRC Staff's positions presented at the January, February, and March 23, 1985 meetings, the draft by Teledyne, and the latest position of Cygna (at that time). The fact that letters were transmitted is of minor significance. The latest position of Applicants in the record prior to the filing of this affidavit was that no problems existed and that any changes being made to suports during the reinspection were being done merely for expediency, and this fact was made public in the Dallas/Fort Worth metroplex newspapers by Applicants.

12. I next address the statement on page 4 of the affidavit, particularly starting at line 13, which states as follows:

> "We remain satisfied that the conclusions and opinions expressed in our affidavits were reasonable, and at most subject to differing professional opinions which would be expected in any highly technical field."

When engineering judgement or industry practice are used as a means of qualifying engineering which is for nuclear plant components, there is no margin for dissenting opinion. And by the way, this is also true for engineering of normal commercial buildings. The statement is therefore a contradiction of sound engineering fundamentals.

13. A statement on page 5 deserves comment, starting on line 15:

> "However, we recognize that other issues regarding the adequacy of the piping systems, and by definition the supports associated therewith, were raised subsequent to the filing of the motions which could have affected the analyses of the supports presented in the affidavit (e.g., by changes in loads, or direction and degree of piping movements)."

These facts were raised prior to the filing of the motions, in fact months before the motion of consequence was filed and was partially the result of the Cygna review insofar as mass participation/mass spacing were concerned, and by CASE as far as additional masses on the piping from supports themselves. Applicants were well aware of these and other questions for many months before the filing of the motions for summary disposition.

The advent of these questions was no shock to CASE since on many occasions I stated that because of the problems I found in the pipe supports, we had doubts concerning areas other than the pipe supports.

It must also be pointed out that the shortcomings discussed in this affidavit are based on specific allegations alone, not the collective effects which have always been a primary concern of CASE in these hearings. So, even though some of the supports may survive on a case-by-case basis analyzed against a specific allegation, they could still fail under the scrutiny of the collective effects.

14. The last sentence on page 5 deserves at least some consideration. Applicants' statement is as follows:

> "For these reasons the ultimate determination regarding the adequacy of piping and supports at Comanche Peak is best resolved in the context of the comprehensive Stone and Webster review, where all outstanding issues can be addressed cumulatively."

This statement is misleading because it presupposes that if Stone & Webster redesigns a support, the controversy over the original support is somehow negated. On the issue of the original allegations, I believe we are entitled to a determination on the merits of the two positions in

controversy, since Applicants' own CPRT Plan precludes any effective evaluation because the main thrust of Applicants' Plan involves the evaluation for "safety significant deficiencies" and by Applicants' thinking, items not conforming to codes, laws, regulations, etc., are not safety significant deficiencies so long as they can perform (in Applicants' opinion) their intended function (<u>see</u> Section B, pages 90 and 91 of 180, 3.4, VII.b.1, Item 2, of November 22, 1985, letter from Applicants' Mr. Counsil to NRC Staff's Mr. Noonan, Re: Response to NRC Staff Evaluation of the Comanche Peak Response Team Program Plan). While two questions were asked by the NRC, one in reference to significant design discrepancy and the other in reference to construction items, Applicants only answered relative to construction deficiencies and we must assume that they both have the same meaning in Applicants' view. ". . . is a deviation in construction of an item which, if uncorrected, would result in the loss of capability of the affected item, structure, or component to perform its intended function."

However, in reference to CPRT Attachment 3.0, Appendix E, Pages 2 and 3 of 19, the following definition appears at B.1.(e), which defines under DESIGN ADEQUACY:

> "'Safety-significant', for purposes of the CPRT Program [Footnote omitted], is defined to mean that the identified discrepancy, if uncorrected, would result in the loss of capability to the affected system, structure or component to perform its intended safety function. For purposes of the CPRT Program, credit is not allowed for redundancy at the component, system, train or structure level."

What Applicants are describing when they make these types of excuses is survivability at the faulted levels. If this were an acceptable procedure under the codes and the laws, there would be no necessity to check for

Levels A, B, or C. If it would survive faulted conditions (Level D), it obviously would at least survive any lesser condition.

In addition, the importance of the compliance with the codes was best stated by Mr. Terao of the NRC Staff; see quotation at page 4 of CASE's 2/4/85 Motion for Reconsideration of Licensing Board's 6/29/84 Memorandum and Order (Written-Filing Decisions, #1: Some AWS/ASME Issues), where he stated:

> "It may be a closed issue from the Cygna standpoint but we are still left with a violation of the code and a violation of the code is important in its own right, because it contributes to the worker's understanding of the extent to which codes are to be followed scrupulously and taken seriously." (Emphases added.)

It must be noted that 10 CFR 50.2, 10 CFR Part 50, Appendix B, ASME Section III, the AISC, not to mention Applicants' FSAR, all intended the purpose for any structure is to function within predictable limits depending on the load case under consideration (for example: load cases A/B, normal and upset; C, emergency; and D, faulted).

Therefore, first we must discuss the original allegations and the interpretation by the Licensing Board as to whether such allegations were with or without merit. Then, as a new phase in the procedure, we may discuss the merits of corrective actions by Stone & Webster.

To avoid a strike-out in these hearings, Applicants have rewritten the rules for the game and in effect have stacked the deck in their favor. They would now like to put blinders on the Board.

D. ANSWERS TO SPECIFIC MOTIONS FOR SUMMARY DISPOSITION

15. Stability of Pipe Supports

The first item discussed by Applicants (affidavit at page 7) is in reference to their motion on stability (see Affidavit of John C. Finneran, Jr. Regarding Stability of Pipe Supports (June 17, 1984)). Before moving forward, I must point out that the original instability problem which I described in my deposition/testimony (CASE Exhibits 669 and 669B -- see below) and during the September 1982 hearings involved both U-bolt and box structures attached to double-pin ended struts (see CASE Exhibit 669 at: page 95, line 25, through page 96, line 23; page 97, line 5, through page 101, line 15; page 103, line 15, through page 105; see also CASE Exhibit 669B: items 4I, 4J, 4M, 4N, 4P 4Q, 4S and 4T, among others). The Applicants finally took corrective action on this problem, without ever admitting, however, that the problem existed. So the problems being discussed in Applicants' affidavit are more relative to stability fixes than stability per se.

The Applicants, in quoting why the NRC did not accept Mr. Finneran's statement ("Affidavit at page 7" shown on page 8) only references the NRC comments as pages 33-36 and generally as follows: "The Staff stated that piping engineers should assure system stability by reviewing the piping and support configurations."

Of more consequence than an NRC desire is what was stated by the NRC Staff's Mr. Terao during the 3/23/85 meeting at Tr. page 25, lines 17-22, which was:

> "The Applicant referenced the ASME code, Subsection NF, Appendix XVII, Paragraph XVII-2221(a) which states, quote,

"'General stability shall be provided for the structure as a whole and for each compression element.'

"end quote."

This section of the code states that not only must a system be stable, each support must also be stable (not necessarily by itself but when considered as a compression element of the system). The main objection of the NRC Staff to the Applicants' stance involves the following statement which appears on page 31, line 24 through page 32, line 4, of the 3/23/85 meeting where Mr. Terao stated:

"The Staff's concerns stem from the fact that many of the pipe support designs at Comanche Peak represent either an unconventional application of the component standard supports which have not previously been proven to be acceptable, or the use of unconventional support designs."

In addition, the following statement offers some insight into why the NRC Staff has adopted its position (see pages 35 and 36 of 3/23/85 meeting):

"The Staff finds that unstable pipe support designs at Comanche Peak do not conform to standard industry practice; that is, the unstable designs are unconventional designs."

Then at lines 22-25 (same source):

"Thus, the Staff finds the Applicants' discussion of industry practice for stability and piping and pipe support designs is irrelevlant."

This statement is also not included in Applicants' affidavit. From page 44, lines 15-18, Mr. Terao continues:

"In fact, that was one of our conclusions, is that the design review required under ANSI N45.2.11 was really not sufficient to catch those kind (sic) of unstable characteristics."

Another quote by Mr. Terao which was overlooked by Applicants is from page 44, lines 19-23:

"It is very unique to Comanche Peak, and it's very difficult in this nuclear industry to have someone look at a support characteristic that no one else has ever looked at before. So it is a very difficult thing to catch."

On page 9 of the affidavit, Applicants again state that my concerns about stability have somehow been minimized because I didn't bring it to the attention of the specific members of Applicants' staff. I will say again, I brought my concerns up with my immediate supervisor (in fact, two of my immediate supervisors when you include his successor), another person who had at least some type of authority (a Mr. Kerliu, who if he was not responsible for individuals had at least some type of responsibility in the reviewing process). Beyond this, I brought it up with my supervisor's manager. With all the negative responses, I could see no use pursuing the matter all the way to the office of Mr. Spence, since normal protocol dictates that you do not go over your bosses' heads.

Moving on to the conclusion (affidavit at page 12), Applicants state:

"Applicants agree with the NRC Staff's assessment that there is not necessarily a safety concern regarding the stability of the various supports discussed in the affidavit."

What the NRC states is that only if the position of the clamp can be assured can they accept the analysis for the support; <u>see</u> affidavit at 7, last paragraph, where Applicants quote Mr. Terao:

". . . we are saying that if the support does not have a positive controlled clamping mechanism to assure that the support cannot slide or rotate along the pipes, then we have difficulty accepting the analysis for that support because of the uncertainties involved in that design."

It is not a carte blanche blessing of this unique design, which is, by its unapproved introduction, in violation of 10 CFR 50.34 (a)(2) and (8).

The stability allegation as originally structured, when considered in the light of the Cygna, NRC Staff, Teledyne, and CASE positions, was not in compliance with 10 CFR 50.34 (a)(2) or (8) which states, respectively, that Applicants' Preliminary Safety Analysis Report was to have contained:

> "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." (Emphases added.)

> "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 . . . "

Beyond this, the above provisions of 10 CFR 50.34 would obviously apply to any fix proposed for such novel, unique or unusual structure. This point will become more obvious below. While Applicants have never seen fit to concede that there was a problem with the original configuration for these supports, such lack of responsible action does not preclude the fact that the original design falls within the NRC classification of unconventional designs. In addition, I doubt if any person now involved in these hearings would suggest that the supports regarding which there were allegations in the 1982 hearings were anything other than non-supports.

16. U-bolts Acting as Two-Way Constraints

Applicants correct the statement in reference to gaps for U-bolts (page 12 of affidavit). This is a simple catalog item which required first grade

arithmetic to determine the proper dimensions. I am therefore amazed that it took the Applicants so long to detect their obvious error.

I also note that Applicants fail to cite from the 3/23/85 meeting; however, I shall make up for their lack of attention to small facts. At page 97, lines 3-6, of the 3/23/85 transcript, NRC Staff's Mr. Fair states:

> "The bottom line conclusion that I had out of this at this point is that the Applicants originally had no basis for making the assumption that these U-bolts provided no lateral support."

On page 13, lines 6-17, Applicants discuss thermal movement as the criteria for further consideration of the potential for lateral constraint. This factor was never considered by the engineer who designed the original support nor would this have been possible without knowing the potential tolerance due to installation which could result in as little as no clearance. For example, see NRC Staff's Mr. Fair at page 96, lines 7-13 of the 3/23/85 trancript where he concluded that Applicant is incorrect in assuming a gap existed, based on his personal measurements in the field. Proceeding on this premise that it has been proven that a zero gap was probable, all U-bolts of this type required evaluation by the analyst since two-way constraint has now been shown to have been a definite probable condition.

As a final thought on thermal movement, Applicants would have the Board believe that if the computer lists a thermal movement of 1/16", the movement is set in granite and will be no more or no less. The fact is the only time the thermal movement can be stated with any degree of accuracy is after start-up under the various combinations of thermal loadings by physically measuring the thermal movement. And even then ten years later you may not get the same readings under the same thermal conditions.

The fact that actual movements are an unknown quantity may be noted in Applicants' own affidavit at page 5, lines 18 through 23, where they state:

> ". . . which could have affected the analyses of the supports presented in the affidavits (e.g.., by changes in loads, or direction and degree of piping movements). (See draft Staff letter presented in the February 26, 1985, meeting, regarding need to include assessment of mass participation and mass point spacing effects.)"

Here we have Applicants admitting that the thermal movements and direction are dependent on the input and I am quite sure that Applicants would concede that all factors which affect thermal movement are not included in the computer input (building thermal movement, for example), and therefore one would be foolish to stake his life on either the direction or magnitude of thermal movements which are the result of the computer output. The computer merely indicates the expected movement relative to the parameters and geometry which are input, and in the case of CPSES (as mentioned above) does not compensate for building thermal movements. In other words, there is no thermal differential (building/pipe) movement considered.

Unfortunately, to complete my analysis of Applicants on stability, it is required to study Applicants' new position on cinched-up U-bolts which will be covered later when I discuss Applicants' position in the chronology established in their Affidavit.

On pages 14 and 15 of the Affidavit, Applicants found that with 1/32" gap for pipe sizes 3" and 4", the number of supports requiring further evaluation was raised from 8 to 22, which translates into 29% of the total number, which is 76 supports. And this number is using allowables established by Applicants' rules (with which I do not agree).

On page 16, second paragraph, Applicants discuss the fact that they learned of at least one U-bolt with no clearance, but claim that it doesn't matter since the analysis for that support assumed zero-inch clearance and therefore at worst the analysis was nonconservative.

However, Applicants forget that they only considered for reevaluation those supports which have a thermal plus seismic movement in excess of the theoretical gap. If there is no gap, then any thermal plus seismic movement affects the support. Additionally, the engineer originally assumed no load in the lateral direction with no concept of the magnitude of such load if zero-inch gap existed. This can only be determined by complex and time-consuming methodology of computer analysis, as Applicants have proven in their motion for summary disposition. It must be noted that for the support movements such determination has been made by computer.

For another maneuver by Applicants, see Applicants' affidavit at page 13, lines 13 through 16, where they stated:

> "Further, consistent with our original commitment, U-bolts in 3 and 4-inch lines where the thermal movement exceeded 1/32" should also have been candidates for removal."

I don't know how Applicants can explain how they determined that all U-bolts had a 1/16" clearance unless they are so set on justification that once they think they have the answer, no further investigation is attempted. On page 17 of their affidavit, last paragraph, Applicants stated:

> "It is evident from the results obtained in the analyses presented in the affidavit that some lateral loads are not much smaller than the rated load. They are, nevertheless, small enough that adequate margins of safety exist. . . the safety factor as determined by ITT Grinnell would range from 2.78 (emergency) to 3.70 (normal/upset). Further, even if allowance is made for the fact that actual material properties employed in the ITTG tests are higher than minimum material properties, the safety factors would still be in excess of 3.0 for normal/upset and 2.2 for emergency."

Applicants are not being explicit in this statement. The support is overloaded to the 1982 and indeed the current (a year after testing) manufacturer's advertised allowable.

The fact that these are much higher allowables than are standardly accepted may be noted in the 3/23/85 meeting, where NRC Staff's Mr. Fair at page 96, lines 2-6, stated:

> "Now, in order to make this conclusion, the Applicants had to do some actual physical testing of a couple of U-bolts to come up with a load rated allowable that was higher than the original manufacturer's allowable."

It must be recalled that the original acceptance of these supports was assumed to be based on the original lower allowables, not based on a clairvoyant knowledge by each of the designers that new higher allowables were forthcoming.

On page 18 of Applicants' affidavit, first paragraph, Applicants state as follows:

> "It should be noted that the U-bolts in the field had material properties greater than the minimum and comparable to the material used in the tests. . ."

This statement is beyond belief, since this is a commerical item which is warehoused, and the only justification that the material is above minimum can be based on the mill report which involves the properties of the particular heat. There are two problems with the mill report: One is that the mill report does not go with each U-bolt throughout its course from initial manufacturing to final installation. The user generally relies on the manufacturer's load data sheet for qualification of these off-the-shelf items. Therefore, the precise physical properties of the U-bolts which are

installed at Comanche Peak can only be stated to have material properties at least to the guaranteed limit. The second problem may be noted in the fact that the actual physical properties are not normally reflected in the mill report. Another point may be noted in the fact that the yield of the material as reflected in the mill report is not the lower yield value which is of interest to the engineer, but is rather the upper yield limit which results from high speed testing for production items. See, for example, CASE's Proposed Findings of Fact and Conclusions of Law (Walsh/Doyle Allegations), Section I, Page I-10, last full paragraph.

It must be known to Applicants that of the hundreds of thousands of U-bolts made by NPSI and ITT Grinnell, the variation in material properties and also physical properties covers a wide range and for any individual U-bolt precise properties are unknown. All that can be stated with any degree of accuracy is that the material properties are at least equal to the advertised minimums at the upper yield point and the physical properties due to mill tolerance is unknown. The only way to determine both properties precisely is to test for mechanical properties and determine dimensions by field measurements for each U-bolt in the plant.

The final paragraph on page 18 of Applicants' affidavit is a statement of an error in the strain listed in the original affidavit. It is appalling to note the errors incorporated in Applicants' original affidavit material which was submitted to convince the Board that Applicants don't make mistakes.

Applicants' conclusions are a contradiction of facts. The purpose of the affidavit accompanying the motion for summary disposition was to prove

to the Board that the use by Applicants' engineering staff of engineering judgement for determining consequences of lateral loads on U-bolts installed as one-way supports was acceptable. Instead, Applicants actually proved that if the original allowable listed by the manufacturer is used, a substantial number of this type of support would fail. And the only way to qualify these supports was to increase the allowable substantially and in addition neglect the effects of friction caused by pipe axial movement. And even at this, they couldn't get all of the supports to work.

In addition, the last statement in the conclusion by Applicants would indicate that at least one support will fail to qualify even at Applicants' generous allowables. However, Applicants are not supplying an answer to this question but are deferring to Stone & Webster which will, no doubt, change the support in the name of "expediency." The fact that this support is doomed is preordained, since the U-bolt must constrain an 18" diameter pipe against almost 5/16" (.305) displacement.

As shown above, for engineers to attempt to outguess a computer's output based on the same information available to the engineer and the computer is not only imprudent, it is outright stupid. To understand the preceding, a computer is essentially a high-speed moron; it can only manipulate the information input. The computer per se does not make errors; the computer operator may. But as we have seen in the past in these hearings, the engineers (particularly at CPSES) are highly susceptible to error.

Applicants, in their summary disposition and specifically in their current affidavit, have proven this by showing that by their own numbers

almost 30% of the supports of this type may fail to qualify as functional. The fact that Applicants identify questionable supports (to say the least) confirms CASE's position that failure to analyze these types of supports for this type of loading means that each support was in fact indeterminate.

17. Applicants on friction

Starting on page 19 of Applicants' affidavit, they attempt to justify the statement ". . . when friction and the normal load are combined the stress ratio actually drops from the .775 calculated for friction alone to .46." Applicants' main premise for assuming the correctness of their assumption is that the base plate is finished to bear. They base this on (again) what the code does not state (see Applicants' affidavit, page 20 at lines 2 through 15, and footnote 10). Applicants state (item (2) on page 20, lines 15 and 16) that this short member was saw cut leaving it plane. The question I have in reference to this statement is: plane what? saw cut?

Finally, on page 20 at item (4), Applicants state that weld spatter will fill in any gaps. To this, I state that relying on weld spatter which is literally absent in the work of qualified welders is a sad commentary on the limits Applicants will go to to salvage the unsalvagable. The AISC 8th Edition has this to say at 1.5.1.5.1: ". . . accurately sawed or cut to a true plane by any suitable means." The key word is "accurately," which is defined in the American Standard Dictionary, Second College Edition, Houghton & Mifflin Company of Boston, "accurately, adverb, 1. in exact

conformity to fact; errorless. 2. deviating only slightly or within acceptable limits from a standard." With this definition, a procedure must be established for those specific supports which require a finished to bear joint. Such joint cannot be created after the fact by decree. Also, Applicants do not state what provisions were provided to determine standard mill tolerances, for example, plate flatness, plate camber, etc. I also note in Applicants' statement that they again overlook the 3/23/85 transcript; however, I shall assist them. For this particular oversight, see NRC Staff's Mr. Fair at Tr. 78, lines 11-18:

> ". . . but the critical point in the specification is that you have to have finished-to-bear item in order to take credit for bearing stresses between the beam and plate.
>
> "And that specifically is the question I asked in the meeting a couple of meetings ago, whether they have any justification for that assumption and did they specify this joint as a finished bearing joint."

A finished bearing joint is precisely that: finished -- not assumed by the fact that a base plate is used, a column is saw cut, and a welder does a sloppy job. Finished means both joint preparation and welding are intended to create a flush matching joint.

Since Applicants had no documentation to produce a special joint condition, they cannot use the existence of such condition as justification for taking reductions which are specific only to such joints. Therefore, in this case, Applicants have shown that indeterminancy does in fact exist for friction loads which are not analyzed for supports at CPSES.

18. Applicants on section properties

While Applicants have no comments other than the one statement (pages 21 and 22 of affidavit) that the NRC Staff has questioned their assessment of flare bevel welds, they make no statement and instead refer the NRC question to Stone & Webster. In addition, this is not new information. CASE raised the same point years ago (see Tr. 6870, 6873, and 6874).

19. Applicants on AWS/ASME

On page 23, lines 11 through 13, of Applicants' affidavit, the following statement occurs:

> "As Mr. Terao noted in the June 8 and 20, 1984, meetings, those 'compensatory' requirements were deleted in the 1978 Winter Addenda."

On page 23, starting with the last word on line 13, Applicants make three statements, apparently related to the above statement, and then draw a conclusion. I can assess the conclusion only based on the contents of the three statements. In this case, however, there are various apparently intended meanings involved, depending on who is interpreting the contents.

For example, take the first of the three sentences, which states:

> "However, as Applicants indicated at those meetings, the large majority of Applicants' designs were accomplished prior to deletion of the 'compensatory' through thickness requirement from our design criteria."

The second statement is as follows:

> "In fact, we incorporated that portion of the 1978 Code Addenda in our design criteria in 1982."

The final statement reads:

> "As evidenced by Attachment 1 to Applicants' original affidavit, following deletion of that provision Applicants were assessing skewed fillet welds in a manner that satisfied both ASME and AWS provisions concerning effective throat assessment."

