ENCLOSURE 2

UNITED STATES
NUCLEAR REGULATORY COMMISSION

IN THE MATTER OF: DOCKET NO:

MEETING BETWEEN KANSAS GAS & ELECTRIC COMPANY
AND THE NUCLEAR REGULATORY COMMISSION

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NATIONWIDE COVERAGE

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

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MEETING BETWEEN

KANSAS GAS & ELECTRIC COMPANY
AND
THE NUCLEAR REGULATORY COMMISSION

AWS STRUCTURAL STEEL WELDING

WOLF CREEK GENERATING STATION

Nuclear Regulatory Commission
Phillips Building
Room P-118
7920 Norfolk Avenue
Bethesda, Maryland

Wednesday, February 27, 1985

The meeting between KG&E and NRC was convened at 9:00 a.m., Hugh Thompson, Director, Nuclear Reactor Regulation, presiding.

CERTIFICATE OF OFFICIAL REPORTER

This is to certify that the attached proceedings before the UNITED STATES NUCLEAR REGULATORY COMMISSION in the matter of:

NAME OF PROCEEDING: MEETING BETWEEN KANSAS GAS & ELECTRIC COMPANY AND THE NUCLEAR REGULATORY COMMISSION

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were held as herein appears, and that this is the original transcript thereof for the file of the United States Nuclear Regulatory Commission.

(sigt) William R. Bloom/sg
(TYPED)

WILLIAM R. BLOOM
Official Reporter
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Reporter's Affiliation

P R O C E E D I N G S

MR. MARTIN: Good morning. I am Robert Martin. I am the Regional Administrator for Region IV.

Darrell Eisenhut and Dr. Denton have asked me to make some opening remarks relative to the purpose of this meeting.

The purpose of this meeting is for Kansas Gas and Electric and its consultants and engineering staff, as appropriate, to make a presentation to the NRC on aspects of the issue of welding deficiencies determined in the miscellaneous structural steel welding at the Wolf Creek facility. The purpose of this meeting is for them to make a presentation on the complete background and scope of the nature of the problem and the corrective actions that have been taken, as well as their implications.

During the course of the presentation I would hope that Kansas Gas and Electric would make their presentation to at least address a few issues to assure themselves that they address certain issues, and let me identify what those issues are in case that should require them to make any last minute modifications in the presentation.

We would like to receive in your presentation, if you will, a history of the entire issue of the miscellaneous structural steel welding with regard to the chronology of events that led to the identification of the issue, including if you will the first time that KG&E was aware of difficulties in this particular area.

We wish for you to address your position as to the issue of whether or not the problems identified during the course of the resolution of this issue have implications that extend to non-miscellaneous structural steel welding, that is, other welding covered by the AWS code, and if it has any implications which extend your views and your position on any implications extending beyond non-AWS welds.

Since the issue, as I think most of the participants in this meeting recognize, was addressed to some extent by enforcement action and that enforcement action addressed the implications to quality program breakdown, similarly we would like you to address the position that you have taken and the basis for it, that the issue does not extend to quality control problems, or whether or not it extends and what actions you have taken to quality control problems in areas of other AWS welding or non-AWS welding, and similarly, to any issues of whether or not any related aspects of the quality assurance breakdown associated with the issue has any implications that extend beyond the AWS area for miscellaneous structural steel, in essence, for the conclusion that your rationale and confidence that (a), miscellaneous structural steel welding is now adequately addressed for Wolf Creek, and any implications that go beyond this.

Certainly an issue associated with this activity was the identification, during the early phases or during at least a phase of this issue, of missing records associated with miscellaneous structural steel welding or at least that which has come to be known as a case of missing records, and we would like you to address the significance of those missing records in terms of any implication to other records relative to the facility.

Again, it's the extension beyond the issue at hand; moreover, whether or not and to what extent the original records currently serve as a basis for your confidence in the existing structures at Wolf Creek.

I think finally, in terms of the overall conclusion of KG&E relative to this, we would like to have some understanding of whether or not KG&E, as the owner utility, the Daniel International Construction and Bechtel and whoever else of your engineering consultants and advisory organizations are now all in accord with the final conclusions that you have drawn relative to structural steel.

I think that in essence addresses at least both the intent of this meeting and the scope of the issues we would hope that you would address during the course of your presentation this morning.

At this point I would like to introduce Darrell Eisenhut, the Deputy Director of NRR, for any further comments that he might have before we turn it over to the utility.

MR. EISENHUT: I think, Bob, you identified most of the issues. I have one point I would make and that is while the technical discussion is largely focusing on the miscellaneous structural steel welding, it is obvious that the real question, at least in my mind, is a much broader question:

What went wrong, how did it go wrong, how do you know you fixed the problem today, what is the structural steel welding problem indicative of and, in the overall broad umbrella, this is rather late in the licensing process for us to be evaluating any kind of a particular issue such as this. It is my understanding information has been evolving over the last couple of months. The real question is how do you, in the first instance, have confidence that the rest of the plant is built in a satisfactory manner?

I think it is fair to say that that will be really the thrust of at least where we're looking. While we certainly want to understand the miscellaneous structural steel welding issue, the questions really we are looking at in a much larger overall umbrella.

I just wanted to emphasize that point. Otherwise, Bob, I think you've covered all the critical issues.

Perhaps we ought to just turn it over to KG&E.

One point I ought to make is that what I think we propose doing this morning is going through your presentation. At the end of that presentation, we would have several things. We would probably ask any elected officials if they or their representatives would have any comments they would want to make, any of the public interest groups if they would have any comments they would want to make. And then the Staff at some point later this morning will have a caucus to discuss among ourselves what we've heard.

As was pointed out earlier, we do have representatives of all of the key NRC offices from the Director on down, of the Regional Office, the Office of I&E and from NRR.

Maybe we ought to just proceed with your presentation.

MR. KOESTER: I believe we will cover all the items you spoke of, Mr. Martin and Mr. Eisenhut. You may at the end want us to elaborate on some of them because we tried to condense this meeting, as was recommended by you last week to Mr. Brown and myself, who is here. Any items you would like further discussion on on any of the items you have discussed, we can do that. But I believe we cover all of the items that you have just brought up during our presentation. But you will have to wait until we get all the way through them before--

MR. EISENHUT: Good.

I want to clear up my comment about condensing. It wasn't meant to condense in size as much as it was to condense the issues. There are a lot of various documents and correspondence floating along, and I thought it would be appropriate for one time to go from the inception of the problem, whatever that was, what was the origin of the issue, how did you discover it, how did it come about, all the way through to the conclusion.

MR. KOESTER: That's what we have tried to do.

MR. EISENHUT: Okay.

MR. KOESTER: Mr. Rathbun.

MR. RATHBUN: As Mr. Koester said, I am Gene Rathbun. I am KG&E's manager of Licensing and Radiological Services.

NRC requested that we come here today to have one last meeting concerning the welding of structural steel at Wolf Creek. That welding was performed in accordance with the American Welding Society code, AWS D1.1, the 1975 edition.

KG&E has a firm commitment to protect the health and safety of the public. That is why we undertook the extensive corrective action program that you will hear about later, to evaluate the acceptability of the structural steel welding at Wolf Creek.

Our secondary inspection program efforts did find minor deviations that gave the appearance of a higher-than-expected reject rate. However, the primary reason for these rejects resulted from the augmented reinspection philosophy that we used.

The vast majority of these deviations would not be rejected by qualified AWS inspector unless they were making the same type secondary inspection that we made.

The fact that KG&E took a more conservative approach during the secondary inspection does not in any way invalidate the primary inspection.

The secondary inspection did identify also a few joints in which some welds had not been made. These primarily resulted from misinterpretations of the welding drawings and not from any inadequacies in the inspection program.

While we strive for perfection, it must be recognized that human errors can and do occur. That is one of the many reasons why we design and build nuclear plants with so much conservatism.

The primary objective of our overall corrective action program was to assure that Wolf Creek is structurally sound and will not fail under accident conditions. We have done that. In doing so we also verified that the welding was done in accordance with the American Welding Society code and our FSAR commitments.

As you hear, we did not limit our reviews in this matter to welding alone. We also looked at other areas to assure that they were also completed in accordance with applicable requirements and our FSAR commitments.

We had three of the leading authorities in structural steel welding independently review our program. Their reviews concluded that we had done a very thorough, conservative assessment. They found nothing to question or invalidate the conclusions we made. You will hear from each of them today.

We firmly believe that the structural steel record is complete and that our plant is constructed, testing is complete, and we are ready to receive the operating license for Wolf Creek.

(Slide.)

