KANSAS GAS AND ELECTRIC COMPANY

GLENN L. KOESTER
VICE PRESIDENT - NUCLEAR

February 18, 1985

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

KMLNRC 85-058
Re: Docket No. STN 50-482
Ref: (1) Letter KMLNRC 84-238, dated 12/31/84 from GLKoester, KG&E, to RCDeYoung, NRC
(2) Letter KMLNRC 85-037, dated 1/21/85 from GLKoester, KG&E, to RCDeYoung, NRC
Subj: Supplemental Information on Structural Steel Welding

Dear Mr. Denton:

Questions raised concerning the structural steel welding at Wolf Creek Generating Station resulted in an extensive evaluation of the AWS welding program. This included an evaluation of the relevant aspects of the various programs from the initiation of purchase orders for procurement of the structural steel and welding materials to final installation and acceptance. Upon completion of this evaluation, KG&E concluded that the structural steel welding at Wolf Creek Generating Station meets AWS D1.1 requirements and, most importantly, that the structural integrity of the buildings has been assured. This evaluation was documented in the report transmitted by References 1 and 2.

As a result of additional questions raised concerning the validity of visual reinspections through paint and its impact on compliance with the code, KG&E initiated additional actions to confirm the conclusions previously stated. These actions included contacting the American Welding Society (AWS) and retaining three independent leading authorities in the field of structural steel welding to review the evaluation as documented in References 1 and 2.

Detailed justification for the reinspection of welds that had been painted subsequent to the initial inspection/acceptance was provided in section VI.E of the evaluation report. In addition KG&E had Roger Reedy of Reedy Associates (Engineering Management Consultants), Doctors Slutter, Fisher, and Yen of Lehigh University (Fritz Engineering Laboratory) and Dr. Geoffrey Egan of APTECH, Inc. to review KG&E's justification for reinspection through paint. The results of their reviews are included as Attachments A, B, C, and D to this letter. All three of these same leading authorities independently came to the same conclusion as KG&E in that the important attributes of the welds



B001
1/1

can be reinspected through paint.

In addition to the issue of reinspection through paint, KG&E also had the same three leading authorities independently review the overall program associated with the welding verification effort documented in References 1 and 2. Again all three concluded that the structural steel welding at Wolf Creek meets or exceeds the structural requirements.

In order to assure that the reinspection program documented in References 1 and 2 does not conflict with the AWS Code, KG&E and Daniel International Corporation (DIC) contacted the American Welding Society (AWS) to discuss the applicability of the AWS Code to reinspection efforts at Wolf Creek. Attachments E and F document the results of these discussions and confirm that the reinspections were not inconsistent with the AWS Code and in fact the Secretary of the AWS Structural Welding Committee recognized the authority of the Architect/Engineer acting as the owner's representative to establish pertinent reinspection criteria.

In conclusion the structural steel welding at Wolf Creek meets the requirements of AWS D1.1 and far more importantly the structural integrity has been assured.

Yours very truly,

Glenn L Koester

Glenn L. Koester
Vice President - Nuclear

GLK:sjm

Attach

xc: PO'Connor, w/o
HBundy, w/o
RDenise , w/a

Attachment A to
KMLNRC 85-058

February 15, 1985

Glenn Koester
Vice President-Nuclear
Kansas Gas & Electric Company
P.O. Box 208
Wichita, KS 67201

Dear Mr. Koester,

It is my opinion, based on the studies I have made on the Wolf Creek site, that the structural welding meets the visual acceptance criteria of AWS D1.1.

**BACKGROUND**

One of the major reasons for the controversy concerning adequacy of welding at the Wolf Creek site is directly related to the use of two different welding inspection philosophies in two different time frames at the site. In this regard, I am only referring to the visual inspection of the physical attributes each weld after completion.

About mid-1981, even though structural welding was 99-100% complete, a new inspection philosophy evolved for the re-inspection of completed welds. This new philosophy, a "no tolerance" philosophy, by its very nature, guaranteed that many welds which had previously been accepted, would be considered to be "inadequate". The "no tolerance" philosophy is contrary to what is taught by AWS (American Welding Society) to candidates for their Certified Welder Inspector (CWI) test. (If this "no tolerance" philosophy were applied to the inspection of steel bridges and buildings welded in accordance with the AWS D1.1 Structural Code, these structures would be found to have many "inadequate" welds.)

The difference in inspection philosophies is as follows:

1. AWS philosophy -

   Welds should be measured and evaluated using good judgement. Weld sizes are designated to the nearest 1/16 inch. Deviations of 1/32 inch or less are irrelevant. Weld lengths are measured with a tolerance of about 1/4 inch. Tolerances are allowed for all evaluations of attributes, including undercut. Visually detected cracks are not allowed, but it is recognized that not all "crack-like" linear indications can be found by visual examination. If the Engineer is concerned because of design consideration about minute linear indications which can not always be found by visual examination, more critical examination methods, such as magnetic particle (MT) or liquid penetrant (PT) will be specified.

2. "No tolerance" philosophy-
All visual evaluations of welds will be made on strict (no judgement allowed) literal interpretation of acceptance criteria. That is, any weld which is undersized, even by less than 1/64 inch is unacceptable. The most critical interpretation is applied for each criteria. Each acceptance is on a "go-no go" basis, with no tolerance. This philosophy is contrary to AWS requirements and will automatically result in the rejection of AWS acceptable welds. The advantage of this philosophy is that any weld accepted this way will always be acceptable, no matter who performs the inspection, and what the inspector's qualifications are.

When inspecting any item, judgement must be used. For example, the inspector must choose the proper measuring tools for the condition to be examined, he must judge whether or not lighting is adequate, determine areas most likely to cause concern, and must judge how and where to make measurements. These judge-ments are taught in AWS Inspector Training courses.

Engineers design structural welds to the nearest 1/16 inch. Therefore weld size measurements should be to the nearest 1/16 inch in accordance with "Rules for Rounding Off Numerical Values" (ANSI Z25.1). This standard provides that a weld 1/32 inch undersized would be rounded off to the next 1/16 inch and therefore accepted as adequate. As discussed above, the "no tolerance" inspection philosophy which evolved at the Wolf Creek site in does not allow rounding-off, and any deviation in size, no matter how insignificant, is documented as inadequate.

The "no tolerance" philosophy was used on the site in order to demonstrate that by "any criteria" the structural welds at Wolf Creek are adequate.

## INSPECTION OF PAINTED WELDS

At the time the "no-tolerance" philosophy evolved almost all structural welds had been completed, inspected, accepted and painted. Because of an inspection record control problem (some inspection records were lost or mis-placed), it was decided that a large number of structural weld joints (each joint may contain a number of welds) would be reviewed. This type of review is consistent with the requirements of 10CFR50 Appendix B which provides that the applicant take measure "to provide adequate confidence that a structure, system, or component will perform satisfactorily in service." The question then becomes whether or not painted welds can reviewed to provide adequate confidence. This reinspection or review is a verification that inspections were performed and not a first time acceptance inspection, and not a requirement of AWS D1.1.

Mr. Moss V. Davis' letter of February 13, 1985 to Mr. John G. Berra points out that secondary inspections of welds are outside the scope of D1.1. The letter further states that secondary inspection of welds should be agreed upon by the owner or the Engineer and the contractor. Obviously the techniques used for the secondary inspection techniques should not be more severe than the original inspection techniques.

It is known and understood in all welding Codes and Standards that magnetic particle inspections are far more severe than visual inspection. (The ASME and AWS Codes make this an obvious conclusion by classification of inspection criteria.) The inspections required of the structural welding in question on site are all visual inspections.

## VISUAL INSPECTION OF WELDS

The weld attributes usually required to be visually inspected are:
- o Weld location (including existence)
- o Length
- o Size
- o Undercut
- o Cracks
- o Cracers
- o Fusion
- o Concavity
- o Convexity
- o Overlap
- o Porosity
- o Arc Strikes (with regard to cracks)
- o Slag and spatter

Obviously, some weld attributes are more important than others. The most important attributes are those related to weld strength or loss of load carrying capability. In this category, I would place the following attributes as most important.

- o Weld location (and existence)
- o Length
- o Size
- o Cracks
- o Craters
- o Undercut
- o Fusion
- o Concavity

The other attributes do not generally affect weld strength and are therefore of less consequence.

With regard to painted welds, the only attributes which the paint may mask are some tight cracks, some tight undercut (a rare occurrence), fine porosity, some arc strikes and some slag and spatter. Arc strikes without cracks can be readily evaluated through paint and slag and spatter on accepted welds is immaterial. AWS D1.1 address slag and spatter as issue only with regard to weld cleanliness in the chapter on Workmanship (paragraph 3.10). Porosity less than 1/16 inch is not even considered relevant by ASME Codes, and larger porosity can be evaluated through paint. If it were ever considered necessary or desirable, tight undercut and cracks could readily be evaluated by a magnetic particle examination through the paint, but this is not a requirement of AWS D1.1. The MT examination will find cracks which are undetectable by the naked eye and is therefore a more severe inspection.

A demonstration was made at the Wolf Creek site to assure that a magnetic particle (MT) examination would detect cracks through a painted weld surface. Even with a heavy paint layer of 10-11 mils, all cracks visually detected in the weld sample prior to painting were detected with MT after painting.

The NRC inspection team reviewed more than 70 random weld joints using both visual and magnetic particle examination methods and found no welds which did not meet the AWS D1.1 acceptance criteria. This sample size, assures with at least a 95/95 confidence level that the welds meet the AWS D1.1 acceptance criteria.

In summary, I feel that based on my review of welds, documentation and reports, the reinspection programs used at the Wolf Creek site adequately demonstrate that the structural welding meets the acceptance criteria of AWS D1.1 and provides adequate evidence that the welds are structurally sound and meet the design parameters specified.

Roger F. Reedy

Roger F. Reedy, PE
Registered Structural Engineer (Illinois)
Member AWS
Member ASCE
Fellow ASME

Attachment B to
KMLNRC 85-058

APTECH IS APPLIED TECHNOLOGY

February 17, 1985

Mr. John Bailey
Kansas Gas and Electric Company
Wolf Creek Generating Station
Post Office Box 309
Burlington, Kansas 66839

Dear Mr. Bailey:

RE: Evaluation of Structural Steel Welding at Wolf Creek - CAR No. 19

At your request I have reviewed the approach developed by KG&E and implemented by Bechtel and DIC to evaluate welds on safety related structural steel at the Wolf Creek Generating Station. This review has concentrated on KG&E's final report on corrective action request (CAR) number 19 (1)* and documents (2) through (6).

My evaluation of the approach developed by KG&E was for convenience divided into the following areas:

1) Impact on FSAR Commitment
2) Impact on Structural Integrity

Some specific comments arising out of my review, and relating to these areas are summarized below:

Impact on the FSAR Commitment

In view of the FSAR commitment by KG&E to work to the requirements of AWS D1.1-75 incorporating (2), (3) and (5), it is entirely appropriate for KG&E as owner to develop a reverification inspection program to provide assurance that the provisions of AWS D1.1 75 are met and to generate the documentation to support that position. In addition, your review of related activities and their control has shown that this is not a generic problem but is confined to the structural steel work, welded to AWS D1.1 and covered by the Miscellaneous Structural Steel weld records. These related activities include:

1) Assurance that all welders and welding procedures were qualified to AWS D1.1.

2) Determination that only acceptable filler metal (in this case E7018) was used.

* Support References are included at the end of this letter.

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3) Evaluation of DIC inspection criteria.

4) Validation of inspections performed with paint on the weld.

5) Qualification and training for reinspection personnel.

All of these contribute to the conclusion that poor original documentation procedures do not lead to poor welds. This was also confirmed by my examination of relevant welds in the Auxiliary Building and the Reactor Building. I was able to examine both painted and unpainted welds and in all cases the welds appear to be good with a generally uniform appearance, indicative of skilled crafts people.

With regard to the ability to reinspect welds after painting, I have already stated that this is the proper approach for KG&E to pursue for the following reasons:

- The discontinuities that are being examined for (i.e. porosity, lack of fusion, etc.) are rather gross imperfections and are readily detected by visual examination. A coating of a few mils thick would not obscure imperfections in the size ranges of 1/16 to 1/8 inch. Even these imperfection sizes are small compared to the size that would compromise structural integrity.