As is apparent, Applicants established three time frames: prior to 1978, after 1978, and a period in between 1978 and 1982. Now the first statement argues that the majority of Applicants' supports were designed in the period before deletion of the compensatory requirement from their design criteria. The statement could be read as meaning that the majority of the designs were completed prior to 1978; in this way, the NRC would be convinced that the designs were executed during a period when analysis included an extra conservatism. But that is not what Applicants have stated. What they have said was ". . . prior to deletion of the 'compensatory' through thickness requirement from <u>our design criteria</u>" (emphasis added). Since the only point at which they had "our design criteria" was at some point in 1982 (and there is no indication that this has ever been deleted), the statement could mean that most of the designs were completed prior to December 1985.

The conclusion which is based on all of the material from page 22, last two lines (starting "To clarify the use. . . ") through statement 3 (quoted above) on page 23, is as follows:

"Thus, the 'compensatory' measures were unnecessary."

As will be noted, the conclusion is not related to the contents of statements one or two, and is only vaguely related to the third statement, but only if statement one refers to the period prior to December 1985.

In any event, I can't waste any more time trying to figure out what Applicants' somewhat knowledgeables are attempting to say or avoid saying. The addition of two law firms apparently hasn't helped them to compose their affidavits.

Aside from the fact that we will address some of the points made on page 23 of the affidavit at a later date /3/ I have the following minimum comments, since we have covered the subject extensively in the past. But in particular, one statement made by Applicants cannot be allowed to pass without at least a cursory response. On page 23, last paragraph, through page 24, line 5, and footnote 12, Applicants include the following comments:

> "Applicants agree that proper consideration should be given to effective throats of skewed fillet welds. We believe we have done so. Although engineers may not always actually calculate the effective throat of each skewed fillet weld in the weld pattern being evaluated e.g., a conservative and simplified design approach was often used for efficiency, /12/ Applicants' practices were appropriate to satisfy Code weld stress requirements."

Quoting from footnote 12:

> "For example, the engineer may ignore the existence of an obtuse fillet weld altogether in calculating weld stresses."

On the one hand, we have Applicants stating that they believe that skewed fillet welds should be properly considered and they believe they have done so, but on the other hand they concede that the engineer may not calculate the weld at each fillet, although by inference he still calculates the weld per se. If the engineer must calculate the weld anyway, why doesn't he do it right? If one calculates an all-around weld with the AWS correction factors, the process is less of a problem than calculating a weld

---

/3/ For example, on page 23 starting with the last word on line 1 and continuing through the rest of the paragraph, the material contained therein is ambiguous to such a point that the conclusions as outlined in the final sentence of that paragraph are not possible.

which neglects the obtuse weld, thereby calculating a three-sided weld with eccentricity corrections. Beyond this, the failure to consider the welds as they actually exist creates a blind spot in two areas relative to the weld itself:

(1) The throat of the obtuse weld may be insufficient to take the shrinkage loads before cracking could occur due to minimum weld violations which could lead to catastrophic crack propagation. The fact that minimum weld violations escaped detection at CPSES is not news, as anyone is aware. For example, just examining the ANI documentation will enlighten anyone as to the potential problems in this area; <u>see</u> Attachment E hereto, CASE Exhibit No. 1,035. And this material is generally in relation to non-skewed welds which are far simpler to detect because factoring of the leg size as is necessary for skewed welds is not required for 90 degree fillet welds.

Applicants' lack of understanding of the serious nature of minimum weld violations is best noted in the exchange that took place during the January 10, 1985, Cygna/Staff/Applicants meeting (<u>see</u> page 5 of CASE's 2/4/85 Motion for Reconsideration of Licensing Board's 6/29/84 Memorandum and Order (Written-Filing Decisions, #1: Some AWS/ASME Issues), discussing Mr. Bush's statement in reference to wash passes for minimum welds:

> "That might make it worse, not better. Because the standard procedure is often to put a wash pass on and that doesn't accomplish much of anything, based on practical experience. I'm not talking, now, about precisely meeting the code."

This is followed by a jewel dropped by Mr. George of TUGCO,

who stated as follows:

> "We have done just that on a lot of welds that were supposedly quarter-inch fillet welds. QC put gauges on them and they come up with findings like those -- in fact 7/32 instead of one quarter; and the corrective action is we go in there and do just what you said. It's been done all over the plant."

To which Mr. Bush added:

> "I know it. At about $1500 a weld."

(2) The acute weld may have a depth over face ratio in excess of the AWS standard 1.4 to 1, thus creating the conditions for internal weld cracking which is not detectable by visual examination. For example, see NRC Staff's Mr. Collins at Tr. 12159. This could also lead to crack propagation.

Another point of interest may be noted on page 24 of the affidavit under "Affidavit at 6". Here Applicants note that 4 of the sample of 13 skewed fillet welds used in the original affidavit contained errors. How could such obvious errors have gone undetected?

Page 25 contains some interesting "what if's." But first a correction is required for Applicants' first statement that:

> "It should be noted that AWS punching shear analysis requirements were introduced to deal with large tubular structures (e.g., offshore platform supports) with relatively large flange width to flange thickness ratios."

The correct origin and purpose of the AWS punching shear requirement may best be learned by going to the source. In this case, the AWS itself at the Commentary, Part A, Section 10.1 (I quote from 1984 edition) stated:

> "Section 10 originally evolved from a background of practice and experience with fixed offshore platforms of welded tubular steels . . . The requirements of section 10 are intended to be generally applicable to a wide variety of tubular structures."

The Applicants must concede that the criteria evolved from and was not

developed for the offshore industry. Also, by the wording of the code, the intent of the usage is for a wide variety of tubular structures, not specifically offshore structures.

Beyond this, and still on page 25, Applicants disregarded the fact that the tube structures most often used in offshore platforms are round, and they commence to presume what ratios could exist if they use the full range of square and rectangular tube steel available in the 1978 edition of the AISC for their offshore rigs, which Applicants determined was a ratio of 64. (As a side thought, do the Applicants really believe that a tube steel with a 12" width and a 3/16" wall thickness is normally used on offshore drilling platforms?) The number Applicants developed from this premise is then compared to a ratio of 16, which is now stated by Applicants to be the largest known ratio to have been used at CPSES (after having originally claimed to the NRC Staff, until challenged by CASE, that 10 was the largest known ratio to have been used at CPSES).

However, Applicants are again in error, since they have made an improper comparison based on an erroneous ratio equal to 64. A ratio of 64 for any tube section in the 7th Edition (or, for that matter, the 8th Edition) is impossible since the maximum width of a member in the 7th Edition is 12" with a thickness of 3/16", resulting in a maximum possible ratio of 32; the maximum width of a member in the 8th Edition is 20" with a thickness of 5/16", still resulting in a maximum possible ratio of 32. In addition, these are available not only for offshore use, they are available for anyone. If one were to utilize Applicants' fallacious reasoning, CASE could then claim that ratios at CPSES have been as high as 32. For example, in just a brief examination of CASE Exhibit 669B (Attachment to Doyle

Deposition/Testimony, accepted at Tr. 3630) at Item 11XX, we note a TS 8x8x1/4 is used (the support which failed at hydro) which has a width over thickness ratio of 16; utilizing Applicants' reasoning, the ratio would be 32. At 14H, Applicants item No. 4 is a 10x6x1/2 which has a width over thickness ratio of 10; utilizing Applicants' reasoning, the ratio would be 20. At 13AA, Applicants' item No. 11 uses a 6x4x3/16 tube which has a width over thickness ratio of (again) 16; utilizing Applicants' reasoning, the ratio would be 32. This is only to mention three supports with width over thickness ratios which would be, utilizing Applicants' fallacious reasoning, greater than 16.

Note 13 on page 25 of Applicants' affidavit would indicate that at least one support failed to qualify to AWS local failure criteria. Here again, we have Applicants confirming CASE's position that unanalyzed supports indicate indeterminancy.

20. Generic Stiffness

The statement made by Applicants on page 27, under "Affidavit at 17", requires review:

> "The analyses relied on in the affidavit indicate that the original generic stiffness assumptions were adequate. However, subsequent analyses of other piping systems, performed in response to NRC questions after the affidavit was filed, indicate that some supports could experience, when actual stiffnesses are assumed, load increases which could cause the support to exceed allowable loads."

Applicants' sample was to prove that the original position of Applicants in reference to generic vs. actual stiffnesses was not a concern, and this sample was to show that since it produced good numbers all the other systems were O.K. However, Applicants mystically defied the laws of

statistics by analysing a system favorable to their own position. The purpose of Applicants' Motion for Summary Disposition was to prove that Applicants' use of engineering judgement was adequate to insure that the supports at CPSES were in compliance with the codes and the law. The purpose was not to show that in some cases it was indicated that Applicants lucked out. If Applicants believed the purpose was to show that engineering judgement works sometimes on a random basis, they have made their point. If they had anything else in mind, they were sadly mistaken.

The statement by Applicants on page 27, starting on line 17, "This does not mean that those supports would not be capable of performing their intended functions," is a contradiction in terms, since Applicants admit that a number of supports would exceed allowables. The intended function of the supports is to operate at the various Levels A, B, C, and D conditions within the allowable deflections and stress levels established for those conditions. It is not the intent of regulations, codes or the Atomic Energy Act to have components operating at Levels A, B, or C conditions to the stress or deflection levels established for Level D. By Applicants' own statement ". . . which could cause the support to exceed allowable loads," Applicants concur with CASE that failure to consider the stiffness of the support pipe system leads to an indeterminate condition for associated supports.

I wonder if Applicants, having now found the error of their ways, will now concede that the only way to really know what the actual loading of the support would be is by doing it on a case-by-case basis, which has been the position of CASE all along.

21. Effects of Gaps

The first reference in the material by Applicants on page 28 of the affidavit is to the AISC 8th Edition definition for hole sizes, which limits the size of standard holes and confirms that, regardless of the fact that such holes have not exceeded the maximum size for oversize holes per code, they are nonetheless oversized.

The second item noted by Applicants quotes from section 1.23.4.1 of the AISC code and is stated to be in reference to what the first item means or refers to. The affidavit states:

> ". . . this section of the AISC Code actually addresses steel to steel connections. This is apparent from paragraph 1.23.4.1, which specifically states that for the connections discussed in the affidavit, i.e., holes in baseplates for anchor bolts in concrete foundations ('column bases' in AISC terminology), even holes larger than those listed in the table may be used."

Applicants' game of semantics is once again in high gear. First, there is no part of AISC which was ever intended to address the tube steel/ Richmond insert method of attachments to structures, since this is a unique design. Applicants are trying to read into the code provisions those requirements which they require to qualify their structures.

Beyond this, the reference in the code to column bases is not AISC terminology for attachments to concrete generally but means precisely what it says: "column bases," which is a vertical load bearing (generally compression element) shape with a plate attached to the lower end to transfer structural loads to the foundation and footing. This cannot be interpreted to include beams attached to ceiling and wall.

To determine the truth of this statement, Applicants need only turn to pages 4-125 through 4-131 of the AISC 8th Edition (see Attachment A). The same information, with the exception of the oversize hole notes at the

bottom of the pages also appears in the 7th Edition of AISC on pages 4-104 through 4-109. This shows the details and the specific hole diameter oversized allowable for the column bases being discussed, and incidentally, on page 4-131 of the AISC 8th Edition shows a beam and alignment member integral with the concrete, and indicates the hole type required for adjustment as being slotted; and slotted holes are controlled as normal to the load, as shown in Attachment A, Sections 1.23.4.4 and .5. And incidentally, at Section 1.23.4.3 of the same citation, the following statement is made: "Oversized holes may be used in any or all plies of friction type connections, but they shall not be used in bearing type connections. And hardened washers shall be installed over oversized holes in an outer ply." (Emphasis added.) The same paragraph for the 7th Edition, located at 5-193 (3), Bolted Parts (1), Oversize Holes, states: ". . . they may be used in any or all plies of friction type connections. Hardened washers . . . " By omission from the above, oversize holes were not permitted in bearing type connections, even in the 7th Edition, since this section tells one where they may be used. Obviously, the connection in question is not in compliance with the code and all discussion relative to the size of the oversized hole is irrelevant, as indicated by this new code clarification.

On this particular problem, I gave a copy of J. Fisher's paper on shear keys and oversized hole situations for column base plates (which is the same information used by the code committee for AISC, of which Mr. Fisher is a member) to Mr. Bachman and Mr. Tereo of the NRC Staff over a year ago.

The code is referring to a totally different animal than exists with the Richmond/tube steel arrangement utilized at CPSES. The code is making

allowance for holes in column bases for specific reasons as follows:

(1) Shear transfer between column bases and foundations occurs through two mechanisms: first, there are normally huge vertical dead loads in columns, and thus friction will allow for a discrete horizontal shear to be transferred prior to engagement of the bolts; second, for larger shear loads which exceed the friction capacity of the joint, shear keys are employed.

(2) If the code referred to any steel member in contact with concrete, they would have so stated and not used the word "column." I am sure that even Applicants' experts have never seen a horizontal column, the key word being "column."

The Richmond/tube steel arrangement has many supports, perhaps most cases where there is no compression between the steel and the concrete, and in fact tension may be the usual condition. Therefore, Applicants' word games are worse than incorrect; they are ridiculous to say the least.

22. OBE v. SSE Conditions

In reference to page 30 of the affidavit, OBE vs. SSE conditions, I have several comments. First, Applicants state two facts as follows: First, that there was "Information received subsequent to filing the affidavit" which necessitated their clarifying their affidavit /4/.

/4/ It is hard to understand what Applicants would have us believe in reference to this statement, since Westinghouse was working on changes to their procedures for the use of damping factors. In fact, as may be seen in Applicants' affidavit at page 34, item (2), in May 1984 Westinghouse was making corrections of errors relative to damping factors at the same time that Applicants were preparing their affidavit. All Applicants had to do was check with Westinghouse to have known about such errors and thus avoid the problem of "information received subsequent to filing . . . "

And then at page 31, Applicants state, starting with the first full paragraph:

> "Regarding the different damping values employed by Westinghouse, at Applicants' request Westinghouse conducted a historical review of the stress problems within their scope of work."

Apparently Applicants went about formulating their motion for summary disposition without checking all the material they were using and without discussion with their vendors.

On page 34 of the affidavit, Applicants at (2) are discussing nine stress problems on 24" and 30" lines which used the wrong damping factors, and they state "This was identified and corrected in May of 1984 . . ." On this point, I would like to comment in two areas. First, Applicants were anticipating fuel loading in the spring of 1983 at the time of the September 1982 hearings (see above), and yet incorrect stress problems were still in place at least until May of 1984. Second, the problem of incorrect damping factors was brought to Applicants' attention long before this time frame; see CASE's 8/22/83 Proposed Findings of Fact and Conclusions of Law (Walsh/Doyle Allegations), Section XXII. Obviously, Applicants failed to monitor the problem.

At page 35, Applicants for at least the second time in the discussion on OBE vs. SSE offer the excuse that they have no problem since the material which was incorrect was (to quote Applicants) "not in final record analysis."

At page 30, Applicants make the following comments:

> "Information received subsequent to filing the affidavit indicates that in some initial and intermediate stress problem calculations, i.e., calculations performed prior to final record calculations, Applicants did not employ Regulatory Guide 1.61 damping values. The affidavit could be read to suggest that even for these analyses Applicants used Regulatory Guide 1.61 values."

Two points that I find curious: what part of codes or regulations allows you to produce less than somewhat correct calculations for non-record analyses? Second, were Applicants trying to suggest that this type of material could be offered to prove that there were no design problems? In reference to the above, the Board has already told Applicants to use due caution in the preparation of documents offered in these hearings in support of a specific point (see Board's 12/28/83 Memorandum and Order (Quality Assurance for Design), from page 52, line 3, through page 53, line 10). In rereading Applicants' original affidavit, I find no mention of "interim" or "intermediate."

At page 36, Applicants make the following comment:

> "While this is not necessarily incorrect, depending on the piping problem it may result in a response which is not appropriate, or at least less conservative than using the other approaches [footnote omitted]."

I love this statement. It shows imagination, if not the results of adding new legal staff assistance. Applicants' statement offers a mirror argument which I add because Applicants probably forgot to, and it would read thus: "While this methodology is incorrect, depending on the piping problem it may result in a response which is not appropriate, or at least less conservative than using the other approaches." This reminds me of the old cliche: is the glass half full or half empty? In this case, one might say that the Applicants are being optimistic, but not necessarily candid. Additionally, how can Applicants devise such clever sentence structure that within 20 words they can cancel the statement they just made that ". . . this is not necessarily incorrect . . . [but] it may result in a response which is not appropriate . . . "?

At the time CASE filed Proposed Findings of Fact on this matter, and indeed until the time Applicants filed their Motion for Summary Disposition, CASE stated that the damping factors were incorrect, and now apparently we have no argument from the Applicants, since they agree with CASE's position as shown above.

23. Differential Displacement of Large Frame Supports

Since Applicants have nothing of consequence to offer in this affidavit, I shall stand mute at this time and rely on our answers to their Motion.

24. Upper Lateral Restraint

On page 39 of their Affidavit, Applicants stated:

> ". . . I note, however, that the conclusion is correct for those analyses performed specifically to address CASE's concern with thermal expansion -- the two analyses performed to assess long term LOCA and MSLB effects, i.e., when the maximum thermal load on the beam and concrete walls occur."

Applicants are at least premature with this declaration, and this is borne out by the NRC Staff's "BNL Review of Texas Utilities Generating Company Comanche Peak Steam Electric Station Upper Lateral Restraint Beam - Steam Generator" (FOIA-85-59, C/364, Attachment D hereto); in the BNL Review of the calculations, it is stated (page 9, Conclusions, item 3):

> "BNL has investigated the effects of the ULRB axial loads developed during MSLB and it was found that significant shear and moment cracking will develop in the concrete compartment walls. It is reasonable to expect that significant cracking will occur both to the interal (sic) (i.e., reactor cavity walls as well as

at the outer wall of the 4th compartment. Some cracking is also expected on the outside compartment walls of the other three compartments."

Therefore, to make the upper lateral restraint beam qualify, Applicants have extended the capability of the beam and concrete to the extent that severe cracking takes place through the biological shield. So Applicants' excitement over the fact that they have beaten CASE may be shortlived, since the proof may be merely academic unless they can now prove that the damaged biological shield will not present a hazard to the public health and safety. See page 10 of the BNL Report, item 6, which states:

"No assessment was made with respect to the effect of the cracks on radiological shielding."

Beginning on page 38 of their affidavit, Applicants expend 7 pages of doubletalk to evade a major point. Applicants on several occasions referred to questionable procedures as relate to tension on the upper lateral restraint anchor bolts. On almost every occasion that Applicants mention this concern, they cite as an intervening factor that this problem was related to tension on the bolts and not the problems related to compression as noted by CASE. This insertion of caveats each time Applicants must cite an error is absurd. Applicants would do well to address the errors as they find them and refrain from qualifying their answers. In this way, Applicants will not wind up again compromising their credibility and CASE will not have to resort to vast expenditures (based on available resources) of time and personnel pointing out Applicants' errors and deceptions.

The main reason the Board offered Applicants a second bite at the apple was to prove that the supports were adequate to the codes and the law, not

to devise schemes to cover CASE's allegations while neglecting the fact that the structure suffers from other diseases.

Applicants' caveats notwithstanding, the above recently detected potentially severe error is notable for several reasons, none of which are covered in Applicants' response.

(1) As shown in the beginning of this affidavit, Applicants were certain that Unit 1 of the plant was completed and were demanding a license in September 1982 (in fact, earlier than this).

(2) The upper and lower lateral restraints are the main structural components which constrain the steam generators horizontally. Prevention of catastrophic failure of these members is critical to the safety of the plant and the public. The unique nature of the support system for the steam generator is such that the loss of the horizontal constraint results in vertical instability.

(3) Since the plant was (in the opinion of the Applicants) complete and the covering of this problem with the restraint anchorages would not have occurred if CASE had not proved that:

   (a) The original calculation did not address certain phenomena (LOCA, for example), and

   (b) When Applicants attempted to qualify this support for the hearings, they utilized limit analysis (plastic design) but their procedure was in error (<u>see</u> Tr. 6043 where Applicants' Mr. Vivirito conceded the error was 5%). At Tr. 6068, Mr. Vivirito admitted the error amounted to 600 kips (which is 300 tons of force). At Tr. 6191, NRC Staff's Dr. Chen agreed

that CASE allegations of error were correctly assessed. In using the correct numbers in Applicants' equations, the support exceeds the yield strength of the material.

(4) As part of the proof requested by the Board in the December 28, 1983, Memorandum, Applicants filed a Motion for Summary Disposition which addressed the upper lateral restraint. As a result, we now come to the third and most sophisticated attempt to qualify the upper lateral restraint. First Applicants produced a time history to reduce the loading used in the calculations and then Applicants resorted to a NASTRAN analysis to take advantage of the energy dissipation through a large portion of the containment. In addition, through a vast effort in theoretical engineering, Applicants have produced perhaps the world's first variable joint fixity condition; <u>see</u> BNL Review, Attachment D hereto, page 10, item 6, where they state the following:

> "The effects of the wall cracks will be to change the end fixity of the ULRB to that of simply supported."

Applicants have done this type of analysis for a simple beam and in all probability have set a world's record for sophistication in analysis of simple beams, because the stress for this type beam is always determined by the simple equation Mc/I plus or minus P/A. But this is the least of Applicants' problems, since in attempting to prove to the Board that the upper lateral restraint was qualified to codes, regulations, and FSAR, the Applicants found that there are questionable areas in the

analytical methodology. This type of problem never occurs using Mc/I plus or minus P/A for fixed-ended simple beams. Beyond this, what is the probability of survival for the ULRB as currently designed by Applicants vs. the same beam and loading conditions designed by conventional methods?

(5) Having noted the questionable areas of analysis which could affect the statements made by Applicants in their affidavit, we are not told what the effects will be, but are rather told that CPRT is looking into the problems and ramifications. My basic conclusion as relates to this problem is how can a simple beam be erroneously analyzed wrong not once, not twice, but on three separate occasions?

