UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of APPLICATION OF TEXAS UTILITIES GENERATING COMPANY, ET AL. FOR AN OPERATING LICENSE FOR COMANCHE PEAK STEAM ELECTRIC STATION UNITS #1 AND #2 (CPSES)	Docket Nos. 50-445 and 50-446

AFFIDAVIT OF MARK A. WALSH

Q: Mr. Walsh, do you have any comments in regards to the Applicants' quality assurance program as discussed in the Applicants' Motion for Reconsideration of Memorandum and Order (Quality Assurance for Design)?

A: Yes, I do.

The first item I would like to address is Applicants' Attachment A, "Summary of Quality Assurance Program for Design of Pipe Supports for Comanche Peak Steam Electric Station."

As I will discuss later, the program they have cited is only on paper. The procedures indicated in their Summary may be implemented at other sites, but not at Comanche Peak (at least while I was there). What is most important is the testimony provided by the Applicants' witnesses in regards to the Walsh/Doyle allegations, as summarized in CASE's 8/22/83 Proposed Findings of Fact and Conclusions of Law (Walsh/Doyle Allegations) and in the Board's 12/28/83 Memorandum and Order (Quality Assurance for Design), along with other information already in the record.

To me, the <u>reality</u> of Applicants' attitude toward 10 CFR, ANSI, and ASME is demonstrated through NCR M-1802 and its Revision 1 (accepted into the record as CASE Exhibits 498 and 497, respectively), dated 11/5/79 and reported by J. Patton. As shown in block (3) of the original NCR, the "Document Violated" was 10CFR50 App. B Pt. V & VIII." On the revision, block (3) indicates the "Document Violated" was "ASME Section III." There are several other changes which were made in the original and Revision 1 of this NCR (for example, block (9), Review/Approval, was signed by two different individuals on two different dates, as indicated in block (10)).

Of particular concern to me and CASE is the Summary (on the second page of the document). The Summary Mr. Patton wrote on the original NCR is as follows:

> "Exhibits I & II attached show loss of control and failure to conform to sections V and VIII of 10CFR50, Appendix B by both craft and Engineering personnel. The drawing control and design change controls as specified in site Procedures has been neglected such that both these documentation packages cannot support these applicable regulatory requirements." (Emphasis added.)

Referring to Revision 1 of the NCR, one will notice that there is no Summary.

This is important because the 12/28/83 Board Order cited the same general issue, Applicants' noncompliance with 10 CFR Part 50, Appendix B, as was cited in the original NCR's Summary. The Applicants have, by their convenient handling of this NCR, dismissed the 10 CFR Part 50 violation, by revising it as shown in Revision 1 of the NCR. The reason for the revision is not given. Rather than adding to the original NCR, the original information was changed. The original NCR was not dispositioned, but the Summary was deleted and the document violated as well as item II.B. was changed (again without explanation). This technique of modifying NCR's will not aid the NRC or the Board in regards to the trending of deficient items or of violations of 10 CFR. Even if the Applicants were now able to explain the reason for a revision to the initial NCR, the explanation from the trending standpoint would be meaningless.

A confusing point shown in these two NCR's is item II.B. In the original NCR, it states "Hole spacing has been changed on the ABRF" (emphasis added); on the revised NCR, item II.B. states "Hilti bolt size has been changed on the ABRF" (emphasis added). No reason is given for this change.

The Disposition of NCR M-1802 R1 (pages 3 and 4 of 28) reveals more information in regards to the procedures utilized by the Applicants regarding nonconforming conditions. Item ID. reveals the problems of who is responsible, for what, and when. On page 15 of 28, the title block of the drawing states "ITT Grinnell, Pipe Hanger Division" (emphasis added). Disposition of item ID. states ". . . welds No. 3 thru 9 are vendor shop welds." (Emphasis added.) On sheet 11 of 28, a request for change is by NPSI fabricator. On page 12 of 28, the reason for change is "can not be fabricated as detailed." The request came from Hal Goodson of the Hangers Department. On page 13 of 28, the CMC is requested by Hal Goodson of the Hanger Eng. Dept. It is my understanding that Hal Goodson was not an engineer and that he was at that time superintendent of the steel fabrication and hanger department. In addition, the disposition of this NCR was accepted by Gibbs & Hill, Inc. to "use as is" as shown on page 5 of 28; this appears to be Gibbs & Hill approval of the method of design and fabrication by ITT Grinnell, NPSI, and/or Brown & Root.

It is a standard industry practice for field engineers to make minor changes to construction documents, and later (less than 90 days) to have those changes approved or disapproved by their home office and if acceptable, those changes are then incorporated on the original drawing. Also, it is an industry practice for field engineers to request changes, and resident engineers (engineers from the home office who have access to the original calculations) to approve changes or disapprove changes and incorporate changes into original calculations prior to construction of the field change. It is not industry practice for construction to request a change in a design document, and have a field engineer, who has no access to the original calculations, approve such a change, as is the case at Comanche Peak.