The American Welding Society-- The American Society of Mechanical Engineers Boiler and Pressure Vessel Code, the ASME Boiler and Pressure Vessel Code is incorporated into the Code of Federal Regulations. Deviations from the Code must be approved by ASME in the form of Code cases and then later adopted by you, the NRC, in the form of revisions to Regulatory Guides 1.84 and 1.85.

The AWS code, however, is not part of the Code of Federal Regulations. This code is used by the engineer in specifying welding in accordance with commitments made by the plant owner in this Final Safety Analysis Report.

Deviations from the code are evaluated on a case-by-case basis by the architect-engineer who either requires physical plant corrections or decides that the conservatisms in the design can accommodate the deviation. Stated plainly here, final approval for AWS deviations is done by the engineer.

(Slide.)

KG&E has discussed the AWS structural steel welding issue several times in meetings with the NRC. These include many discussions that we held onsite with Region IV personnel, at the enforcement conference that Mr. Martin referred to that was held in October at the Region IV headquarters. Here in Bethesda we met in late November with NRR and Region IV personnel, and as recently as our completion status meeting that we held two weeks ago today in Region IV.

KG&E had completed the extensive AWS welding corrective action program we committed to in the enforcement conference.

Documentation on AWS welding has been provided to Region IV in several forms. First, there was reports filed pursuant to 10 CFR 50.55 (e). There was an interim report and the final report that are shown on the Vugraph.

There was also a final report on our corrective action program which was submitted on December 31st, 1984, and then three weeks later we updated that report with some additional information.

Additionally through verbal requests with Region IV personnel we have provided supplemental information in three different letters that are listed there on the Vugraph.

We are ready today to summarize that information and to answer your questions. We intend for the presentation to be informal, and will entertain your questions at any time. We will pause at the end of each presentation for questions and would prefer, however, that you ask your questions then.

(Slide.)

To help you understand the flow of the rest of the presentation and in the interest of possibly saving some premature questions, I will now briefly run over the agenda.

First as you see, the introduction phase. The

NRC portion is complete, and mine is almost complete.

Next, a discussion on general design philosophy will be given by Jim Ivany of Bechtel, and he will give an overview of the plant's layout, its structural design, and the conservatisms in that design.

Secondly, Bill Rudolph will describe pertinent features of the quality assurance program and the precursors which led up to the KG&E structural steel welding corrective action program, and the elements that are contained within that corrective action program. This material is basically background, not too much of it dealing with the structural steel welding issue.

After that, John Berra is going to describe the history of our AWS welding and explain what AWS welding requirements are, and then outline KG&E's corrective action management plan.

Next, Jerry Brown will describe the engineering evaluations done by Bechtel that he led.

Mr. Koester, vice president, Nuclear, will then introduce our three welding consultants, and then the consultants, who are Mr. Reedy and Drs. Fisher and Egan, will summarize their independent review activities.

Finally, Mr. Koester will summarize our entire presentation.

Are there any questions on the material that I have provided?

(No response.)

Hearing none, I would like to introduce Jim Ivany. He is the SNPPS project civil engineering supervisor for Bechtel.

MR. IVANY: Good morning. My name is Jim Ivany. I am the civil engineering supervisor of the SNPPS project for Bechtel Power Corporation. My presentation will provide you with a brief description of the facility and buildings, and the conservatisms associated within the design philosophy for the weld and structural steel connections.

(Slide.)

The Wolf Creek Generating Station is located near Burlington, Kansas, and uses a nominal 1150-megawatt pressurized water reactor to generate power. It is one of the standardized nuclear power plants that form part of the SNPPS concept.

The power block-- Here is a slide which is in your handout showing the general site features including the main power block and the essential service water system pumphouse which I'll be talking about later.

(Slide.)

The power block consists of several buildings, not all of which are safety-related Category 1 structures. This slide shows shaded the Category 1 structures which are the reactor building, the fuel building, the auxiliary building, the diesel generator building, and the control building.

In addition, one site-related structure is safety-related and that is the essential service water system pumphouse.

These buildings are all reinforced concrete structures with exterior walls at least two feet thick and roofs at least 18 inches thick. The reactor building shell and dome are additionally post-tensioned with prestressed tendons.

The structural steel framing of these buildings consists primarily of steel beams which support internal floors and building roofs. Loads transmitted to the steel framing include those from floors, equipment, piping, heating and ventilation equipment, electrical raceways, laydown components and personnel.

These loads are transferred to the walls or steel columns down to the foundation. The steel framing is connected together and to embedded steel pipes which are mounted in concrete walls and slabs.

(Slide.)

This shows you a sample floor plan looking down at the floor which shows the relationships between steel beams, columns, exterior and interior concrete walls.

In the six Category 1 buildings there are over 11,000 structurally significant connections. These are connections which provide support for safety-related equipment and building components. Of these, approximately 44 percent were shop-welded by the structural steel fabricators in their facilities; 35 percent were field-bolted together in place by the constructor at the job site. The remaining 21 percent consisted of field-welded connections which are the subject of today's discussion.

In order to view the criticality of structural steel welding in its proper perspective I would like to review some of the conservatisms which are inherent to structural welded connections in nuclear facilities.

Typically no credit was taken for this design margins when designing or analyzing welded connections. It is important to point out that the engineering evaluation performed by Bechtel to resolve the AWS field welding issues did not utilize these margins which I am going to discuss.

(Slide.)

The standards used by engineers and constractors in the construction industry for welding of steel are AWS D1.1 and AISE specifications. These are shown in a little bit more detail in the slide.

These codes are applied by the engineer in the context of his design. As such, variations from code requirements in the constructed facility are readily evaluated by the engineer, and the code allows for these assessments of structural integrity.

The allowable stresses specified in these codes are the same for nuclear facilities as they are for commercial structures. The conservatism in this approach becomes evident once the bases for allowable stresses are explained.

(Slide.)

Allowable shear stresses for fillet welds which are the predominant type used for structural steel at Wolf Creek are set at 30 percent of the weld metal ultimate tensile strength. The ultimate shear strength of fillet welds is in the range of 65 to 75 percent of the ultimate tensile strength.

(Slide.)

This relationship is shown graphically on this next slide which shows a stress-strain diagram for the base material used for structural steel and for the weld metal that is used for structural steel welds at Wolf Creek. The allowable stresses that are used for design and analysis are set at a level which is below the ultimate capacity of the steel or the weld metal.

It is also important to point out that these stress-strain curves and ultimates are minimum values. The basic allowable stress is based on a percentage of the minimum values.

(Slide.)

Allowable stresses are specified at a level below ultimate capacity for several reasons, including the following load definition and variations in materials and construction.

Load definition for commercial structures is a matter of safety and economy. For nuclear facilities, load definition is a matter of safety and conservatism.

For commercial structures, load definition is based on a realistic assessment of maximum imposed loads and are typically extracted from local building codes which use national building codes such as the Uniform Building Code. These loads include components such as live loads, wind loads, snow loads, and seismic loads.

Although these loads are not expected to be exceeded during the life of the structure, the potential for overloading does exist. The use of allowable stresses provides for a significant margin to accommodate potential overloads and variations in loads.

For nuclear structures load definition is based on a conservative and complete assessment of maximum imposed loads and are developed based on a much more rigorous investigation of expected loads than local building codes.

Live loads, often in excess of 200 pounds per square foot, are based on maximum concentrated laydown loads during the life of the plant as well as during the construction phase.

Wind and snow loads are based on recurrence intervals which are more severe than for commercial structures.

Seismic loads are determined from geologic data, site investigations and detailed analytical models.

In addition, loads due to abnormal events such as pipe breaks and loads due to extreme environmental events such as tornadoes are also included as part of the design criteria for nuclear facilities.

As such, the design loads are more conservatively and completely defined and are therefore never expected to be exceeded during the life of the plant, and the potential for overload is virtually non-existent, even though the same allowable stresses as for commercial structures are used.

Therefore, based on load definition alone, the use of code allowable stresses provides for a significantly greater margin of conservatism in nuclear power plant facilities than in other types of construction.

The second item I mentioned is variations in materials and construction, which include material properties, construction tolerances, procedural departures, track performanceability, all of which are inherent to the erection of any type of structure. However, the quality control programs that are in effect at nuclear facilities assure that these variations are kept to a minimum.

In addition, code allowables as I mentioned previously are based on minimum material strengths whereas actual material strengths are always higher.

(Slide.)

In the case of AWS welded connections, the weld rod typically exceeds the minimum strength requirements by 20 percent or more.

What that means is that the ultimate strength is 20 percent or more higher than that curve shows.

This same basic allowable stress is used for the design analysis when we did the evaluation for this particular AWS field welding issue. We did not increase our allowables based on actual material strengths. These are conservatisms that were retained.