CASE originally stated that problems existed with the upper lateral restraint. While it is true that our reference basically was related to compression, it was the tenacity of CASE's efforts on the first calculation, the second calculation, and in fact the third calculation which ultimately led the applicants to find that the NRC also questioned the third calculation. Now Applicants concur that their calculation may be incorrect for the problems related to tension in the upper lateral restraint anchor bolts. Although Applicants fail to mention it, in addition to the problems discussed new problems related to the radiological protection have been induced. Therefore, the CASE allegation that the upper lateral restraint was indeterminate is in fact accurate (which certainly has been proved in the area of tension).

25. Cinched-Up U-bolts

On page 47 of the affidavit, Applicants state as follows:

> "In light of the definition of stability employed by the NRC Staff, i.e., motion of support into a position not analyzed or analyzable, and Applicants' decision to apply that standard in subsequent analyses, the conclusion -'thus behaving stably'- would no longer be consistent with the use of the Staff's definition."

For an organization to employ at a minimum four major legal firms, Applicants seem to have a problem expressing themselves. Beyond this, from the way I read their statement, it appears that Applicants are saying that while they disagree with the NRC definition of stability, they will do as the NRC requires.

Beyond this, by inference, Applicants seem to state that they have or do not have a stability problem based solely on the definition of the word "stability," which is obviously absurd since stability in engineering requires no definition to exist de facto nor to be recognizable by competent engineers. The definition was required for the hearings merely to explain for the record why a concern existed in the first place, not to indicate that the NRC Staff, Teledyne, and Cygna had simultaneously uncovered a new physical phenomenon. The phenomenon has always existed, but was rarely if ever institutionalized as it has been at CPSES. Therefore, a definition was not normally required in the past. However, since Applicants installed the unique design at CPSES, a method of describing it became a necessity. This is yet another example of Applicants' somewhat knowledgeable staff off on another tangent designed to confuse the issue. The above becomes apparent when one reads the statement by Mr. Terao of the NRC Staff at page 44 of the March 23, 1985, meeting in Texas:

". . . it's very difficult in this nuclear industry to have someone look at a support characteristic that no one else has ever looked at before."

Obviously, from the above statement, definition was required.

No one wishes Applicants to do anything other than qualify the supports. If the Applicants can show that the NRC, Cygna, Teledyne, and CASE are all incorrect, let them do so, because to disagree and have facts to back up this disagreement and proceed to rework the plant is somewhat less than prudent and in fact can only be seen as complete irresponsibility.

On page 49, Applicants make the following statement:

"In summary, we conclude that in general torque applications to the U-bolt pipe assemblies can potentially result in high but acceptable local pipe stresses and can further result in high stresses on the U-bolt. In some instances pipe stresses, calculated on an elastic bases (sic), may be unacceptably high even for torque values comparable to those required for stability. In such instances, the supports would be candidates for modification."

In addition, Applicants continue as follows:

"Applicants make these revisions because the analysis of individual supports performed subsequent to filing and in accordance with the commitment in the Affidavit, indicated that for a small number of supports (on large pipes with relatively thin schedule walls) the pipe stresses may exceed Applicants' acceptance criteria [footnote omitted] at torques predicted to be necessary for stability. This condition will be further addressed in the context of the Stone & Webster review."

On the above, I have a number of comments, but first I would like to comment on the structure of the above quote. It appears from the statement made by Applicants that in their original affidavit they had committed to an analysis of all of the individual supports, whereas in fact they had only committed to a program of retorquing.

It was the Applicants' purpose in filing their Motion for Summary Disposition to prove to the Board that consideration of the effects of cinching were not required, since no combination of geometry, piping, or loading conditions would result in any problems (see Applicants' Statement of Material Facts As to Which There Is No Genuine Issue Regarding Consideration of Cinching U-Bolts, page 7, lines 8 through 11, where Applicants state "(3) Stresses in piping due to preload values expected in the field in conjunction with other loads imposed will not result in any adverse impact." Further, at lines 23, continued to page 8, lines 1 and 2: "To provide further assurance of acceptable preload values, Applicants have committed to an inspection program to assure that every cinched-down U-bolt on a single strut or snubber (a total of 380) is torqued to a level at which the assembly will be stable in the absolute truest sense, i.e., no rotation and axial movement, if any, is towards the strut.")

In Attachment 3 to Applicants' Motion for Summary Disposition at page 5, last three lines, the following statement is contained: "The pipe will not have stresses that exceed acceptable limits if preload torquing is restricted to the recommended values." On page 63, last three lines (same source), it states as follows: "From the above discussion, it can be concluded that the pipe stresses induced in the pipe due to cinching of the U-bolt will not exceed acceptable limits." From Applicants' Motion for Summary Disposition, pages 4 and 5, the Applicants provide the following comments on the purpose of the Motion for Summary Disposition: ". . . provide evidence of the acceptability of stresses on pipes caused by thermal

expansion in local areas around cinch d U-bolts." And on page 5, lines 3-6 ". . . no genuine issue of material fact exists in respect to the allegations and the Board should find that the Applicants are entitled to a judgement as a matter of law." At lines 11-13, Applicants state "Applicants conducted a survey of a torque on a respresentative sample of cinched-down U-bolts. From the data Applicants established that to bound field conditions . . . " From the Affidavit of Messrs. Finneran and Iotti, the following comes to light: from their answer on page 2, the purpose of the Affidavit is outlined, and at page 3, item 4 "The local (and global if any) stresses induced in the piping by the cinching down practice . . . " The Applicants' answer to this question appears on page 3 at 3: "Provide evidence that the use of U-bolt cinching is appropriate to eliminate potential local instability without introducing adverse effects in the piping and U-bolt itself."

The evidence was compiled from tests and finite element analyses in reference to the tests from the original affidavit of Messrs. Finneran and Iotti; page 35, lines 16-18 contains this statement: "The test results indicate no unacceptable stresses in the pipes for the preload condition." The key statement in the affidavit in reference to the evidence compiled to support Applicants' position that no problems existed with the pipe supports or piping may be noted on page 51 of their original affidavit, lines 16 through the end of the page, which is as follows:

> "The bottom line, however, is that the testing program and finite element analysis have demonstrated that cinching of U-bolts as done at CPSES and generally by the industry produce no adverse effects on piping and supports for the range of pretorque values which are either representative of the worst conditions encountered at the plant or required to ensure stable behavior of the U-bolt assembly."

But on page 76, Applicants state: "However, as previously noted, Applicants will inspect every U-bolt on a single strut or snubber (a total of 380) and ensure that each U-bolt is torqued to the torque value set forth in Table P." Finally, at page 77, item 2, Response, Applicants state:

> "Stress results obtained from the finite element analysis of the U-bolt support piping assembly associated with the anticipated support and piping loads as well as recommended preload values are within acceptable limits. The supports as well as the pipe will not experience any gross distortion or loss of function."

From the above, several points are perfectly clear:

(a) Applicants had no reservations that the support type under discussion (cinched U-bolts) did not cause an overstressed condition in either the pipe or the U-bolt.

(b) The only commmitment made by Applicants was in relation to an inspection (not an analysis) program to ensure that the torque values recommended by Applicants were in fact existing in the field or the U-bolt would be retoruqed.

(c) All of Applicants' statements in all of their original Motions for Summary Disposition are now suspect as a result of the gross errors found in those Motions for Summary Disposition covered in this affidavit.

Applicants' November 12, 1985, affidavit contains several other points that deserve mention, as discussed below.

On page 50, last two lines and continued on page 51, Applicants make the following statements:

> "These changes are prompted by the fact the (sic) during the initial preparation of Table I, outside surface secondary circumferential stresses were subtracted inadvertently from the inside surface total circumferential stresses."

Are we to assume that the use of the word "initial" means that the summary disposition process is also an iterative procedure, or was this yet another case of failure to check once the initial result proves acceptable to Applicants? I have said it before and I will say it again: Is it mere coincidence that the vast majority of errors committed by Applicants favor their position?

At the bottom of page 51, in the Conclusion, Applicants state: "The conclusions set forth in the [original] affidavit remain valid." In view of all of the above, I shall refrain from comment on this point.

In reference to the material covered under cinched-up U-bolts, it must be noted that two of CASE's allegations are confirmed by the revelation in these corrections to Applicants' Motion for Summary Disposition: the cinched-up U-bolt problem being one, and the fact that in a number of cases, as pointed out by Applicants themselves, stability cannot be achieved without exceeding allowables. While we still disagree with the ludicrous allowables derived by Applicants, it must be borne in mind that the problems noted by Applicants are based on these super allowables.

26. Axial Restraints

First, let me state again that the purpose of Applicants' Motion for Summary Disposition was to prove to the Board that for all potential loading conditions and geometries there would be no supports of this specific type which would exceed allowables for each of the Motions for Summary Disposition as shown above.

On page 53 of the affidavit, Applicants have the following statements starting on line 5:

> "Accordingly, we no longer consider application of the ASME code provisions concerning self-limiting loads, as was done in the Affidavit, to be appropriate. Therefore, we have assessed the implications of not applying Code provisions applicable to self-limiting loads, i.e., use of stress allowables equal to three times the normal allowable, in the analyses of these restraints. We now conclude that although modelling the restraints in question as purely axial restraints (i.e., no rotational constraint) is adequate for assessing pipe global stresses, such modelling may not be adequate to assess the adequacy of the support itself or the local stress conditions in the trunnions at their interface with the pipe (i.e., weld to pipe and trunnion stresses) [footnote omitted]."

It must be mentioned in any event that Applicants' flagrant use of 3Sm to triple allowables is generally in error, as was noted by a recent code clarification. See Cygna's 11/26/85 letter 84056.095, attaching their latest revisions to open items, and attached pages 18 and 19 (Attachment F hereto) on friction, regarding change by ASME re: 3Sm (1983 Edition, NF-3121.2). This is not a change in the definition of primary stress, it's a clarification. In practice, restraint of the thermal growth of the pipe has always been considered to be a primary stress industry-wide. And thermal loads inducing pipe expansion have always been part of load condition A.

As relates to the above statement, Applicants have more caveats than candor, but regardless, Applicants now find that the allowables they were using to qualify these suports are incorrect. CASE has always contested the use of 3Sm.

At the bottom of page 53 and continued to page 54, the following is noted by Applicants: "When normal Code allowables are applied, 12 out of 114 double trunnion supports exceed allowables." It must be observed that

this equals 10% of this type of support installed at CPSES which are inadequate.

In the middle of page 54, Applicants explain how, by performing an elastoplastic analysis, they showed that the ductility of the support redistributed loading and the support accepted the load. This whole dissertation is irrelevant, since the code does not allow one to venture into areas of esoteria to prove that structures that have failed by normal code criteria may still hang on by a thread. Beyond this, the majority of those who are involved in these hearings have always been aware that there is a reserve between the allowables and the ultimate; however, this reserve is a no man's land. This is simple to understand when one realizes that the design of nuclear power plants and their components are qualified to at least two load conditions: A/B and C; and for some supports, a third case, Level D. Therefore, if a support can be proven to qualify at Level D, it obviously will not fail at the lower load levels.

Here we have in yet another area Applicants concurring with CASE's position that this type of support which is not analyzed for rotation or otherwise addressing the rotational problem becomes indeterminate. In fact, as Applicants point out, 12 out of 14 axial restraints are overstressed. CASE never predicted the numbers of supports which would fail, but have always indicated that the possibility for failure existed. We are happy that Applicants now concur.

One must keep in mind that Applicants are using allowables which they created in their own image. Now, to understand the problem, one will find when examining the numbers developed to prove an existing item is safe, one

need only examine the fudge factor which results from bias. Aside from this, the code has already clarified the status of constraint of free-end displacement (see Cygna's 11/26/85 letter 84056.095, page 19, line 4, et seq., Attachment F hereto). The new code clarifies the interpretation of constraint of free-end displacement (Cygna says "change the definition;" we disagree on this point, since generally constraint of the pipe per se has always been considered to induce a primary stress). This clarification also applies to items 15 and 16 of this affidavit.

I find it difficult to understand how, during the 1983 hearings, Mr. Reedy was stating that a code revision was coming out with an editorial change to state that thermal stress in supports would not need consideration. But Mr. Reedy failed to mention that an editorial change would also clarify the fact that constraint of free-end displacement would be considered to be a primary stress.

27. Richmond Inserts

One statement I can make regarding Applicants' comments beginning on page 57 is that here again Applicants are revising their allowables from the original affidavit which accompanied their Motion for Summary Disposition downward from an interaction of 1.75 to 1.33.

One other minor point I might make is that it seems rather senseless to discuss the particulars of a problem when the concept is erroneous from the beginning. In the case of the Richmond insert/tube steel issue, the AISC clearly dictated the restrictions for oversized and slotted holes, as is

shown in Section 5-193 (3) of the 7th Edition. But the clarification of the 8th Edition is more to the point (see Attachment A, Section 1.24.4.3), which states: "Oversized holes . . . but they shall not be used in bearing type connections." Beyond that, Table 1.5.2.1, Note 1, limits the use of A307 bolts and A36 threaded material to static loads only (see Attachment A). Also see NF 4721, Attachment C, which limits oversized holes to friction joints only at (d)(1), but only restricts the slots in the bearing connections relative to direction, which must be normal to the load. Beyond that, I can only say that this issue remains open for the time being.

28. In reference to the box beams covered on page 61

This issue also remains open and CASE will have much to say on this at a later date, even though we could have commented on the fact that the Applicants admit that some of the stresses are higher (although they don't say how much higher) than assumed when they filed their original affidavit. This issue they also defer to Stone & Webster.

29. Overall Perception

As a final statement on the revelations contained in Applicants' affidavit, I would like to comment on the overall perception which is conveyed. With over 35 years of experience within the engineering discipline, I find it impossible to accept the possibility that all of the errors and oversights which effectively negate at least 8 of Applicants'

Motions for Summary Disposition are the result of massive incompetence. One has to keep in mind that in Applicants' efforts to provide the proof for the Board of the adequacy of the various supports, Applicants utilized the staffs of no less than 6 major corporations /5/, in addition to their own rather formidable staff. Beyond this, Applicants had the legal staffs of at least two major law firms /6/ to guide them as relates to the legalities and the wording of the contents of the motions for summary disposition, again, in addition to their own substantial in-house legal staff. The only conclusion available to me at this point in time is that Applicants were aware that at least some of their arguments rested on "somewhat" shaky ground to say the least.

In considering the above, I must state that it is not in the interest of plant safety or in fact public safety to allow those involved in the motions for summary disposition to exercise any control over decisions being made in relation to the Stone & Webster/CPRT corrective action program, considering the fact that they were involved in past decisions that proved to be either results of gross incompetence or otherwise.

/5/ Gibbs and Hill, ITT Grinnell, NPSI, Ebasco, Westinghouse Corporation, and Cygna Energy Services indirectly.

/6/ Bishop, Liberman, Cook, Purcell & Reynolds; and Worsham, Forsythe, Sampels & Wooldridge.

Attachments:

A -- AISC Eighth Edition, pages 4-125 through 4-131, Table 1.5.2.1, Sections 1.23.4.1, 1.23.4.3, 1.23.4.4, 1.23.4.5 -- (pages 35, 36, and 54 of this affidavit)

B -- ASME Section III, Subsection NF, Appendix I, Table 1-7.1 -- (page 9 of this affidavit)

C -- ASME Section III, Subsection NF, paragraphs 3260, 3261, 3262.2, 3262.3, 3262.4, 4721, etc. -- (pages 8, 9, and 54 of this affidavit)

D -- BNL Review of Texas Utilities Generating Company Comanche Peak Steam Electric Station Upper Lateral Restraint Beam - Steam Generator, FOIA-85-59, C/364 -- (pages 40, 41, and 43 of this affidavit)

E -- CASE Exhibit No. 1,035, SIS Report G-044, Hartford Steam Boiler Inspection and Insurance Company (ANI), 5/26/83 -- (page 30 of this affidavit)

F -- Cygna Energy Services November 26, 1985 letter 84056.095, cover letter and pages 18 and 19 of 11/20/85 Revision 2 of Pipe Supports Review Issues List -- (pages 51 and 53 of this affidavit)

I have read the foregoing affidavit, which was prepared under my personal direction, and it is true and correct to the best of my knowledge and belief.

Jack J. Doyle

Date: 12-14-85

STATE OF Massachusetts

COUNTY OF Worcester

On this, the 14th day of December, 1985, personally appeared Jack Doyle, known to me to be the person whose name is subscribed to the foregoing instrument, and acknowledged to me that he/she executed the same for the purposes therein expressed.

Subscribed and sworn before me on the 14th day of December, 1985.

Lorraine E. Doyle

Notary Public in and for the State of Massachusetts

My Commission Expires: My Commission Expires September 4, 1992

Manual of

STEEL CONSTRUCTION

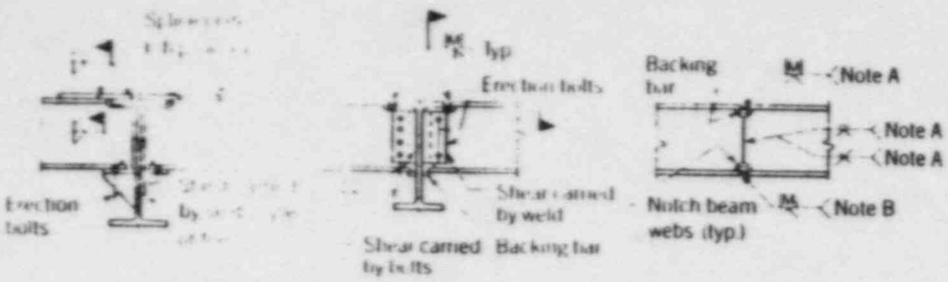
EIGHTH EDITION

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

CASE ATTACHMENT A

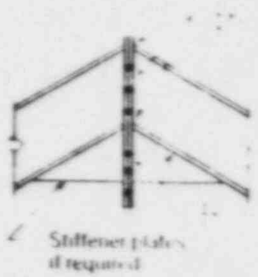
SUGGESTED DETAILS
Beam framing

WELDED MOMENT SPLICES

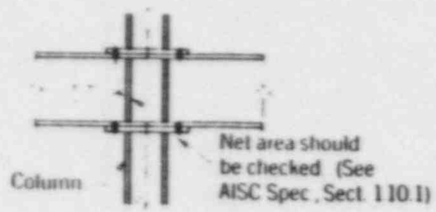


Note A Joint preparation depends on thickness of material and welding process.
Note B Invert this joint preparation if beam cannot be turned over.

MOMENT SPLICE AT RIDGE (FIELD BOLTED)

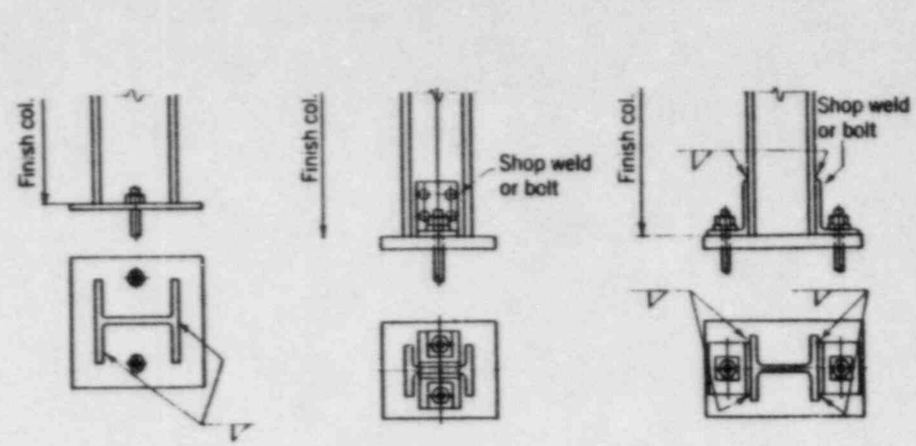


*BEAM OVER COLUMN (WITH CONTINUITY)

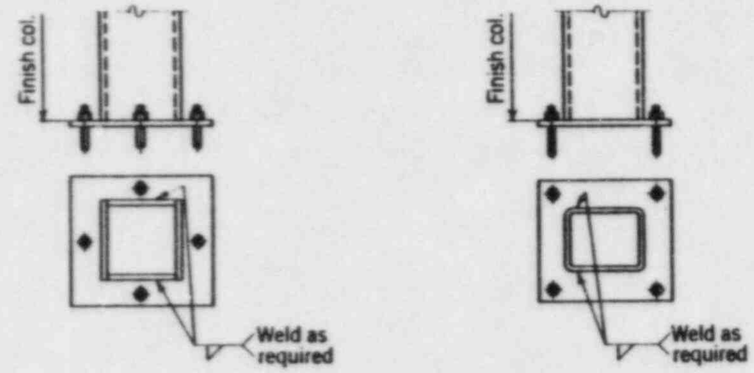


*For Plastic Design see Spec. Sect. 2.6.

SUGGESTED DETAILS
Column base plates

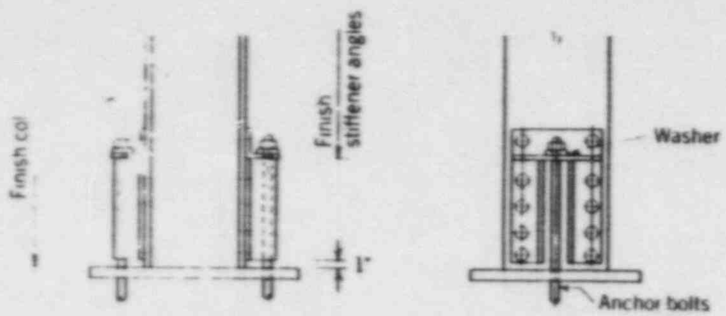


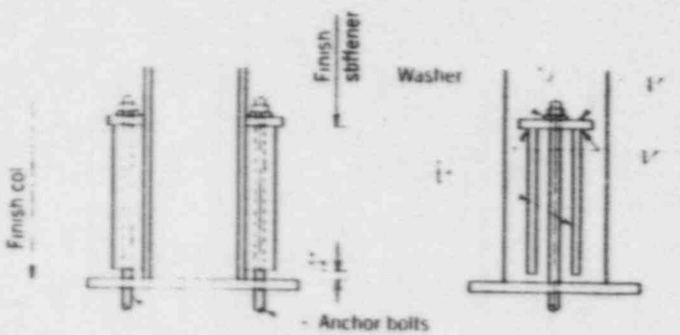
Base plate detailed and shipped loose when required.