The abuse of CMC's has the consequence of avoiding the proper trending and prompt correction of deficiencies, as is further shown when one observes CMC 17671, page 18 of 28 of NCR M-1802 R1. This CMC was generated by the Hanger Dept. The reason for the change is "misfabrication by NPS." (Emphasis added.)

The abuse of CMC's is compounded when one looks at the approval box. On page 11 of 28, for example, the approval came from G. M. Chamberlain. It is my understanding that Mr. Chamberlain had no previous engineering experience prior to Comanche Peak or the proper qualifications for an engineer.

It is not apparent from the information contained in the NCR who or what organization provided the material and who welded the material together, since the initial drawing was an ITT Grinnell drawing, a CMC claimed that NPSI had misfabricated it, yet a Brown & Root superintendent of the fab shop initiated another CMC -- all against the same support.

It appears that the original NCR was correct that the Applicants were in violation of 10 CFR Part 50, Appendix B.

It also appears that the NRC Senior Resident Inspector at Comanche Peak, NRC Staff witness Robert Taylor, was aware of the construct/design philosophy at Comanche Peak and was concerned about it, as indicated in his 1979 Trend Analysis for Comanche Peak (NRC Staff Exhibits 192-195) and in his supplemental testimony for the June 1982 hearings (NRC Staff Exhibit 180, Supplemental Testimony of William A. Crossman, Robert C. Stewart and Robert G. Taylor Regarding Annual Assessments of Applicants' Performance (Contention 5)). On page 16 of his Testimony, Mr. Taylor stated, in part:

> "What I had begun to see, but had difficulty proving, was that the Brown & Root construction philosophy was to build something any way they wanted to and then leave it up to the engineer to document and approve the as-built condition. If the engineer refused, he was blamed for being too conservative and not responsive to the client's needs."

And on page 17, he stated, in part:

> "In a couple of cases, I had been able to show them that their people were essentially incompetent even though they had been through the site training and had been certified as competent. I saw a desire on the part of the licensee to turn this situation around in the important areas of electrical and piping installation. However, too often an installation was clearly constructed other than as originally designed and had been approved by the licensee's on-site engineering arm as fulfilling requirements. In effect, the engineer had approved a nonconforming installation in advance of QC being called. QC was then signing for the as-built condition and the underlying problem was not addressed." (Emphases added.)

It is evident from the preceding that Mr. Taylor was aware that this had been, and probably still was, the practice at Comanche Peak as far back as 1979; however, this is contrary to the impression he gave during the May 1983 hearings.

The preceding is important also in that it clearly indicates that this had been the practice at Comanche Peak not just regarding pipe supports, but "in the important areas of electrical and piping installation" (emphasis added) as well. Jack Doyle and I have not addressed these areas, but there is every reason to believe that problems of the same magnitude exist in those areas as exist regarding pipe supports.

It should also be noted that the Applicants' FSAR Section 17, at page 17.1-39, at the time of the Walsh/Doyle allegations, stated:

> "A nonconformance report is utilized for the identification, documentation, dispositioning, and verification of deficiencies in characteristics, documentation, or procedures which render the quality of an item unacceptable or indeterminate."

And on the same page, it will be noted, it states that:

> "Procedures require 'trending' of nonconformance and deficiency reports to identify trends adverse to quality."
>
> (See FSAR page 17.1-39, May 31, 1979, attached to CASE's Motion to Supplement the Record (In Regard to Walsh/Doyle Allegations).)

It does not state that trending is done on CMC's.

On July 11, 1983, Applicants changed their FSAR to document in writing that NCR's no longer have to be used for all nonconforming conditions. FSAR page 17.1-39, Amendment 41, July 11, 1983 (attached to CASE's Motion to Supplement the Record (In Regard to Walsh/Doyle Allegations) states:

> "Procedures require 'trending' of deficiencies identified on nonconformance reports, deficiency reports, and inspection reports to identify trends adverse to quality."

It does not state that trending is done on CMC's.

Q: On page 26 of their pleading, Applicants address the role that you and Mr. Doyle fulfilled at Comanche Peak. Do you have any comments regarding this?

A: Yes. I believe the perception provided by the Applicants, while partially correct, is deceiving. While Jack Doyle and I may have analyzed 1084 individual supports, it appears that Applicants are trying to show that we only identified "a few design deficiencies in supports" (page 26, second full paragraph). Although this may be the Applicants' perception, it is far from the truth. When I testified back in July of 1982, I had only one day of preparation, and I did not recall all the problems at Comanche Peak. Since then I have recalled other deficiencies, which have not been addressed before.