These are just a few of the conservatisms associated with margins in the design of welded connections. Additional factors of safety can also be demonstrated by examining areas such as enveloping of multi-site earthquakes, consequence considerations for non-critical connections, and conservatisms in the design methodology.

I hope the foregoing sheds some light on the reality of conservatisms which are the basis for safe design of welded connections in structural steel for nuclear facilities.

Large design margins exist in the ability of connections to resist loads prior to compromising structural integrity. When considering these elements, factors of safety against failure can easily exceed 500 percent per connection.

(Slide.)

In summary, when taken all together, the conservative code allowables, the conservative definition of loads, the conservative use of minimum material strengths and minimized variations in materials and construction, when combined with conservative enveloping of multi-site earthquakes, conservative design methodology and consequence considerations for non-critical connections, we realize that there are very large factors of safety against failure in these structures.

Are there any questions at this point?

MR. EISENHUT: Let me ask you a question on something you said earlier.

You said there were about -- I forget the number, on the order of 11,000 or so structural steel welds.

MR. IVANY: Structural steel connections in the Category 1 structures.

MR. EISENHUT: And of course the weld number is much higher. And I thought you said the real focus is on some fraction of those.

MR. IVANY: Okay. There are 11,000 structural steel connections in the six Category 1 buildings. Okay? Of those, a percentage are shop-welded by the fabricators in their facility. They are not field-welded connections. The angle would be shop-welded to a beam by the fabricator.

A remaining percentage is field-bolted together in the field with bolts, no welding at all. Okay? Those two things account for 79 percent of those 11,000 significant connections in the structures.

The remaining 21 percent are field-welded connections. They represent the population that was investigated in our inspection program.

MR. EISENHUT: And at some point in some step of the presentation you will explain then -- if the question is really a question over missing records of field welding, you will explain why you don't have a similar concern over any other aspect, for example, missing records of welded joints or missing records of shop-fabricated welds, et cetera.

I take it from what you are saying that you have made the jump now from the 11,000 connections down to the about 20 percent. At some point you ought to explain why you are convinced that the question of the missing records only relates to these field structural steel welds.

MR. IVANY: That will be addressed in one of the future presentations and if it is not sufficiently clear, we will clarify it after we are done, if there are any specific questions.

MR. DENTON: I think you are giving us a lot of background that we're all familiar with, but let me ask one question about three things that Darrell mentioned.

You have looked at conservatisms in shop welds, field welds and bolted connections. Are they the same conservatisms in all three classes? Or how would you characterize the level of conservatism?

MR. IVANY: I would characterize the level of conservatism in them all to be equal. These are conservatisms in design philosophy for structural steel. Some of the specific things about welding obviously don't relate to bolting, and so on, but these conservatisms in terms of load definition, variations of material and construction as well as the other items down here are not specific to the field welding issue except that they all apply.

They also apply to a lot of other areas of design.

I don't know if that answers your question.

When you design concrete structures you've got similar conservatisms which are inherent to the design philosophy.

MR. LIAO: You show a stress-strain curve in which you have a basic elemental stress. Can you explain to us the basis of specifying the basic allowable stress?

MR. IVANY: The basic allowable stress is a function of the ultimate tensile strength of either the base metal or the weld material.

MR. LIAO: In this case what exactly--

MR. IVANY: Thirty percent of the ultimate tensile strength of the welded material. That's the allowable tensile stress.

If there are no other questions, Mr. Bill Rudolph, the quality assurance manager at Wolf Creek, will now provide you with details relating to the quality assurance program and corrective action.

MR. DENISE: Jim, you've mentioned twice about the enveloping seismic loads. Do you recall what the controlling plant site was in the SNPPS design?

MR. IVANY: The SNPPS design envelope was a safe shutdown earthquake as committed to in the Final Safety Analysis Report is at a level of .20g. Wolf Creek is at .15g.

Mr. Bill Rudolph.

MR. DENTON: Could we ask a question of a previous speaker?

Just to follow up Dick's question about the seismic design, how important is seismic design loads in this particular issue we're discussing today?

MR. IVANY: It depends on where you are. Obviously there's a lot of structural steel in these buildings. Certain structural steel is controlled by seismic considerations, and for certain structural steel, seismic considerations are not the critical load combination. It might be pipe break; it might be heavy equipment laydown areas during construction.

There are some areas that are controlled-- As a percentage-- I would not want to, you know, estimate right here what percentage of the structural steel welding is controlled by seismic.

MR. DENTON: Well, can you take a case where there was a missing weld and discuss to what extent these conservatisms apply at that place? Take the pressurizer. Weren't there some missing welds on the pressurizer?

MR. IVANY: Okay.

MR. DENTON: Could you characterize to what extent your previous general discussion would apply in that location?

MR. IVANY: Okay.

The pressurizer support-- Specifically the most critical loading condition for the pressurizer support I believe was a combination of pipe break, loss of coolant pipe break at the pressurizer which created overturning and torsional loads on the pressurizer combined with safe shutdown earthquake loads.

And the conservative definition of "loads"-- I would have to go back to Westinghouse to see how much conservatism there is in their pipe break loads and their safe shutdown loads. They're the NSSS supplier. But we use their loads which are typically enveloped for several different conditions, so those loads in themselves may in fact be enveloped. I can't speak for them because I didn't generate those loads.

We did not reduce those loads or cut back on that load definition, so we have a conservative load definition. The code allowables we used based on the loading combination would be a function of that basic allowable stress.

We used minimum material strengths for evaluating the -- setting the allowables at a level below ultimate.

The quality control program that is in effect verified minimum variations in materials and construction.

We did not take advantage of enveloping of multi-site earthquakes. We used the envelope that Westinghouse gave to generate the safe shutdown earthquakes. Conservative design methodology is always there.

That's the conservative assumptions made in the actual analytical approach.

The consequence considerations we did not utilize, in other words, the consequences of one of those connections actually failing. We did not use that margin.

So basically all of these things apply in different contexts to structural steel welding. Those margins are there.

MR. KNIGHT: A point of clarification.

You mentioned the SNPPS seismic design level being .2g, the site specific for Wolf Creek being .15g.

Do I remember correctly there are some site-specific structures that are designed at a lower level?

MR. IVANY: That's correct.

MR. KNIGHT: Are those all concrete or are they structural steel?

MR. IVANY: The essentially service water pumphouse is a reinforced concrete structure but it does have some internal steel framing with field-welded connections.

MR. KNIGHT: Those were designed to the .15g level, site-specific?

MR. IVANY: Correct.

MR. DENISE: Jim, let me help you a little bit with Harold's question.

He was trying to find out from the Bechtel side when you designed the structure, when you were given those loads of pipe whip or pipe break forces combined with seismic forces, whether you knew -- Let me take the pressurizer supports -- whether you knew which one of those loads controlled, or what percentage came from which part, not the question of whether Westinghouse gave you a conservative number or anything but you did apply those loads to the structure and you designed the structure.

MR. IVANY: Correct.

MR. DENISE: If you had 50 percent of the load that was due to pipe break and 50 percent of the load that was due to seismic, or seismic was more controlling, I think that's the thrust of the question.

MR. IVANY: We've got a number of load combinations for all structures that we have to consider. We consider all load components you are talking about there. When we get loads from, for example, Westinghouse for the pressurizer, what we would do, we don't get just one load, we get a series of loads and load combinations from them.

We then take those loads and make them a part of the design criteria for that structural steel, combining it with whatever other loads we know of. We establish a load set of combinations.

Typically if there are load combinations that appear to be at the more critical level, we would analyze all those load combinations on the steel to ensure that all of them are satisfied. Only one may be critical for a particular connection, and for the next connection over it might be a different load combination that governs. But we would take into consideration all of the load components and combinations that Westinghouse supplied us, together with any additional loads that we have in that area.

I don't know if that answers the questions.

MR. DENISE: Let me see if I make sure I understand you.

I think the answer that I'm receiving is that you don't have any of that specific information here today to speak to any particular joint today, but if we were interested in following it up, you could tell from your records what went into the design and look at it on a joint-by-joint basis. There is no general answer that you can give that applies.

MR. IVANY: Yes.

MR. DENISE: Is that correct?

MR. IVANY: That's a general answer but that's correct. We can look at any particular joint, any particular connection, and get the full basis for the load definition on them, and what is critical.

(Slide.)

MR. RUDOLPH: My name is Bill Rudolph, and I am the Quality Assurance Manager for Wolf Creek.

I am going to discuss the corrective action program to nullify KG&E quality assurance, which led to a detailed management action plan for the resolution of the AWS welding concerns at Wolf Creek.

MR. EISENHUT: Before you go on, the standard question I ask QA managers, can you describe, first, how long you have been a QA manager, what your background is?