Notes: 1. Hole sizes for anchor bolts are normally made oversize to facilitate erection as follows:
Bolts ¾ to 1"⌀ — $^5/_{16}$" oversize
Bolts 1 to 2"⌀ — ½" oversize
Bolts over 2"⌀ — 1" oversize
2. The stability of a column with its loading should be considered at all stages of erection and its base designed accordingly for anchors and base plate.

SUGGESTED DETAILS
Column base plates



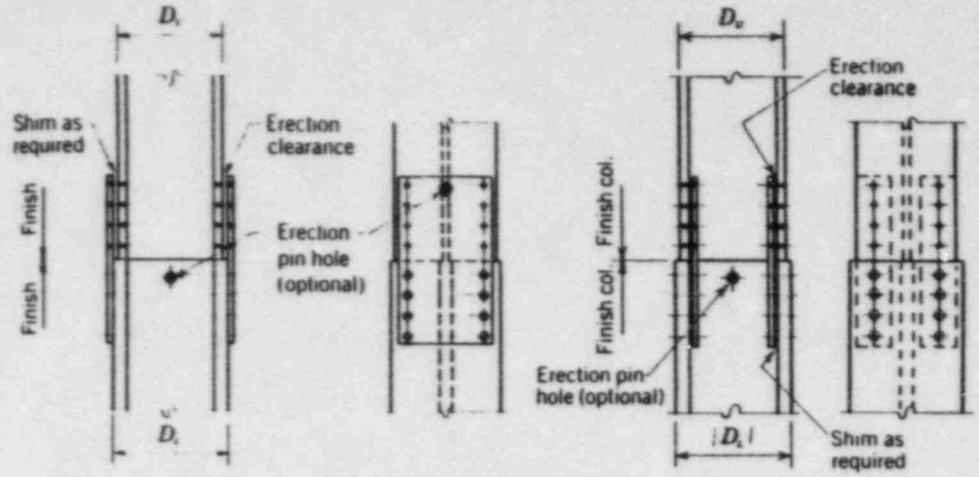


Base plates are normally detailed and shipped loose

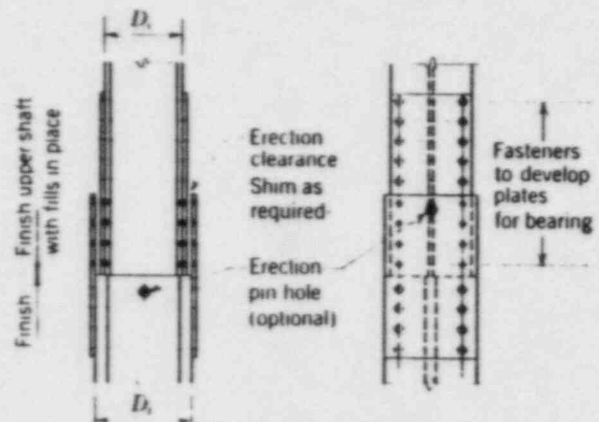
Notes: 1. Hole sizes for anchor bolts are normally made oversize to facilitate erection as follows:
Bolts [illegible] to 1 [illegible] — 5/16" oversize
Bolts 1 to 2 [illegible] — ½" oversize
Bolts over 2 [illegible] — 1" oversize

2. The stability of a column with its loading should be considered at all stages of erection and its base designed accordingly for anchors and base plate.

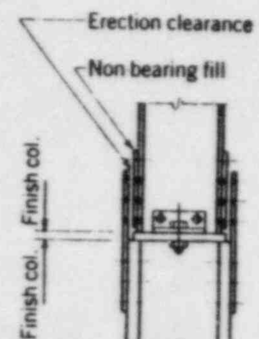
SUGGESTED DETAILS
Column splices
Riveted and bolted



DEPTH OF D_U AND D_L NOMINALLY THE SAME



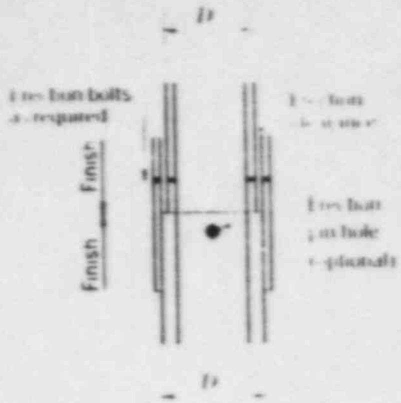
DEPTH D_U NOMINALLY 2 IN. LESS THAN D_L

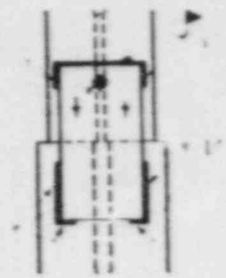


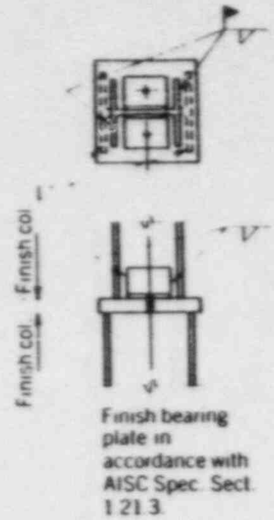
BUTT PLATE

Note: Erection clearance = ⅛ in.

SUGGESTED DETAILS
Column splices
Welded

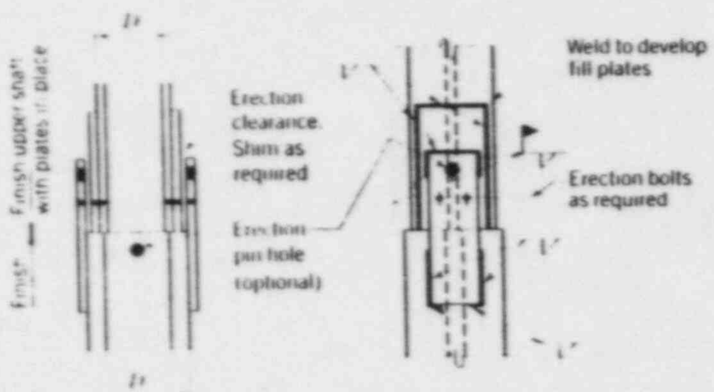






DEPTH OF D_U AND D_L
NOMINALLY THE SAME

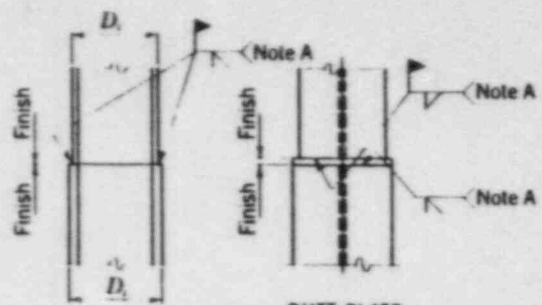
BUTT PLATE



DEPTH D_U NOMINALLY
2 IN. LESS THAN D_L

Notes: Erection cl[illegible] in.
When D_U and D_L are nominally the same and thin fills are required, shop may attach splice plate to upper section and provide field clearance over lower section.
Stability of upper shaft, with its loading, should be considered until the final welding is completed.

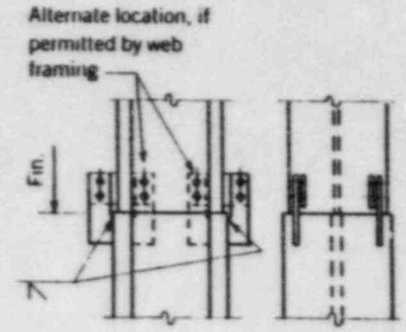
SUGGESTED DETAILS
Column splices
Welded



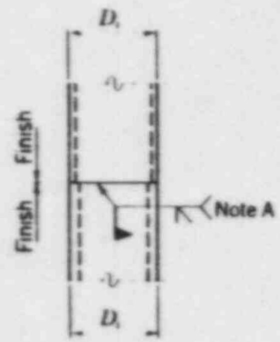
BUTT PLATE

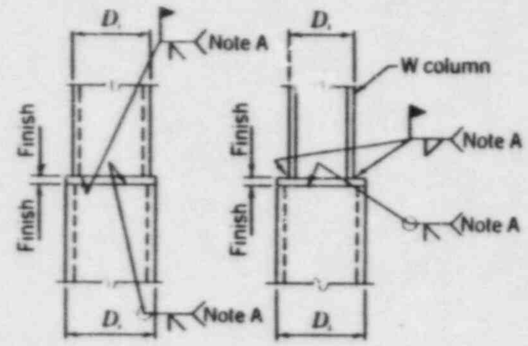
DEPTH OF D_U AND D_L
NOMINALLY THE SAME

DEPTH D_U NOMINALLY
2 IN. LESS THAN D_L



ERECTION AID AND
STABILITY DEVICE





BUTT PLATE

DEPTH OF D_U AND D_L
NOMINALLY THE SAME

DEPTH D_U NOMINALLY
2 IN. LESS THAN D_L

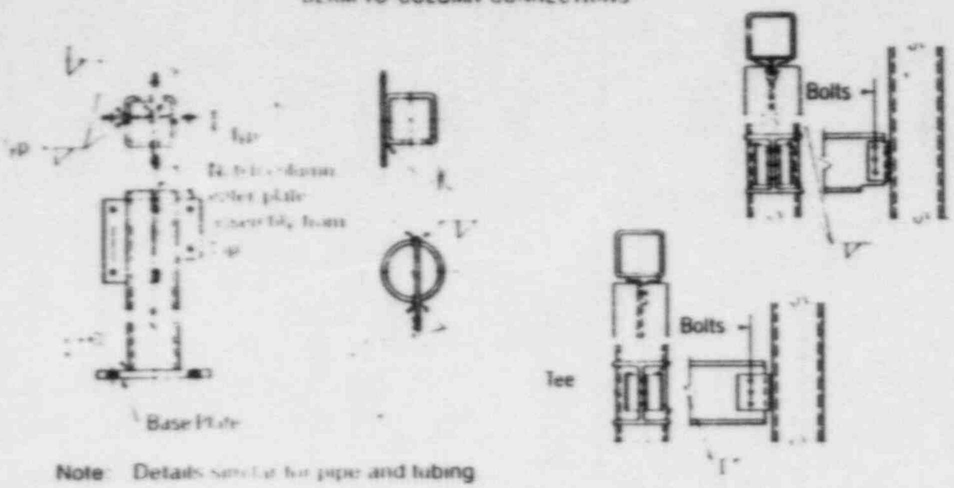
Note A: Use fillet welds or partial penetration weld whenever possible.

Finish bearing plates in accordance with AISC Spec. Sect. 1.21.3.

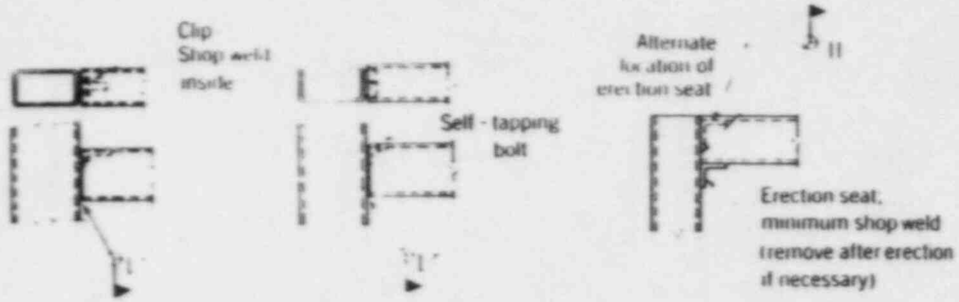
SUGGESTED DETAILS

Miscellaneous

STRUCTURAL TUBING AND PIPE BEAM-TO-COLUMN CONNECTIONS

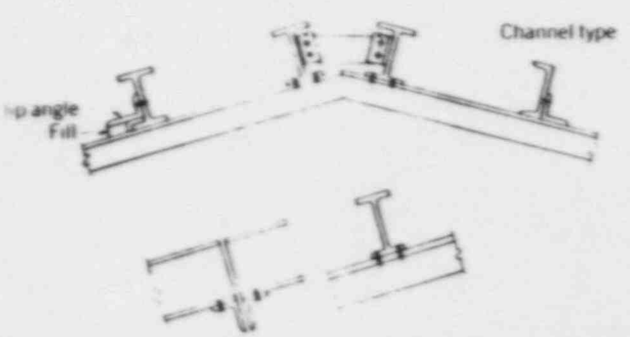


Note: Details similar for pipe and tubing

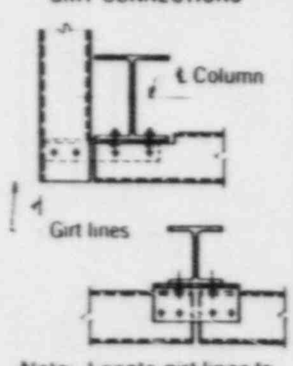


Note: Connections within tubes and pipe may be difficult or impossible to erect

PURLIN CONNECTIONS



GIRT CONNECTIONS

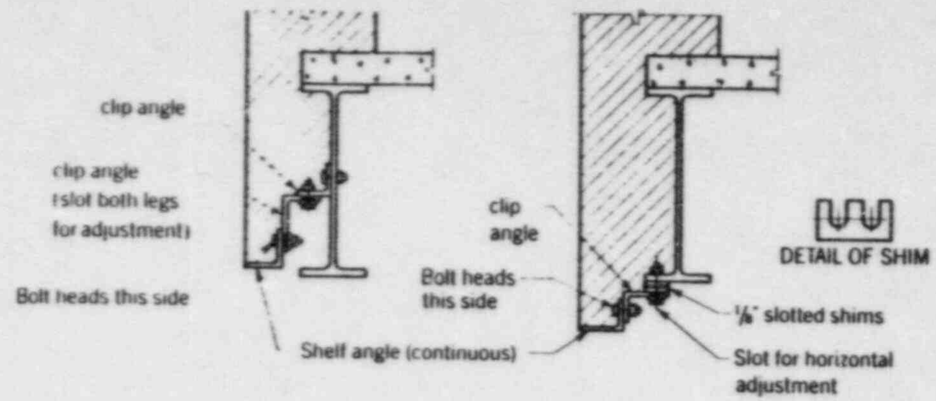


Note: Locate girt lines to avoid blocking girts when possible

SUGGESTED DETAILS

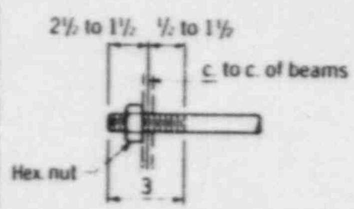
Miscellaneous

SHELF ANGLES WITH ADJUSTMENT



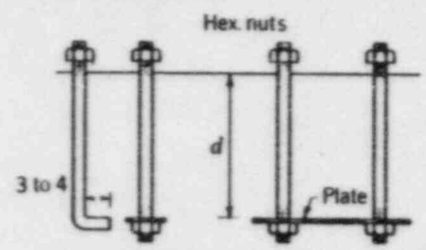
Notes: Horizontal adjustment is made by slotted holes; vertical adjustment may be made by slotted holes or by shims.
For tolerance allowance in alignment, see AISC Code of Standard Practice.

TIE RODS AND ANCHORS



Note: Length of rod o. to o. should be specified in multiples of 3 in.

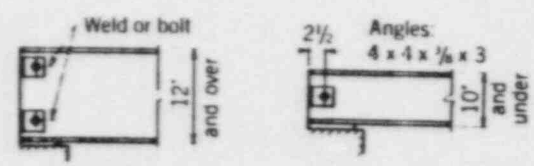
Tie Rods



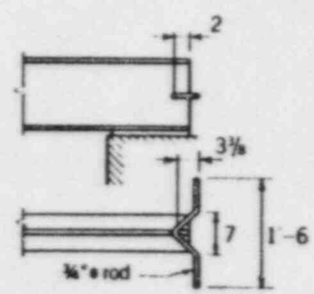
Note: Dimension *d* should be based on design req't for uplift

Anchor Bolts

Swedge Bolts



Angle Wall Anchors



Government Anchor

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.

TABLE 1.23.4

MAXIMUM SIZES[a] OF FASTENER HOLES, INCHES

Nominal Fastener Diameter (d)	Standard Hole Diameter	Oversized[b] Hole Diameter	Short-Slotted[b] Hole Dimensions	Long-Slotted[b] Hole Dimensions
$\leq \frac{7}{8}$	$d + \frac{1}{16}$	$d + \frac{3}{16}$	$(d + \frac{1}{16}) \times (d + \frac{1}{4})$	$(d + \frac{1}{16}) \times 2\frac{1}{2}d$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + \frac{1}{16}$	$d + \frac{5}{16}$	$(d + \frac{1}{16}) \times (d + \frac{3}{8})$	$(d + \frac{1}{16}) \times 2\frac{1}{2}d$

[a] Sizes are nominal.

[b] Not permitted for riveted connections.

gouges greater than $\frac{3}{16}$-inch that remain from cutting shall be removed by grinding. All re-entrant corners shall be shaped notch-free to a radius of at least $\frac{1}{2}$-inch.

1.23.3 Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes will not be required unless specifically called for on the drawings or included in a stipulated edge preparation for welding.

1.23.4 Riveted and Bolted Construction—Holes

1.23.4.1 The maximum sizes of holes for rivets and bolts shall be as stipulated in Table 1.23.4, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, may be used in column base details.

1.23.4.2 Standard holes shall be provided in member-to-member connections, unless oversized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Oversized and slotted holes shall not be used in riveted connections.

If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least $\frac{1}{16}$-inch smaller than the nominal diameter of the rivet or bolt. Holes in A514 steel plates over $\frac{1}{2}$-inch thick shall be drilled.

1.23.4.3 Oversized holes may be used in any or all plies of friction-type connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

1.23.4.4 Short-slotted holes may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

LES. INCHES

...tted[b] ... ions	Long-Slotted[b] Hole Dimensions
$(d + {}^{1}/_{4})$	$(d + {}^{1}/_{16}) \times 2^{1}/_{2}d$
$1^{5}/_{16}$	$1^{1}/_{16} \times 2^{1}/_{2}$
$(d + {}^{3}/_{8})$	$(d + {}^{1}/_{16}) \times 2^{1}/_{2}d$

cutting shall be removed by
...tch-free to a radius of at least

...t edges of plates or shapes will
...the drawings or included in a

...les

...ets and bolts shall be as stipu-
...uired for tolerance on location
...sed in column base details.

...n member-to-member connec-
...ted holes in bolted connections
...tted holes shall not be used in

...r than the nominal diameter of
...unched. If the thickness of the
...he rivet or bolt plus ⅛-inch, the
punched and reamed. The die
...b-drilled holes, shall be at least
...e rivet or bolt. Holes in A514

...ny or all plies of friction-type
...g-type connections. Hardened
...an outer ply.

...n any or all plies of friction-type
...sed without regard to direction
...ength shall be normal to the di-
Washers shall be installed over
...n-strength bolts are used, such

TABLE 1.23.5

MINIMUM BOLT TENSION, KIPS*

Bolt Size, Inches	A325 Bolts	A490 Bolts
½	12	15
⅝	19	24
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102
1⅜	85	121
1½	103	148

* Equal to 0.70 of specified minimum tensile strengths of bolts, rounded off to nearest kip.

1.23.4.5 Long-slotted holes may be used in only one of the connected parts of either a friction-type or bearing-type connection at an individual faying surface. Long-slotted holes may be used without regard to direction of loading in friction-type connections, but shall be normal to the direction of the load in bearing-type connections. Where long-slotted holes are used on an outer ply, plate washers or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than $^{5}/_{16}$-in. thick and shall be of structural grade material, but need not be hardened. If hardened washers are required to satisfy Specification provisions for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

1.23.5 Riveted and High-Strength-Bolted Construction—Assembling

All parts of riveted members shall be well pinned or bolted and rigidly held together while riveting. Use of a drift pin in rivet or bolt holes during assembling shall not distort the metal or enlarge the holes. Holes that must be enlarged to admit the rivets or bolts shall be reamed. Poor matching of holes shall be cause for rejection.

Rivets shall be driven by power riveters, of either compression or manually-operated type, employing pneumatic, hydraulic, or electric power. After driving, they shall be tight and their heads shall be in full contact with the surface.

Rivets shall ordinarily be hot-driven, in which case their finished heads shall be of approximately hemispherical shape and shall be of uniform size throughout the work for the same size rivet, full, neatly finished, and concentric with the holes. Hot-driven rivets shall be heated uniformly to a temperature not exceeding 1950° F; they shall not be driven after their temperature has fallen below 1000° F.