The first such item is the reduction in sectional properties in a tube steel member at a Richmond insert. To anchor the tube steel with a threaded rod to the Richmond insert, a hole is drilled through the tube steel. This hole has decreased the section properties of the tube steel section at the connection. I informed NPSI of this condition while I was at Comanche Peak, but to the best of my knowledge and belief, the -eduction of tube steel properties was not considered in the design process.

To illustrate, referring to Attachment A to my affidavit, which I have prepared, Figure 1 is the cross-section of a tube steel section without the hole drilled in it. Its properties for a TS 4x4x1/2 are: Area = 6.14 in.^{2}; moment of inertia = 11.4 in.^{4}; and section modulus of 5.7 in.^{3} (using 7th Edition steel properties).

In Figure 2 of the Attachment is the same tube steel shape, but at the Richmond insert where a 1-5/8" diameter hole has been drilled to accommodate the threaded rod, its properties are as follows: Area = 6.14 -(2)(1/2)(1-5/8) = 4.52 in.^{2}; moment of inertia $\simeq$ 11.4 -(2)(1/2)(1.625)$(1.75)^2$ = 6.423 in.^{4}; and the section modulus is 6.423/2 = 3.211 in.^{3}.

It is apparent that there is a reduction in properties at the connection of a Richmond insert. The most significant reduction is that of the section modulus. To illustrate this problem, see Figure 3 of Attachment A. This is a 2 span continuous beam with two concentrated loads with dimensions as shown. The moment in the beam at support point B is (3/16)(10)(5) = 9.375 kip-ft. The bending stress, using a full tube steel section modulus is (9.375)(12)/5.7 = 19.7 ksi at support point B.

The bending stress in the tube steel using the actual section modulus at support point B is (9.375)(12)/3.211 = 35 ksi. The allowable bending stress for A500 Grade B tube steel is (.67)(36) = 24 ksi. Therefore, this member would be highly overstressed.

Another item I have not previously mentioned is overstressed liner plate attachments. NPSI has attached to the liner plate with an assembly similar to what is shown in Figure 4. Plate A is the member that is being overstressed. I became aware of this problem about Decmeber 1981 or January 1982. I informed the NPSI group that plate A had a bending stress of 60 ksi when analyzed with normal operating loads, and returned the support package back to them.

One month later, the same support package came to the STRUDL group to be reanalyzed. I once again informed NPSI of the overstressed plate.

This time I was instructed not to model in the plate. I was told that the plate was not NPSI's but Gibbs & Hill's responsibility, and that Gibbs & Hill had accepted it although NPSI knew it was overstressed.

Other problems that I remember, which I saw while on my trips through the plant were:

(1) A cantilever with only two anchor bolts loaded about its weakest direction. (Cygna also saw this condition on a cable tray support.)

(2) On an axial restraint, I had analyzed for ITT Grinnell, using their procedures, the lug showing the greatest amount of load had a distance between the lug welded to the pipe and the supporting member of approximately 1/16 in. gap.

(3) The dry wall in the control room is hung with wire and I do not consider this to be seismically qualified.

(4) An HVAC duct within the Containment Building rises a considerable length without any axial restraint. I question the capacity of the sheet metal to resist buckling in the event of a seismic motion.

Another point not previously discussed is the number of supports evaluated by myself and Jack Doyle. While I do not know the exact amount, I do know that it was more than the Applicants have stated. This could be verified by using the log book I used to maintain as the group leader, and I believe was maintained when I decided not to keep that position wit out proper compensation.

If one were to audit the log book, one would see that Jack Doyle and I did the majority of the problems (there were five of us, not including Gary Krishnan, the supervisor.) If one looked at the production before I became group leader and compared it to the production after I became group leader and before Jack Doyle became a member of the group, the average for the group went from about five problems a week to about eleven problems a week which we handled. The five a week was based on counting each support which had two-way restraints as being two supports; the eleven a week counted each support with two-way restraints as a single support.

In addition, Jack Doyle preferred to do the more difficult problems; for example, Gary Brown's Space Frame that was discussed during the LOCA issue, or taking difficult problems from the only TUSI employee so that she could do the easier ones (which also made her average look better).

I mention this primarily to point out that one doesn't <u>have</u> to sacrifice quality to increase production. Even though Jack Doyle and I were concerned about quality, we were able to do our work and still take a few plant tours and identify some specific problems.

Q: Do you have any comment regarding Applicants' proposal that additional hearings be held on the closed Walsh/Doyle items?

A: Yes. The Applicants should have been serious when they had the opportunity to present information to this Licensing Board in regards to the Walsh/Doyle allegations and should have gotten all their information in to show their position and not stuck with generalities. If this is the attitude they have for these hearings, the Applicants will most likely maintain that attitude when they get an operating license, if they get one. If they were not serious about these hearings, they likely will not be serious about operating the nuclear power plant.