MR. RUDOLPH: Absolutely.

MR. EISENHUT: Were you the QA manager at the time the problem occurred as well as in the regime now?

MR. RUDOLPH: Let me tell you my credentials and the time period that they pertain to, and I think the presentation will answer that question.

MR. EISENHUT: If I am getting ahead, go ahead.

MR. RUDOLPH: My credentials are not part of my presentation.

I have a Bachelor of Arts Degree in physics and mathematics that I received in 1971 from a small college in Pennsylvania.

I have a Master of Arts Degree in education from the University of Pittsburgh, which I received in 1974.

I have a Master of Science Degree in nuclear engineering, which I received from Carnegie Mellon University in 1975-76 time period. I forget. I could find out real fast by looking at my resume.

(Laughter.)

I have a senior reactor operator's certification. I am a lead auditor, certified ANSI N.45.223, and I have been in the business for about 10 years, four of which have been in the quality assurance business. I have been quality assurance manager at Wolf Creek since April of 1983.

So I have two Master's Degrees, two Bachelor's Degrees.

MR. EISENHUT: As we go through, you will index the history of the problem in a later part of the presentation?

MR. RUDOLPH: If my presentation doesn't satisfy your curiosity, I can provide you more information.

(Slide.)

Federal law, specifically 10 CFR 50, Appendix B, requires every application for a construction permit and operating license to establish a quality assurance program, to be applied in their design, fabrication, construction, testing, and operation of their facility.

KG&E's quality assurance program has established and implemented these requirements to provide the utmost confidence that Wolf Creek will operate safely and reliably.

One of the main elements of the Wolf Creek quality assurance program pertains to the prompt identification, control, and resolution of hardware and programmatic deviations. Multiple levels for the identification, control, and resolution of hardware and programmatic deviations exist at Wolf Creek and extend from the quality assurance program implemented by major site contractors to the quality assurance program implemented by Kansas Gas & Electric.

Initially, a comprehensive system or plan and periodic audits and surveillances implemented at KG&E to verify compliance with all aspects of the quality assurance program and to determine the effectiveness of that program. The individuals performing these evaluations have sufficient authority and organizational freedom to identify problems, initiate, recommend, or provide corrective actions, and to verify the effective implementation of these corrective actions.

Whenever a hardware or programmatic deviation is identified, the responsible organization is required by our QA program to initiate the appropriate documents to resolve the concern. These documents have specific titles based on the type of deviation.

For example, hardware deviations are addressed on nonconformance reports. Generic broad scope deviations, either hardware or programmatic in nature, are typically resolved by initiating a document called a corrective action request, or simply what I will refer to in this presentation as a CAR.

These corrective action documents and the programs that govern their use are formally structured and systematically applied and represent a significant contribution to our overall quality assurance program.

(Slide.)

As a result of welding deviations identified at another nuclear plant, a variety of actions were taken by our constructor, Daniel International Corporation, which subsequently led to the initiation of several Daniel corrective action reports to address these generic deviations. These corrective action reports were limited in scope and did not address some of the welding concerns, which will be discussed today.

In July of 1984, the NRC established a task force to assure that that timely completion of the NRC's construction inspection program at Wolf Creek. During these inspections the NRC developed some concerns with the resolution of two Daniel corrective action reports associated with AWS D-1.1 safety-related structural steel welding.

As a result, KG&E performed a reevaluation of these Daniel corrective action reports. This reevaluation involved document reconciliation and limited weld reinspections. As a result, a potential 50.55(e) was reported to the NRC.

This reevaluation plus additional substantial comments provided by senior NRC task force members resulted in the development and implementation of a comprehensive corrective action program. This corrective action program, known as KG&E QA Corrective Action Request 19, or simply CAR-19, was issued in the KG&E construction organization for their action. KG&E construction then developed a comprehensive management action plan to resolve the findings of CAR-19.

MR. MARTIN: Let me back you up to your first box. I am going to play some of your words back on you.

You have the corrective action reports, weld deviations and record retrievability. What timeframe are we talking about? Are we -- is this the appropriate point to address, if you will, the history of when did you get your first indications of MSSW problems?

MR. RUDOLPH: That time period is February 1983.

MR. MARTIN: 2/83?

MR. RUDOLPH: That is correct.

MR. MARTIN: Okay.

MR. EISENHUT: You also, as I understood, said the reason you undertook that was because of information that had been developed at another facility?

MR. RUDOLPH: Yes, sir.

MR. EISENHUT: So I take it, then, in the absence of that information from the second facility, other facility, you would not have been going down this path, even in the first place?

MR. RUDOLPH: Yes, we would have.

MR. EISENHUT: Then there must be another origin of --

MR. RUDOLPH: There are precursors to that.

MR. MARTIN: Is this the time to discuss those precursors?

MR. RUDOLPH: We can if you wish.

MR. MARTIN: Do you have it in your presentation to discuss the precursors later?

MR. RUDOLPH: Absolutely.

MR. KOESTER: The backup information we have with you.

MR. MARTIN: I think this is the time then.

Quite frankly, I recognize that typically in the discussion of a licensee's QA program that you use words like "prompt," "comprehensive," "to look at all aspects," and "appropriate resolution."

Now, I am looking at something that is a chronology, a February of '83 to a July of '84 timeframe, and then on through.

What I would like to go back is look at the precursors, and then let's talk about how did it work and where did it not work so that we can start this discussion that we asked about before.

How do you know that it has not pervaded into other areas of your activities, and what assurance do you have and therefore can convey to us that it has not or has, and what you have done in those areas?

MR. RUDOLPH: We can do that.

MR. KOESTER: I was just asking some of my folks if we thought any of the other presenters covering this -- and what we will show you here now I am sure Region IV has seen it. I think it was in our enforcement conference presentation.

MR. RUDOLPH: We presented this information. I know I have personally two times before.

(Slide.)

There are precursors, and then beginning in February of '83 my presentation addresses what happened from that point in time to the present.

The precursors prior to February of '83 were initiatives that KG&E took which in and of themselves identified and resolved concerns which were indirectly related to the concerns associated with AWS D-1.1 welding of structural steel.

However, an important point to make is had we not done this precursor investigation and work, we still would have identified the problem in the March-April time period of '83, and I think that will become apparent.

MR. EISENHUT: Can you just help me, though, with this point? How would that -- how would you have gone down this path to resolve these? You are starting in early '83. If you had not gone down this path, because of the problem of the facilities and then the NRC task force, how would you have gotten on that path?

MR. RUDOLPH: I will answer that by going through this slide.

MR. DENTON: Why don't you walk us through this, assuming that most of us have not seen this?

MR. RUDOLPH: I would be happy to do it.

MR. DENISE: Denton.

MR. DENTON: I would be interested to know what the other facility was. Is that a facility built by Daniel, so that you don't keep it a mystery?

MR. RUDOLPH: Let me begin with my presentation, and I will fill in these gaps that you want to be informed about.

In September of 1980, a concern was identified at another project, another project with Calloway. It was built -- it was designed by SNPPS, the same participants.

The concern involved undersized socket welds which are a type of fillet weld, as you know.

We performed a sample inspection as a result of being informed of this concern to determine if a similar concern existed at Wolf Creek. This is good QA practice. We were aware of the concern. We wanted to become knowledgeable of that concern, as is apparent in our facility.

At that same time, Daniel initiated a corrective action report called Daniel CAR-7, which was issued as a result of this sample inspection program. A 100 percent reinspection of socket welds on small bore piping was made prior to June of 1980.

The reason we did that is because the inspection technique had been changed. The inspection technique on socket welds had been changed from a 180-degree inspection technique to a 360-degree inspection technique.

MR. DENISE: Let me clarify to be sure that we all understand what you said. There was a 100 percent reinspection of the small bore welds which were made before June '80?

MR. RUDOLPH: Yes, sir.

MR. DENISE: The reinspection was not before June '80; the reinspection was --

MR. RUDOLPH: It was made prior to June.

In March of 1981. This slide says March of 1980. That is a typographical error. It should be 1981.

Some mechanical, structural, and electrical deficiency reports were written by Daniel. While installing fireproofing, they noticed that there was some concern in those three areas.

The mechanical and structural deficiency reports were closed in May of 1981 because of the corrective actions taken and the significance, which was minor in that degree. The electrical deficiency reports, or the problems identified on those deficiency reports were addressed by a subsequent Daniel corrective action request which was called CAR-9.

MR. EISENHUT: I am still thinking about the first item.

When you had undersized socket welds and went through the overall reinspection program with a 100 percent, as discussed, did you at that time submit a 50.55(e) report or any kind of report to the NRC?

MR. RUDOLPH: Absolutely.