Surfaces of high-strength-bolted parts in contact with the bolt head and nut shall not have a slope of more than 1:20 with respect to a plane normal to the bolt axis. Where the surface of a high-strength-bolted part has a slope of more than 1:20, a beveled washer shall be used to compensate for the lack of parallelism. High-strength-bolted parts shall fit solidly together when assembled and shall not be separated by gaskets or any other interposed compressible materials. When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. They shall be free of dirt, loose scale, burrs, and other defects that would prevent solid seating of the parts. Contact surfaces

ASME BOILER AND PRESSURE VESSEL CODE
AN AMERICAN NATIONAL STANDARD
ANSI ASME BPV-III-1-NF

SECTION III 1977 EDITION

CASE ATTACHMENT B

NUCLEAR POWER PLANT COMPONENTS

DIVISION 1

SUBSECTION NF

component supports

ASME BOILER AND PRESSURE VESSEL CODE
AN AMERICAN NATIONAL STANDARD

ANSI/ASME BPV-III-1-A

SECTION III

Rules for Construction of Nuclear Power Plant Components

DIVISION 1 — APPENDICES

1977 EDITION

JULY 1, 1977

ASME BOILER AND PRESSURE VESSEL COMMITTEE
SUBCOMMITTEE ON NUCLEAR POWER

THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS
United Engineering Center 345 East 47th Street New York, N.Y. 10017

APPENDIX I

Design Stress Intensity Values, Allowable Stresses, Material Properties, and Design Fatigue Curves

TABLE I-7.1
ALLOWABLE STRESS VALUES, *S*, FOR FERRITIC STEELS FOR CLASS 2 AND 3 COMPONENTS

Nominal Composition	P-No.	Gr. No.	Product[1] Form	Spec. No.	Grade or Type	Class	Notes
Carbon Steels							
C	1	1	Wld. Pipe	SA-155	C45	1	(3)(13)
C	1	1	Plate	SA-285	A	...	(13)
C	1	1	Wld. Pipe	SA-672	A45	...	(3)(13)
C	1	1	Smls. Pipe	SA-106	A	...	(13)
C	1	1	Wld. Pipe	SA-155	C50	1	(3)(13)
C	1	1	Plate	SA-285	B	...	(13)
C	1	1	Wld. Pipe	SA-672	A50	...	(3)(13)
C	1	1	Wld. Pipe	SA-155	C55	1	(3)(13)
C-Si	1	1	Wld. Pipe	SA-155	KC55	1	(3)(13)
C-Si	1	1	Wld. Pipe	SA-155	KCF55	1	(3)(13)
C	1	1	Plate	SA-285	C	...	(13)
C-Mn	1	1	Wld. & Smls. Pipe	SA-333	1	...	(3)(13)
C-Mn	1	1	Wld. & Smls. Tube	SA-334	1	...	(3)(13)
C	1	1	Plate	SA-414	C	...	(13)
C-Mn-Si	1	1	Plate	SA-442	55	...	(13)
C-Si	1	1	Plate	SA-515	55	...	(13)
C-Si	1	1	Plate	SA-516	55	...	(13)
C	1	1	Wld. Pipe	SA-671	CA55	...	(3)(13)
C-Mn-Si	1	1	Wld. Pipe	SA-671	CE55	...	(3)(13)
C	1	1	Wld. Pipe	SA-672	A55	...	(3)(13)
C-Si	1	1	Wld. Pipe	SA-672	B55	...	(3)(13)
C-Si	1	1	Wld. Pipe	SA-672	C55	...	(3)(13)
C-Mn-Si	1	1	Wld. Pipe	SA-672	E55	...	(3)(13)
C-Mn-Si	1	1	Shape, Plate & Bars	SA-36	...	...	(6)(13)
C-Si	1	1	Forg.	SA-181	I	...	...
C-Si	1	1	Casting	SA-216	WCA	...	(4)
C-Si	1	1	Forg.	SA-266	...	1	(13)
C-Mn	1	1	Forg.	SA-350	LF1	...	...
C-Si	1	1	Casting	SA-352	LCA	...	(4)
C-Si	1	1	Cast Pipe	SA-660	WCA	...	(4)
C-Si	1	1	Wld. Pipe	SA-155	KC60	1	(3)(13)
C-Si	1	1	Wld. Pipe	SA-155	KCF60	1	(3)(13)
C-Mn-Si	1	1	Plate	SA-442	60	...	(13)
C-Si	1	1	Plate	SA-515	60	...	(13)
C-Si	1	1	Plate	SA-516	60	...	(13)
C-Si	1	1	Wld. Pipe	SA-671	CB60	...	(3)(13)
C-Si	1	1	Wld. Pipe	SA-671	CC60	...	(3)(13)
C-Mn-Si	1	1	Wld. Pipe	SA-671	CE60	...	(3)(13)
C-Si	1	1	Wld. Pipe	SA-672	B60	...	(3)(13)
C-Si	1	1	Wld. Pipe	SA-672	C60	...	(3)(13)
C-Mn-Si	1	1	Wld. Pipe	SA-672	E60	...	(3)(13)
C-Mn	1	1	Smls. Pipe	SA-106	B	...	(13)
C-Mn-Si	1	1	Wld. & Smls. Pipe	SA-333	6	...	(3)(13)
C-Mn-Si	1	1	Wld. & Smls. Tube	SA-334	6	...	(3)(13)
...	1	1	Bar	SA-695	B35	...	...
...	1	1	Bar	SA-696	B	...	...
C	1	1	Wld. Tube	SA-178	C	...	(13)
C	1	1	Smls. Tube	SA-210	A-1	...	(13)
C-Si	1	1	Wld. Pipe	SA-155	KC65	1	(3)(13)
C-Mn-Si	1	1	Wld. Pipe	SA-155	KCF65	1	(3)(13)
C-Si	1	1	Casting	SA-352	LCB	...	(4)
C-Si	1	1	Plate	SA-515	65	...	(13)
C-Mn-Si	1	1	Plate	SA-516	65	...	(13)
C-Si	1	1	Wld. Pipe	SA-671	CB65	...	(3)(13)

TABLE I-7.1
ALLOWABLE STRESS VALUES, *S*, FOR FERRITIC STEELS FOR CLASS 2 AND 3 COMPONENTS

Min. Yield Strength	Min. Ult. Tensile Strength	Allowable Stress, ksi (Multiply by 1000 to Obtain psi), F[illegible] Metal Temperatures, F, Not to Exceed									
		100	200	300	400	500	600	650	700	750	800
											Carbon Steels
24.0	45.0	11.2	11.2	11.2	11.2	11.2	11.2	11.2	10.9	...	...
30.0	48.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	11.6	...	...
27.0	50.0	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.1	...	...
30.0	55.0	13.7	13.7	13.7	13.7	13.7	13.7	13.7	13.2	...	...
36.0	58.0	12.6	12.6	12.6	12.6	12.6	12.6	12.6	12.6	...	...
30.0	60.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	14.3	...	...
32.0	60.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	14.3	...	...
35.0	60.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	14.3	...	...
37.0	60.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	14.3	...	...
35.0	65.0	16.2	16.2	16.2	16.2	16.2	16.2	16.2	15.2	...	...

TABLE I-7.1 (CONT'D)
ALLOWABLE STRESS VALUES, *S*, FOR FERRITIC STEELS FOR CLASS 2 AND 3 COMPONENTS

Min. Yield Strength	Min. Ult. Tensile Strength	Allowable Stress, ksi (Multiply by 1000 to Obtain psi), For Metal Temperatures, F, Not to Exceed									
		100	200	300	400	500	600	650	700	750	800
High Alloy Steels (Cont'd) Precipitation Hardened Steels (Cont'd)											
115.0	140.0	35.0	35.0	35.0	34.1	33.3	32.8	32.6	32.2	...	...
125.0	145.0	36.2	36.2	36.2	35.2	34.5	34.0	33.7	33.4	...	...

Method of Examination	*Quality Factor*
(a) Visual	0.80
(b) Magnetic Particle	0.85
(c) Liquid Penetrant	0.85
(d) Radiography	1.00
(e) Ultrasonic	1.00
(f) Magnetic Particle or Liquid Penetrant plus Ultrasonic or Radiography	1.00

(5) Not for welded construction.
(6) The use of these materials shall be limited to materials for supports and hangers and for tanks covered in Subsections NC and ND. The allowable stress in the short transverse (VT) direction for plates shall be limited to one-half of the listed values.
(7) *S* values and P-Numbers for steel fittings shall be the same as those assigned to the material from which the fittings are made. Fittings shall be made only from materials listed in this Table. However, the materials requirements of the referenced material specification need be met only as required by the fitting specification.
(8) These stress values apply to material that has been age-hardened at 1150 F.
(9) These stress values apply to material that has been age-hardened at 1100 F.
(10) These stress values apply to material that has been age-hardened at 1075 F.
(11) Grade B, C, or D of SA-302 may be specified.
(12) Any grade of the plate specification may be specified.
(13) For External Pressure Chart reference, see Table I-14.0.

CASE ATTACHMENT C

ASME BOILER AND PRESSURE VESSEL CODE
AN AMERICAN NATIONAL STANDARD

ANSI/ASME BPV-III-1-NF

SECTION III

Rules for Construction of Nuclear Power Plant Components

DIVISION 1 — SUBSECTION NF

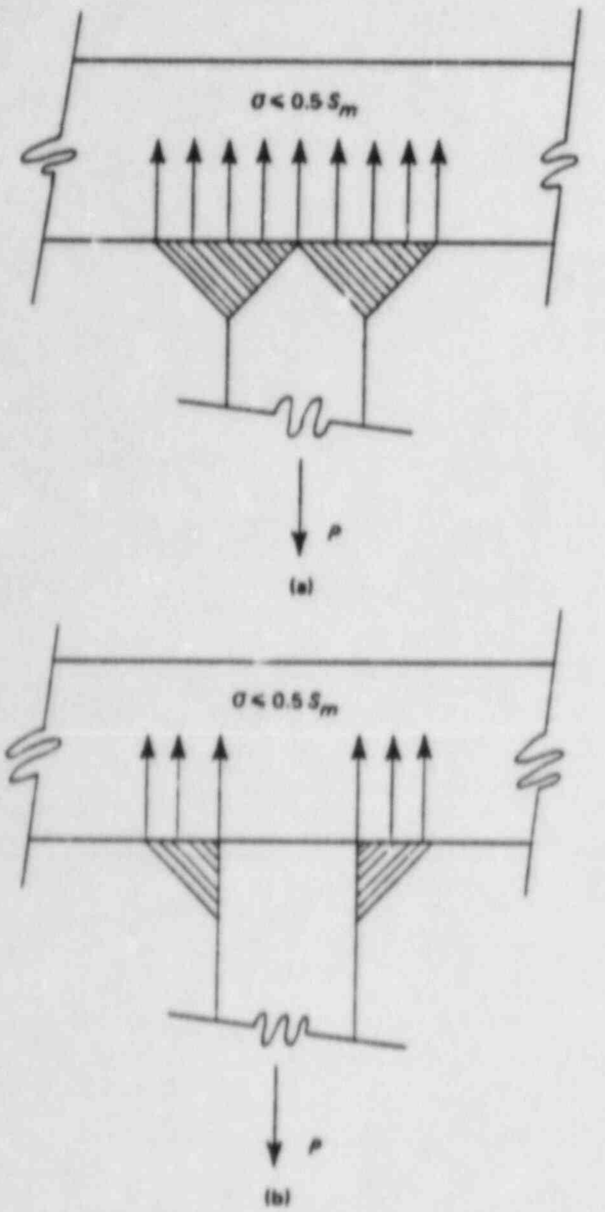
Component Supports

1977 EDITION

JULY 1, 1977

ASME BOILER AND PRESSURE VESSEL COMMITTEE
SUBCOMMITTEE ON NUCLEAR POWER

THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS
United Engineering Center 345 East 47th Street New York, N.Y. 10017



S_m = Design stress intensity value
σ = Maximum tensile stress in the through-thickness direction of plates and elements of rolled shapes, evaluated at the contact point of the weld area with the element surface
P = Applied load

FIG. NF-3226.5-1 ILLUSTRATIONS OF MAXIMUM DESIGN STRESS IN THROUGH-THICKNESS DIRECTION OF PLATES AND ELEMENTS OF ROLLED SHAPES

NF-3240 DESIGN OF COMPONENT STANDARD SUPPORTS BY ANALYSIS

The requirements of NF-3220 or NF-3230 apply as applicable.

NF-3250 EXPERIMENTAL STRESS ANALYSIS

Component supports of all types may be designed by experimental stress analysis in accordance with Appendix II.

NF-3260 DESIGN BY LOAD RATING

NF-3261 Procedure for Load Rating

The procedure for load rating shall consist of imposing a total load on one or more duplicate full size samples of a component support equal to or less than the load under which the component support fails to perform its required function. A single test sample is permitted but, in that case, the load ratings shall be derated by 10%. Otherwise, tests shall be run on a statistically significant number of samples.

NF-3262 Load Ratings in Relation to Service Loadings

The load ratings for Service Loadings for which Level A, Level B or Level C Limits have been designated shall be determined by means of the equations in the following subparagraphs. For Level D Limits, see Appendix F.

NF-3262.1 Nomenclature. The symbols used in this paragraph are defined as follows:

T.L. = support test load equal to or less than the load under which the component support fails to perform its specified support function
F_{all} = allowable value for the type of stress in XVII-1100 of Appendix XVII
S = allowable stress value at the design temperature (NF-3112.2) from the applicable table of Appendix I
S_u = specified minimum tensile strength of the material used in the support as given in the applicable table of Appendix I

NF-3262.2 Plate and Shell Supports. The load ratings for plate and shell supports for the Service Loadings shall be determined by the following equations:

Level A Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{S}{S_u} \quad (1)$$

Level B Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{S}{S_u} \quad (2)$$

Level C Limits

$$\text{load rating} = \text{T.L.} \times 1.2 \frac{S}{S_u} \quad (3)$$

NF-3262.3 Linear Type Supports. The load ratings for linear type supports for the Service Loadings shall be determined by the following equations:

Level A Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{F_{all}}{S_u} \quad (4)$$

Level B Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{F_{all}}{S_u} \quad (5)$$

Level C Limits

$$\text{load rating} = \text{T.L.} \times 1.33 \frac{F_{all}}{S_u} \quad (6)$$

NF-3262.4 Component Standard Supports. The load ratings for component standard supports for the Service Loadings shall be determined by the following equations:

Level A Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{S \text{ or } F_{all}}{S_u} \quad (7)$$

Level B Limits

$$\text{load rating} = \text{T.L.} \times 1.0 \frac{S \text{ or } F_{all}}{S_u} \quad (8)$$

plate and shell

Level C Limits

$$\text{load rating} = \text{T.L.} \times 1.2 \frac{S}{S_u} \quad (9a)$$

linear type

Level C Limits

$$\text{load rating} = \text{T.L.} \times 1.33 \frac{F_{all}}{S_u} \quad (9b)$$

NF-3280 DESIGN OF BOLTS

NF-3281 Level A and Level B Service Limits

The number and cross-sectional area of bolts required for the design loadings of NF-3112 shall be determined in accordance with the procedures of Appendix XVII. The allowable bolt design stress values shall be the yield strength values of Table I-13.3 of Appendix I multiplied by the applicable design factors of Table XVII-2461.1-1.

NF-3290 DESIGN OF WELDED JOINTS

NF-3291 Permissible Types of Welded Joints in Plate and Shell Type Welded Supports

Revision 10/75

(a) All welded joints in plate and shell type supports shall be continuous and shall be one of the types shown in Fig. NF-3291(a)-1 and described in (1) through (8) below:

(1) full penetration butt welded groove joint, sketch (a)

(2) double ~~fillet~~ groove welded ~~lap~~ butt joint, sketch (b)

(3) full penetration groove welded tee joint, sketch (c)

(4) full penetration groove welded corner joint, sketch (d)

(5) ~~full~~ fillet welded ~~tee~~ joints, sketches (e)-(k)

~~*(6)* angle joints, sketch (f)~~

(7) fillet welded joint between a surface and a closed tubular section or a closed section, sketches (g) and (h)

(8) fillet welded joint between the edge of a plate and the end surface of a closed tubular section or a closed formed section, sketch (i)

When angle joints are used for connecting a transition in diameter to a cylinder, the angle, α, of Fig. NF-3291(a)-1, sketch (f), shall not exceed 30 deg.

(b) A tapered transition having a length not less than three times the offset between the adjacent surfaces of abutting sections, as shown in Fig. NF-3291(b)-1, shall be provided at joints between sections that differ in thickness by more than one-fourth of the thickness of the thinner section or by more than 1/8 in. (3.2 mm), whichever is less. The transition may be formed by any process that will provide a uniform taper. The weld may be partly or entirely in the tapered section or adjacent to it. This paragraph also applies when there is a reduction in thickness within a spherical shell or cylindrical course or plate.

(c) When the use of backing rings will result in undesirable conditions such as severe stress, corrosion, or erosion, the requirements of NF-4240 shall be met.

NF-3291.1 Design Stress Intensity and Allowable Stress Limits for Welded Joints. The limits of design stress intensity for welded joints for plate and shell type supports shall not exceed the applicable design stress intensity value or allowable stress value for the base metal being joined or the electrode being used.

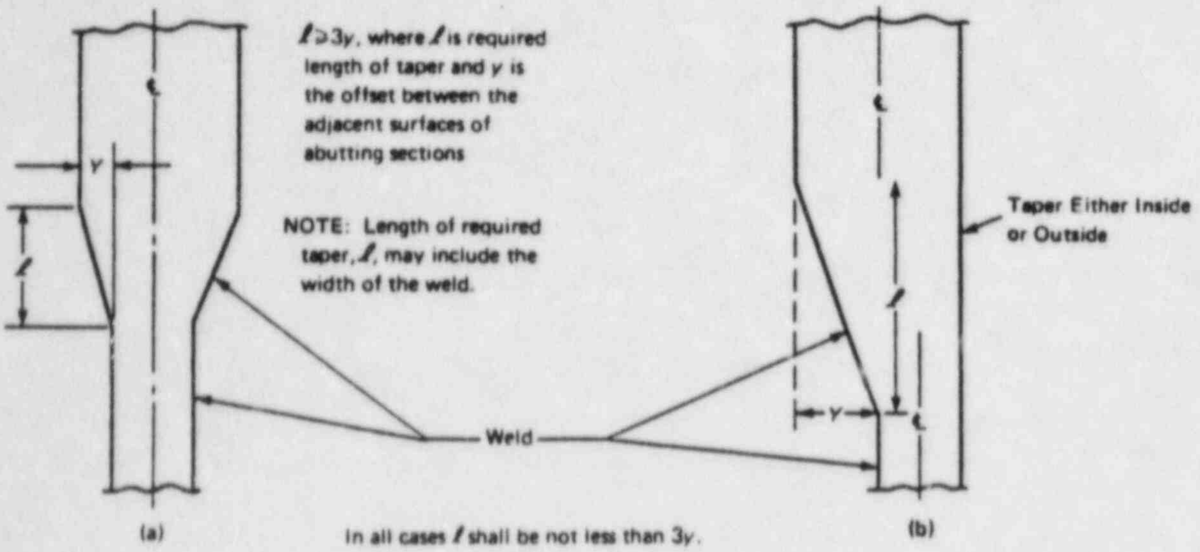


FIG. NF-3291(b)-1 BUTT WELDING OF PLATES OF UNEQUAL THICKNESS

Temperature differences between the component and its support and, where applicable, expansion or contraction of a vessel produced by internal or external pressure, shall be considered.

NF-3292 Permissible Types of Welded Joints in Linear Type Welded Supports

The permissible types of welded joints used in linear type supports shall be as stipulated in XVII-2450.

NF-3292.1 Allowable Stress Limits. The allowable stress limits for welds in linear type supports shall be as set forth in Table NF-3292.1-1.

NF-3293 Permissible Types of Welded Joints in Component Standard Supports

The permissible types of welded joints used in component standard supports and connections shall be as stipulated in NF-3291(a) and NF-3292.

NF-3293.1 Design Stress Intensity and Allowable Stress Limits for Welded Joints. The limit of design stress intensity or of allowable stress for welded joints for component standard supports shall not exceed the applicable design stress intensity value or allowable stress value for the base metal being joined. Temperature differences between the component and its support and, where applicable, expansion or contraction of a component produced by internal or external pressure, shall be considered.

NF-3300 DESIGN OF CLASS 2 COMPONENT SUPPORTS

NF-3310 GENERAL REQUIREMENTS

NF-3311 Acceptability

The requirements for acceptability of Class 2 component support design are given in (a) through (d) below.

(a) The design shall be such that the design stresses will not exceed the limits given in this Subarticle. Table NF-3132.1(b)-1 indicates the rules to be used for the various classes and types of design procedures. The applicable table of allowable stresses for a given material to be used with a specific design procedure is stipulated in Table NF-2121-1.

(b) The design procedure shall be one of those referenced in Table NF-3132.1(b)-1 applicable to Class 2 component supports.

(c) The design details shall conform to the rules of this Subarticle.

(d) For configurations where compressive stresses occur, the potential for critical buckling shall be considered.

TABLE NF-3292.1-1
ALLOWABLE STRESS LIMITS FOR
LINEAR COMPONENT SUPPORT WELDS—ALL CLASSES

Kind of Stress	Stress Limits, ksi	Base Metal T.S. Range, ksi
Tension and compression parallel to axis of any complete penetration groove weld	Same as for base metal	
Tension normal to effective throat of complete penetration groove weld	Same as allowable tensile stress for base metal	
Compression normal to effective throat of complete or partial penetration groove weld	Same as allowable compressive stress for base metal	
Shear on effective throat of complete penetration groove weld and partial penetration groove weld	Same as allowable shear stress for base metal	
Shear stress on effective throat of fillet weld regardless of direction of application of load; tension normal to the axis on the effective throat of a partial penetration groove weld and shear stress on effective area of a plug or slot weld. The given stresses shall also apply to such welds made with the specified electrode on steel having a yield stress greater than that of the *matching* base metal. The allowable stress, regardless of electrode classification used, shall not exceed that given in the table for the weaker *matching* base metal being joined.	18	45-60
	21	61-70
	24	71-80
	27	81-90
	30	91-100
	33	101-120
	36	121-135

NF-3320 DESIGN OF PLATE AND SHELL TYPE SUPPORTS BY ANALYSIS

NF-3321 Stress Limits

NF-3321.1 Design Loadings. The stress[3] limits are satisfied for the Design Loadings (NA-2142.1) stated in the Design Specifications if the requirements of Eqs. (1), (2), and ~~(3)~~ are met.

$$\sigma_1 \le 1.0S \qquad (1)$$

$$\sigma_1 + \sigma_2 \le 1.5S \qquad (2)$$

$$\cancel{\sigma_3 \le 0.5S} \qquad \cancel{(3)}$$

where

σ_1 = membrane stress, which is the average stress across the solid section under consideration. It includes the effects of discontinuities but not local stress concentrations.

σ_2 = bending stress, which is the linear varying portion of the stress across the solid section under consideration. It excludes the effects of discontinuities and concentrations.