I was serious when I made my allegations and I assumed the Applicants were equally serious since they hired Mr. Reedy and others to be their expert witnesses to address our allegations. I also spent a considerable amount of time preparing for the hearings, helping write the Proposed Findings of Fact, and participating in the hearings. I am not being paid as a consultant for doing this, and I do not want to have to go through this another time.

Q: Is there anything further you would like to discuss?

A: Yes, in regards to AWS (pages 33-37 of Applicants' pleading). The Applicants continue to state that they are not required to meet the requirements of AWS. But the Applicants are required to provide a safe plant, no matter what Code is used. In regards to the recapping of the 382 deficient fillet welds on NPSI supports, Mr. Doyle is correct in his assessment of the shrinkage problem with minimum fillet weld size. The AWS welding code at Section 2.7.1 (see Attachment B to this Affidavit) requires the minimum fillet weld to be placed in a single pass, and this is based only on thickness of base metal. The Commentary to the Code, at Section 2.7.1, provides the reasoning for the minimum size fillet weld, and this reasoning compliments the statements made by Mr. Doyle. It should also be noted, all additional passes must be the same minimum size.

This minimum fillet weld size for the first pass, and all additional passes, is a major problem because of cracking of the weld or base metal. To the best of my knowledge, the Brown & Root welding procedures will not accommodate this minimum size, particularly the 1/4" fillet weld in a single pass.

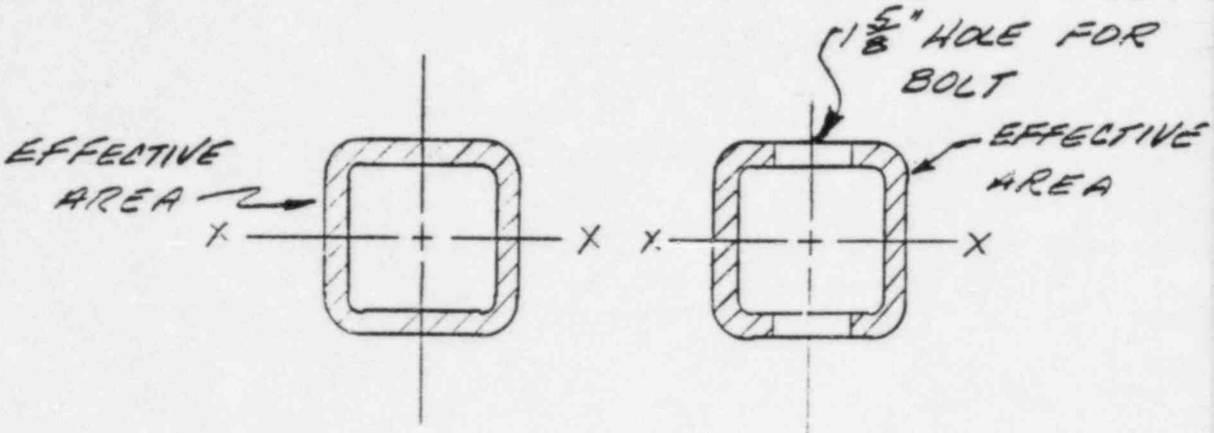


FIG 1

FIG 2

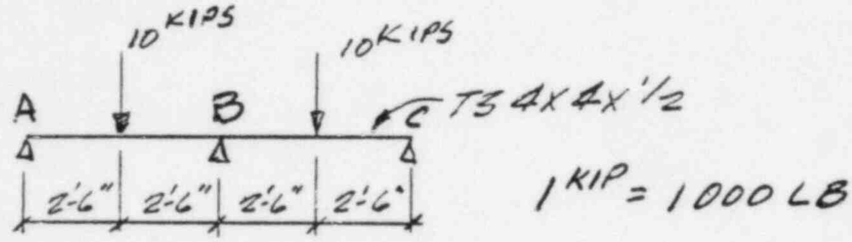
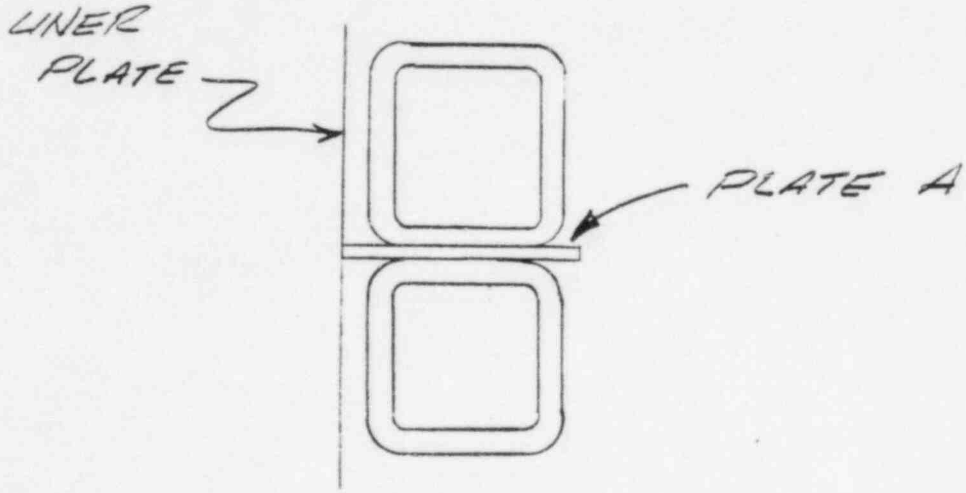