MR. EISENHUT: Help me also understand the corrective action reports. Are those reports the kind of reports -- have you submitted those to the NRC?

MR. RUDOLPH: I don't typically submit deficiency reports and nonconformance reports to the Commission unless they are requested specifically. The residents have full capability to -- of course, as you are aware of -- look at those deficiency reports or nonconformance reports at their leisure.

MR. EISENHUT: And at that time did you focus on what the cause of the problem of the deficiencies in the socket welds was? Do you know whether -- when you did the evaluation and had the corrective action report, did you focus on what the cause of the problem was?

MR. RUDOLPH: Yes. The Daniel corrective action report that was generated as a result of these deficiency reports identified the cause, took the appropriate corrective action, and also took action to prevent recurrence.

So it was a full scope application of the QA program.

The actual cause -- John Berra, who was Vice President of Daniel -- do you remember or recall what the actual cause of those deficiency --

MR. BERRA: No, I don't. They weren't -- I know they didn't relate to missing records. I don't recall.

MR. RUDOLPH: Like I said before, the deficiency reports were based on the fact that the inspection philosophy had changed. That was the cause.

MR. EISENHUT: The cause of -- can you help me? Very simply, when you did this you found, I take it, some deficiencies, a certain percentage of deficiencies as a result of the 100 percent reinspection of that group of welds.

Can you give me an indication, roughly? Was it a small number of problems found, a large number of problems found?

MR. RUDOLPH: What we did -- I don't know if this will address your concern, but what this basically ends up doing is my organization generated the surveillance, which is the next block in the progression of events. That surveillance went back and looked at these deficiency reports in those three areas -- the mechanical, structural, and electrical areas.

The intent in writing that surveillance report was to identify any adverse trends. There were no adverse trends identified in the mechanical-structural area; however, there was an adverse trend in the electrical area, and that adverse trend was corrected as a result of Daniel initiating CAR-9.

MR. DENTON: I think with the benefit of

hindsight, was there anything in that review that could have led you to forecast the structural problem?

MR. RUDOLPH: No.

MR. DENTON: So you don't think that that relates to the next problem that cropped up?

MR. RUDOLPH: That is correct. I do not believe that they relate to the next.

MR. DENTON: Maybe if you go through the chronology, which goes back quite a ways, you might tell us where it does begin. So if we had known then what we know today, we could have fixed it years ago.

I guess that is what I am interested in, in your conclusion as to whether that was possible or whether it was one of these things that really wasn't known.

MR. RUDOLPH: As I go down through here, I think you will see the transition and the sequence of events, thought processes, and so forth, that led us into discovering this miscellaneous structural steel situation.

So on September 1981, Daniel initiated CAR-9, which were part of the corrective actions associated with our surveillance report, KG&E's Surveillance Report No. S-372.

In the August time period, 1982, Daniel initiated another corrective action report, CAR No. 19, that required a 100 percent reinspection of fillet welds made prior to

April 1, 1981 on ASME and special scope pipe hangers.

What we did in April of 1981 is we went and retrained our quality control inspectors, and that CAR was generated as a result.

At that point in time -- and this is the transition, gentlemen, I think you are concerned about -- at that point in time DIC Daniel then gently questioned, if we had a concern in the ASME and special scope areas, did we also have basically the same concern in the AWS D-1.1 area, and that is the critical point. Everything prior to February of 1983 were precursors. We went from socket welds to fillet welds to ASME and special scope welding.

Then logically one would assume, or could assume, if applying judiciously the quality assurance program, do we have a concern in another area? But that is the transition. We asked ourselves and we took the initiative and investigated another area, and that other area was AWS D-1.1.

Is that clear on that transition?

MR. KNIGHT: To clarify for me, so I would understand that, one, there were some deficiencies in welds of various types; two, you discerned that you weren't finding those as readily as you should have been finding them. That led you to retrain inspectors.

Is this what I am hearing?

MR. RUDOLPH: Yes. We retrained our inspectors, basically because we had some concerns in the area of fillet welding.

MR. KNIGHT: Okay. But the two facts are pertinent. One, there were -- the crafts were performing, if you will, some unacceptable welding or welding with deficiencies?

MR. RUDOLPH: Not to a significant level.

MR. KNIGHT: Okay, but some?

MR. RUDOLPH: Yes. The answer is --

MR. KNIGHT: I am not debating that point. I just want to be clear in my mind. Okay?

And the larger point really being, and the whole reason we have inspectors, is that they were not being picked up with the acuity, if you will, that we would desire, and you then retrain and at that juncture would have assumed that you now had the process under control?

MR. RUDOLPH: Yes.

MR. KNIGHT: That was prior to -- but that is the point I would like clarified. Was it prior to or in some close conjunction, where you said, well, we had better look at some other areas?

MR. RUDOLPH: That is correct. About a year -- April of '81.

MR. KNIGHT: Okay, and then at that juncture, you started looking at other areas?

MR. RUDOLPH: We began looking at other areas, in essence, in August of '82. In other words, we looked at ASME and special scope, and then we decided to look at AWS D-1.1, and that occurred in 1983, February of 1983. We performed --

MR. KNIGHT: So this was a transition period, and during that year and a half or so you were looking --

MR. RUDOLPH: Well, we continued to construct and do our job as we had.

MR. THOMPSON: Would you give me some feel for the scope of this training that you put your inspectors back through? Was it a week long? How many inspectors were involved? What level of deficiencies were you trying to address?

MR. RUDOLPH: I can't specifically address that, John. Berra may be able to.

MR. BERRA: I don't recall the number. It was all the welding inspectors at the jobsite. I don't know how many there were at that time. They were put through a program, a certified program, in accordance with ANSI-45.2.6, which determines how you -- the qualification for inspectors.

So it was one of our certified programs, according to AMSE, that we put the inspectors through.

MR. THOMPSON: So you ran the program? Daniel?

MR. BERRA: Daniel ran the program.

MR. THOMPSON: And how long a program was it? Was it a week long, two weeks?

MR. BERRA: It would not have been two weeks long, but I don't know how many days it was.

Remember that these inspectors were already welding inspectors, and it was certain aspects of the inspection criteria that was unique.

So this was not taking, you know, somebody like me, who was not a welding inspector, and trying to turn him into one. This was taking inspectors and enhancing his training.

MR. THOMPSON: Did you go back and look at their previous qualifications and training to validate or verify for yourselves that these people did have those prerequisites that you thought they had; that is, have assurance that these inspectors had the appropriate training and that they just had one area of deficiency and that therefore your training only needed to be focused on that one particular area?

MR. BERRA: The retraining was not inspector specific. It was project specific. In other words, we didn't train Joe for one thing and Harry for another.

There were some changes in the inspection criteria we were using, and therefore we trained all inspectors to that criteria.

Previously those inspectors, according to ANSI, their education, their background information, prior experience information, and their examination information is part of their file in accordance with that code in certifying those people to inspect.

So we have all that data on their background already.

MR. THOMPSON: So you are saying you did not reverify, is that correct?

MR. BERRA: We did not recontact their colleges and high schools.

MR. THOMPSON: Do you originally contact the colleges and high schools, or do you accept that on face value?

MR. BERRA: There is a verification process. I personally can't tell the exact steps that took place there, but there is a verification of experience and education.

MR. RUDOLPH: Let me address from a general context how. I think where you are coming from is do you have reason -- or do we, the utility, have reasonable assurance that the people that are doing our quality control inspections -- do they possess the requisite requirements within ANSI N-45.2.6?

The answer to that question is we do have that assurance, and I will tell you why we do.

When an individual is initially certified, he has minimum educational background requirements to meet and also experience level and capability requirements to meet. Those are established right from ANSI N-45.2.6.

When an individual is brought into the program and certified -- qualified and certified as an inspector, there are two things that happen. The organization that certifies them does a background check to make sure that they can meet those requisite requirements.

In addition to that, the instructor in this particular case, Daniel, has a quality assurance program consisting of a quality assurance organization which audits that process to assure the management of Daniel and the management of KG&E that that process is being performed in accordance with the requirements.

In addition to that, a second layer of auditing occurs in that my organization not only looks at the auditors within the Daniel organization but looks at the certification process that those auditors looked at, and the certification process, independent of those auditors, are audited by my organization.

So in the case that we are talking about here, we are talking about not decertifying previously certified auditors or inspectors. We are talking about enhancing their certification; in other words, providing them additional training, which allows them to do inspections to a larger number of attributes, inspection attributes.

Now, there is another point that will address your concern. These people were not certified or allowed to do inspections for five years without additional training and without recertification. So there is a continuous process to assure ourselves and for Daniel to assure themselves that the people who are doing these inspections are adequately trained and qualified and certified.