σ_3 = maximum tensile stress at the contact surface of a weld producing a tensile load in a direction through the thickness of plate and rolled shape elements, as shown in Fig. NF-3321.1(c)-1

S = allowable stress value from the applicable Table of Appendix I as referenced in Table NF-2121(a)-1

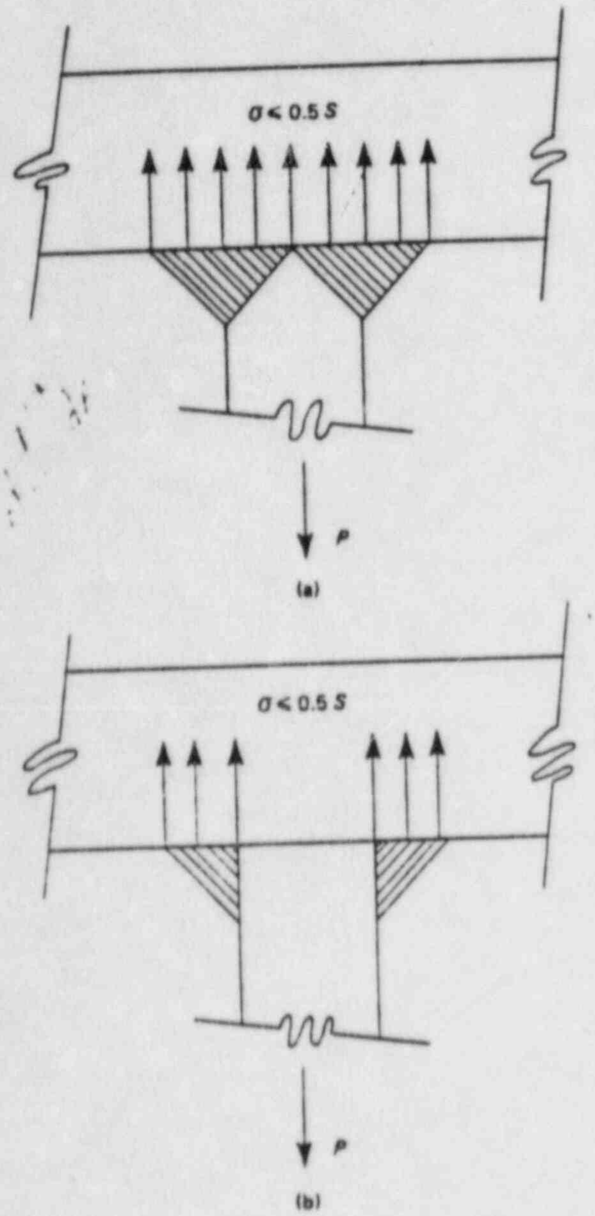
NF-3321.2 Service Loadings

(a) Level A Service Limits. Level A Service Limits are satisfied for the Service Conditions [NA-2142.2(b)] for which these limits are designated in the Design Specifications if the requirements of Eqs. (1), (2), ~~and (3)~~ of NF-3321.1 are met.

(b) Level B Service Limits. Level B Service Limits are satisfied for the Service Conditions [NA-2142.2(b)] for which these limits are designated in the Design Specifications if the requirements of Eqs. (1), (2), ~~and (3)~~ of NF-3321.1 are met.

(c) Level C Service Limits. Level C Service Limits are satisfied for the Service Conditions [NA-2142.2(b)] for which these limits are designated in the

[3] Stress means the maximum normal stress.



σ = Maximum tensile stress in the through-thickness direction of plates and elements of rolled shapes, evaluated at the contact point of the weld area with the element surface

P = Applied load

FIG. NF-3321.1(c)-1 ILLUSTRATIONS OF MAXIMUM DESIGN STRESS IN THROUGH THICKNESS DIRECTION OF PLATES AND ELEMENTS OF ROLLED SHAPES

Design Specifications if the requirements of Eqs. (1) and (2) of NF-3321.1 are not exceeded by more than 20%~~, and if the requirement of Eq. (3) of NF-3321.1 is met.~~

(d) Level D Service Limits. Level D Service Limits are satisfied for the Service Conditions [NA-2142.2(d)] for which these limits are designated in the Design Specifications if the requirements of ~~Eq. (3) of~~ NF-3321.1, and Eqs. (4) and (5) below, are met.

$$\sigma_1 \leq \text{lesser of } 1.5S \text{ or } 0.4S_u \qquad (4)$$

$$\sigma_1 + \sigma_2 \leq \text{lesser of } 2.25S \text{ or } 0.6S_u \qquad (5)$$

where

S_u = specified minimum ultimate tensile strength of material, Table ~~I-12.1~~ I-3.0

Other terms are as defined in NF-3321.1.

NF-3330 DESIGN OF LINEAR TYPE SUPPORTS BY ANALYSIS

The design rules and stress limits which must be satisfied for the Design and Service Loadings are as given in NF-3230.

NF-3340 DESIGN OF COMPONENT STANDARD SUPPORTS BY ANALYSIS

The design of component standard supports shall be in accordance with NF-3320 or NF-3330.

NF-3350 DESIGN BY EXPERIMENTAL STRESS ANALYSIS

Component supports may be designed by experimental stress analysis in accordance with Appendix II.

NF-3360 DESIGN BY LOAD RATING

Component supports may be designed by load rating in accordance with the requirements of NF-3260.

NF-4640 HEAT TREATMENT AFTER REPAIR BY WELDING

NF-4641 Rules Governing Heat Treatment After Repair by Welding

Component supports, parts, and materials that have been repaired by welding shall be postweld heat treated in accordance with the requirements of NF-4620.

NF-4700 REQUIREMENTS FOR BOLTED CONSTRUCTION

NF-4710 BOLTING AND THREADING

NF-4711 Thread Engagement

The threads of all bolts or studs shall be engaged for the full length of thread in the nut.

NF-4712 Thread Lubricants

Any lubricant or compound used in threaded joints shall be suitable for the service conditions and shall not react unfavorably with any support element material. Contact surfaces within friction type joints shall be free of oil, paint, lacquer, or galvanizing.

NF-4713 Removal of Thread Lubricants

All threading lubricants or compounds shall be removed from surfaces which are to be welded.

NF-4720 BOLTING

NF-4721 Bolt Holes

Bolt holes shall meet the requirements of (a) through (e) below.

(a) Holes for nonfitted bolts shall be $\frac{1}{16}$ in. (1.6 mm) larger than the nominal diameter of the bolt for bolt sizes up to and including 1 in. (25 mm) and $\frac{1}{8}$ in. (3.2 mm) larger than the nominal diameter of the bolt for sizes larger than 1 in. (25 mm).

(b) Except as specified in (c) below, holes may be punched provided the thickness of the material is not greater than the nominal diameter of the bolt plus $\frac{1}{8}$ in. (3.2 mm). Holes shall be subpunched and reamed, drilled, or thermally cut when the thickness of the material is greater than the nominal diameter of the bolt plus $\frac{1}{8}$ in. (3.2 mm). Thermal cutting shall not be used unless the load bearing surfaces are machined or ground smooth. The die for all subpunched holes shall be at least $\frac{1}{16}$ in. (1.6 mm) smaller than the nominal diameter of the bolt.

(c) Bolt holes in material over $\frac{1}{2}$ in. (13 mm) thick having a specified minimum yield strength greater than 80.0 ksi (552 MPa), shall be drilled.

(d) Oversized, short-slotted, and long-slotted bolt holes may be used with high strength bolts $\frac{1}{2}$ in. (13 mm) in diameter and larger except as restricted in (1), (2), and (3) below.

(1) Oversized holes shall not exceed $\frac{3}{16}$ in. (4.8 mm) larger than bolts $\frac{7}{8}$ in. (22 mm) and less in diameter, $\frac{1}{4}$ in. (6 mm) larger than bolts 1 in. (25 mm) in diameter, and $\frac{5}{16}$ in. (8 mm) larger than bolts $1\frac{1}{8}$ in. (28 mm) and greater in diameter. They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed oversized holes.

(2) Short-slotted holes shall not exceed $\frac{1}{16}$ in. (1.6 mm) wider than the bolt diameter and shall not have a length exceeding the oversize diameter allowed in (1) above by more than 1/16 in. (1.6 mm). They may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type connections but shall be normal to the direction of the load in bearing-type connections. Hardened washers shall be installed over exposed short-slotted holes.

(3) Long-slotted holes shall not exceed $\frac{1}{16}$ in. (1.6 mm) wider than the bolt diameter and shall not have a length which exceeds $2\frac{1}{2}$ times the bolt diameter. In friction-type connections, the long-slotted holes may be used without regard to direction of loading provided the stress on the bolts does not exceed 75% of the allowable working stress given in NF-3000. In bearing-type connections, the long diameter of the slot shall be normal to the direction of loading. Long-slotted holes may be used in only one of the connected parts of either a friction-type or bearing-type connection at an individual faying surface. Structural plate washers or a continuous bar not less than $\frac{5}{16}$ in. (8 mm) in thickness shall be used to cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall meet the requirements of NF-3000.

(e) For bolts not subjected to shear, the limits for oversized and slotted holes in (d) above may be increased if structural plate washers or continuous bars which meet the requirements of NF-3000 are provided.

NF-4722 Bolted Connections

(a) Surfaces of bolted parts in contact with the bolt head and nut shall not have a slope of more than 1:20 with respect to a plane normal to the bolt axis. Where the surface of high strength bolted part has a slope of more than 1:20, a beveled washer shall be used to compensate for the lack of parallelism.

(b) Bolts loaded in pure shear shall not have threads located in the load bearing part of the shank unless permitted by the Design Specifications.

NF-4723 Precautions Before Bolting

All parts assembled for bolting shall have contact surfaces free from scale, chips, or other deleterious material. Surfaces and edges to be joined shall be smooth, uniform and free from fins, tears, cracks, and other defects which would degrade the strength of the joint.

NF-4724 Bolt Tension

All high strength structural bolts shall be tightened to a bolt torque not less than that given in the Design Specifications. Tightening shall be done by the turn of nut method or with properly calibrated wrenches. Bolts tightened by means of a calibrated wrench shall be installed with a hardened washer under the nut or bolt head, whichever is the element turned in tightening. Hardened washers are not required when bolts are tightened by the turn of nut method, except that hardened washers are required under the nut and bolt head when the bolts are used to connect material having a specified yield point less than 40.0 ksi (276 MPa).

NF-4725 Locking Devices

All threaded fasteners, except high strength bolts, shall be provided with locking devices to prevent loosening during service. Elastic stop nuts (when compatible with service temperature), lock nuts, jam nuts, and drilled and wired nuts are all acceptable locking devices. Disk and helical spring lock washers shall not be used as locking devices. Upset threads may serve as locking devices.

CASE ATTACHMENT D

BNL Review of Texas Utilities Generating Company
Comanche Peak Steam Electric Station
Upper Lateral Restraint Beam - Steam Generator

Contributors

M. Reich, C.J. Costantino, S. Sharma, C. Miller
and A.J. Philippacopoulos

FOIA-85-59
C/364

INTRODUCTION

On August 14 and 15, 1984, an audit was conducted by NRC and BNL at the offices of Gibbs and Hill, Inc. (G & H), New York, on the subject of the upper lateral support beams (see Figures 1 and 2) for the steam generator compartment. At this meeting, various aspects of the structural analysis and design of the steel beam and concrete compartment were investigated. At the conclusion of the meeting, G & H was asked to provide additional information which was needed to complete the review. This material was received during the final week of August. Additional material and clarifications were required during the review process. These were obtained via telephone converstations and meetings held at BNL in Upton, NY, at NRC in Bethesda, MD and Gibbs and Hill in New York City. Specific dates for these meetings are November 5, 1984, November 13, 1984 and December 5, 1984. The BNL evaluations have now been completed and the specific findings are summarized below.

LOAD COMBINATIONS

Three separate load conditions were considered in the structural analysis, two LOCA loadings (at 0.5 and 216 seconds) and a Main Steam Break at 324 seconds. These load conditions were composed of combinations of pressure, temperature, dead, live, mechanical loads including primary system seismic loads, and seismic loadings from compartment motions. The last of these, seismic from compartment motions, were obtained from the peak acceleration output determined from the dynamic analysis of the stick model of the RCB. The other loadings, having been previously investigated by the MEB of NRC, and thus were not considered as part of this audit. All load components for each load condition are summarized on pages 2 and 3 of the Letter Report GTN-69363 from G & H to Texas Utilities, dated August 21, 1984.

NONLINEAR FINITE ELEMENT STUDY

A detailed nonlinear finite element model (FEM) analysis of the upper lateral beam and concrete compartment walls was performed using the EBASCO version of the NASTRAN computer program. The nonlinearity in the problem was introduced by allowing concrete cracking to occur as the load combination was applied to the structure. Since the problem contains nonlinearities, all

applied loads, including seismic loads, must be applied at the same time to properly assess the extent of cracking. This was done by G & H.

Seismic loads were applied to the G & H model in three coordinate directions, two horizontal and one vertical and were scaled such that 100% of the seismic load was applied in the NS direction, 40% in the EW direction and 40% in the vertical direction. Normally, seismic analyses are performed by applying the dynamic loads independently in opposite directions. For this nonlinear analysis, such an approach leads to the requirement of performing nine separate analyses. G & H performed two such analyses for the seismic loadings.

The FEM used by G & H was developed for one-half of the concrete compartment, since the compartment is symmetric about the NS direction. The model extends from an elevation of about 819' at the bottom to 884' at the top. Boundary conditions applied to the top and bottom nodes of the FEM are considered reasonable in view of the actual construction of the compartment. Along the centerline, however, the node points were restrained in the EW direction (by using roller supports in the NS direction). The use of this boundary condition leads to the result that the EW seismic loading condition cannot be correctly treated, since the model is artificially restrained. For the other loading types, namely pressure and temperature, this boundary condition is reasonable, since both the applied loadings are symmetric for these cases.

Thus, considering all the loads applied to FEM, two questions remain on the adequacy of the calculations associated with the seismic loads; namely, only two seismic load combinations were considered, and the boundary conditions used invalidate one component calculation. To evaluate the impact of these two restrictions on the nonlinear analysis, the results of previously obtained linear finite element seismic analyses were investigated. These analyses were performed by G & H using STARDYNE and hence did not suffer from the two restrictions mentioned above. Concrete cracking effects were not considered in these runs, however. The results of these runs indicate that

the stresses developed by the seismic loads are very small as compared to the stresses developed by pressure and thermal loadings. Therefore, it is our judgement that the deficiency in the analyses associated with the seismic loadings are not significant. It should be noted that BNL arrived at this conclusion using the results of G & H's TARDYNE output together with a mesh diagram used for that run.

Another aspect of the numerical studies of interest to the audit team has to do with the verification of the modifications inserted into the NASTRAN Computer Code inserted to perform the nonlinear cracking analysis of concrete. The analytic formulation of the model was ascertained and based on BNL's experience with concrete crack modeling it was found to be reasonable. In addition, EBASCO was asked to perform two other computer model studies to further verify the applicability of the code to this particular type of loading problem. These had to do with the problem of bending behavior of members with significant axial loads.

NASTRAN VERIFICATION RUNS

After the audit of August 14 and 15, 1984, EBASCO performed the evaluation of several check problems. The results of these were sent to BNL early in September. After studying the output and after consultation with NRC staff, it was decided to further investigate one of these problems in greater detail. The specific problem (identified as CA29 in the EBASCO submittal), and involves the prediction of load/deflection/cracking behavior and failure conditions for a RC beam rigidly held at both ends and centrally loaded.

A detailed finite element grid was developed by EBASCO, consisting of 200 elements, 10 layers through the thickness and 20 divisions along the half length. BNL used the same mesh and boundary conditions, and ran a comparable

analysis using the NFAP program (developed at BNL). The results of these two runs can be described as follows:

1. At low loads of (about 15 kips in the linear region, before the development of substantial concrete element cracking), the two runs yield comparable load-deflection output.

2. At higher loads (about 22 to 24 kips), the BNL/NFAP results predict diagonal tension cracking failure occurring near the supports for the beam. This result is very close to the value of ultimate capacity predicted by ZSUTTY'S equation, which is based on statistical evaluation of extensive experimentally determined data points. (Bazant & Kim, paper 81-38, ACI Journal September - October 1984). It should be noted that this failure load could also be deduced from the ACI Code.

3. The failure pattern predicted by NFAP indicates that multiple cracking will develop in some of the elements of the mesh, prior to failure of the beam.

4. The results from NASTRAN indicate no failure even at a load of 32 kips where the run was terminated.

5. The predicted crack pattern for this NASTRAN run (Sept. 84) did not indicate any multiple cracking. In fact, the brief review of the output from all four sample problems presented by EBASCO did not show any mulitple crack patterns.

At this stage, a second meeting was held between BNL and EBASCO to discuss these comparisons. This meeting was attended by R. Iotti and H. Chang who represented EBASCO/G&H and S. Sharma and M. Reich from BNL. The outcome of this meeting was that EBASCO would check the NASTRAN results and screen the problem to validate the EBASCO predicted results. At a subsequent meeting

(November 5, 1984) at BNL, the results of the new computer runs for (EBASCO CA29) were presented by H. Chang. The written report given to BNL is attached to Appendix 1 of this paper. These results can be summarized as follows:

1. EBASCO was able to show a failure load of 24 kips (comparable to test data) only by assuming a tensile strength of 120 psi, which is four times lower than the strength that would be normally assumed for the concrete. (It should be noted that in the September run, the concrete tensile strength used in NASTRAN was 546 psi.)

2. The EBASCO results show that failure is associated with shear failure accompanied by very large displacements. This is contrary to the results of other studies, including the BNL/NFAP results, as well as experiments. Diagonal tension failure in concrete is typically a brittle failure at normal displacements.

3. The results of the EBASCO runs indicate that the assumed value of the shear retention factor has no impact on the computed failure loads (this factor was varied from 0.2 to 0.4). This is not supported from studies reported in the literature, nor from the BNL/NFAP results. An increase in of shear retention factor from 0.2 to 0.4 should lead to an increase in ultimate capacity of the beam of about 20%.

4. The multiple crack pattern shown by EBASCO for this new run indicates cracks orthogonal to each other. This result would not normally be expected and needs further explanation.

On the basis of these results, BNL concludes that the EBASCO formulation of the concrete cracking model as implemented in their NASTRAN version leads to incorrect results for this verfication problem. Therefore, the adequacy of the results from the code for the steam generator compartment cracking prediction are considered questionable.

EVALUATION OF UPPER LATERAL SUPPORT BEAMS (ULRB)

The four upper lateral restraint beams in the steam generator compartments are subjected to high thermal loads especially for the Main Steam Line Break (MSLB) case. G & H was requested to provide calculations showing that an upper bound estimate of the axial and bending stresses in the ULRB would still comply with AISC-1.6-1a requirements. These computations were included in the G & H Letter Report GTN-69363 mentioned previously. A review of the data,however, shows that the stresses were calculated for a peak temperature of 282°F which corresponds to a LOCA temperature loading. For the MSLB, the peak temperature reaches a value of 355°F according to G & H and EBASCO. For this latter case BNL carried out computations assuming rigid walls and found that the upper bound estimate (for the case when the beam is assumed fixed at both ends) of stresses would not be in compliance with AISC-1.6-1a. It should however be noted, that this assumption is unrealistically conservative and if reasonable if conservative assumptions are made concerning the concrete wall flexibility, the stresses in the beam will be reduced significantly and they then satisfy AISC-1.6-1a. A sample calculation is provided in Appendix 2. These calculations show, that even if only through thickness compressive flexibility of the wall is considered, the beam stresses would reduce to allowable levels. In reality the walls are even more flexible because of bending and shear deformations, and thus, the beam stresses will be substantially lower.

In all of the discussed calculations above, the assumptions were that the beam is rigidly held by the concrete walls, i.e., no end rotations are possible. This condition would change if the walls crack around the beam end supports. The beams could then be considered as simply supported instead of fixed. The net effect of this condition would be a further reduction in the thermal stresses. However, for the case of seismic loads this change of end fixity would increase the bending stresses by approximately a factor of two. This is due to the fact that the maximum bending moment in a centrally loaded beam with simply-supported ends is twice that of a beam with fixed ends. BNL, therefore, evaluated the adequacy of the ULRB under seismic laods assuming simply supported conditions. Both, the deflections as well as the stresses in the beam were found to be much below the allowable limits. It

should be noted, however, that in carrying out this latter evaluation, BNL utilized the loads provided by G & H who obtained them from Westinghouse. The modeling assumptions used by Westinghouse especially with regards to gap-impact phenomena between the steam generator and the ULSB were not evaluated.

EVALUATION OF CONCRETE WALLS

G & H and EBASCO used the modified NASTRAN results to establish the adequacy of the reinforced concrete walls for the four steam generator compartments. In particular G & H provided calculations to show that the concrete walls of the steam generator compartments satisfied ACI shear strength criteria. These calculations are however based on stresses obtained from the NASTRAN runs. Since BNL had questions pertaining to the adequacy of the modified NASTRAN code, especially with regards to the shear failure mode, it was decided to evaluate the walls using simplified engineering concepts.

In performing these evaluations, flexibility calculation for the walls were made assuming that the walls deformed as plates fixed along the intersection lines with supporting, floors or walls. The reactor cavity walls were found to be much more flexible than the outer compartment walls which are stiffened by various intersecting floors. Because of this flexibility the interior wall will deflect and substantially reduce the axial thermal thrust induced by the ULSB. Based on the reduced thrust it was however still found that the reactor cavity walls undergo substantial flexural and shear (punching type) cracking. This type of failure is not likely to occur at the outer walls of compartments 1, 2 and 3, because of the intersecting floors (although some cracking of the concrete will probably also occur there). In compartment 4, the intersecting floors are outside of the punching shear, zone, and hence, cracking cannot be precluded there. It should be noted that thermal stresses are self limiting and consequently once this failure occurs, the beam expands axially, and the total thrust is corresponding reduced. Actually, the beam only has to expand approximately 1/3 of an inch to completely relieve its stresses. It is therefore judged that the cracking will be of a localized nature and will not significantly affect the overall structural integrity of the compartment walls.

Because of the localized stress and deformation condition around the beam supports, the structural response away from the supports should not be affected. Thus, with respect to the question pertaining to missing rebars at a much lower elevation, it is BNL judgement that the missing rebars will have negligible effect on the evaluations carried out for this report.

ANCILLARY INVESTIGATIONS

In addition to the above items, several other questions were raised during the audit to which G & H was asked to respond. These had to do with what are considered to be relatively minor problems and those which, based upon engineering judgement and experience, could be relatively easily quantified.