FIG 3



ANSI/AWS D1.1-82

**An American National Standard
Approved by
American National Standards Institute
January 25, 1982**

Structural Welding Code—Steel

Sixth Edition

**Superseding
AWS D1.1-81**

Prepared by
AWS Structural Welding Committee

Under the Direction of
AWS Technical Activities Committee

Approved by
AWS Board of Directors

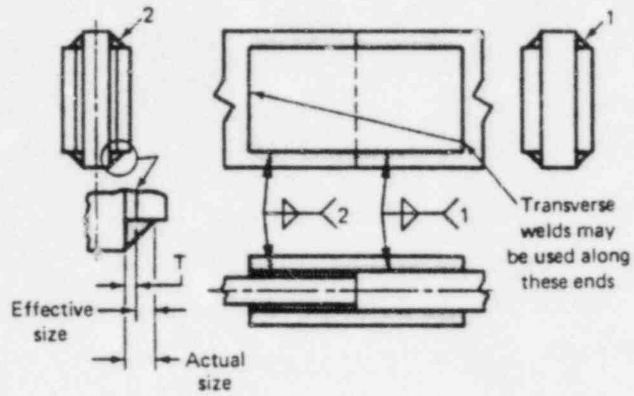
Effective January 1, 1982

AMERICAN WELDING SOCIETY, INC.
550 North LeJeune Rd., Miami, FL 33126

Table 2.3.1.4
Effective throats of flare groove welds

Flare-bevel-groove welds	Flare-V-groove welds
All diam bars	
5/16 R	1/2 R*

Note: R = radius of bar.
*Except 3/8 R for GMAW (except short circuiting transfer) process with bar sizes 1 in. (25.4 mm) diam and over.



Note: The effective area of weld 2 shall equal that of weld 1, but its size shall be its effective size plus the thickness of the filler T.

Fig. 2.4.2—Fillers less than 1/4 in. thick

Part C
Details of Welded Joints

2.6 Joint Qualification

2.6.1 Joints meeting the following requirements are designated as prequalified:

(1) Conformance with the details specified in 2.7 through 2.10 and 10.13.

(2) Use of one of the following welding processes in accordance with the requirements of Sections 3, 4, and 8, 9, or 10 as applicable: shielded metal arc, submerged arc, gas metal arc (except short circuiting transfer), or flux cored arc welding.

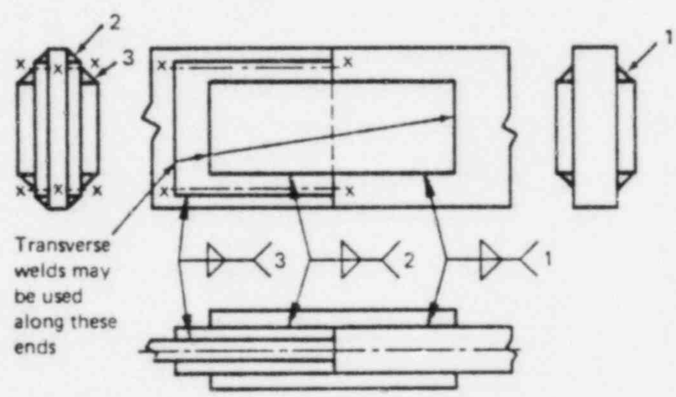
2.6.1.1 Joints meeting these requirements may be used without performing the joint welding procedure qualification tests prescribed in 5.2.

2.6.1.2 The joint welding procedure for all joints welded by short circuiting transfer gas metal arc welding (see Appendix D) shall be qualified by tests prescribed in 5.2.

2.6.2 Joint details may depart from the details prescribed in 2.9 and 2.10 and in 10.13 only if the contractor submits to the Engineer his proposed joints and joint welding procedures and at his own expense demonstrates their adequacy in accordance with the requirements of 5.2 of this Code and their conformance with applicable provisions of Sections 3 and 4.