MR. THOMPSON: So to identify the training need that they have previously, it was based on the development of a new inspection technique or deficiencies that they had? I am still not quite clear.

MR. RUDOLPH: The change in inspection technique precipitated the need for us to do training to qualify and certify those inspectors to that technique.

MR. THOMPSON: But these people were accepting welds that had deficiencies in them, I guess --

MR. RUDOLPH: No, I don't believe that was the case. No, sir.

MR. EISENHUT: I guess I am lost a little bit here. As you went down the sequence of events, starting from the socket welds through a number of other iterations, it appears you are telling me that there was a reject rate, so to speak, after the inspection performed on the welds; that is, a certain number of them?

MR. RUDOLPH: A certain number of rejects.

MR. EISENHUT: I guess one thing I would like to get at a later time is the kinds of percentages we are talking about, the various different efforts down the line where the welds just weren't up to snuff.

I think that is Hugh's point. The welder was accepting their doing it. He was accepting it. It was being inspected, yet it was going through the system and at a later time found to be -- through a 100 reinspection in some areas, found to have problems.

And then if I follow the sequence of things -- and this really goes back to Harold Denton's questions -- in hindsight, when you look at this as a family, is it saying that -- hindsight is always 20/20 -- should we now be saying if you looked at all these as indicative of what I will call the welding problem in a number of areas, not ultimately in just the MSSW area, but it was clear the number of welds didn't stack up to the standard ultimately?

MR. KOESTER: May I interrupt for just one moment?

I believe before we are through here today that we will speak thoroughly to -- when we go back on a 100 percent inspection.

. If we do not satisfy that requirement when we get through with our presentations, then we can discuss this further, but I do think on original inspections and subsequent inspections after that, that there is a different philosophy that I think anybody might take.

I don't know whether -- Bill, do you....

MR. RUDOLPH: The answer to the question -- I think that answer will be given in the presentation and become obvious. If it doesn't, then I will try to elaborate on it, or some other member of the staff will elaborate on it.

MR. KOESTER: Not only here, but on any reinspection or secondary inspection that you do.

MR. EISENHUT: Right.

Please don't misunderstand, I am not questioning the judgment as much as I am trying to understand the judgment.

MR. RUDOLPH: I appreciate that.

I am trying to recall what I have read on the subject because I wasn't there at the time.

The fillet welds, I believe -- which some people don't refer to as fillet welds because they refer to them as socket welds and small bore piping, but they look like a fillet weld -- the method used in taking the measurements was a 4-point measurement. We changed that to a 100 percent measurement. Although, as I will point out, not required by ASME or any other code to take 100 percent, we changed that to that.

That type of measurement would and did yield different results than the 4-point measurement. That was the socket weld program at that time.

We went to a 360-degree -- I don't believe it was called 180 versus 360, but a 4-point versus a 360. You take the gauge and you give it four points and 90 degrees of each other, approximately, and look for high and low points, and that is how you get your size and contour measurements, as compared to 360-degree bolt leg size and contour measurements.

And, yes, when they did change, they did yield deficiencies. We would have anticipated that. As to the severity of the deficiencies, I could only -- did they result in a significant finding? I don't recall the results of that, whether it was considered --

MR. KOESTER: The findings were not significant in the context of significance as we apply to other pressure vessels.

MR. EISENHUT: When you compare the two methods, you would not expect a significant difference?

MR. RUDOLPH: There is a methodology that is applied, and I think that will be addressed in the presentation.

MR. TAYLOR: Jim Taylor here.

You were doing 360 degrees fillet measurements, and the deficiency most noted again was undersized in local areas? We are still dealing with undersized?

MR. BERRA: These welds were seen under visual inspections, also.

MR. TAYLOR: This was with a fillet gauge?

MR. BERRA: Yes.

MR. MYERS: I am the project manager for Bechtel Power Corporation for all the SNPPS projects, including Wolf Creek, and a large part of the presentation today will cover what the inspection philosophy required by a particular code is. That is generally the inspection philosophy used in what we will call for argument purposes a primary inspection, the first time something is inspected.

Then there is an inspection philosophy that is developed by the architect engineer, the constructor and the owner for secondary inspections. They are, as John Berra described, almost always more stringent than that required by the code.

For example, 4-point socket weld inspections were required initially. We did 360. You will find later on in AWS welding inspections that were done over again we did a great deal more than AWS trains its inspectors to do.

So the deficiencies that we are finding are absolutely expected because we are using a more stringent requirement, and I think that many of the questions you have about this will be answered when Mr. Berra, Mr. Brown, and so on, make their talks. It is all in there.

MR. EISENHUT: Maybe that is good. Maybe we ought to go back and let you walk through this.

I would ask you to at least make it clear to me when you walk through when it was that you gave us the 50.55(e) notification. You have indicated one here that you sent in and it was approved, and ultimately --

MR. RUDOLPH: The ultimate --

MR. EISENHUT: -- this was in December of 1984.

MR. RUDOLPH: The ultimate identification, and calling into the NRC of a potential 50.55(e) was on September 18, 1984.

MR. EISENHUT: 1984, and then the question I was looking at as I went through was, you started with socket weld problems. Was that ever called to the NRC?

Perhaps we ought to let you walk through.

MR. RUDOLPH: Licensing can --

MR. MAYNARD: The socket weld issue was submitted to the NRC as a 50.55(e) item.

MR. EISENHUT: Was it right about that time?

MR. MAYNARD: Yes, September 1980.

MR. EISENHUT: Thank you.

MR. DENTON: Maybe we will let you go through each box without interruption, and then we will pause and come back.

Why don't you take off on that?

MR. RUDOLPH: Okay, let's pick up where we left off at February of 1983.

As a further initiative, Daniel reinspected -- performed a reinspection in all Q buildings and identified an unacceptable percentage of welds that were deficient in the auxiliary control and fuel buildings.

That precipitated as a result in March of '83 a Daniel CAR-29, which was the hardware-oriented corrective action report. That report was initiated to obtain corrective actions of these deficient welds noted in February of '83.

And the reason I think again will be discussed in our presentation. Why these documents were written I think will be very clearly stated in the remainder of the presentation.

At that time we called in a potential 50.55(e), which was later withdrawn as a result of -- in October of '83 as a result of the analysis of those deficiencies.

In August of '83, Daniel initiated CAR No. 31, which was a result of putting together turnover packages for the fuel building. In other words, in the typical turnover of building and structures to the owner a document reconciliation occurs, and through that document reconciliation it was identified in the fuel building that there were some missing MSSWRs. There were also some missing MSSWRs associated with the reactor and the essential service water pumphouse.

The MCRs in those areas that were identified as being -- or the records that were identified as being missing were noted on nonconformance reports, which is a typical corrective action vehicle for problems of that type.

As a result of DIC's CAR-31 and after a late July discussion with the NRC senior project management at Wolf Creek, KG&E initiated a document reconciliation task force to determine which structural steel welds had missing inspection documentation. That was August 13.

Four days later, on the 17th, we also initiated a limited inspection verification plan to obtain an accurate assessment of the as-build condition, having missing MSSWRs. These inspections were performed in accordance with the approved Daniel quality program.

As a result of that inspection process, we called in a potential 50.55(e) to the NRC on September 18, and we initiated CAR No. 19 and issued that corrective action request on October 17.

That basically takes you through the sequence of events which initiated our corrective action request and led to the resolution of the concern.

MR. DENTON: I didn't want to cut off questions. I wanted to give you a chance to get through it.

So you had an early indication of fillet weld problems in 1980. Now, did that lead directly, in your view, to the '83 random inspection of structural steel fillet welds? Is that the process that --

MR. RUDOLPH: Yes. The review led to this random inspection process simply because we identified the concerns in ASME and special scope, and it was natural to look at other areas.

MR. DENTON: You also had missing records earlier?

MR. RUDOLPH: That is the point I am attempting to make here. We got down as a result of these undersized socket welds to February of '83. That reinspection initiated the Corrective Action Request No. 29.

Now, the critical point is, in August of '83, when we were turning over buildings, we were identifying missing MSSWRs. At that point we initiated that corrective action request, and through the initiative of that corrective action request and with inputs from your people from Region IV, we did other inspections.

Those other inspections, in reality, if nothing had occurred from February of '83 to 1980, would have identified the same concerns.

So basically to summarize, we were leaning -- we were going toward the same focal point. If we hadn't taken Road X, we would have obtained the same results by taking Road Y.

MR. EISENHUT: I guess, then, a lot of that flows from the discussions in the corrective action reports. So how do you propose to manage this effort?

As I recall, I don't think those corrective action reports that you referred to here have been submitted to the NRC.