- Carbon manganese steel welded with E7018 weld rod is probably one of the easiest combinations to produce high quality welds. Carbon Manganese steels are readily weldable and do not harden significantly with welding thermal cycles as would alloy steels. With proper rod control (which is demonstrated in your review) the likelihood of weld cracking is low. This is confirmed by the results of the inspection of the uncoated steel in which few cracks and lack of fusion imperfections were discovered.

- The detection of size variances (either over or under) will not be impacted by the presence of paint or coatings.

- Missing weld elements would be rather obvious even where coatings are present.

I understand from discussions with KG&E that USNRC Region 1 made a site visit and performed a sampling inspection on more than 60 relevant joints. This inspection included examination by UT and MT, before and after paint removal and the results were positive. These data should be requested from Region 1 and used to support your position.

In view of the fact that we are now using twenty - twenty hindsight and are sensitized to the need to perform detailed inspections the defect rates are relatively low in those categories of attributes that were classed as defects (about 3% on a joint basis which would be much less on a total weld basis).

Normal reinspection detection rates come in at around 2% on a weld basis. We recently performed a review of previously accepted welds in Class I piping and established a reinspection call rate at about 1%.

The focus of your program on structurally significant details has enabled you to evaluate those situations that are most important. It is worth emphasizing that the extent of CAR No. 19 is limited to about 21% of these structural details. The other details are either shop welded or bolted.

I believe that with your re-examination program, the related activities referred to earlier and the confirmation that examination under paint is effective, you have met the extent of (4) and complied with your commitment in (7).

Structural Integrity

Since we have concluded that defective paper work does not necessarily indicate a defective weld, the real question is, "What is the impact on structural integrity of the imperfections discovered in the reinspection?".

Bechtel has evaluated those situations where the stresses could exceed the design stress because of geometry indications (missing welds, undersize, underrun) and in all cases the calculated stress are less than those that would be required to fail a weld (i.e. the weld capacities are in no way approached under the design loads). I concur with Bechtel's approach, but would point out that it is conservative (i.e. greater margins will be available in the actual joint than indicated by the Bechtel analysis).

The first factor contributing to the conservatism is that for the governing allowable stresses, the specified minimum properties are used whereas actual properties of as deposited welds will usually run 20-25% higher than the specified minimums. This means that based on actual properties deviations from allowable stresses at up to 20-25% would not violate design criteria based on actual properties.

The second factor relates to the consequences of exceeding the design allowable stress in one weld, or for that matter all welds, in a connection that contains several welds as many of these joints do. There are of course none. In the joint one weld may be overstressed, however, the structural integrity of the joint is not impaired at all. It is important to re-emphasize this fact. The integrity of a structural detail is not affected by the imperfections detected in the reinspection program. If this was more generally recognized, we would be faced with far fewer reverification exercises in nuclear facilities.

A further fact that contributes to the conservatism in the Bechtel analysis is that where undersize has been measured to be intermittent in the actual detail, in the analysis it has been attributed to the complete weld length.

A question may arise about the integrity of those welds that are:

1) uninspectable (because of access) and

2) could not be evaluated for alternate load paths

There are 83 joints in this category and the approach chosen by Bechtel is to demonstrate that the expectation is that in only one joint would the design stress be exceeded. This is derived from the frequency of those structural joints that exceed the design stress. Remembering, as noted above, that small amounts of undersize are attributed to the complete weld it may be instructive to consider this on a weld basis.

Assuming an average number of welds per joint of 4 and the same liklihood of exceeding the design stress in a weld as in a joint, the following table provides the probability that 1, 2, 3 and 4 welds would exceed the design stress:

| Number of Welds In a 4 Weld Joint Detail That Exceed Design Stress | Probability | |
|---|---|---|
| | A | B* |
| 1 | $3.17 \times 10^{-2}$ | $8.7 \times 10^{-3}$ |
| 2 | $1.0 \times 10^{-3}$ | $7.6 \times 10^{-5}$ |
| 3 | $3.2 \times 10^{-5}$ | $6.6 \times 10^{-7}$ |
| 4 | $1.0 \times 10^{-6}$ | $5.7 \times 10^{-9}$ |

* This column is based on a 0.87% rate which excludes the polar crane radial stops.

These numbers illustrate the very remote liklihood of all welds in a joint exceeding the design allowable stress at the same time and further confirm that structural integrity is assured. On this basis, I would expect a timely closeout of CAR 19 because there is no safety impact and hence it is not reportable under 10 CFR 50.55(e).

In the foregoing, I have tried to emphasize the important facts related to the closeout of CAR 19. I think you would agree that there is no safety issue and the documentation problem did not spill over to other related areas. There are, however, a few points that may be worthwhile making, particularly if you have to present all of the work that has been done to date, to the management of KG&E.

First the question of cracks may be raised. What is the liklihood of having cracks in uninspectable areas?

The only cracks that have been observed were from construction loading of beam seats and not attributable to welding (1). The review of weld procedures, filler metal control, and welder records indicate that the welding was not out of control. Usually when something goes wrong with the welding process to cause cracking, the cracking is quite extensive and obvious at the toes of welds. Moreover, the A36 structural steel and A516 embed plates are easy-to-weld carbon manganese steels not prone to cracking. These steels are widely used in other industries in which the rigorous quality assurance requirements of our commercial nuclear program are not adopted. These industries include bridges, multi-story buildings, offshore platforms and pressure vessels. Our record in these industries would confirm that integrity margins are available in welded structural steels. On this basis I would conclude that there is no potential for structural degradation due to the presence of cracks.

Further confirmation of this fact is provided by the good inherent toughness of these materials at the minimum operating temperature of the steel. This would preclude crack initiation and propagation from pre-existing cracks.

The thoroughness and detail of the reinspection program undertaken by KG&E attests to the commitment that you have already made to safety at the Wolf Creek Nuclear Generating Station.

In the rather short period that I have had to review your approach to the resolution of CAR 19, I have probably not done justice to the extensive work already done by KG&E, Bechtel, DIC and other consultants on this matter. I hope, however, that I have been able to grasp the main points of this issue and if you would like to discuss any of the comments I have made, please feel free to contact me.

Kind Regards,

Geoff Egan

Geoffrey R. Egan

GRE/nw

REFERENCES

1) Kansas Gas and Electric Company Final Report
Corrective Action Request No. 19

2) Technical Specification for Erecting Miscellaneous Metal
for the Standardized Nuclear Unit Power Plant System
Bechtel Specification No. 10466-C132Q

3) Technical Specification for Contract for Erection of Structural
Steel for the Standardized Nuclear Power Plant System
Bechtel Specification No. 10466-CR2Q

4) AWS Structural Welding Code AWS D1.1-75

5) Daniel Internationsl Corporation, Inspection of Welding Process
Procedure No. QCP-VII-200

| DATE | REVISION |
|---|---|
| 3-30-77 | |
| 10-28-77 | 1 |
| 2-01-78 | 2 |
| 10-18-78 | 3 |
| 11-08-78 | 4 |
| 11-18-80 | 6 |
| 1-21-81 | 7 |
| 3-12-81 | 8 |
| 12-17-81 | 9 |
| 6-29-81 | 12 |
| 9-22-83 | 17 |
| 12-17-84 | 21 |

6) Letter from C. M. Herbst (Bechtel) to G. L. Fouts (KG&E) date
2-15-85 regarding Structural Steel Joint Sketches

7) Final Safety Analysis Report
SNUPPS Section 3.8.3.6.3.3.

Attachment C to
KMLNRC 85-058

**LEHIGH UNIVERSITY**
Bethlehem, Pennsylvania 18015

**Fritz Engineering Laboratory**
Building 13

February 14, 1985

Mr. John A. Bailey
Wolf Creek Generating Station
Kansas Gas and Electric Company
P. O. Box 309
Burlington, Kansas 66839

Re: Visual Inspection of Painted Fillet Welds

Dear Mr. Bailey:

Dr. Fisher and I have reviewed the paper prepared by Bechtel Power Corporation regarding their position on the "Visual Inspection of Painted Fillet Welds". Dr. Yen of our staff has also reviewed this and provided comments on the paper. We all agree that the important characteristics of the welds can be evaluated with the paint thickness of 14 mils (±) on the members.

The evaluation must be made on the basis that certain problems that could occur in welding can be ruled out because they do not exist or are not important for the type of welds and materials involved. We are concerned only about inspection items that might reduce the strength of connections. Tests made on welds from the Hope Creek Plant (Fritz Engineering Laboratory Report 200.81.240.3) revealed that even very large amounts of porosity in the welds reduced the strength of connections by only a small amount. Large porosity of the type present in welds from the Hope Creek Plant could be detected through paint. Fine porosity of a size that could not be observed through paint is of no importance in evaluating the strength of these connections.

We feel confident that the inspection results to date demonstrate that the quality of welding on the buildings was more than adequate to provide the strength required in the building connections. If there are inspection items such as fine porosity, minor undercutting or cracking in welds produced by joint restraint that can not be detected through paint, these items are not apt to reduce the strength of connections sufficiently to be of concern. The redundancy in the completed structure is also available to provide alternate load paths if necessary in the event that a connection of lower than expected strength exists.

Sincerely yours,

Roger G. Slutter

Roger G. Slutter

RGS/df
cc: Richard Ivy
John W. Fisher

Attachment D to
KMLNRC 85-058

LEHIGH UNIVERSITY
Bethlehem, Pennsylvania 18015

Fritz Engineering Laboratory
Building 13

December 10, 1984

Mr. Richard Ivy
Kansas Gas and Electric Company
P.O. Box 208
Wichita, Kansas 67201

Dear Mr. Ivy:

Re: Structural Steel Welds at
Wolf Creek Generating Station

We have reviewed the problems associated with the structural welds in the structures at the Wolf Creek Generating Station. Dr. Slutter was on the site on November 1 and 2, 1984 to observe firsthand some of the weld deviations, the method of inspection, inspection records, and problems encountered in completion of the inspection program. The problems encountered at this site are not unlike structural welding problems that we have seen at other nuclear power plants. The problems at Wolf Creek are perhaps more frustrating but less serious than similar problems at other sites. The approach being used by Bechtel as summarized in "Weld Deviation Evaluation Methodology" dated November 26, 1984 has also been reviewed.

The examination of the welds in this reinspection program is very thorough, as evidenced by the documentation on every connection. The thoroughness of the inspection has revealed some problems that require evaluation from a structural analysis point of view and a much larger number of instances where deviations from AWS D 1.1 - 1975 are reported that do not constitute structural deficiencies. It appears from the latest summary of inspection and evaluation received from Bechtel (dated November 27, 1984) that no significantly deficient joints have been found.

We have the following comments on the various categories of problems that have been found in the reinspection:

1. Missing Welds

   Obviously the missing welds should be replaced if they are needed to resist design loads. Some of these welds such as the beam to beam seat welds may not be required, and replacement should not be necessary. Where they are inaccessible and cannot be replaced, an appropriate analysis of the other load paths should be provided.

2. Undersize, Unequal Leg, and Underlength Welds

   The approach that is being used to evaluate these types of conditions using the smallest weld dimension is very conservative. Welds that are no more than 1/16 in. undersize will have adequate strength on the basis of the latest code recommendations. The allowable stresses being used by Bechtel from the Seventh Edition AISC provide a conservative basis for evaluation.

3. Oversize and Overlength Welds

   These deviations are not generally a problem to be concerned about. There are some instances where the additional amount of weld causes the connection to provide more restraint than intended. The original design actually specified this additional welding. In these structures the additional weld metal should not cause problems. End rotation and the resulting connection deformation can result in cracking of the welds if the additional weld increases the bending stiffness of the connection and decreases ductility.

4. Cracked Welds Between Beam and Beam Seat

   These cracks resulted from rotation of the end of the beam as concrete slabs were poured and additional dead load was placed. The cracking does not indicate a deficiency in the connection since the weld is not needed. The cracked welds that were detected were probably undersize because of the rolled edges of the members being joined.

5. Return Welds That Are Overlength But Undersize

   The purpose of this weld is to produce a proper termination for the vertical we'd. It is not necessary that it meets AWS 1.1 - 1975 size requirements, since it is not needed structurally. The added length can increase capacity in some instances. The primary objective of end returns is to minimize prying and distortion at the root of the primary weld.