2.7 Details of Fillet Welds

2.7.1 The details of fillet welds made by shielded metal arc, submerged arc, gas metal arc (except short circuiting transfer), or flux cored arc welding to be used without joint welding procedure qualifications are listed in 2.7.1.1 through 2.7.1.5 and detailed in Figs. 2.7.1 and 10.13.5.



Notes:
1. The effective area of weld 2 shall equal that of weld 1. The length of weld 2 shall be sufficient to avoid overstressing the filler in shear along planes x-x.
2. The effective area of weld 3 shall at least equal that of weld 1 and there shall be no overstress of the ends of weld 3 resulting from the eccentricity of the forces acting on the filler.

Fig. 2.4.3—Fillers 1/4 in. or thicker

2.7.1.1 The minimum fillet weld size, except for fillet welds used to reinforce groove welds, shall be as shown in Table 2.7.

2.7.1.2 The maximum fillet weld size permitted along edges of material shall be

(1) the thickness of the base metal, for metal less than 1/4 in. (6.4 mm) thick (see Fig. 2.7.1, detail A).

(2) 1/16 in. (1.6 mm) less than the thickness of base metal, for metal 1/4 in. (6.4 mm) or more in thickness (see Fig. 2.7.1, detail B), unless the weld is designated on the drawing to be built out to obtain full throat thickness.

transfer), or flux cored arc welding processes are listed in 2.8.2 through 2.8.8 and 3.3.1 and may be used without performing the joint welding procedure qualification prescribed in 5.2, provided the technique provisions of 4.21 or 4.22, as applicable, are complied with.

2.8.2 The minimum diameter of the hole for a plug weld shall be no less than the thickness of the part containing it plus 5/16 in. (8.0 mm), preferably rounded to the next greater odd 1/16 in. (1.6 mm). The maximum diameter of the hole for a plug weld shall not be greater than 2-1/4 times the depth of filling.

2.8.3 The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

2.8.4 The length of the slot for a slot weld shall not exceed ten times the thickness of the part containing it. The width of the slot shall be no less than the thickness of the part containing it plus 5/16 in. (8.0 mm), preferably rounded to the next greater odd 1/16 in. (1.6 mm), nor shall it be greater than 2-1/4 times the depth of filling.

2.8.5 Plug and slot welds are not permitted in quenched and tempered steels.

Table 2.7
Minimum fillet weld size for prequalified joints

Base metal thickness of thicker part jointed (T)		Minimum size of fillet weld*		
in.	mm	in.	mm	
$T \leq 1/4$	$T \leq 6.4$	1/8**	3	Single-pass welds must be used
$1/4 < T \leq 1/2$	$6.4 < T \leq 12.7$	3/16	5	
$1/2 < T \leq 3/4$	$12.7 < T \leq 19.0$	1/4	6	
$3/4 < T$	$19.0 < T$	5/16	8	

*Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, particular care should be taken to provide sufficient preheat to ensure weld soundness.

**Minimum size for bridge applications is 3/16 in.

2.8.6 The ends of the slot shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

2.8.7 The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

2.8.8 The depth of filling of plug or slot welds in metal 5/8 in. (15.9 mm) thick or less shall be equal to the thickness of the material. In metal over 5/8 in. thick, it shall be at least one-half the thickness of the material but no less than 5/8 in.

Legend for Figs. 2.9.1 through 2.10.1

Symbols for joint types
- B–butt joint
- C–corner joint
- T–T-joint
- BC–butt or corner joint
- TC–T- or corner joint
- BTC–butt, T-, or corner joint

Symbols for base metal thickness and penetration
- L–limited thickness–complete joint penetration
- U–unlimited thickness–complete joint penetration
- P–partial joint penetration

Symbols for weld types
- 1–square-groove
- 2–single-V-groove
- 3–double-V-groove
- 4–single-bevel-groove
- 5–double-bevel-groove
- 6–single-U-groove
- 7–double-U-groove
- 8–single-J-groove
- 9–double-J-groove

Symbols for welding processes if not shielded metal arc
- S–submerged arc welding
- G–gas metal arc welding
- F–flux cored arc welding

The lower case letters, e.g., a, b, c, etc., are used to differentiate between joints that would otherwise have the same joint designation.

ATTACHMENT B

Commentary on Structural Welding Code –Steel

Third Edition

Prepared by
AWS Structural Welding Committee

Under the Direction of
AWS Technical Activities Committee

Approved by
AWS Board of Directors

ATTACHMENT B

Foreword

This commentary on AWS D1.1-81, Structural Welding Code—Steel, has been prepared to generate better understanding in the application of the Code to welding in steel construction.

Since the Code is written in the form of a specification, it cannot present background material or discuss the Committee's intent; it is the function of this commentary to fill this need.