1. G & H was asked to provide calculations to show that in fact the concrete walls of the compartment are stiff enough such that the pressure rise time of LOCA loading is slow enough to eliminate dynamic effects. Such was in fact the case, with a dynamic load factor calculated to be about 1.0.

2. Calculations were requested to show that local buckling and torsional loading effects on the upper lateral restraint beam do not lead to a reduction in stress allowables. They do not. It should be noted that the stress resultants used for these calculations were taken from the EBASCO/NASTRAN nonlinear output and as such may be in error. The moment resultants shown to us were small (on the order of 4% of the allowable for the MSLB case). Thus, even if they are in error, they should not influence the overall conclusions.

3. Additional calculations were requested to show that stresses in the anchor bolts and adjacent reinforcing rods at the beam-wall support junction are of no concern. These calculations are included in the referenced G & H document (sheets 64-69) and are satisfactory.

4. The same peak temperatures were used in the upper and lower restraint beams in the FEM analyses of the compartment. G & H was asked to estimate the magnitude of stresses that could be developed in the walls from the small differences in temperatures of the two beams that could develop during LOCA and Main Steam Break loadings. The approximate analyses performed are felt to be reasonable, and the results indicate that the effects of the differential temperatures are small.

CONCLUSIONS

Based on the discussions above BNL has arrived at the following conclusions:

1. The ULRB can satisfactorily support the high thermal loads developed the LOCA and MSLB. Since the other load components are small, the ULRB is considered satisfactory.

2. The calculations presented for the nonlinear concrete cracking of the Steam Generator Compartment are considered unverified, due to questions arising from the sample problem CA29 output from the modified EBASCO/NASTRAN Code.

3. BNL has investigated the effects of the ULRB axial loads developed during MSLB and it was found that significant shear and moment cracking will develop in the concrete compartment walls. It is reasonable to expect that significant cracking will occur both to the interal (i.e., reactor cavity walls as well as at the outer wall of the 4th compartment. Some cracking is also expected on the outside compartment walls of the other three compartments.

4. It is our engineering judgement that cracking will be localized around the beam supports and shoul, not significantly impair the structural performance of the walls and of the URLB.

5. The effects of the wall cracks will be to change the end fixity of of the ULRB to that of simply supported. Based on seismic loads provided by the applicant, the modification in end conditions will have little impact on the seismic response.

6. No assessment was made with respect to the effect of the cracks on radiological shielding.

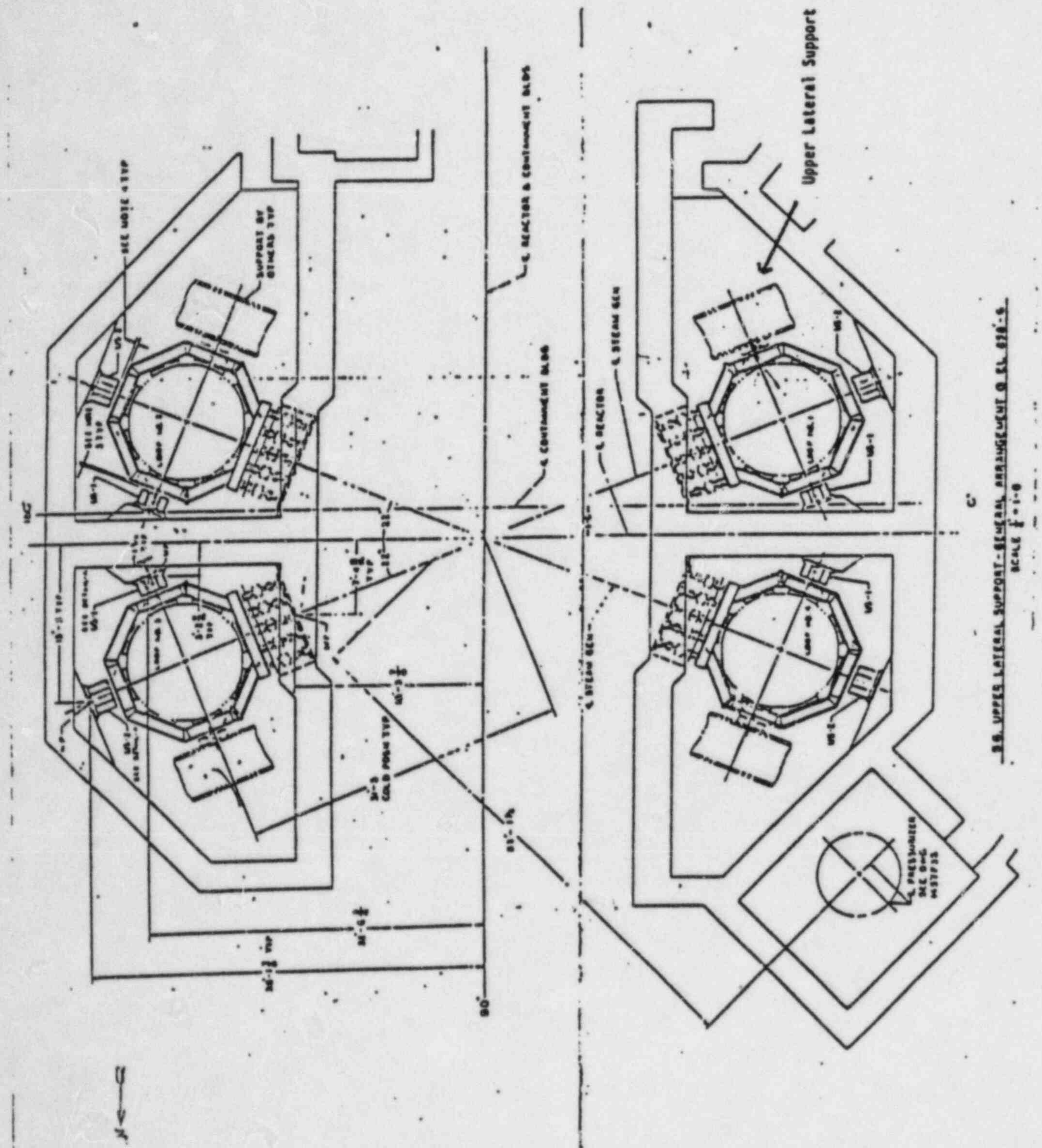


Figure 1 - Plan Detail of Steam Generator Cavities With Supports.

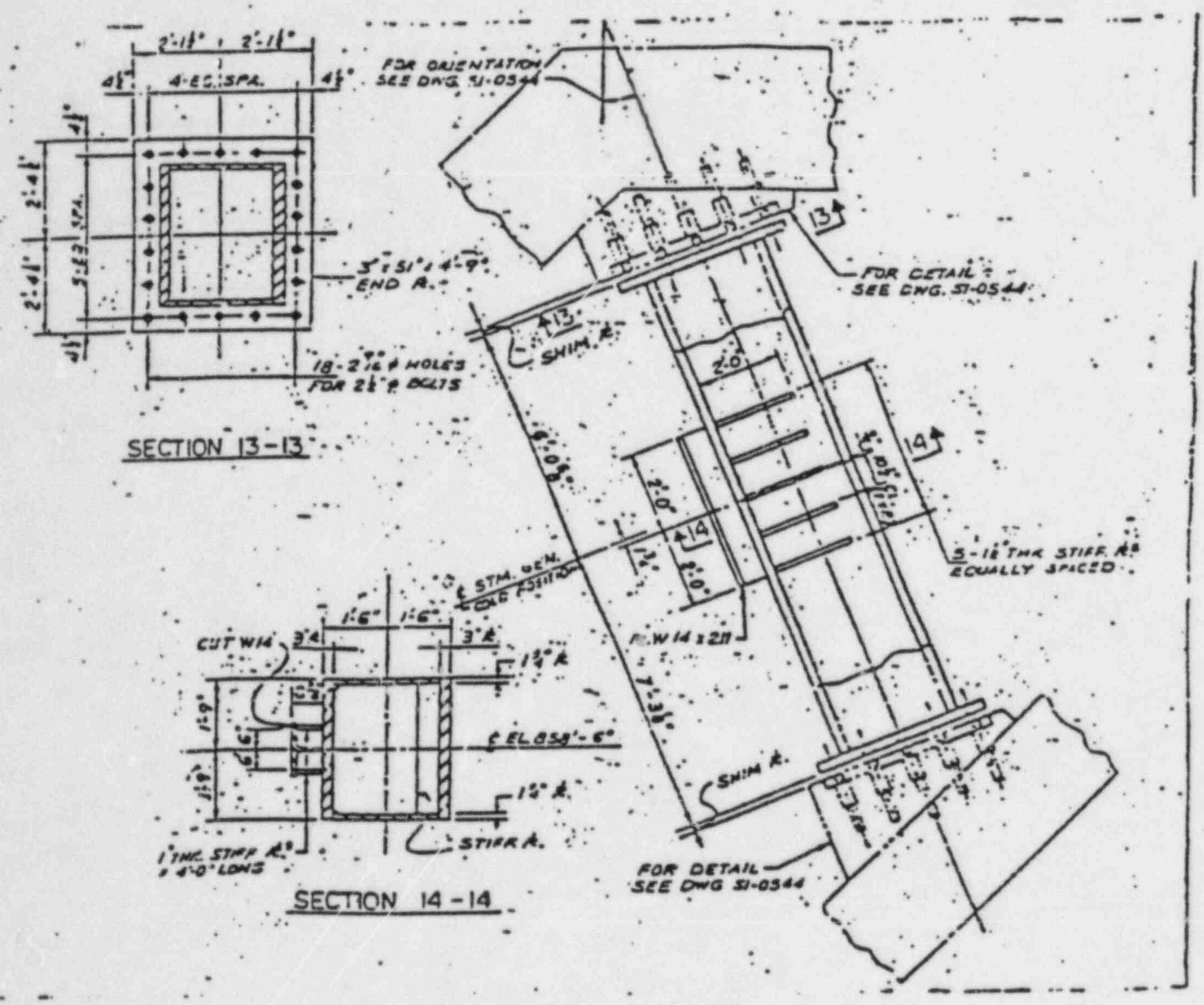


Figure 2 - Typical Steam Generator Upper Lateral Support Detail.

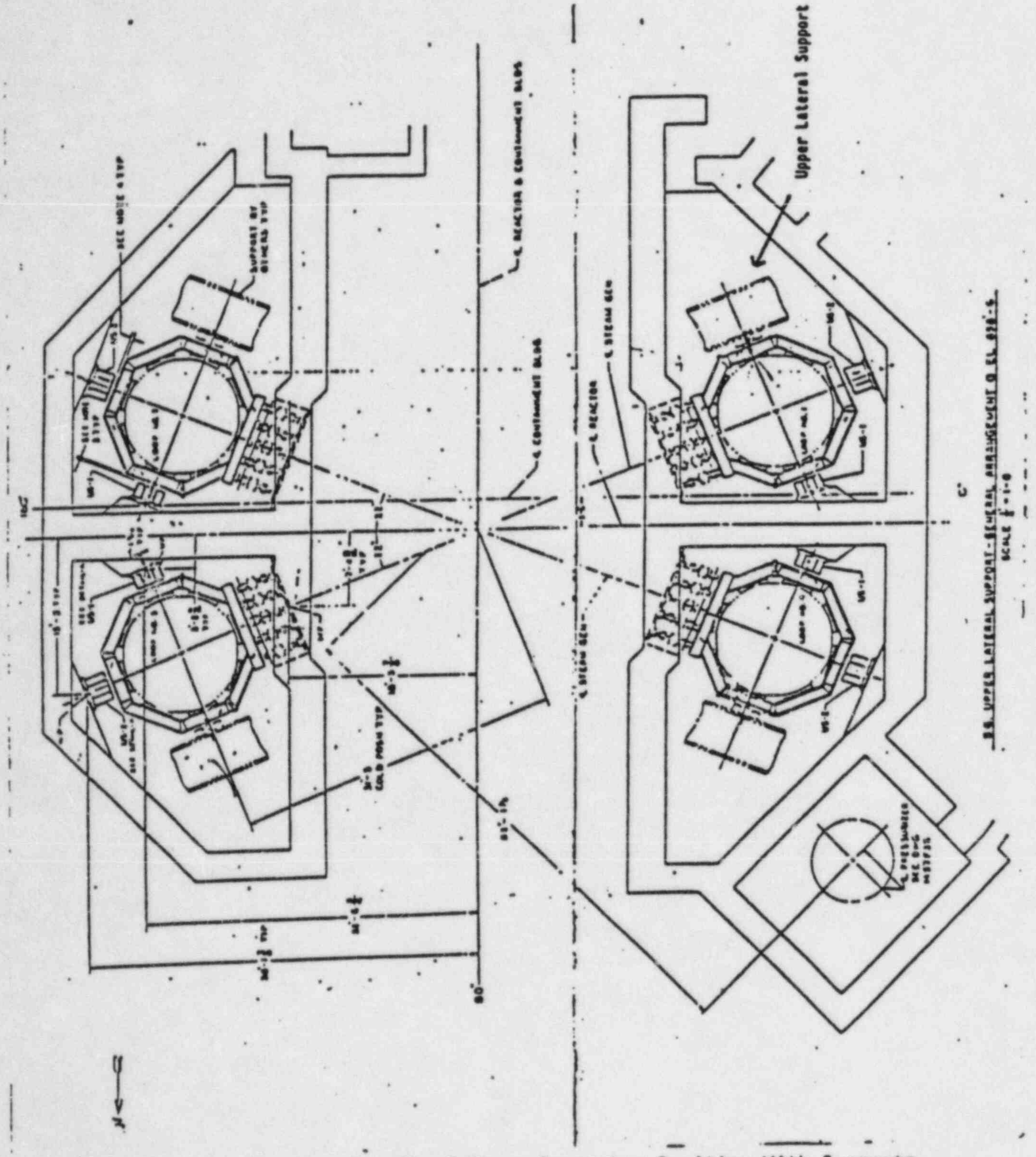


Figure 1 - Plan Detail of Steam Generator Cavities With Supports.

APPENDIX 1

VERIFICATION PROBLEM CA29
CONCRETE SHEAR FAILURE MODE STUDY

Submitted by EBASCO - November 5, 1984

INTRODUCTION

This is a continuation of verification problem CA29. Verification problem CA29 is a reinforced concrete beam fixed at both ends. Due to symmetry, half of the beam is modelled by two hundred CQDCA elements and forty CROD elements, the latter simulating the two layer reinforcements (Fig 1).

The beam is loaded at its center with a series of concentrated loads of 8, 16, 24, and 32 kips. The dimensions of the beam are 8 inches wide, 10 inches thick, and 78 inches long. The following properties of steel and concrete are used in performing the analysis.

Concrete tensile strength	= 546 psi *
Concrete elastic modulus	= 3,800,000 psi
Concrete Poisson's ratio	= 0.15
Steel elastic modulus	= 30,000,000 psi

Brookhaven indicates that this beam should have a shear failure when the concentrated load is approximately 22 kips. The shear failure is exhibited by opening up two cracks in some of the concrete elements and the vertical beam displacements become very large.

However, from our results of the analysis performed by the EBS/NASTRAN program, this phenomenon is not observed even with concentrated load at 32 kips, e.i., no concrete element has two cracks and the beam vertical displacements are still reasonable. This raises the question of whether the EBS/NASTRAN can correctly predict the shear failure mode by opening up the second crack in the concrete elements.

By reviewing the theory implemeted in the program, it is determined that the theory allows the opening of the second crack and can thus predict the shear failure mode. Brookhaven concurs the correctness of the theory. The program has been verified against many experimental data and manually computed data, hence the possibility of a coding error in the program is very small. Our review of the coding has indicated no error. Thus the difference between our results and Brookhaven's may be due to differences in parameters used.

There are three parameters in the program which will affect the shear failure mode. The first is the angle limitation on the

* This strength is varied as one of the parameters in this study.

opening of the second crack. The second is the shear interlocking factor. And the third is the tensile strength of the concrete used in the analysis. These three parameter are discussed in detail in the followings.

ANGLE LIMITATION ON THE OPENING OF THE SECOND CRACK

The program as developed is intended to be a designing tool. In designing a concrete structure, it is usually assumed that the tensile strength of the concrete is zero. In order to prevent spurious opening of a second crack and render the nonlinear iteration scheme stable, the progarm adopted a criterion that the angle between the first crack and the second crack should be larger than 31.7 degree. Otherwise, only one crack is assumed. This criterion is derived from the rule that the stress parallel to the first crack is in compression and the compressive stress is assumed to be larger than the shear stress along the first crack (see attached notes).

In order to see whether this criterion will affect the shear failure mode, CA29 was re-ran with the angle limitation criterion removed. The results of this run togather with the run for which this criterion is not removed are shown in Table 1. It is seen that the results have no difference whether the anlge limitation criterion is removed or not.

SHEAR INTERLOCKING FACTOR

When the concrete has one crack, the shear stiffness across the crack is reduced, but does not completely vanish because of the interlocking effect of concrete. The reduced shear stiffness is obtained by multiplying the original uncracked shear stiffness by a constant which is called the shear interlocking factor. The shear interlocking factor is usually taken to vary between 0.2 to 0.4. The progarm has adopted a default shear interlocking factor of 0.2.

In order to see whether the shear interlocking factor has any effect on the shear failure mode, CA29 was re-ran with a shear interlocking factor of 0.4. The results are tabulated in Table 1. It is seen that the shear interlocking constant of 0.2 gives very close results to those given by the shear interlocking constant of 0.4.

CONCRETE TENSILE STRENGTH

The concrete tensile strength originally used in CA29 is 546 psi.

The opening of the second crack is very much dependent on the concrete tensile strength used in the analysis. The second crack will open if the diagonal tension obtained from the combined effect of the stress parallel to the first crack and the shear stress across the first crack is larger than the concrete tensile strength.

In order to see the effect of the concrete tensile strength on the shear failure mode, three runs have been performed. The first run uses a concrete tensile strenth of 60 psi and four loading cases: 10, 12, 14, and 16 kips. The beam fails at the concentrated load equal to 12 kips. The failure is a shear failure mode which is exhibited by very large vertical displacements. Most of the concrete elements have two cracks. The second run uses a concrete tensile strength of 120 psi and four loading cases: 22, 24, 26, and 28 kips. This time, the beam fails at the concentrated load equal to 24 kips. The third run uses a concrete tensile strength of 180 psi and four loading cases: 30, 32, 34, and 36 kips. For this run, the beam fails at the concentrated load equal to 34 kips. The results of these three run are tabulated in Table 2.

As an example, the crack angles of the ten concrete elements near the fixed end are shown in figure 2. Please note that these crack angles occur with very large displacements. Thus the example serves only as an indication of how many cracks in the element and their approximate orientation.

CONCLUSION

From above discussion, it is seen that the effect of the angle limitation criterion and the value of the shear interlocking constant on the opening of the second crack and the shear failure is negligible.

The most important parameter in the opening of the second crack is the concrete tensile strength used in the analysis. Once the second crack opens, shear failure follows immediately. The program pridicts the shear failure by analyzing the condition in each element. From a macroscopic point of view, we can ask the question: what is the ratio of the beam nominal shear stress to the concrete tensile strength which will cause the beam to fail in shear. The program predicts a value from 1.18 to 1.25 (see Table 2), with the lower value at higher axial stresses. The theoretical value for the simplest case of pure shear is 1.0. Therefore, the prediction of the program is in consistant with the conventional beam analysis method.

From above discussion, we conclude that the program correctly pridicts the shear failure mode by opening up the second crack in the concrete. The reason we did not observe the shear failure in concrete for CA29 is that the concrete tensile strength of 546 psi used in the analysis is higher than the value of 181 psi which will cause the beam to fail at a loading of approximately 34.0 kips.

TABLE 1.

THE EFFECT OF
ANGLE LIMITATION CRITERION AND SHEAR INTERLOCKING CONSTANT

RUN NO.	LOAD kips	TENSIL STRENGTH psi	ANGLE LIMITATION CRITERION	SHEAR INTERLOCK. FACTOR	MAXIMUM DISPLACEMENT inches	MAXIMUM REINF.STRESS ksi
1	32.0	546.0	yes	0.2	0.231	28.57
2	32.0	546.0	no	0.2	0.231	28.57
3	32.0	546.0	no	0.4	0.225	28.98

Angle limitation criterion: yes means the angle between the first crack and the seccond crack must be larger than 31.7 degree. No means this angle can be any value.