If they are a key -- and I am looking ahead at the slides -- obviously, the --

MR. RUDOLPH: Those corrective action reports have been evaluated by members of the Commission. Region IV personnel have looked at those. They have looked at them on a number of occasions, once during the task force -- or a number of times during the task force and subsequent to that.

MR. EISENHUT: I wasn't referring that they weren't. Certainly, they are available, though, in the plant. But since they are a key element in the sequence of how it follows it on, I think you ought to package those up and submit them along with your slides, as I understand it, which is not part of the briefing package and will have to be made part of the record.

MR. RATHBUN: Those are submitted to Region IV.

MR. EISENHUT: My question was: have they been submitted?

MR. RATHBUN: I am sorry, I am thinking of our packaging of the results of the corrective action reports. That is more important.

MR. RUDOLPH: The information that I am just giving you here has been provided to the NRC Region IV personnel.

MR. MARTIN: Harold asked for it to be put in the docket.

MR. EISENHUT: It is key pieces of the record. For a complete record, I say you ought to -- it would probably be preferable if you packaged up those and submitted them as part of the docket.

MR. KOESTER: Do you want the entire CAR-19?

MR. RATHBUN: You have the reports.

MR. KOESTER: They are already submitted.

MR. EISENHUT: We will come back to it. I just wanted to make sure that this information -- for example, the slides, as I understood it, had not even been submitted to the NRC.

MR. RUDOLPH: It had, to Region IV.

MR. KOESTER: Region IV has that slide.

MR. EISENHUT: It should be part of the record. Thank you.

MR. DENTON: I want to pursue the missing welds a little further.

MR. KOESTER: We will get to the missing welds.

MR. RUDOLPH: That is a part of the other presentation.

MR. KOESTER: You are getting ahead of our presentation, Mr. Denton.

MR. DENTON: I don't want to go to the technical aspects of this yet, but in this framework these welds were made a long time ago, obviously. They were inspected and accepted a long time ago. So they were originally made and inspected, I guess, by 1980.

Is that correct?

MR. RUDOLPH: I believe so, yes.

MR. DENTON: So in effect all of the structural steel had been accepted by KG&E in that timeframe. Is that right?

MR. RUDOLPH: No, the structural steel was accepted as part of the building turnover process.

Is that right?

MR. BERRA: The buildings were turned over-- The first one was turned over in late February of 1984. The buildings are turned over, the transfer responsibility, as a complete unit. The first one-- I've got it in my slide. The first turnover occurred in the first quarter of '84, I believe.

MR. EISENHUT: The welds physically were welded in the field -- Is it not true? -- most of them prior to 1981, at least?

MR. BERRA: Yes, sir. Right.

MR. EISENHUT: And therefore, it is not also true that the welder who welds them certainly inspects his work and his work is inspected as time goes along?

MR. BERRA: They were welded and inspected, the majority of welds, in the '77 to '81 time period.

MR. RATHBUN: Let get on with the presentation. These questions are going to be answered.

MR. DENISE: I just wanted to be sure that Harold gets the answer that he asked for, and I think that you are affirming that what Darrell says is true, that the welds were made and inspected in the 1978 to 1981 timeframe.

When Harold asked the question, did KG&E accept these back then, he was getting an answer, No, we didn't accept them until later. But your agents accepted them before, and then they presented them to you in 1984, and then you accepted it. So they were accepted as acceptable by 1981 by all the folks who had to accept them, absent turnover from your contractor or your agent.

MR. RUDOLPH: That's correct.

MR. KOESTER: Thank you, Mr. Denise, for that excellent answer that we should have made.

MR. DENISE: You're welcome.

MR. KOESTER: I'm sorry we didn't understand your question, Mr. Denton.

MR. RUDOLPH: Moving on to the program objectives:

(Slide.)

KG&E quality assurance initiated what we call CAR No. 19 to resolve the concerns identified.

There are four program objectives associated with CAR No. 19. The first objective was to document a consolidated project plan for the identification, evaluation and resolution of safety-related AWS D-1.1 welding deviations.

Secondly, to assure by objective evidence that AWS D-1.1 safety-related structural steel welding complies with all quality criteria.

Third, to assure that the inspection documentation for safety-related structural steel reflects the appropriate information and is available, complete, and traceable to the Item 4 activity.

And lastly, to evaluate other data on the AWS D-1.1 safety-related activity for compliance with the FSAR and the design and construction quality assurance program manual.

(Slide.)

There are five findings associated with CAR No. 19. These findings required corrective-action implementation. These corrective actions were both hardware and programmatic oriented. A more detailed discussion of these corrective actions will be provided by Mr. John Berra during his presentation of the KG&E management action plan which responded to CAR No. 19.

The first finding referred to missing weld documentation.

The second finding referred to various weld deviations.

The third finding pertained to welds not made or missing material.

The fourth finding involved the presence of weld inspection documentation without the presence of the weld.

And lastly, the fifth finding pertained to a verification of completed corrective action associated with KG&E QA surveillance report S-372.

Simply what this represented was to go back and pull the corrective actions that had been taken and satisfy ourselves that that was in fact complete and on the record.

(Slide.)

Upon initiating CAR No. 19 on October 17th of '84, I assigned two experienced auditors from my quality assurance organization on a full-time basis to follow the resolution of the CAR-19 findings.

The results of these independent audit and surveillances indicate that the KG&E management plan for the resolution of the AWS D-1.1 welding concerns was effectively implemented and that the corrective actions taken satisfactorily resolved the CAR-19 findings.

(Slide.)

In summary, the corrective actions recommended in corrective action 19 were readily adopted by KG&E Construction, the organization responsible for resolving the concern. The five findings of CAR-19 were transformed into a detailed management action plan consisting of 51 separate action items which exceeded the CAR-19 recommendation.

This resulted in a much more comprehensive treatment of the AWS D-1.1 welding concern than was recommended in KG&E's CAR-19.

If you wish to refer to the detailed logic plan it is provided in your handout right after this slide.

The comprehensiveness can also be demonstrated by mentioning that virtually all safety-related, structurally-significant welding, with and without inspection records, was performed; in other words, a virtual 100 percent re-inspection occurred.

In addition, an evaluation of other AWS D-1.1 safety-related welding rograms was performed and the evaluation occurred of other safety-related programs beyond AWS D-1.1 welding. The results that these evaluations confirmed was that these activities were programmatically controlled and effectively implemented.

If there are no other questions, or if there are continuations--

Yes?

MR. DENTON: To what extent is this program -- these findings that you made related to differences between the American Welding Society requirements for record retention, inspection, and so forth versus ASME?

MR. RUDOLPH: That will be addressed as part of our presentation.

MR. DENTON: Well, coming back to your findings chart where you made some significant findings in the ASW area, why do you think, from your overview standpoint, that they weren't found in other parts of the plant? Because the other parts of the plant weren't controlled by ASME standards?

MR. RUDOLPH: Yes. There are much more rigorous controls applied to other types of welding activities, specifically ASME welding activities.

MR. DENTON: Is that the sole answer? I mean is it the people who do the job?

MR. RUDOLPH: If you will permit us to continue with our presentation, these root-cause factors will be identified and explained in detail.

MR. DENTON: So we can learn some lessons from it, suppose we had to build another plant. What would you do differently to keep this from reoccurring?

MR. RUDOLPH: Quite frankly, the record retrievability program which has been enhanced to prevent recurrence would be readily adopted in its current condition as it exists right now on the site. We had a quality assurance program breakdown associated with record retrievability which is one of the root causes which will be explained.

Other than that program enhancement which has already been completed, I think it is readily apparent from some of the other presentations that the causal effects of record retrievability were there and did need to be enhanced and have been enhanced.

The missing welds, the missing material also represented a QA program breakdown.

From a quality perspective, all the controls incumbent in the program to prevent these things are in fact there, and I think again in our presentation if will be stated why, in my opinion, I don't believe we need any additional corrective action, either corrective action immediately or corrective action to prevent recurrence.

MR. EISENHUT: If I could ask another general question, not just in CAR-19 but in previous ones, would you answer Harold's question the same way? That is, the lessons learned from the other ones that go back to the earlier issues all the way through 31, are there similar kinds of findings there? Or how do they relate to these kinds of findings?

MR. RUDOLPH: As part of our lessons-learned process, I evaluated or had evaluated every corrective action request initiated by my organization. In the same manner we also had evaluated for similar root-cause effects every corrective action request generated by the Daniel Corporation. And there were no adverse findings upon the reevaluation of those corrective-action requests that would have directly related to CAR-19 and the management action plan which was adapted and worked through.

. MR. EISENHUT: I guess that's all right. The only think, when you say it is directly related, indirectly related?

MR. RUDOLPH: I'm sorry.

MR. EISENHUT: You think they are independent and didn't relate to one another?

MR. RUDOLPH: That's correct.