6. Lack of Fusion and Undercut

   These problems are very few in number and are being satisfactorily handled in the analysis.

7. Beam Seat Missing

These may not be needed but an analysis of each one is being made. It is assumed that seats will be provided if needed.

8. Fit-Up Gap with Undersize Weld

This is a rare occurrence considering structures involved. Proper analysis of this is being made by Bechtel.

9. Inaccessible Welds

Since there are no significant structural deficiencies among the exposed welds inspected, it is reasonable to assume that the inaccessible welds are similar.

The general problem of weld size should be considered in terms of the expected statistical variation of weld dimensions in typical structural welding where the AISC allowable stresses are applicable. Enclosed are Fig. a through Fig. e showing the statistical variation of the 1/4 in., 3/8 in., and 1/2 in. welds used to develop the AWS and AISC specification provisions. These curves show the deviation in weld sizes that are to be expected with production welds. The variation of weld capacity that resulted from the AWS-AISC fillet weld study in 1968 was in part due to the variation in weld size that existed with the test sample. These were normal production welds, and similar deviations will exist with all welds. Figure 19.3 in Structural Steel Design shows the shear strength based on nominal weld size. It is clear that part of the reason for the variation in capacity is based on the weld size variation.

When a weld is found to be undersize by measurement, it is not significant unless it falls below the range indicated by the curves. The AWS Specification does not address the problem of deviations, and disposition of undersize welds must be done using the type of analysis that Bechtel has proposed. The fact that they are using actual weld sizes in calculations is conservative, since the specifications used the lower bound of the test data which included weld undersize.

Weld size deviations on the return welds does not require analysis. These welds are not intended to increase the strength of the connection, although some additional strength does result from the addition of these welds. The main function of return welds is to increase the ultimate strength of the structure by delaying end tearing of the weld and improving the ductility of the connection. These welds need not be held to exact dimensions but should be large enough to provide a satisfactory weld termination.

The analysis work being done by Bechtel is based on elastic design with reference to the Seventh Edition of the AISC Manual of Steel Construction. This approach is conservative compared to the ultimate strength method available in the Eighth Edition and the current approach used in LRFD design as given in Load and Resistance Factor Design Criteria for Connectors*. One of the provisions of the earlier specification that is very conservative and not applicable to weld capacity is the allowable stress for base metal in shear given as $F_v = 0.4\ F_y$. This limit state was arbitrarily adopted in 1969 and is not related in any way to weld capacity. This is only now being corrected in the AISC Specifications. The attached copy of Table J2.3 shows the proper limit state conditions that are used in the LRFD Specification. Steps are now underway to change the allowable stress provisions for shear on the weld leg to $0.3\ F_u$ in place of the value $0.4\ F_y$. Typical increases in allowable loads for eccentric connections that one can expect to result from using the ultimate strength analysis outlined in the Eighth Edition of the AISC Manual can be seen by comparing the results given in Table III on page 4-31. With a weld length of 11.5 in., the C-shaped weld and the outstanding angle vertical welds are similar to the welded example shown on page 661 of the second edition of Structural Steel Design. The ultimate strength analysis of the clip angle to plate welds provides an 8% increase in load. The C-shaped welds of the clip angles to beam web are permitted to carry 22% more load using the ultimate strength method. This can also be seen by comparing the standard angle connection loads in the Seventh and Eighth Editions of the AISC Manual.

The AISC provisions for the design of this type of connection are very conservative even when one uses the ultimate strength method. The minimum factor of safety for a connection designed by the ultimate strength method is given as 3.33 on page 4-74 of the Eighth Edition of the AISC Manual. The usual factor of safety in weld design for single load vectors is 2.33. The more conservative design for this type of connection recognizes that minor deviations such as found in the connections at Wolf Creek Generating Station will occur. These deviations are not uncommon, and this is recognized by the AISC provisions. In particular, the weld size variations are typical where fillet welds are used. The higher factor of safety in use for eccentric joints recognizes that other deviations are likely.

We do not believe that a structural problem exists with the Wolf Creek welds once the obvious problem of missing welds has been corrected. In the November 27, 1984 summary, Bechtel reports only 17 joints requiring rework due to overstress of 1620 joints evaluated. This is a very low percentage in view of the conservative approach being used in the analysis. A less conservative approach might result in an even smaller number of joints requiring rework.

*Load and Resistance Factor Design Criteria for Connectors, by J. W. Fisher, T. V. Galambos, G. L. Kulak, and M. K. Ravindra, Journal of the Structural Division ASCE, Vol. 104, No. ST9, September 1978.

In any event we feel that Bechtel's approach in considering the inspection reports and their subsequent analysis is adequate and sufficiently conservative for the type of structures and the type of connections involved. The overall quality of the welds based on the inspection data and observations that we have made exceeds the requirements for structural welding for this type of construction.

We would be pleased to examine other Bechtel dispositions when they are available. We agree with the procedure being used.

Sincerely yours,

John W Fisher

John W. Fisher
Professor of Civil Engineering
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JWF:RGS:rag

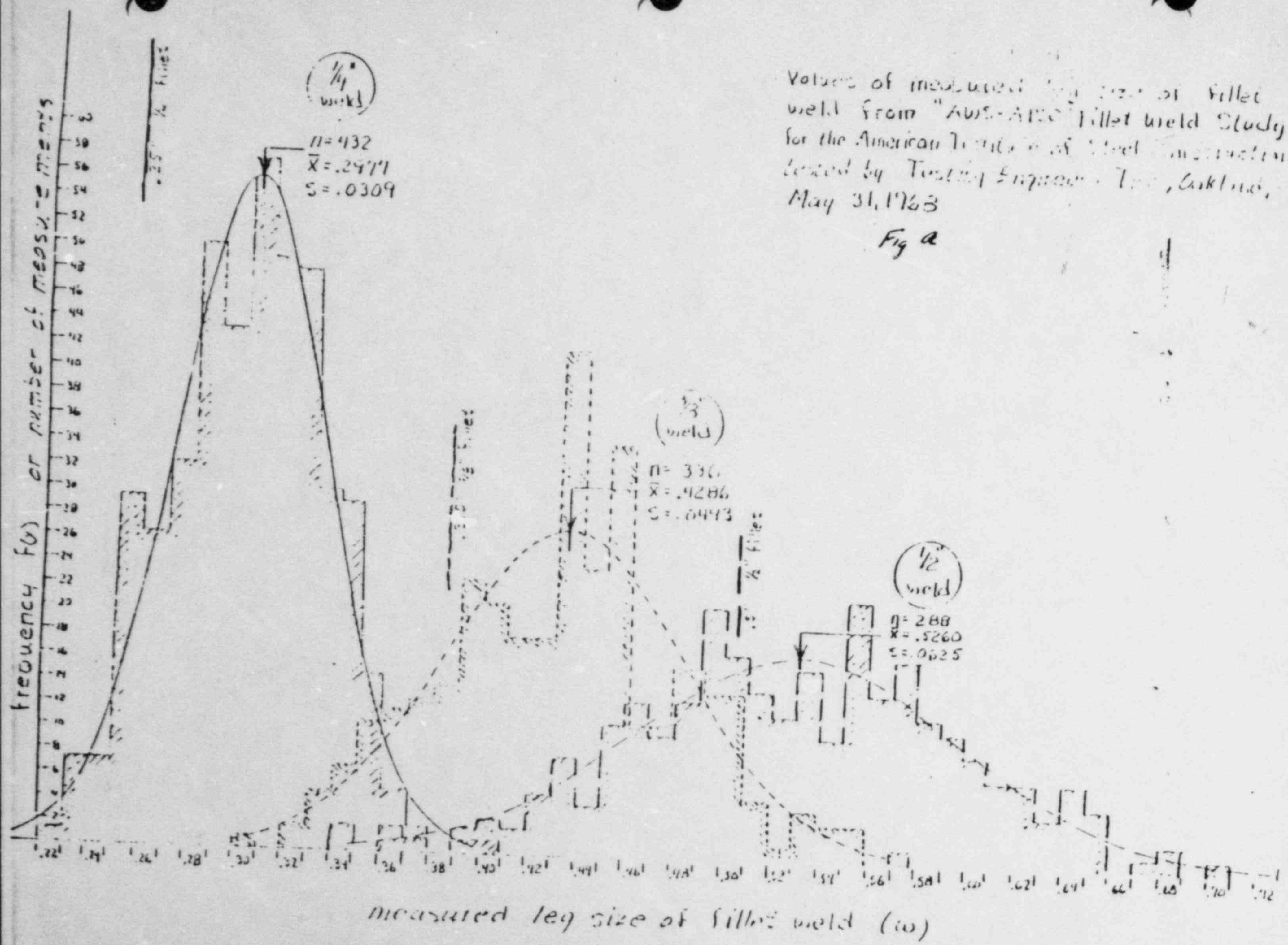
Enclosures

cc: J. A. Bailey

Table J2.3
Design Strength of Welds

| Types of Weld and Stress[a] | Material | Resistance Factor $\phi$ | Nominal strength $F_{BM}$ or $F_w$ | Required Weld strength level[b,c] |
|---|---|---|---|---|
| **Complete Penetration Groove Weld** | | | | |
| Tension normal to effective area | Base | 0.90 | $F_y$ | "Matching" weld be used |
| Compression normal to effective area | Base | 0.90 | $F_y$ | Weld metal with a strength level equal to or less than "matching" may be used |
| Tension or compression parallel to axis of weld | Base | 0.90 | $F_y$ | |
| Shear on effective area | Base<br>Weld elect. | 0.90<br>0.80 | $0.60F_y$<br>$0.60F_{EXX}$ | |
| **Partial Penetration Groove Welds** | | | | |
| Compression normal to effective area | Base[e] | 0.90 | $F_y$ | Weld metal with a strength level equal to or less than "matching" weld metal may be used |
| Tension or compression parallel to axis of weld | Base[e] | 0.90 | $F_y$ | |
| Shear parallel to axis of weld | Base[e]<br>Weld elect. | 0.75<br>0.75 | $0.60F_u$<br>$0.60F_{EXX}$ | |
| Tension normal to effective area | Base[e]<br>weld Electrode | 0.90<br>0.80 | $F_y$<br>$0.60F_{EXX}$ | |
| **Fillet Welds** | | | | |
| Stress on effective area | Base[e]<br>Weld elect. | 0.75<br>0.75 | $0.60F_u$<br>$0.60F_{EXX}$ | Weld metal with a strength level equal to or less than "matching" weld metal may be used |
| Tension or compression parallel to axis of weld[d] | Base[e] | 0.90 $F_y$ | | |
| **Plug or Slot Welds** | | | | |
| Shear parallel to faying surfaces (on effective area) | Base[e]<br>Weld elect. | 0.75<br>0.75 | $0.60F_u$<br>$0.60F_{EXX}$ | Weld metal with a strength level equal to or less than "matching" weld metal may be used |

[a] For definition of effective area, see Section J2.
[b] For "matching" weld metal, see Table 4.1.1, AWS D1.1.
[c] Weld metal one strength level stronger than "matching" will be permitted.
[d] Fillet welds and partial penetration groove welds joining component elements of built-up members, such as flange to web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds.
[e] The design of connected material is governed by J4.