Suggestions for application as well as clarification of Code requirements are offered with specific emphasis on new or revised sections that may be less familiar to the user.

Since the publication of the first edition of the Code, the nature of inquiries directed to the American Welding Society and the Structural Welding Committee has indicated that there are some requirements in the Code that are either difficult to understand or not sufficiently specific, and others that appear to be overly conservative.

It should be recognized that the fundamental premise of the Code is to provide general stipulations applicable to any situation and to leave sufficient latitude for the exercise of engineering judgment.

Another point to be recognized is that the Code represents the collective experience of the Committee and while some provisions may seem overly conservative, they have been based on sound engineering practice.

The Committee, therefore, believes that a commentary is the most suitable means to provide clarification as well as proper interpretation of many of the Code requirements. Obviously, the size of the commentary had to impose some limitations with respect to the extent of coverage.

This commentary is not intended to provide a historical background of the development of the Code, nor is it intended to provide a detailed resume of the studies and research data reviewed by the Committee in formulating the provisions of the Code.

Generally, the Code does not treat such design considerations as loading and the computation of stresses for the purposes of proportioning the load-carrying members of the structure and their connections. Such considerations are assumed to be covered elsewhere, in a general building code, bridge specification, or similar document. As an exception, the Code does provide allowable stresses in welds, fatigue provisions for welds in bridges and tubular structures, and strength limitations for tubular connections. These provisions are related to particular properties of welded connections.

The Committee has endeavored to produce a useful document suitable in language, form, and coverage for welding in steel construction. The Code provides a means for establishing welding standards for use in design and construction by the owner or his designated representative. The Code incorporates provisions for regulation of welding that are considered necessary for public safety.

The Committee recommends that the owner or owner's representative be guided by this commentary in application of the Code to his welded structure. The commentary is not intended to supplement Code requirements, but only to provide a useful document for interpretation and application of the Code; none of its provisions are binding.

Comments or inquiries pertaining to this commentary or to the Code are welcome. They should be addressed to: Secretary, Structural Welding Committee, American Welding Society.

ATTACHMENT B

Preface

It is the intention of the Structural Welding Committee to revise the commentary on an annual basis so that commentary on the changes to the Code can be promptly supplied to the user. In this manner, the commentary will always be current with the edition of the Structural Welding Code—Steel with which it is bound.

Changes in the commentary from the first edition have been indicated by a single vertical line that appears in the margin immediately adjacent to the paragraph affected. Changes to tables and figures, as well as new tables or new figures, have not been so indicated.

ATTACHMENT B

Commentary on Structural Welding Code—Steel

1. General Provisions

Note: All references to numbered paragraphs, tables, and figures, unless otherwise indicated, refer to paragraphs, tables, or figures in AWS D1.1, Structural Welding Code—Steel. References to paragraphs, tables, or figures in this commentary are prefixed with a C. Hence, Fig. 8.8.5 is in AWS D1.1, while Fig. C8.8.5 is in this commentary.

1.1 Application

The Structural Welding Code, hereinafter referred to as the Code, provides welding requirements for the construction of steel structures. It is intended to be complementary with any general code or specification for design and construction of steel structures.

This Code was specifically written for use in the construction of buildings, bridges, or tubular structures, but its provisions are generally applicable to any steel structure.

When using the Code for other structures, owners, architects, and engineers should recognize that not all of its provisions may be applicable or suitable to their particular structure. However, any modifications of the Code deemed necessary by these authorities should be clearly referenced in the contractual agreement between the owner and the contractor.[1]

1.2 Base Metal[2]

The ASTM A6 and A20 specifications govern the delivery requirements for steels, provide for dimensional tolerances, delineate the quality requirements, and outline the type of mill conditioning.

Material used for structural applications is usually furnished in the as-rolled condition. The Engineer should recognize that surface imperfections (seams, scabs, etc.) acceptable under A6 and A20 may be present on the material received at the fabricating shop. Special surface finish quality, when needed in as-rolled products, should be specified in the information furnished to the bidders.

1.3 Welding Processes

Certain shielded metal arc, submerged arc, gas metal arc (excluding the short circuiting mode of metal transfer across the arc), and flux cored arc welding procedures in conjunction with certain related types of joints have been thoroughly tested and have a long record of proven satisfactory performance. These welding procedures and joints are designated as prequalified and may be used without tests or qualification (see 5.1 and 5.2).

Prequalified provisions are given in Section 2, Prequalified Joint Details; Section 3, Workmanship; and Section 4, Technique. Section 4 includes welding procedures, with specific reference to preheat, filler metals, electrode size, and other pertinent requirements. Additional requirements for prequalified joints in tubular construction are given in Section 10.