TABLE 2

THE EFFECT OF
CONCRETE TENSILE STRENGTH ON SHEAR FAILURE

beam cross-sectional area A = 80 square inches

RUN NO.	TENSILE STRENGTH FT, psi	FAILURE LOAD P, kips	NOMINAL SHEAR STRESS TAU=P/A, psi	RATIO TAU / FT
1	60.0	12.0	75.0	1.25
2	120.0	24.0	150.0	1.25
3	180.0	34.0	212.5	1.18

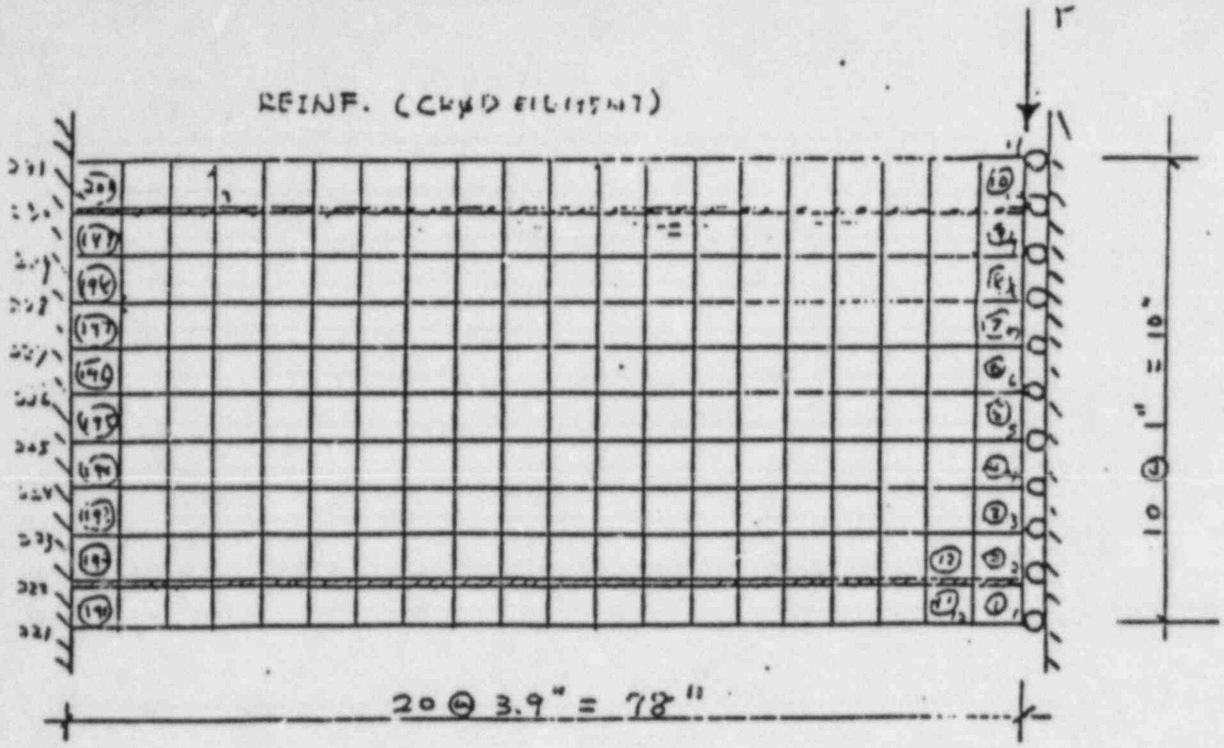


Fig. 1 CA29 - Finite Element Beam Model

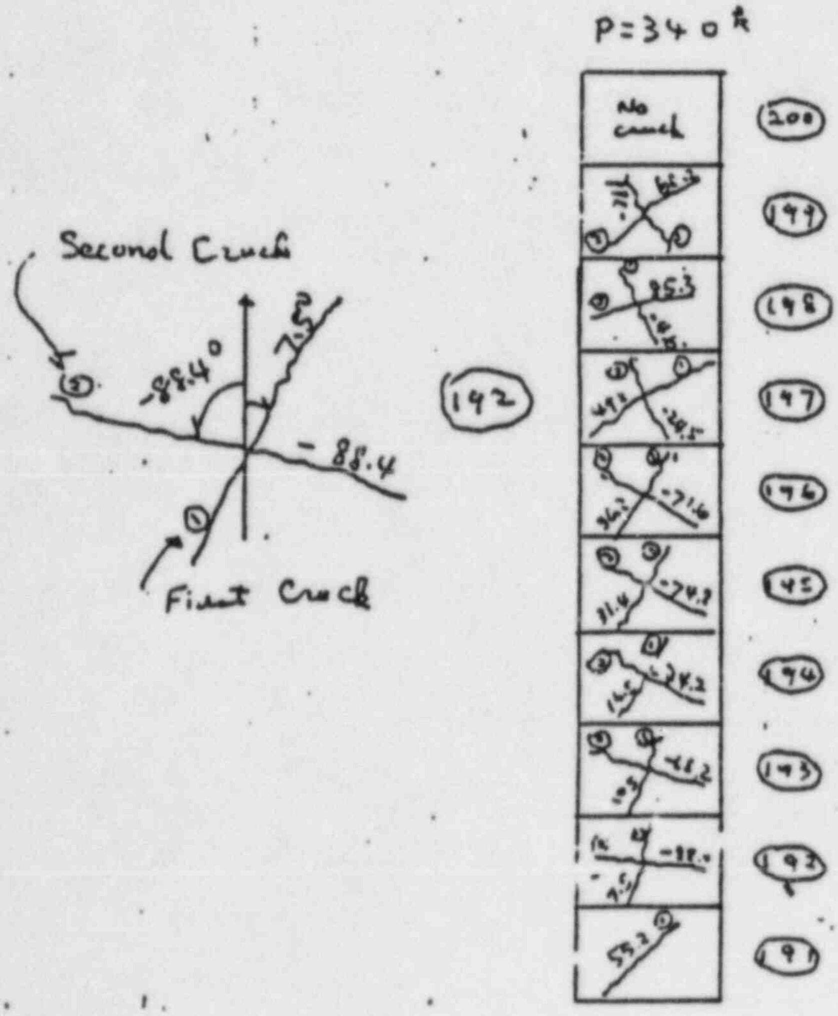


Fig. 2 Concrete cracking pattern

EBASCO SERVICES INCORPORATED

NEW YORK

BY H Cheng DATE 4/19/80 SHEET ___ OF ___

CHKD. BY [illegible] DATE 4/21/80 OFS NO. ___ DEPT. NO. ___

CLIENT ___

PROJECT ___

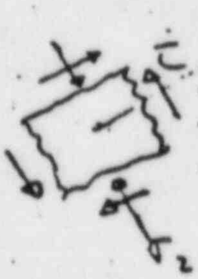
SUBJECT Cracking Analysis

Criterion for the openning of the second crack for elements with one existing cracking

θ_1 – cranking angle for the first crack

compute the principle angle & stress for the second crack

θ_{p2} & σ_{p2}

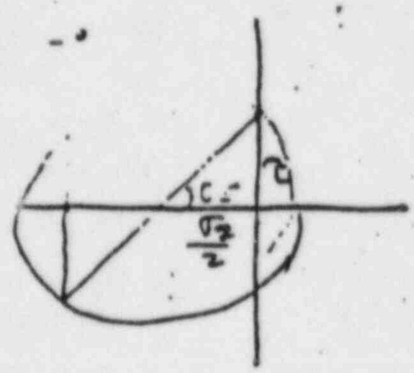


The criterion for second crack to open will be

$(\sigma_{p2} > \sigma_0)$ and $|\theta_{p2} - \theta_1| > \theta_0$

θ_0 can be obtained by condition

$(\sigma_2 < 0)$ & $|\sigma_2| \geq \tau$



$\theta_0 = \frac{1}{2} \tan^{-1} \frac{\tau}{\sigma_2/2} = \frac{1}{2} \tan^{-1} 2 = 31.717475°$

FORM 551 REV 7-75

APPENDIX 2

For the case involving the main steam break the maximum compressive load with ends restraint would be

$$P = A\alpha(\Delta t)E$$
$$= 357 \times 6.632 \times 10^{-6} \times (340-70) \times 28,000$$
$$= 17912 \text{ kips.}$$

If the wall were perfectly rigid, this would amount to an axial stress

$$f_a = P/A = \frac{17912}{357} = 50.17 \text{ ksi.}$$

The allowble (F_a) for this beam has been calcualted (See sheet No. 56 attachment to GIN-69363, August 21, 1984) to be 45 ksi. Thus, under this assumption the upper lateral beam would not satisfy AISC 1.6-1a requirement.

The assumption of completely rigid wall is, however, unrealistically conservative. The walls in reality are flexible members and thus will move due to the axial thrust.

In order the assess the safety of the upper lateral support under conservative estimates but without going through a detailed complex model, BNL has performed the following calculations based on the assumptions given below:

(1) Walls at outer surfaces are fixed and are 6 feet thick. This thickness assumption is based on the fact that only the area adjacent to the beam support is modeled for the analysis.

(2) The axial force due to thermal expansion acts perpendicular to the wall.

(3) No in-plane displacement is allowed in the walls. This further increases their stiffness by a factor "ϕ" equal to 1.2 which is caused by the Poisson effect.

The specific calculation are performed in an interative manner as follows:

<u>Initial Axial Force</u>

$$P = 17912 \text{ kips}$$

Assuming conservatively that this force is distributed uniformly at the base plate (area 65" x 66") - wall intersection, the compressive stress in the concrete is

$$\sigma_s = \frac{17912}{65x66} = 4.175 \text{ ksi}$$

The compressive strength f_c' for the concrete is 5.0 ksi. Then, σ_c corresponds to (see Fig. 1) a uniaxial strain $\epsilon = 0.00125$. Thus the concrete strain (including the Poisson stiffening effect ϵ_c is,

$$\epsilon_c = \frac{.00125}{1.2} = .001 \quad ,$$

ϵ_c is assumed to be distributed linearly through the walls thickness. Therefore, the wall will yield by an amount δ, given by

$$\delta = .001 \times \frac{72}{2} = 0.036 \text{ in.}$$

This will reduce the axial force in the beam by an amount $R = 2\delta \times \frac{A \times E}{L}$

$$= 0.72 \times \frac{357 \times 28000}{163.2}$$

$$= 4410 \text{ kips}$$

Modified Upper Lateral Support Loads

The new beam axial force $P' = P - R = 17912 - 4410$
$= 13502$ kips,

and the compressive stress in the concrete $\sigma_c' = \frac{13502}{65 \times 66} = 3.147$ ksi.

This corresponds to a compressive strain $\varepsilon_c' = 0.00085/1.2 = 0.00071$.

The wall will thus yield, $\delta' = .00071 \times \frac{72}{2} = 0.0255$ in.

Axial force reduction $R' = 2 \times 0.0255 \times \frac{357 \times 28000}{163.2} = 3124$ kips.

Repeating the above iterative procedure, we finally arrive at a upper lateral beam axial force = 14000 kips.

The final axial stress f_a is $P/A = \frac{14000}{357} = 39.2$ ksi. From CL 1.6 - 1a of AISC manual

$$f_a/F_a + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}'}\right) F_{bx}} + \frac{C_{mx} f_{by}}{\left(1 - \frac{f_a}{F_{ey}'}\right) F_{by}} \leq 1.0$$

$F_a/F = \frac{39.2}{45}$, while the other two factor in the above equation are given in sheet No. 60 (attachment to GIN-69363, August 21, 1984) as 0.015 and 0.033 respectively. Thus the left hand sum is .918 <1.

Since the above is obtained by conservative assumptions, it is the BNL opinion that upper lateral restraint beam will satisfy the AISC code requirements.

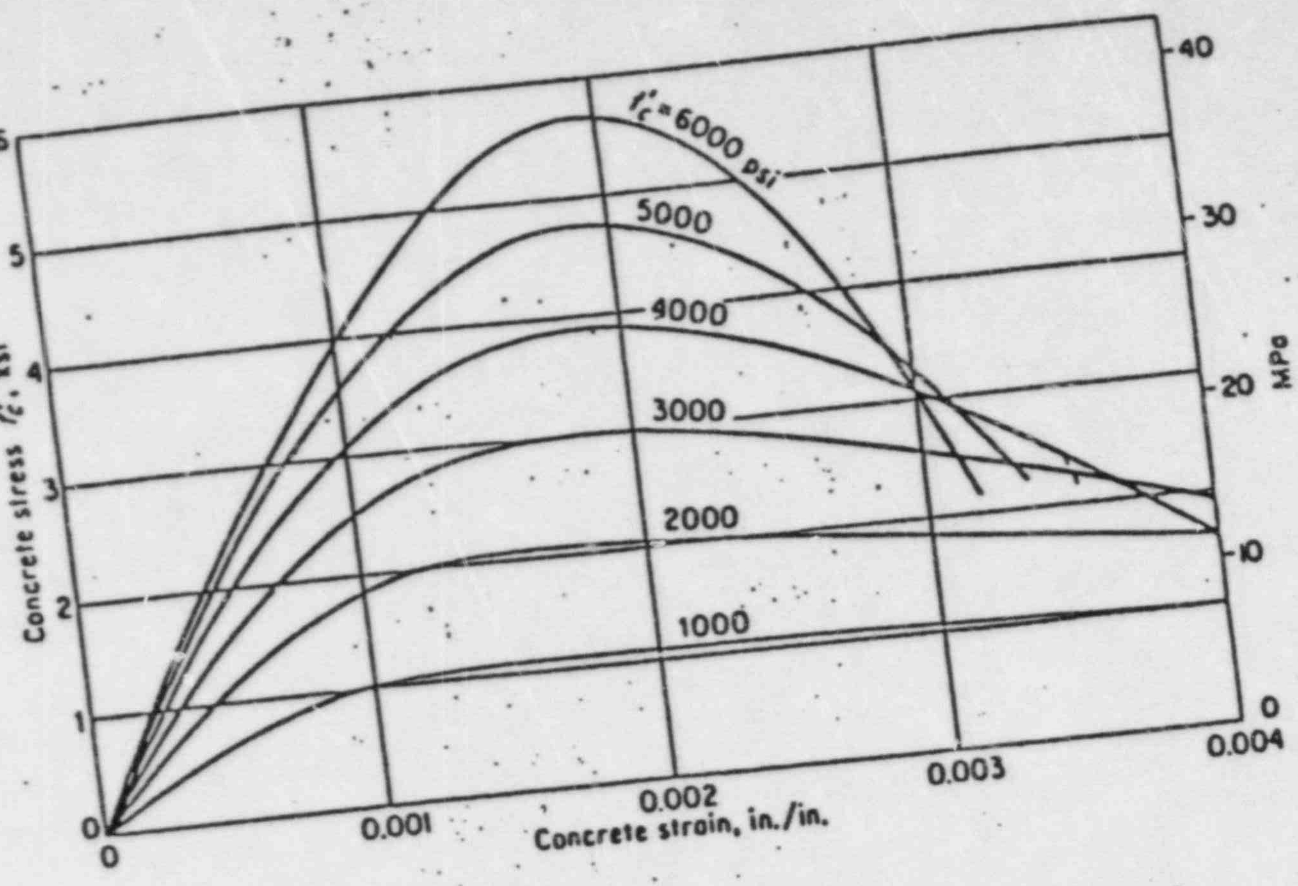


FIGURE 1. Typical Concrete Stress-Strain Curves in Compression.[1]

CASE ATTACHMENT E

- CASE EXHIBIT NO. 1,035

SIS REPORT

THE HARTFORD STEAM BOILER INSPECTION and IN[illegible]RANCE COMPANY
HARTFORD, CONNECTICUT 06102

G-044

TO: Gordon Purdy Q. A. Manager	DATE 5/26/83	SHEET 1	OF 2
FROM: Marvin Coats, Lead ANI	H.O./BRANCH OFFICE Houston		

ORGANIZATION: Brown & Root, Inc.

LOCATION	STREET	CITY	COUNTY	STATE	ZIP CODE
CPSES		Glen Rose	Somervell	Texas	76043

PERSON CONTACTED (GIVE NAME AND OFFICIAL TITLE):

CONTRACT/P.O. NO. BS 042007

REASON FOR VISIT: Full time contract

COPIES SENT TO: ☐ H.O. Eng Claim, SIS ☐ Chief Inspector ☒ Regional Manager, SIS ☒ Other (Specify): ANI file.

Subject: Component Supports

Re: Meeting 5/24/83 B. Baker, G. Purdy, M. Coats, B. Walker.

At the referenced meeting we discussed several ANI concerns about the present methods in place to identify problems with supports and subsequent rework or repair to resolve those problems. Per request, I am documenting those concerns and your proposed remedial action as I understand them.

(1) Brown & Root Q. A. has recognized generic deficiencies in support fabrication and subsequent inspection (e.g. undersized fillet welds). Corrective action has been implemented procedurally in CP/QAP 12.1 which dictates a final "walkdown" of each support by QC to verify configuration, weld size, pipe to hanger clearance, etc. Final hanger package review by QES and ANI is predicated on this documented re-inspection. This final inspection has resulted in thousands of NCR's which causes duplication of walkdowns and a loss of perspective in NCR processing. In view of above, Brown & Root has adopted a policy of Welding Engineering personnel inspecting supports to final drawings prior to the final Q C inspection. Noted discrepancies are worked on process sheets rather than identified on NCR's based on a rationale that the support is still in process. This is an effort to reduce

☐ OVER

SIGNED

initiation of NCR's and better as. .re that Q C will perform final inspections on acceptable fabrication. This policy is understandable but is not supported by the content of Section 16 of the Q. A. Manual.

(2) Repair Process Sheets generated to build up undersize welds are being transmitted to craft with an information copy of the vendor certified drawing. Even though the RPS virtually stands alone and the drawing serves only to provide location & material information Section 7 of the Q. A. M. specifically precludes use of an uncontrolled drawing for fabrication and installation activities.

(3) Full fillet on Class 1 support primary members should be identified in process and not left to be identified during the final walkdown.

Your proposed action of the above is as follows:

(1) Prepare a Q. A. M. revision for submittal to the ANIS to provide for policy outlined in item 1 above.

(2) R.P.S.'s will be issued with controlled drawing attached.

(3) Q C I identification of full fillet welds will be proceduralized to assure implementation.

Your assistance in resolving the above is appreciated.

M Coto

CASE ATTACHMENT F

Rcd. 11/27/85

CYGNA
ENERGY SERVICES
101 California Street, Suite 1000, San Francisco, CA 94111-5894 415 397-5600

November 26, 1985
84056.095

Mr. W.G. Counsil
Executive Vice President
Texas Utilities Generating Company
Skyway Tower
400 North Olive Street, L.B. 81
Dallas, Texas 75201

Subject: Review Issues List (RIL)
Texas Utilities Generating Company
Comanche Peak Steam Electric Station
Independent Assessment Program - All Phases

References: See Attachment A

Dear Mr. Council:

Enclosed are revisions to the mechanical systems, electrical/I&C, cable tray supports, conduit supports, pipe supports, and design control Review Issues Lists (RILs). All significant changes are noted by a revision bar in the right margin. Most of the revisions were made to provide changes in accordance with information requests made in the open items letters (References 2, 3, 5, and 6) and questions or comments on the CPRT plan (References 1 and 4). The cable tray supports RIL was also revised to provide clarification and expansion of existing issues. A similar revision is in progress for the pipe stress RIL. The current revisions to each discipline RIL are as follows:

Discipline	Revisions	Cygna letter reference
Pipe Stress	1	84056.093
Pipe Supports	2	84056.092
Mechanical Systems	3	84056.088
Electrical/I&C	3	84056.090
Cable Tray Supports	12	84056.094
Conduit Supports	3	84056.094
Design Control	2	84056.085

ENERGY
SERVICES

If there are any questions please call at your convenience.

Very truly yours,

N.H. Williams

N.H. Williams
Project Manager

Attachments

cc: Mr. V. Noonan (USNRC) w/attachments
Ms. A. Vietti-Cook (USNRC) w/attachments
Mr. S. Treby (USNRC) w/attachments
Mr. W. Horin (Bishop, Liberman, et al.) w/attachments
Mr. J. Redding (TUGCO) w/attachments
Mr. J. Finneran (TUGCO) w/attachments
[redacted]
Mr. D. Pigott (Orrick, Herrington & Sutcliffe) w/attachments
Mr. F. Dougherty (TENERA) w/attachments
Mr. R. Ballard (Gibbs & Hill) w/attachments
Mr. R. Kissinger (TUGCO) w/attachments
Mr. J. Beck (TUGCO) w/attachments

PIPE SUPPORTS
Review Issues List

Summary: In performing the pipe support design review for Phases 2, 3 and 4, Cygna utilized certain engineering standards from ITT Grinnell, NPSI, and PSE when they were referenced in a particular calculation. Cygna did not review all the guidelines or standards from each organization and has returned those that were used.

Status: In order to complete our design process reviews, Cygna requests a controlled copy of the pipe support engineering guidelines/standards from ITT Grinnel, NPSI, and PSE.

28. Use of A563 Grade A Nuts With High Strength Bolting

References:
1. Communication Report between Rencher (TUGCO) and Minichiello (Cygna) dated 3/16/84, Item 1.
2. L. M. Poppelwell (TUGCO) letter to N. H. Williams (Cygna) dated 4/19/84, Item 1.

Summary: ASTM specification A563 recommends that Grade A nuts be used with A307 (low strength) bolting. However, as noted by TUGCO, their designers, when not using high strength nuts, will specify double nuts, with both nuts snugged. Cygna's scope of review confirmed this statement.

Status: This issue is closed.

29. Friction Loads

References:
1. Cygna Phase 3 Final Report, TR-84042-01, Revision 1, Appendix G., Pipe Support Observation PS-08.
2. Juanita Ellis (CASE) Letter to Administrative Judge P.B. Bloch (ASLB) dated 6/13/85. "Further Clarification of CASE's Position Regarding Applicants' Use of 3 Sm".

Summary: Loads due to friction were not included in the support design of pipe supports at CPSES when the piping thermal movement was 1/16" or less.

Status: Open, further TUGCO response is required. The observation on the omission of friction loads in pipe support design

2

PIPE SUPPORTS
Review Issues List

(PS-08) with small thermal movements was previously concluded as invalid based on the considerations of industry practice, the TUGCO sample reanalysis, and the factors of safety available for normal conditions. This conclusion was reached with the due consideration of the 1974 Edition of the ASME Code as the Code of Record. However, the latest edition of the ASME Code--namely, the 1983 Edition, paragraph NF-3121.2--significantly changed the definition of primary stress due to constrained free end displacement and appropriate allowable stresses. (See Table NF-3523 (b)-1.)

The code change has significantly altered some of the technical points upon which Cygna relied to invalidate the observation. Consequently, Cygna requests further response from TUGCO and considers this issue open.

However, it should be noted that Cygna did review fifteen (15) pipe support calculations within Cygna's Phase 3 and 4 scope, which had negleted the effect of friction load. Cygna found that all the supports are acceptable with the inclusion of friction loads in combination with the original design loads. However, load changes due to the consideration of Mass Participation effects (Review Issue 5) may affect those conclusions.

30. MS-1-003-007-C72K, Revision 10

Reference:

1. N.H Williams (Cygna) letter to J.B. George (TUGCO), 84056-013 dated 7/31/84. "Pipe Support Review Questions", Question No. 10.
2. L.M. Popplewell (TUGCO) letter to N.H. Williams (Cygna), dated 8/30/84.
3. L.M. Popplewell (TUGCO) letter to N.H. Williams (Cygna), dated 9/17/84.
4. Communications Report between Van Amerogen/Rencher/Kerlin (TUGCO) and Minichiello (Cygna) dated 9/11/84. Item No. 1.

Summary: Due to insufficient dimensioning in the subject drawing (Section J-J), Cygna has concerns about the design of the connection and particularly about the plate stresses of