Values of measured leg size of fillet weld from "AWS-AISC" Fillet Weld Study for the American Institute of Steel Construction tested by Testing Engineers Inc, Oakland, May 31, 1968

Fig a

Original values of measured leg size of fillet weld from "AWS-AISC Fillet Weld Study for the American Institute of Steel Construction tested by Testing Engineers Inc. Oakland, Calif.
May 31, 1968

Fig b

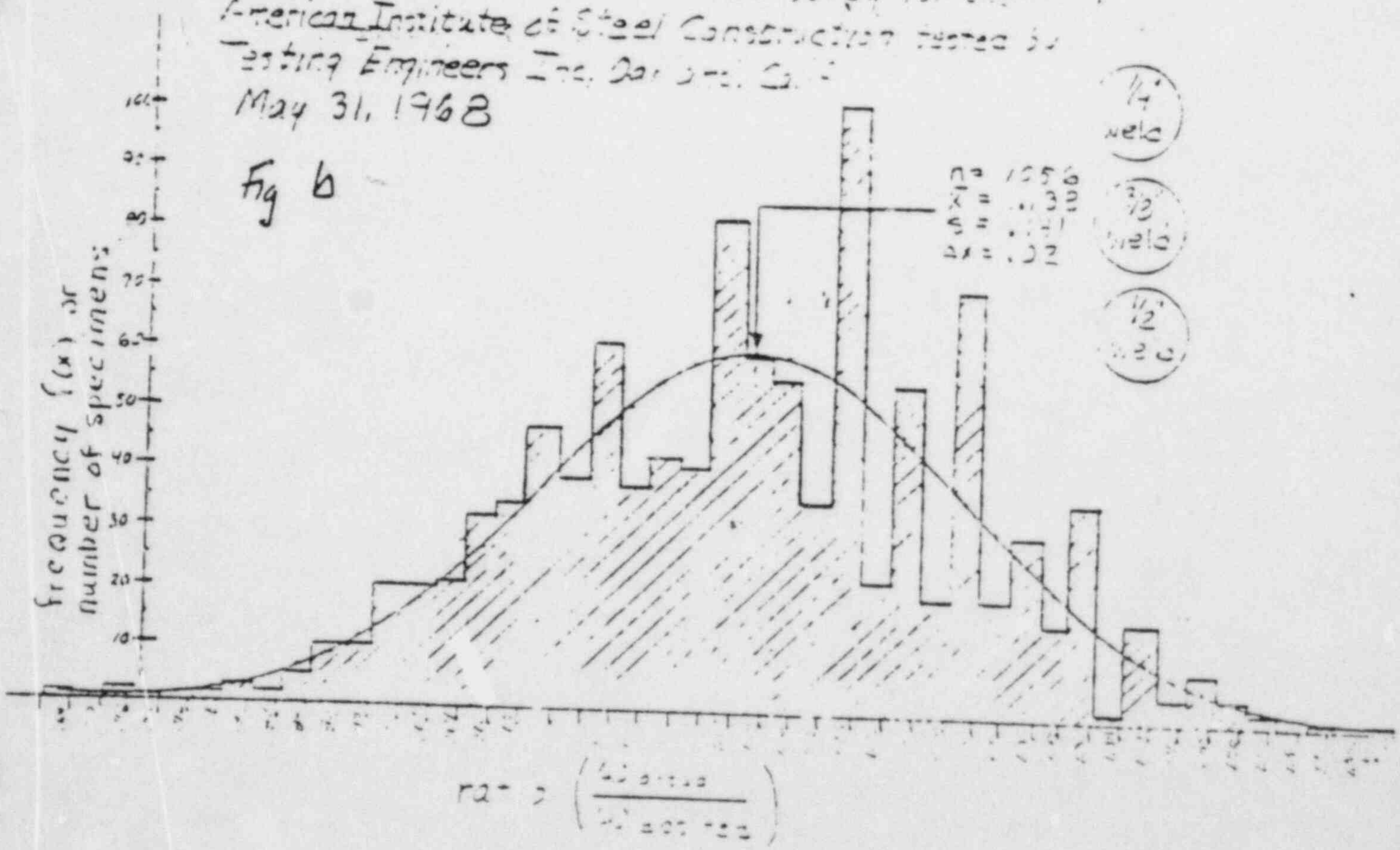



Fig. C

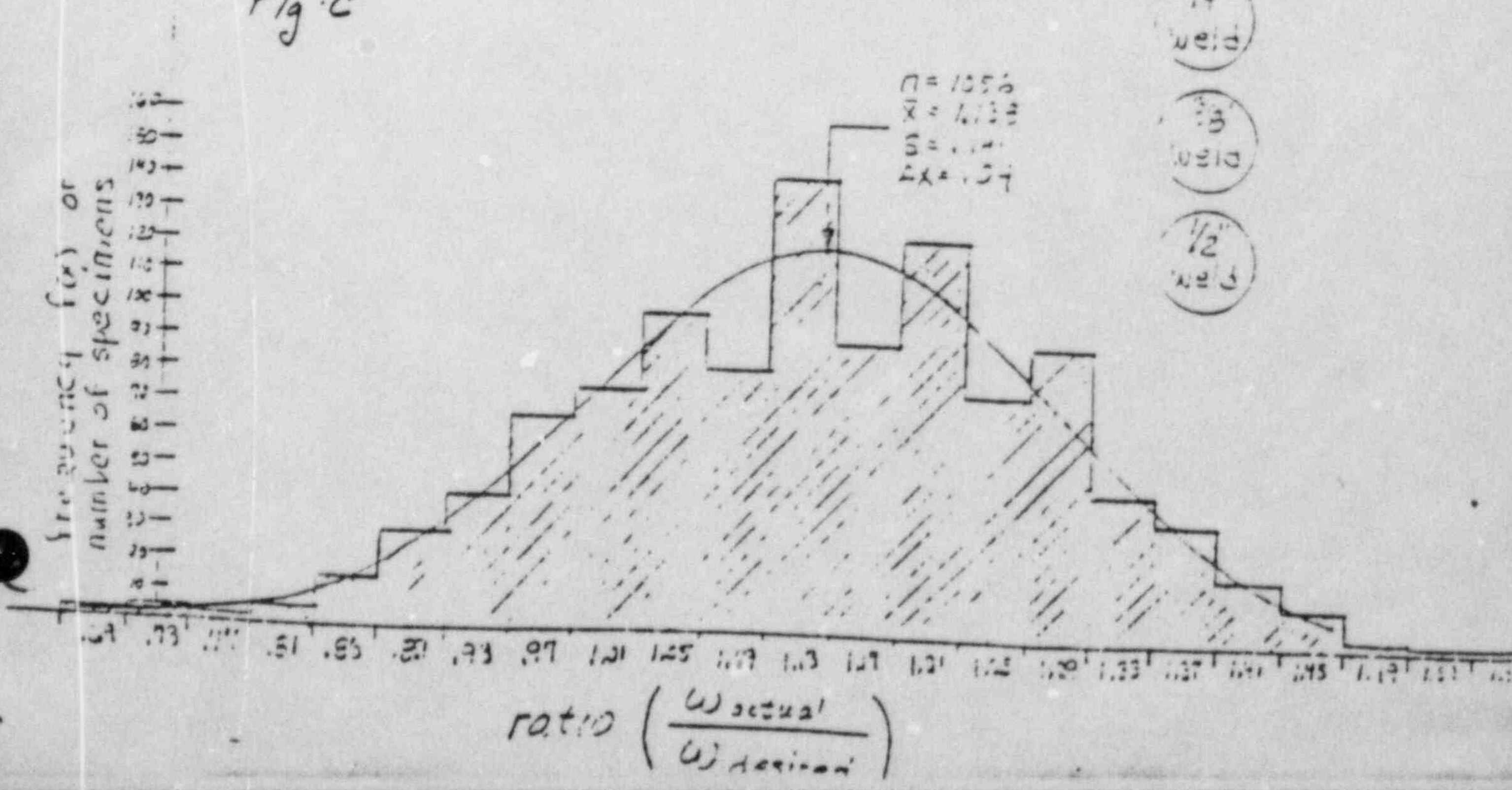
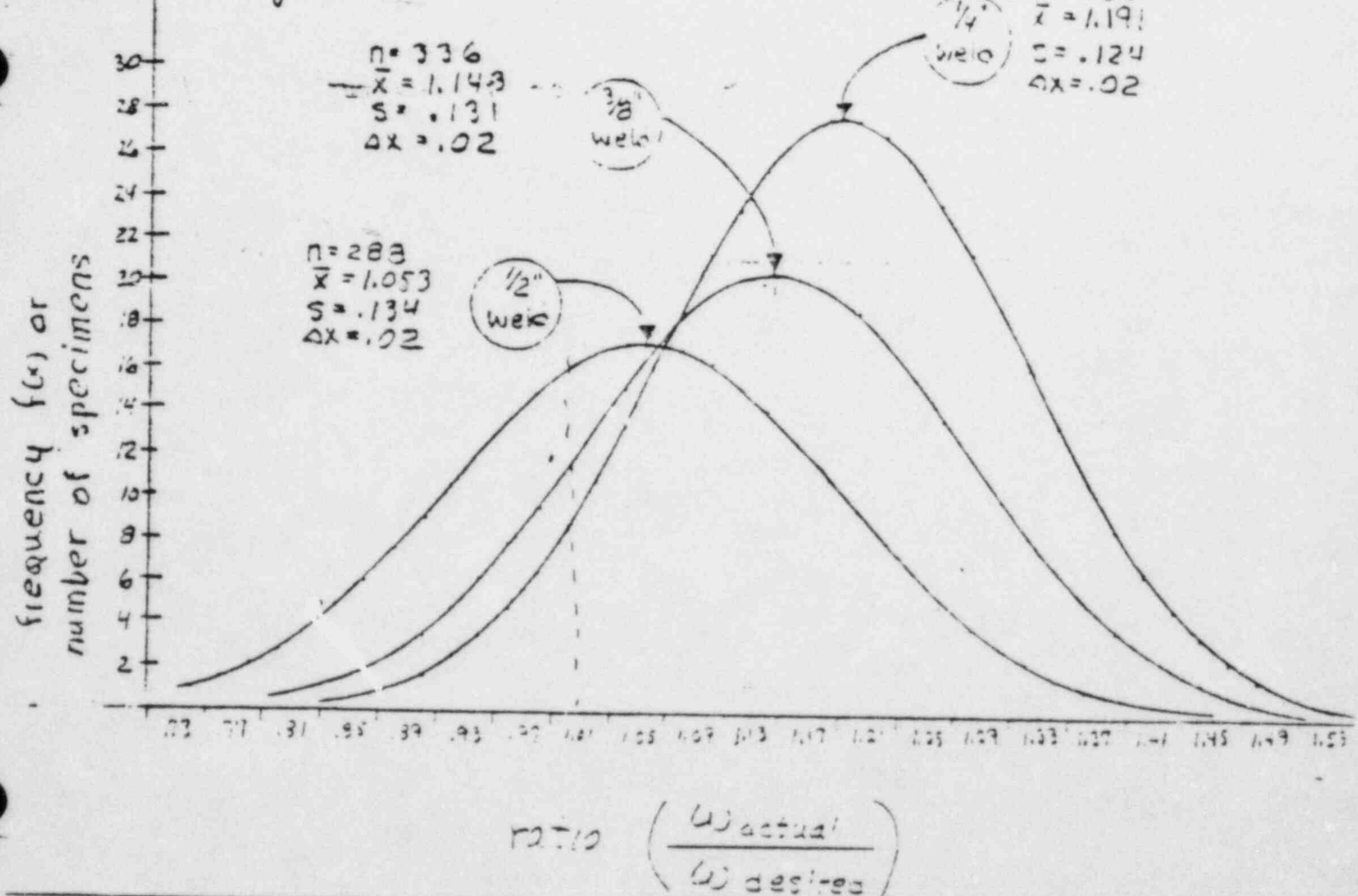


Fig d




Correspondence from Mr W C Cadwell, Asst Ch. Eng of Caterpillar Tractor Co
Peoria, Ill Dec 22, 1964

Of 925 fillet welds checked, from 1/8" to 1/2"

688 (74.4%) from nominal (1.0) to 25% oversize (1.25)

96 (10.4%) exceeded 25% oversize (1.25)

141 (15.2%) under nominal size (1.0)

from this data ;

15.2%, corresponds to 1.
10.4%, corresponds to 1.

$x_1 = \bar{x} - K_1 S$

$1.0 = \bar{x} - 1.028 S$

and

$x_2 = \bar{x} + K_2 S$

$1.25 = \bar{x} + 1.259 S$

from this we get $\bar{x}$ = 1.1
S = .