The use of prequalified joints and procedures does not necessarily guarantee sound welds. Fabrication capability is still required together with effective and knowledgeable welding supervision to consistently produce sound welds.

The Code does not prohibit the use of any welding process. It also imposes no limitation on the use of any other type of jont; nor does it impose any procedural restrictions on any of the welding processes. It provides for the acceptance of such joints, welding processes, and procedures on the basis of a successful qualification by the contractor conducted in accordance with the requirements of the Code (see 5.2).

1. As used in this commentary, contractor designates the party responsible for performing the welding under the Code. The term is used collectively to mean contractor, fabricator, erector, manufacturer, etc.

2. Since all steel specifications approved by the Code for use in buildings, bridges, and tubular structures are listed in 10.2, the general provisions for approved base metals will be discussed in C10.2. As an exception, specific provisions applicable only to buildings or bridges are discussed in C8.2 or C9.2 respectively.

2. Design of Welded Connections

2.1.3 The engineer preparing contract design drawings cannot specify the depth of groove (S) without knowing the welding process and the position of welding. The Code is explicit in stipulating that only the effective throat (E) is to be specified on design drawings for partial joint penetration groove welds (2.1.3.1). This allows the contractor to produce the effective throat by assigning a depth of preparation to grooves shown on shop drawings as related to his choice of welding process and position of welding.

The root penetration will generally depend on the angle subtended at the root of the groove in combination with the root opening, the welding position, and the welding process. For joints using bevel- and V-groove welds, these factors determine the relationship between the depth of preparation and the effective throat for prequalified partial joint penetration groove welds.

2.5 Partial Joint Penetration Groove Welds

A partial joint penetration groove weld has an unwelded portion at the root of the weld. This condition may also exist in joints welded from one side without backing, and, therefore, the Code considers them partial joint penetration groove welds except as modified in Section 10 (10.12.4).

The unwelded portions are no more harmful than those in fillet welded joints. These unwelded portions constitute a stress raiser having significance when fatigue loads are applied transversely to the joint. This condition is reflected in the applicable fatigue criteria.

However, when the load is applied longitudinally, there is no appreciable reduction in fatigue strength. Irrespective of the rules governing the service application of these particular grooves, the eccentricity of shrinkage forces in relation to the center of gravity of the material will result in angular distortion on cooling after welding. This same eccentricity will also tend to cause rotation in transfer of axial load transversely across the joint. Therefore, means must be applied to restrain or preclude such rotation, both during fabrication and in service.

2.7.1 Minimum Fillet Weld Sizes for Prequalified Joints. The Code specifies the minimum fillet weld size and requires that this size be made in a single pass. This provision is intended to ensure sufficient heat input in order to reduce the possibility of cracking in either the heat-affected zone or weld metal, especially in a restrained joint. The minimum size applies if it is greater than the size required to carry design stresses.

The intent of Table 2.7 is further clarified as follows: Base metal thickness of 3/4 in. (19 mm) and under are exempt from preheat in accordance with Table 4.2. Should fillet weld sizes greater than the minimum sizes be required for these thicknesses, then each individual pass of multiple-pass welds must represent the same heat input per inch of weld length a provided by the minimum fillet size required by Table 2.7.

2.8 Plug Welds and Slot Welds

Plug and slot welds conforming to the dimensional requirements of 2.8, welded by techniques prescribed in Appendix A, and using materials approved by 8.2, 9.2, or 10.2, are considered prequalified and may be used without performing joint welding procedure qualification tests.

2.9.4--2.10.5 Corner Joint Details. The Code permits an alternative option for preparation of the groove in one or both members for all bevel- and J-groove welds in corner joints as shown in Fig. C2.9.4.

This provision was prompted by lamellar tearing considerations permitting all or part of the preparation in the vertical member of the joint. Such groove preparation reduces the residual tensile stresses, arising from shrinkage of welds on cooling, that act in the through-thickness direction in a single vertical plane, as shown in prequalified corner joints diagrammed in Figs. 2.9.1 and 2.10.1. Therefore, the probability of lamellar tearing can be reduced for these joints by the groove preparation now permitted by the Code. However, some unprepared thickness, "a," as shown in Fig. C2.9.4, must be maintained to prevent melting of the top part of the vertical plate. This can easily be done by preparing the groove in both members (angle β).

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.

Mark A. Walsh
(Signed) Mark A. Walsh

Date: ____________________

STATE OF Texas

COUNTY OF Dallas

On this, the 30th day of January, 1984, personally appeared Mark A. Walsh, known to me to be the person whose name is subscribed to the foregoing instrument, and acknowledged to me that he executed the same for the purposes therein expressed.

Subscribed and sworn before me on the 30th day of January, 1984.

Cindy Carter
Notary Public in and for the
State of Texas

My Commission Expires: 12/11/85