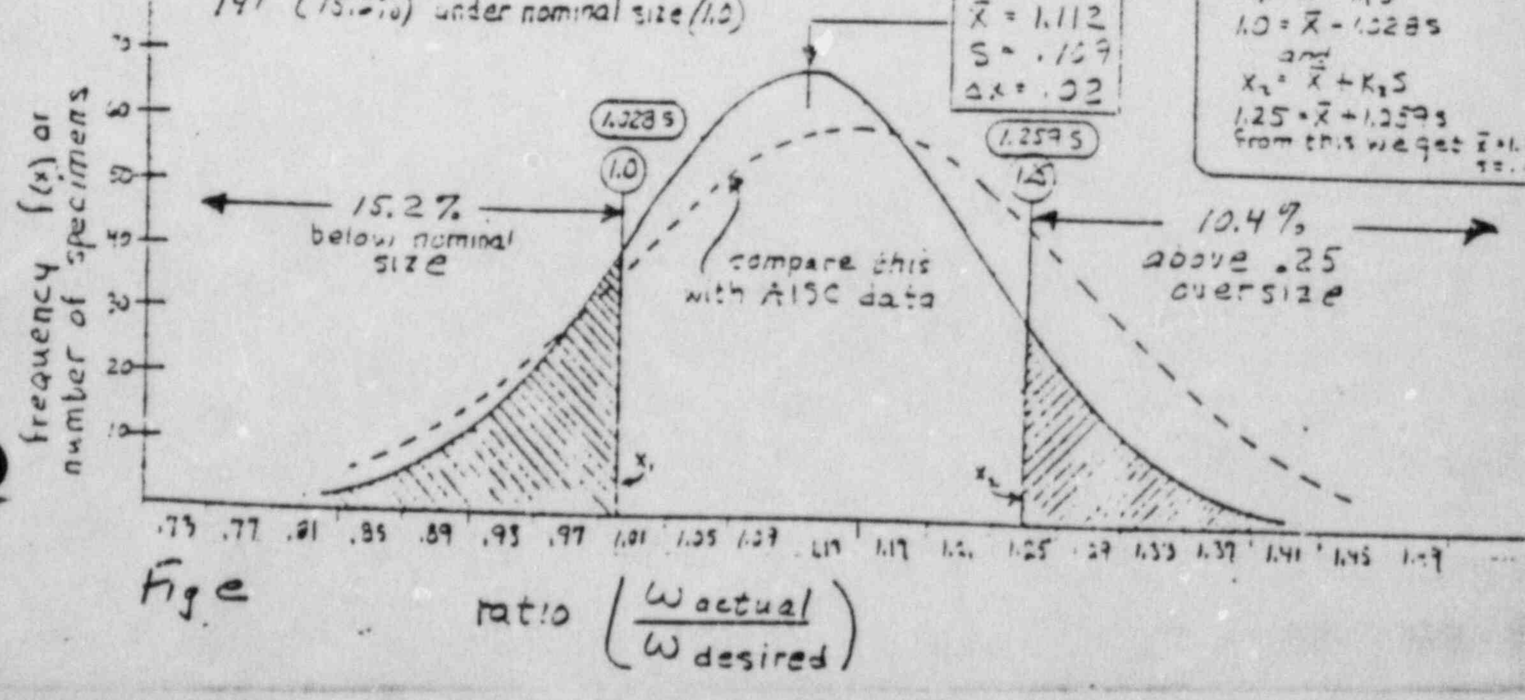



Fig e

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JOURNAL OF THE STRUCTURAL DIVISION

# Load and Resistance Factor Design Criteria for Connectors[a]

By John W. Fisher,[1] Theodore V. Galambos,[2] Fellows, ASCE, Geoffrey L. Kulak,[3] and Mayasandra K. Ravindra,[4] Members, ASCE

## Introduction

Design criteria based on the Load and Resistance Factor Design (LRFD) approach must include a treatment of connections. This report will focus on development of the criteria necessary for the principal fastening elements (welds, high-strength bolts, and ordinary bolts) and will include illustrations of the application of these elements in common types of joints. Comparison will be made with results achieved using working stress design.

As developed in Ref. 11, the LRFD method can be synthesized as

$$\phi R_n \geq \sum_{i=1}^{j} \gamma_i Q_{im} \qquad (1)$$

The left-hand side of Eq. 1 is the resistance of the member or structure ($R_n$ is the nominal resistance and $\phi$ is a "resistance factor"), while the right-hand side gives the effects of the load on the member or structure. Considering, for example, only dead load and live load, Eq. 1 would be written

$$\phi R_n \geq \gamma_D Q_{Dm} + \gamma_L Q_{Lm} \qquad (2)$$

in which $Q_{Dm}$ and $Q_{Lm}$ are the mean dead and live load effects, respectively; and $\gamma_D$ and $\gamma_L$ are the corresponding load factors. The principal purpose of this paper is to develop expressions for the parameters $\phi$ and $R_n$ in Eq. 1.

Note.—Discussion open until February 1, 1979. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 104, No. ST9, September, 1978. Manuscript was submitted for review for possible publication on May 15, 1978.

[a]To be presented at the October 16–20, 1978, ASCE Annual Convention & Exposition, held at Chicago, Ill.

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The development will be based on the use of first-order probabilistic methods.

The fundamental requirements for a well-designed connection can be considered to be:

1. Adequate Strength—It is generally considered good practice that the connections be somewhat stronger than the parts being joined. Thus, if failure should occur, it will take place in the members rather than in the connections thereby ensuring that ample warning (e.g., large deflections) will precede failure.

2. Adequate Ductility—Care must be taken in proportioning the elements of the connection to ensure that ductile behavior will result. Of course, such undesirable phenomena as buckling of plate elements, brittle fracture, lamellar tearing, and excessive local distortion must be avoided. Provision of adequate ductility will mean that the structure containing the connection will have capacity for distortion before failure and will allow for the redistribution of loads. The provision of adequate ductility is a requirement generally less well-defined or understood than that of adequate strength.

3. Economy—As for all structural components, it is desirable that connections be economical of material and be as simple as possible in fabrication.

In working stress design, specifications (13) customarily specify allowable stresses and give rules regarding buckling problems and the like. Although not necessarily obvious, most allowable stresses for fastening elements and most rules for proportioning connections are, in fact, based on ultimate strength considerations. "Traditional" design of connections is much closer to the LRFD approach than most users of these specifications perhaps realize.

## Calibration of Connector Design Requirements

The load factors, $\gamma_i$, and the resistance factor, $\phi$, in Eq. 1 depend upon a "safety index," $\beta$, that is obtained by calibration to existing standard designs (11). Thus, it is intended that successful past practice will be the starting point for LRFD. For beams and columns, it has been found that a value of $\beta$ = 3.0 provides a good estimate of the reliability inherent in current design. This value has been taken also as the basis for LRFD criteria for all other types of structural members. In view of the desirability that connections have a higher degree of reliability than the members they join, the safety index $\beta$ for connections should be somewhat larger than this value of 3.0.

The calibration procedure used here is the same as that followed for beams and columns (11). It will be carried out for various combinations of dead and live load and will cover welds, high-strength bolts, and ordinary bolts.

The safety index $\beta$ is defined (11) as

$$\beta = \frac{\ln \dfrac{R_m}{Q_m}}{\sqrt{V_R^2 + V_Q^2}} \qquad (3)$$

in which $R_m$ and $Q_m$ are the mean values of the resistance and the load effect; and $V_R$ and $V_Q$ are the corresponding coefficients of variation. Detailed definitions of these quantities can be obtained from Ref. 11.



**Welds.**—The weld types used for structural purposes are primarily the groove weld and the fillet weld. In the case of groove welds, the forces acting are usually tensile or compressive. Tests have shown that complete penetration groove welds of the same thickness as the connected part are capable of developing the full capacity of that part. Since it is normal to use weld metal that is at least as strong as the base metal, this means that the properties of the base metal will govern the design. Thus, when complete penetration groove welds are used, design can be based on the properties and behavior of the member in which the connection is being made.

The ultimate strength of fillet welds subjected to shear (the usual case) is dependent upon the strength of the weld metal and the direction of the applied load. The weld may be parallel to the direction of the load (a "longitudinal" fillet weld), transverse to the direction of the load (a "transverse" fillet weld), or at any angle in-between. Regardless of the orientation, the welds fail in shear, although the plane of rupture varies. All experimental studies have shown that longitudinal fillet welds provide lower strength but higher ductility than transverse fillet welds (1,2,7). Since in complex joints it is not always possible to define the direction of loading on the weld and since the longitudinal fillet welds provide the lower bound to weld strength, they will be used here to provide the basis for design recommendations. The results can then be applied in general to fillet welds without reference to the direction of loading.

Early tests on low carbon steels connected by manual arc longitudinal fillet welds showed that the ultimate shear strength on the minimum throat area was 65%-85% of the tensile strength of the deposited material (4,6,12). These early studies also showed that shear yielding was not critical in fillet welds because the material strain-hardened without large overall deformations occurring. Thus, the yield point of fillet welds is not considered a significant parameter.

More recent tests on a wide range of steels connected with "matching" electrodes have provided data on strength and its variability (2,3,8,9). (For many of these tests, data were not obtained on the tensile strength of the deposited weld metal; only the shear strengths were obtained.) Blodgett gives results for 127 samples of weld metal for which the minimum specified tensile strength is 62 ksi (unpublished). The mean tensile strength value, $(\tau_u)_m$, was 66.0 ksi, the standard deviation, $\sigma_{\tau_u}$, was 2.56 ksi, and the coefficient of variation, $V_{\tau_u}$, was 0.039. For a sample of 138 specimens of E70 electrode weld metal (minimum specified tensile strength 72 ksi), Blodgett determined $(\tau_u)_m$ = 74.9 ksi, $\sigma_{\tau_u}$ = 2.67 ksi, and $V_{\tau_u}$ = 0.036. Unpublished studies by Nash and Holtz for the same category gave $(\tau_u)_m$ = 86.8 ksi, $\sigma_{\tau_u}$ = 9.88 ksi, and $V_{\tau_u}$ = 0.247 with a sample size of 40. Blodgett also obtained data from tests on weld metal made with E80, E90, and E110 electrodes. Table 1 summarizes all of the data from Blodgett's report. It is worth noting that Blodgett also obtained results for E70 electrode weld metal that were higher than those listed and comparable to the values found by Nash and Holtz. For a sample of 128 specimens made using E7024 and E7028 electrodes (minimum specified tensile strength 72 ksi), Blodgett obtained values $(\tau_u)_m$ = 85.4 ksi, $\sigma_{\tau_u}$ = 4.77 ksi, and $V_{\tau_u}$ = 0.056.

Until more data are available, it seems reasonable to use the lower bound results listed in Table 1 as the basis of the formulation herein. The value of the ratio of the actual tensile strength of weld metal to its minimum specified tensile strength will be taken as 1.05 with a coefficient of variation of 0.04.

This will be considered to apply to all electrode classifications being considered, i.e., E60 through E110.

Fig. 1 shows a distribution of the ratio of fillet weld shear strength to weld electrode tensile strength for a sample of 133 specimens. The weld shear strength, $\tau_w$, is that for the appropriate matching electrode using the values described herein. These data provide the following results: $(\tau_w)_m = 0.84$, $\sigma_{\tau_w} = 0.09$, and $V_{\tau_w} = 0.10$.

TABLE 1.—Fillet Weld Strength

| Electrode group (1) | Minimum specification tensile stress, in kips per square inch (2) | Sample size (3) | Mean tensile stress, $(\tau_u)_m$ (4) | Standard deviation, $\sigma_{\tau_u}$ (5) | Coefficient of variation, $V_{\tau_u}$ (6) | Tensile stress /specification tensile stress (7) |
|---|---|---|---|---|---|---|
| E6010, E6011, E6027 | 62 | 127 | 66.0 | 2.56 | 0.039 | 1.06 |
| E7014, E7018 | 72 | 138 | 74.9 | 2.67 | 0.036 | 1.04 |
| E8018-X | 80 | 136 | 87.9 | 4.34 | 0.049 | 1.10 |
| E9018-X | 90 | 16 | 100.2 | 4.32 | 0.043 | 1.11 |
| E11018-X | 110 | 72 | 116.9 | 4.68 | 0.040 | 1.06 |

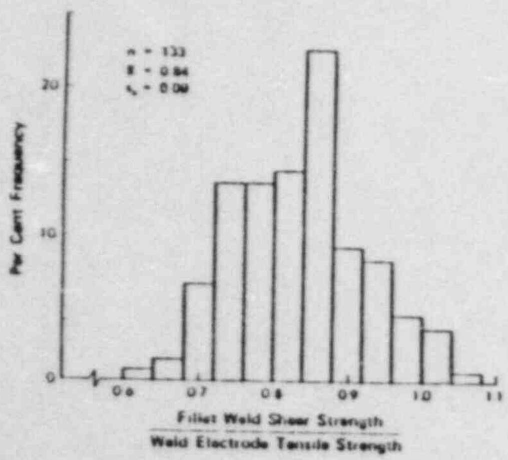


FIG. 1.—Relationship of Weld Shear Strength to Electrode Tensile Strength

The shear strength to tensile strength ratio and its coefficient of variation will be used to evaluate the safety index, β. The mean shear strength of fillet welds can be expressed as

$$(\tau_w)_m = \left(\frac{\tau_w}{\sigma_u}\right)_m \left(\frac{\sigma_u}{F_{EXX}}\right)_m F_{EXX} = 0.84 \times 1.05\, F_{EXX} \quad (4)$$



The coefficient of variation of the resistance, $V_R$, required for the solution of Eq. 3 is defined as (11)

$$V_R^2 = V_M^2 + V_F^2 + V_P^2 \quad (5)$$

in which the coefficients of variation on the right-hand side of the equation represent the uncertainties in material strength, fabrication, and a "professional" factor, respectively.

The variation in the professional assumptions reflect the accuracy with which the forces acting on the fasteners are estimated. The exact determination of these forces is highly complex and they are usually assigned according to a distribution that fulfills the static equilibrium requirements only. However, for a ductile structure, the principles of the lower bound theorem of plasticity are valid. Thus, as no error is made in statics and weld material is provided to resist the forces assigned, the joint will be safe. There is, therefore, no variability of the professional assumptions: the assigned, statically correct forces will be resisted. Accordingly, the term $V_P$ in Eq. 5 is set at zero.

Variation in fabrication reflects the variation of the weld length and throat thickness from those assumed in the design. At the present time, there are not enough data available to obtain $V_F$ quantitatively. A value $V_F = 0.15$ will be assumed for fillet welds. This implies that there is a 50% probability that the actual shear area will be within ± 10% of the area assumed. This is believed to be a conservative assumption.

The coefficient of variation of the material strength from the statistical data available for fillet weld strength is

$$V_m^2 = \frac{V_{\tau_w}^2}{\sigma_u} + \frac{V_{\sigma_u}^2}{F_{EXX}} = (0.10)^2 + (0.04)^2 = 0.0116 \quad (6)$$

Also needed for the calibration is the weld size required by the 1978 American Institute of Steel Construction (AISC) Specification (13). Using Part 2 of the Specification, the design criterion for a load combination of dead and live load is

$$1.7\, A_w \times 0.3\, F_{EXX} = 1.7\, c\,(D_c + L_{rc}) \quad (7)$$

in which $A_w$ = the cross-sectional area through the throat of the weld; $D_c$ = the code value of dead load; $L_{rc}$ = code live-load value as reduced for area; and $c$ is an influence coefficient transforming load intensity to member force. [Note that the load factor (1.7) appears on both sides of Eq. 7; the result obtained here using Part 2 of the Specification are identical to that which would have been obtained using Part 1, allowable stress design, of that same specification.] The mean resistance of a fillet weld designed according to the 1978 AISC Specification is therefore

$$R_m = A_w(\tau_w)_m = \frac{c\,(D_c + L_{rc})(\tau_w)_m}{0.3\, F_{EXX}} = 2.93\, c\,(D_c + L_{rc}) \quad (8)$$

and the corresponding coefficient of variation is

$$V_R = \sqrt{V_M^2 + V_F^2} = \sqrt{0.0116 + 0.0225} = 0.185 \quad (9)$$

Substitution of $R_m$ (Eq. 8), $V_R$ (Eq. 9), $Q_m$, and $V_Q$ (Ref. 11) into the expression

TABLE 2.—Safety Index β for High-Strength Bolts and Fillet Welds

| Dead load,* $D_c$, in pounds per square foot (1) | Tributary area, $A_T$, in square feet (2) | Safety index, β | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | Fillet welds (3) | A325 bolts tension (4) | A490 bolts tension (5) | A325 bolts shear (6) | A490 bolts shear (7) | A325 bolts friction (8) | A490 bolts friction (9) |
| 50 | 200 | 4.20 | 4.81 | 4.74 | 5.86 | 5.23 | 1.46 | 1.32 |
| | 400 | 4.44 | 5.28 | 5.31 | 6.36 | 5.77 | 1.58 | 1.44 |
| | 575 | 4.33 | 5.19 | 5.23 | 6.30 | 5.70 | 1.46 | 1.32 |
| | 800 | 4.56 | 5.58 | 5.72 | 6.69 | 6.15 | 1.61 | 1.48 |
| | 1,000 | 4.70 | 5.83 | 6.03 | 6.95 | 6.43 | 1.71 | 1.58 |
| 75 | 200 | 4.53 | 5.50 | 5.62 | 6.61 | 6.05 | 1.59 | 1.46 |
| | 400 | 4.73 | 5.96 | 6.24 | 7.10 | 6.61 | 1.70 | 1.56 |
| | 720 | 4.50 | 5.71 | 6.00 | 6.88 | 6.39 | 1.47 | 1.33 |
| | 1,000 | 4.67 | 6.02 | 6.41 | 7.19 | 6.75 | 1.58 | 1.45 |
| 100 | 200 | 4.73 | 5.99 | 6.29 | 7.13 | 6.66 | 1.68 | 1.55 |
| | 400 | 4.91 | 6.41 | 6.89 | 7.57 | 7.17 | 1.78 | 1.64 |
| | 600 | 4.82 | 6.34 | 6.86 | 7.52 | 7.13 | 1.68 | 1.55 |
| | 750 | 4.68 | 6.15 | 6.65 | 7.35 | 6.94 | 1.56 | 1.42 |
| | 1,000 | 4.80 | 6.38 | 6.96 | 7.57 | 7.21 | 1.64 | 1.51 |

*Live load is 50 psf for all cases.

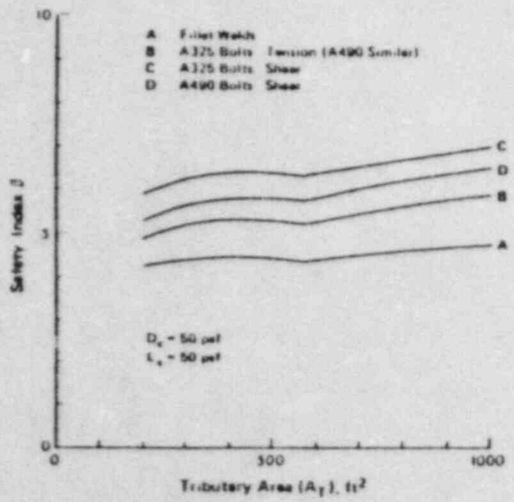


FIG. 2.—Safety Index for Various Connectors



for the safety index β (Eq. 3) can now be performed for a variety of dead- and live-load intensities and for various values of the tributary area. Table 2 lists values of β for the basic code live-load value of $L_c$ = 50 psf and for dead-load intensities of 50 psf, 75 psf, and 100 psf and for tributary areas ranging from 200 sq ft-1,000 sq ft. A plot of β versus tributary area is shown in Fig. 2 for $D_c$ = 50 psf. Examining the tabulated values, it is apparent that β for the whole domain of variables does not change much, the range being from β = 4.20 to β = 4.91. [The safety index has also been examined for higher live-load intensities (75 psf and 100 psf). The minimum value for $L_c$ = 75 psf is β = 5.10 and for $L_c$ = 100 psf it is β = 5.77.]

**High-Strength Bolts.**—A relatively large amount of data concerning the strength characteristics of high-strength bolts are available. The results are scattered throughout a large number of references but these have been well summarized in a publication sponsored by the Research Council on Bolted and Riveted Structural Joints and this will be the principal reference cited in this section (5).

*Direct Tension.*—The mean resistance of a high-strength bolt in direct tension is

$$R_m = \left(\frac{\sigma_u}{F_u}\right)_m A_s F_u \quad \text{(10)}$$

in which $\sigma_u$ = the ultimate tensile strength of the bolts; $F_u$ = the specified minimum tensile strength; and $A_s$ = the tensile stress area of the bolt. The following data are available (5): $(\sigma_u/F_u)_m$ = 1.20 for A325 bolts and 1.07 for A490 bolts; $V_{\sigma_u}/F_u$ = 0.07 for A325 bolts and 0.02 for A490 bolts.

It will be assumed that $V_p$ = 0 (as for fillet welds) and that $V_F$ = 0.05 (reflecting the good control characteristics of bolt manufacturing). In addition, the area of the bolt $A_b$, corresponding to the nominal diameter will be used. This is about 75% of the tensile stress area for bolt sizes commonly used in structural work. Using these data, for A325 bolts:

$$R_m = 0.90\, A_b F_u; \quad V_R = 0.09 \quad \text{(11a)}$$

for A490 bolts: $R_m = 0.80\, A_b F_u$; $V_R = 0.05$ (11b)

The term $A_b$ can be obtained from the 1978 AISC Specification where $1.7(A_b F_t) = 1.7\, c\,(D_c + L_{rc})$ or

$$A_b = \frac{c}{F_t}(D_c + L_{rc}) \quad \text{(12)}$$

in which $F_t$ = the allowable tensile stress as given in the Specification.

The resistance terms of Eq. 11 can now be written as, for A325 bolts:

$$\left.\begin{aligned} R_m &= 0.90\,\frac{F_u}{F_t}\,c\,(D_c + L_{rc}) \\ \text{for A490 bolts:}\quad R_m &= 0.80\,\frac{F_u}{F_t}\,c\,(D_c + L_{rc}) \end{aligned}\right\} \quad \text{(13)}$$

In general terms, Eq. 13 can be expressed as

$$R_m = \left(\frac{\sigma_u}{F_u}\right)_m \frac{A_t}{A_s}\frac{F_u}{F_t}\, c\,(D_c + L_{rs}) \quad \text{(14)}$$

The safety index β (Eq. 3) can now be determined for high-strength bolts acting in tension. The values of $Q_m$ and $V_Q$ are defined in Ref. 11, while $R_m$ is given by Eq. 13 or 14 and $V_R$ by Eq. 11. The specified minimum tensile strength, $F_u$, for A325 bolts up to 1 in. in diameter is 120 ksi and 150 ksi for A490 bolts up to 1-1/2 in. in diameter. The allowable tensile stress, $F_t$, is 44 ksi for A325 bolts and 54 ksi for A490 bolts.

Table 2 lists the values of β determined for this case and they are also shown in Fig. 2 for the particular case of $D_c = L_c = 50$ psf. For A325 bolts, the safety index varies from 4.81 to 5.42 and for A490 bolts it ranges from 4.74 to 6.95.

*Shear.*—The mean resistance of a high-strength bolt acting under a force tending to shear it through a right cross section is

$$R_m = \left(\frac{\tau_u}{\sigma_u}\right)_m \left(\frac{\sigma_u}{F_u}\right)_m A_b F_u m \quad \text{(15)}$$

in which $\tau_u$ = the shear strength; $\sigma_u$ = the tensile strength of the bolt; $F_u$ = the specified minimum tensile strength of the bolt material; $m$ = the number of shear planes in the joint; and $A_b$ = the cross-sectional area of the bolt. The statistical data available for the ratio of bolt shear strength to bolt tensile strength are (5): $(\tau_u/\sigma_u)_m = 0.625$ and $V_{\tau_u}/\sigma_u = 0.053$. These are applicable for both A325 and A490 bolts. The data to be used for the ratio of bolt tensile strength to specified minimum tensile strength are the same as given previously for bolts in tension and are different for the two grades of fasteners. Thus, for A325 bolts:

$$R_m = 0.625 \times 1.2\, A_b F_u m = 0.75\, A_b F_u m; \quad V_R = 0.10 \quad \text{(16a)}$$

and for A490 bolts:

$$R_m = 0.625 \times 1.07 \times A_b F_u m = 0.67\, A_b F_u m; \quad V_R = 0.07 \quad \text{(16b)}$$

In a fashion similar to the development of Eq. 12, the bolt shear area required by the 1978 AISC Specification can be developed as

$$A_b = \frac{c}{F_v m}(D_c + L_{rs}) \quad \text{(17)}$$

in which $F_v$ = the allowable shear stress given in the Specification. The resistance terms of Eq. 16 can now be written as, for A325 bolts:

$$\left.\begin{aligned} &R_m = 0.75\,\frac{F_u}{F_v}\,c(D_c + L_{rs}) \\ &\text{or for A490 bolts:}\quad R_m = 0.67\,\frac{F_u}{F_v}\,c\,(D_c + L_{rs}) \end{aligned}\right\} \quad \text{(18)}$$

In general terms, Eq. 18 can be expressed in the form



$$R_m = \left(\frac{\tau_u}{\sigma_u}\right)_m \left(\frac{\sigma_u}{F_u}\right)_m \left(\frac{F_u}{F_v}\right) c\,(D_c + L_{rs}) \quad \text{(19)}$$

As noted for the case of high-strength bolts in tension, the specified minimum tensile strength will be taken as 120 ksi for A325 bolts and 150 ksi for A490 bolts. The permissible shear stresses according to the 1976 Research Council on Riveted and Bolted Structural Joints Specification and the 1978 AISC Specification are 30 ksi and 21 ksi for A325 bolts (no threads in a shear plane and threads intercepting a shear plane, respectively), with the corresponding figures of 40 ksi and 28 ksi for A490 bolts. The ratios of these shear stresses are approximately the same as the ratio between the gross bolt area and one taken through the root of the threaded portion of a bolt. Thus, the safety index, β, for the two cases will be nearly the same.

The values of β for high-strength bolts loaded in shear are given in Table 2 and are shown in Fig. 2 for the case of $D_c = L_{rs} = 50$ psf. Over the range examined, β varies from 5.86 to 7.58 for A325 bolts and from 5.23 to 7.21 for A490 bolts. It is worth noting that the safety index for high-strength bolts loaded in shear is significantly higher than that for fillet welds.

*Friction.*—High-strength bolts may be used in joints where it is desirable that slip not occur under the working loads. The contribution provided by one bolt to the total slip resistance is

$$P_s = m(k_s)_m (T_i)_m \quad \text{(20)}$$

in which $m$ = the number of slip planes; $k_s$ is a slip coefficient reflecting the type and condition of the faying surface; and $T_i$ = the clamping force provided by the bolt. A good deal of information is known about the slip coefficient and the clamping force and their distributions (5).

The mean value of the clamping force and its distribution depend upon the strength of the bolt and upon the method used for installation (calibrated wrench or turn-of-nut). In either method, the clamping force is to be a minimum of 0.70 times the specified minimum tensile strength of the bolt material, $F_u$, times the tensile area of the bolt, $A_s$. Using the data for bolts installed by the turn-of-nut method (5):

$$(T_i)_m = 1.20 \times 0.70\, F_u \times \frac{1.20}{1.03} A_s = 0.98\, A_s F_u \quad \text{(21)}$$

in which 1.20/1.03 is the ratio of the mean tensile strength of all A325 bolts to the mean tensile strength of the particular lot of bolts used in these tests (both as compared to $F_u$). The coefficient of variation corresponding to Eq. 21 is 0.12 which is obtained by using 0.08 as the variation in the ratio of the actual clamping force to that specified (1.20), 0.07 as the variation in the ratio 1.20/1.03, and 0.05 as the assumed variation due to fabrication uncertainties.

For A490 bolts installed by the turn-of-nut method, the expression equivalent in meaning to Eq. 21 is (5)

$$(T_i)_m = 1.26 \times 0.70\, F_u \times \frac{1.07}{1.10} A_s = 0.86\, A_s F_u \quad \text{(22)}$$

with a coefficient of variation equal to 0.10.

The ... coefficient obtained from a sample of 312 specimens of A7, A36, A440, and FE 37 and Fe 52 (European) steels is 0.336 with a coefficient of variation of 0.07 (5). Similar data are available for a number of other cases. For example, grit-blasted A514 steel has a slip coefficient of 0.331 with a coefficient of variation of 0.04.

The value of the slip resistance expressed by Eq. 20 can now be further quantified. Considering bolts installed by the turn-of-nut method and steels such as A36 with clean mill scale, for A325 bolts:

$$P_s = 0.33\, m A_s F_u; \quad V_R = 0.24 \qquad (23a)$$

and for A490 bolts:

$$P_s = 0.29\, m A_s F_u; \quad V_R = 0.24 \qquad (23b)$$

The 1978 AISC Specification presents the requirements for friction-type connections in terms of an allowable shear stress (even though the bolts are not actually acting in shear):

$$F_v A_b m = c\,(D_c + L_{cc}) \qquad (24)$$

Solving for $m$ and using a value of 0.75 for the ratio of tensile stress area to gross bolt area, $A_s/A_b$, the strength terms in Eq. 23 become, for A325 bolts:

$$\left.\begin{aligned} &P_s = 0.25\,\frac{F_u}{F_v}\,c\,(D_c + L_{cc}) \\ &\text{or for A490 bolts:}\quad P_s = 0.22\,\frac{F_u}{F_v}\,c\,(D_c + L_{cc}) \end{aligned}\right\} \qquad (25)$$

In general terms, Eq. 25 can be written as

$$P_s = (k_s)_m (T_i)_m \frac{A_s}{A_b}\frac{F_u}{F_v}\,c\,(D_c + L_{cc}) \qquad (26)$$

The specified minimum tensile strengths, $F_u$, are again 120 ksi, for A325 bolts and 150 ksi for A490 bolts. The values given by the AISC Specification for $F_v$ are 17.5 ksi for A325 bolts and 22 ksi for A490 bolts. The values of the safety index, β, for joints of A36 (or similar) steel with clean mill scale faying surfaces and using either A325 or A490 bolts installed by the turn-of-nut method are tabulated in Table 2. A plot of values for the case of $D_c = L_c = 50$ psf is shown in Fig. 2. Over the range examined, the safety index varies from 1.46 to 1.78 for A325 bolts and from 1.32 to 1.64 for A490 bolts.

As expected, the values of the safety index are low for bolted, friction-type connections as compared to the other cases considered. This is because the consequences of failure of a friction-type bolted connection are less severe than the failure of high-strength bolts in shear or tension or of fillet welds in shear. A separate value of the safety index should be established for each of the serviceability limit states (bolts in friction-type connections) and strength limit states (bolts in tension or shear and fillet welds).

The value of β = 4.5 will be selected for the strength limit state. This reflects quite accurately the values obtained for fillet welds, except for some cases of high live- to dead-load ratios, and will be conservative for high-strength bolts. It would be in order to select two different values of β for these two cases, fillet welds and high-strength bolts. Although it would be more economical in terms of material used, two values of β would increase the design complexity.

For the serviceability state, β = 1.5 will be used. Based on the cases examined, this represents a reasonable value.



### Determination of Resistance Factor

The resistance factor, φ (Eq. 1), can be expressed as (11)

$$\phi = \frac{R_m}{R_n}\exp(-\alpha\,\beta\,V_R) \qquad (27)$$

in which $R_m$ = the mean resistance; $R_n$ = the nominal resistance as expressed by the design criteria; and α is a numerical factor equal to 0.55 (11). The terms β and $V_R$ have been defined previously. The sections following will establish the values of the resistance factor for the various fastener conditions.

**Fillet Welds.**—The nominal resistance of a fillet weld in shear is customarily taken as 0.6 times the specified minimum tensile strength of the deposited weld metal. This is based on an assumption that the fillet weld is in pure shear and that the distortion energy theory describes the condition of plastic flow. (The "exact" number is $1/\sqrt{3}$ or 0.577.) Calling the throat area of the weld, $A_w$, the nominal resistance is then

$$R_n = 0.6\,F_{EXX} A_w \qquad (28)$$

The mean resistance of the weld is

$$R_m = A_w (\tau_u)_m \qquad (29)$$

As described in the development of the safety index for fillet welds, β = 4.5, $(\tau_u)_m = 0.88\,F_{EXX}$, and $V_R = 0.19$. Substitution of these values and the expressions given by Eqs. 28 and 29 into the expression for the resistance factor (Eq. 27) gives a value φ = 0.93.

**High-Strength Bolts: Tension.**—The nominal resistance of a high-strength bolt in tension is (5)

$$R_n = A_s F_u \qquad (30)$$

and the mean resistance, as given earlier, is $R_m = 1.20\,A_s F_u$ for A325 bolts and $R_m = 1.07\,A_s F_u$ for A490 bolts. For these two fasteners, it was found that $V_R = 0.09$ for A325 bolts and $V_R = 0.05$ for A490 bolts. Again using β = 4.5, it can be determined from Eq. 27 that φ = 0.97 for A325 bolts in tension and φ = 0.94 for A490 bolts in tension.

**High-Strength Bolts: Shear.**—The nominal resistance of a high-strength bolt in shear is (5)

$$R_n = 0.625\,A_b F_u \qquad (31)$$

and the mean resistance, as developed in Eq. 16, is $R_m = 0.75\,A_b F_u m$ for A325 bolts and $R_m = 0.67\,A_b F_u m$ for A490 bolts. The values of $V_R$ were found to be 0.10 for A325 bolts and 0.07 for A490 bolts. Using a value of β = 4.5, the resistance factor (Eq. 27) is φ = 0.94 for A325 bolts and φ = 0.89 for A490 bolts.

**High-Strength Bolts: Combined Shear and Tension.**—For a fastener subjected to both tension and shear, the following relationship has been recommended (5):

$$S^2 + (0.6T)^2 = \phi(0.6\,A_b F_u)^2 \quad (32)$$

in which $S$ is the factored shear force; $T$ is the factored tensile force; and $A_b$ represents either the bolt area through the shank or through the root of the threads, depending upon the actual location of the failure surface.

The resistance factor, $\phi$, can be established from

$$\frac{R_m}{R_n} = \left(\frac{R_{exp}}{R_n}\right)_m \left(\frac{\tau_u}{F_u}\right)_m \quad (33)$$

and $$V_R^2 = \frac{V_{R_{exp}}^2}{R_n} + \frac{V_{\tau_u}^2}{F_u} + V_p^2 + V_f^2 \quad (34)$$

in which $R_{exp}/R_n$ is the ratio of the experimental strength to the nominal strength according to the interaction equation (Eq. 32 with $\phi = 1.0$). The statistical data for the ratio are $(R_{exp}/R_n)_m = 1.05$ and $V_{R_{exp}}/R_n = 0.10$. Using these data and the previously developed information, $V_p = 0$, $V_f = 0.05$, $(\tau_u/F_u)_m = 1.20$ or 1.07 for A325 or A490 bolts, and $(V_{\tau_u}/F_u) = 0.07$ or 0.02 for A325 or A490 bolts, $\phi$ can be determined using Eq. 27 as 0.91 for A325 bolts and 0.85 for A490 bolts.

**High-Strength Bolts: Friction.**—The nominal frictional resistance provided by the clamping action of one high-strength bolt is

$$R_n = m\,k_s(A_s \times 0.7\,F_u) \quad (35)$$

and the mean resistances and coefficients of variation are as given by Eq. 23. The value of $V_R$ was found to be 0.24 for both fasteners. Using these data and the value $\beta = 1.5$, the resistance factor is found from Eq. 27 to be $\phi = 1.15$ for A325 bolts and $\phi = 1.01$ for A490 bolts. In both cases, it has been assumed that the bolts are installed by the turn-of-nut method and that the faying surfaces are in the clean mill scale condition.

**Modified Resistance Factor.**—The use of two different values of the safety index ($\beta = 3$ for members and $\beta = 4.5$ or 1.5 for fasteners) introduces some operational difficulties that must be resolved. Writing Eq. 2 in terms of the dead- and live-load intensities, $D_m$ and $L_m$:

$$\phi R_n \geq \gamma_E(c_D\gamma_D D_m + c_L\gamma_L L_m) \quad (36)$$

in which $\gamma_E$ = the load factor representing uncertainties in the analysis. From Ref. 11:

$$\gamma_E = \exp(\alpha\,\beta\,V_E) \quad (37)$$

$$\gamma_D = 1 + \alpha\,\beta\sqrt{V_A^2 + V_D^2} \quad (38)$$

$$\gamma_L = 1 + \alpha\,\beta\sqrt{V_B^2 + V_L^2} \quad (39)$$

Using the values $V_A = 0.04$, $V_D = 0.04$, $V_B = 0.20$, $V_L = 0.13$, and $V_E = 0.05$ (Ref. 5), the load factors $\gamma$ can be established for the three values of $\beta$. These are tabulated in Table 3.



For beams, columns, and other main structural components ($\beta$ = ., the use of $\gamma_E = 1.1$, $\gamma_D = 1.1$, and $\gamma_L = 1.4$ has been recommended for use in the LRFD format (11). While $\gamma_E = \gamma_D = 1.1$ would still be appropriate for both categories of fasteners, a value of $\gamma_L = 1.2$ should probably be chosen for fasteners in friction-type connections and $\gamma_L = 1.6$ should be used for all other fasteners. However, rather than using different load factors for these cases, the effect of the different $\beta$ factors can be imposed on the value of $\phi$ to be used. For the category described in Table 3 as "Connections—All Others," this means that

$$\phi R_n \frac{1.09\,(1.09\,c_D D_m + 1.39\,c_L L_m)}{1.13\,(1.14\,c_D D_m + 1.59\,c_L L_m)} \geq 1.1\,(1.1\,c_D D_m + 1.4\,c_L D_L) \quad (40)$$

The ratio on the left-hand side of this inequality varies only from 0.86 to 0.90 as the live-load to dead-load effect $(c_L L_m/c_D D_m)$ goes from 2 to 0.25. The corresponding variation for the category "Connections—Friction" is from 1.18 to 1.12 over the same range. Since the variation is not large in either case, it is recommended that the resistance factor, $\phi$, be modified for connections as follows: $\bar{\phi} = 0.88\,\phi$ when $\beta = 4.5$ and $\bar{\phi} = 1.15\,\phi$ when $\beta = 1.5$.

**TABLE 3.—Load Factors for Various Safety Index Values**

| Safety index (1) | Load Factors $\gamma_E$ (2) | $\gamma_D$ (3) | $\gamma_L$ (4) |
|---|---|---|---|
| $\beta$ = 3.0 (members) | 1.09 | 1.09 | 1.39 |
| $\beta$ = 1.5 (connections—friction) | 1.04 | 1.05 | 1.20 |
| $\beta$ = 4.5 (connections—all others) | 1.13 | 1.14 | 1.59 |

The modified resistance factors for the various cases considered are therefore, for fillet welds: $\bar{\phi} = 0.88 \times 0.93 = 0.82$. For high-strength bolts:

1. Tension: A325 $\bar{\phi} = 0.88 \times 0.97 = 0.85$ and A490 $\bar{\phi} = 0.88 \times 0.94 = 0.83$.
2. Shear: A325 $\bar{\phi} = 0.88 \times 0.94 = 0.83$ and A490 $\bar{\phi} = 0.88 \times 0.89 = 0.78$.
3. Tension and shear: A325 $\bar{\phi} = 0.88 \times 0.91 = 0.80$ and A490 $\bar{\phi} = 0.88 \times 0.85 = 0.75$.
4. Friction joints: A325 $\bar{\phi} = 1.15 \times 1.15 = 1.32$ and A490 $\bar{\phi} = 1.15 \times 1.01 = 1.16$.

Clearly, it is desirable to reduce the number of values to be used for the resistance factor to a minimum. It is recommended that $\bar{\phi} = 0.80$ be used for all cases involving the strength limit state, i.e., fillet welds, and high-strength bolts in tension, shear, or combined tension and shear and that $\bar{\phi} = 1.15$ be used for the serviceability limit state, i.e., slip-resistant joints using high-strength bolts. The value selected for the strength limit state is somewhat unconservative for A490 high-strength bolts in shear and for A490 bolts in combined tension and shear. It should be recalled, however, that the value of the safety index

$\beta = 4.5$ as conservative for all cases involving high-strength bolts. The value $\phi = 1.15$ selected for the serviceability limit state is conservative, reflecting the fact that bolts will not always be installed by the turn-of-nut method.

## Related Connector Problems

**Slip-Resistance Connections: Check for Strength.**—When it is considered necessary that connected parts not slip into bearing under service loads, the connection will be designed as a friction-type joint using the criteria already developed for that case. It must be recognized, however, that such a design does not automatically ensure that the criteria established for a bearing-type connection will also be met. Therefore, if the serviceability limit state (slip) is being examined, the strength limit state (both shear strength and bearing capacity) must also be checked.

**Ordinary Bolts.**—It has been customary in the past to apply the same design rules to ordinary bolts [American Society for Testing and Materials (ASTM) A307] as those specified for high-strength bolts (ASTM A325 and A490). Very little data about the strength of ordinary bolts are available and it is therefore recommended that the same procedure be followed, i.e., the LRFD procedures developed for high-strength bolts be considered valid also for ordinary bolts. Of course, ordinary bolts should not be prescribed for friction-type connections since the level of their clamping force is both uncertain as to magnitude and probably highly variable.

**Bolts—Bearing Capacity of Connected Material.**—The bearing capacity of the connected material immediately adjacent to a bolt is a design problem usually associated with the fastener. Strictly speaking, it should be assigned to the member but it will continue here to be related to the fastener.

The nominal resistance in bearing has been established as (5)

$$R_n = e\,t\,F_u \le 3\,t\,d\,F_u \qquad (41)$$

in which $F_u$ = the specified minimum tensile strength of the plate material; $d$ = the bolt diameter; $e$ = the end distance of the bolt; and $t$ = the governing plate thickness (the thinner of the two thicknesses in a lap joint or the least of the sum of the thicknesses of the two outer plies or the thickness of the enclosed ply in a butt joint). Eq. 41 is applicable as long as $e/d$ is not less than 1.5.

The following statistical data relate to Eq. 41 (5): Number of tests = 27; ratio of mean test to predicted values = 0.99; and coefficient of variation = 0.11. With respect to $F_u$, the following data are available (11): Ratio of mean to specified ultimate tensile strength = 1.10 and coefficient of variation = 0.11. From these data, $V_R = 0.16$. Using Eq. 27 and the value $\beta = 4.5$, $\phi = 0.99 \times 1.10 \exp(-0.55 \times 4.5 \times 0.16) = 0.73$.

Modifying this to account for the use of the higher safety index, $\bar{\phi} = 0.88 \times 0.73 = 0.64$.

## Summary and Conclusions

This paper develops the nominal resistance term and resistance factor for each of the commonly used connectors in structural steel. The statistical information necessary for the development is also presented. The work shows that current design values for different connectors provide substantially different levels of reliability.



## Acknowledgments

The work that resulted in this paper was sponsored by the American Iron and Steel Institute (AISI)—Committees of Structural Steel Producers and Steel Plate Producers as AISI Project 163 "Load Factor Design of Steel Buildings." The members of the Advisory Task Force, I. M. Viest (Chairman), W. C. Hansell (Engineering Supervisor), L. S. Beedle, C. A. Cornell, E. H. Gaylor, J. A. Gilligan, I. M. Hooper, W. A. Milek, Jr., C. W. Pinkham, and G. Winter, have been most helpful with their encouragement and advice.

## Appendix.—References

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The increased use of high-strength steels and the need to refer to them in specification provisions resulted in further studies on fillet welded connections.[19.4] Since fillet welds may be made with electrodes whose mechanical properties are not equal to those of the base metal, the study evaluated the influence of type of electrode, size of fillet weld, type of steel, and type of weld. All test specimens were designed to fail in the welds, even though the mechanical properties of the weld metal exceeded those of the base metal.

The study indicated that when longitudinal fillet welds were made with electrodes that "matched" the connected steel, the weld strength varied from 60 to 85 per cent of the electrode tensile strength as illustrated in Fig. 19.3. The study indicated that the failure plane generally was at an angle

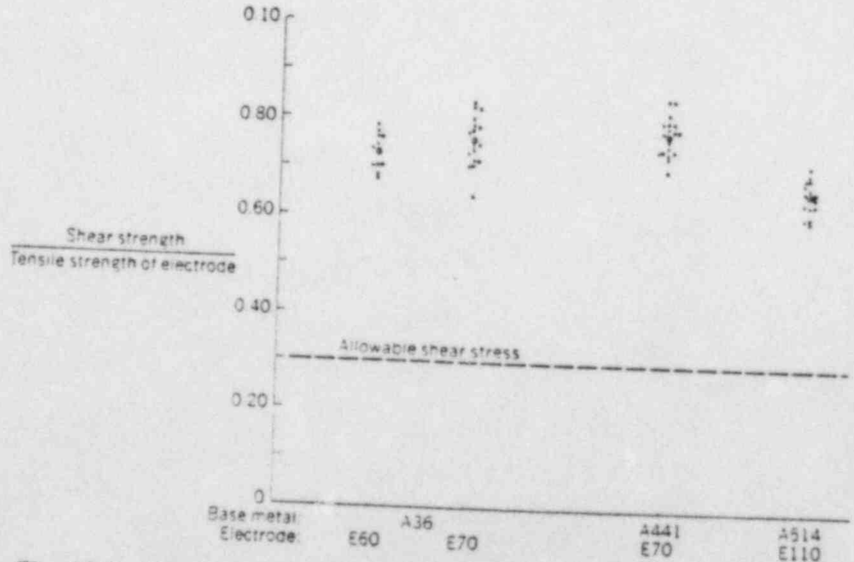


Fig. 19.3 Shear strength of longitudinal fillet welds with matched base metal.

less than 45° to the plane of a leg. Thus, use of the minimum throat thickness is conservative.

Since weld metal may be deposited on base metal with different mechanical properties, combinations of strong base metal with weaker weld metals and vice-versa were also evaluated.[19.4] The results are summarized in Fig. 19.4. This revealed that the effect of dilution upon weld strength was not great.

Where plate bending is not a problem, tests of welds subjected to combined bending and shear have indicated a varying factor of safety against weld failure. The results of tests on vertical weld groups are plotted in Fig. 19.5. As the ratio of eccentricity to weld length ($e/L$) varies from 0.06 to 2.4, the

Attachment E to
KMLNRC 85-058

DANIEL INTERNATIONAL CORPORATION
DANIEL BUILDING
GREENVILLE, SOUTH CAROLINA 29602
(803) 298-2500

February 13, 1985

Dr. Moss V. Davis
American Welding Society
550 N. W. LeJeune Road
Miami, FL 33126

Subject: Secondary Inspection in Accordance with AWS D1.1-75 and Subsequent Issues

Dear Sir:

Daniel International recognizes that AWS D1.1-75 and subsequent revisions require that "welded joints shall not be painted until after the work has been completed and accepted" (3.10.1). Further, it is our understanding that D1.1 is applicable to inspections performed during the fabrication and erection process and does not address subsequent, secondary inspections over the life of the structure. Therefore, when it is desired to perform secondary inspections of structures, it is necessary to develop inspection procedures, and results evaluation criteria specific to that structure.

In light of the above, we submit the following inquiries:

1. Does AWS D1.1 address secondary inspections over the life of the structure?

2. If AWS D1.1 does not address such secondary inspections, what parties are recommended to develop parameters for such inspections?

John G. Berra
Vice President - Operations

Attachment F to
KMLNRC 85-058

# AMERICAN WELDING SOCIETY

*Founded in 1919 to Advance the Science and Technology of Welding*

February 13, 1985

Mr. John G. Berra
Vice President - Operations
DANIEL INTERNATIONAL CORPORATION
Daniel Building
Greenville, SC 29602

Subject: Secondary Inspections in Accordance with AWS D1.1-75 and Subsequent Issues

Reference: Daniel International Corporation Inquiry Dated February 13, 1985

Dear Mr. Berra:

This is in response to your inquiry concerning secondary inspections in accordance with AWS D1.1-75 and subsequent issues.

INQUIRY 1: Does AWS D1.1 address secondary inspections over the life of the structure?

INQUIRY 2: If AWS D1.1 does not address such secondary inspections, what parties are recommended to develop parameters for such inspections.

REPLY 1: No. Inspection (secondary inspection) of welded joints that have been accepted after fabrication or erection, or both, is not covered by AWS D1.1.

REPLY 2: Inspection (secondary inspection) of accepted welds subsequent to the fabrication and erection is not covered by Code provisions and such inspections and criteria for acceptance would have to be as agreed upon by the owner or the Engineer (the owner's representative) and the contractor.

We trust this answers your questions regarding this matter. Should you have any further questions, please do not hesitate to contact me.

Sincerely yours,

Moss V. Davis

Moss V. Davis, Secretary
AWS Structural Welding Committee

MVD:jw
File: D1-30.1
D1e/SC5