MR. DENTON: Are we switching to a new speaker now?

MR. KOESTER: Yes.

One of the questions I was going to ask, how long do you gentlemen-- We have quite a few speakers yet, and we have three-- Mr. Rudolph's speech was originally eight minutes long, and he talked for 53. If this occurs we are going to be here until around 4:30 this afternoon. That's fine with us. We will stay as long as you will stay with us, but we would definitely like to have you hear the independent reviews that were made by people other than us.

Mr. Berra has 25 minutes, and--

MR. DENTON: Let's take a ten-minute caucus break, and that will allow us to look at the agenda and decide how to go from here.

MR. KOESTER: I can tell you exactly how long each one of these gentlemen plans on speaking.

MR. DENTON: I think it is going to be an all-day affair.

MR. KOESTER: That is fine with us. We will stay here tomorrow. I expected to anyway.

MR. DENISE: Before we break for that, let me see if I understand what you're saying.

I expect John Berra to go through a detailed explanation of the welding and the management plan that corrected the welding. This focuses on AWS. Is that correct?

MR. BERRA: It focuses on AWS and other programs.

MR. DENISE: Other welding programs?

MR. BERRA: Other welding and non-welding programs.

MR. DENISE: The engineering evaluation by Mr. Brown, he will focus on the AWS structural steel. Is that correct?

So the independent reviews by Mr. Reedy, Dr. Fisher and Dr. Egan are focused on the AWS D-1.1 application as it is performed at Wolf Creek. Is that correct? And it is not broadened into-- It is not broadened into ASME, QA, QC, et cetera?

MR. KOESTER: For these two gentlemen --

here.

MR. DENISE: We want to know who is going to show it.

MR. KOESTER: AWS as well as other programs. And I think each of us will try to speak to each one of those. Even Mr. Rudolph I thought did, too.

MR. DENISE: My main concern was we are about to take a cut at the agenda rearrangement and we need to understand what is--

MR. KOESTER: I don't think that is fair to the applicant to make an agenda cut. Specifically we were told that discussion was on KG&E's resolution of issues.

MR. DENISE: The agenda rearrangement.

MR. EISENHUT: Well, let's take a break.

(Recess.)

MR. BERRA: Gentlemen, in my presentation I am going to cover two topics: one, the structural steel welding history of Wolf Creek, not the CAR-19 structure but the structural steel welding history of Wolf Creek, and then I will cover KG&E's CAR-19 management plan.

In the management plant discussion I will address the AWS issue, other welding issues potentially related, and other programs potentially related to the structural steel problem that was identified by CAR-19. Those will be in the management plan part of my presentation.

(Slide.)

As I stated earlier in response to a question, here we have the Category 1 safety-related buildings and structures at Wolf Creek. These are the approximate timeframes of start of structural steel erection and completion for each building. Because of the construction sequence, some minor pieces of the steel may have been installed in late '83 or such, but this is the majority of the time when the structures were installed.

As I stated previously, it essentially began in the fourth quarter of 1977, and they were essentially complete by the fourth quarter of 1981. It was during this timeframe that not only was the structural steel erected but the majority of the structural steel welding associated with that was also performed.

In addition, the inspections and documentation associated with that installation was created during these time periods.

The process of transferring from the contractor, Daniel, to KG&E the responsibility for the buildings and that documentation is called a "turnover," and the turnover for these various buildings started in -- actually February 27th of 1984 with the fuel building.

The actual accumulation of the documentation that was generated starting almost seven years earlier was put in packages beginning in 1983 for turnover that occurred in 1983. And as Mr. Rudolph pointed out, in that package putting-together process the records were shown to be less than 100 percent retrievable.

(Slide.)

The code used for structural steel at Wolf Creek is AWS D-1.1, 1975 edition. The major activities covered by AWS were the design of welded connections, the workmanship, the filler material requirements, weld procedure qualification, welder qualifications and inspection criteria.

AWS does not specifically address qualifications of inspectors nor the creation and/or retention of inspection records. In general, the documentation requirements of Wolf Creek are determined by Reg. Guide 1.28 and ANSI 45.2, quality assurance program requirements for nuclear power plants. And the inspectors' qualifications are addressed in ANSI 45.26, qualifications of inspection, examination and testing personnel for nuclear power plants.

I repeat, the documentation for AWS does not require documentation of inspection nor inspection records to be generated or kept. Its records that need to be generated pertain to the weld procedure qualification and the welder qualification.

The project construction and inspection

procedures that were actually used to perform the work and do the inspection at the project incorporate AWS, ANSI, and other applicable codes, regulations and design data.

The inspection that was performed in accordance with AWS during the '77 - '81 time period included a visual inspection of all welds and magnetic particle examination for 10 percent of certain weld details as specified by the architect-engineer. I will repeat that because it has been the source of some confusion.

It is not 10 percent of all welds; it is 10 percent of certain details as specified by the architect-engineer.

AWS does not specify which joints to go magnetic particle examination on, nor does it specify the percentage. This was specified by the architect.

The visual weld inspection was documented on miscellaneous structural steel weld records referred to as MSSWRs.

(Slide.)

At the time of each weld inspection an MSSWR was completed. It contains the drawing number, joint number, the area which is a designator used for a portion of the building, the location within the building, the base material piece or heat number, rod withdrawal data which is the information relative to the welder pulling out his electrode, the filler material heat number and lot number, the weld procedure utilized for that weld, the welder identification number, and the quality inspector who inspected that weld.

As previously stated, these inspections were performed in accordance with project inspection procedures that did incorporate AWS inspection criteria.

(Slide.)

That inspection criteria is summarized on this slide.

This is a scaled-down example of a connection at Wolf Creek. It is a connection of a beam to a simulated embed plate. It has two clip angles, each containing two welds, one welding the clip angle and the beam, one welding the clip angle to the embed plate.

There are also two welds welding the beam to the embed plate and these two welds are not typically required. They are put on this as an example, so I can have just one sample up here to show. It is kind of difficult to see the fillet welds in here.

So this connection referred to as a joint -- we use those two terms interchangeably, joint and connection -- contains, as you can see, more than one weld. This one contains six welds. The average connection at Wolf Creek has between four and five welds.

MR. DENISE: Do you want to point out, John, any other things like the return?

MR. BERRA: Yes, I will. It is in the CAR-19 part when I get into that.

Each of these welds is visually examined for AWS inspection criteria. A welder walking up to this joint in the field-- And I want to repeat that this is scaled down. It's a little hefty as it stands, but typically this clip would be 18 inches in length. Some would be shorter, some would be longer, but typically 18 inches rather than this, but I wouldn't be able to pick it up here.

What the inspector does when he approaches this which of course would be at some condition in the field-- The embed plate would be embedded in the concrete. The inspector would either walk up to it or use scaffolding to get to it. He would look at each arm, in this particular case, the six welds for smatter, slag, arc strikes, porosity, overlap, profile, fusion, craters, cracks, undercut, size, length, location and presence, that the six welds are there.

Now if he noted a deviation, the weld would either be repaired at that time or presented to the architect-engineer for evaluation.

The inspection aids specified in the AWS code are suitable gauges for size and contour and strong light magnifiers or other such devices that may be found helpful for a visual inspection for cracks and other discontinuities.

The most common type of weld at Wolf Creek is the fillet weld, of which there are six on this sample. This is a set of gauges that the inspector would use to fulfill the requirement of suitable gauges for size and contour.

These welds were specified by me, not the architect-engineer, to be 5/16th of an inch on this sample. The methodology used to apply this suitable gauge is not defined in the AWS code.

The methodology used in the '77 - '81 timeframe would be for the inspector, once performing all these visual looks for those attributes, would be to make a judgment as to the size of that weld. He knew it was supposed to be 5/16ths by the drawing. By looking at some of the material used in the weld he can pick up some of the attributes without measurement.

If it is a four-inch beam, he knows he's got a four-inch weld on the top if it runs full length. He knows the thickness of the clip and therefore if the weld is using up that thickness, he knows the size of the weld by visual rather than gauge measurement.

He looks at the rest of the weld that he can't pick up with strictly visual and does some high- and low-point checks for the weld size using this gauge, applying it as such. (Indicating.) Of course he doesn't have to hold it up like I do.

(Laughter.)

This gauge also has a device for measuring concavity. Now he would look on there at that timeframe, look to see if it looked concave to him, look whether it goes in or sticks out. If he thought it was concave he would take this, apply it to the point that he thought was concave to check if it met the requirements of his inspection.

As you can see with this type of inspection, it is very subjective in nature and it is not surprising that a different judgment concerning minor deviations might occur from inspector to inspector. However, due to the built-in design margins previously explained by Mr. Ivany and that will again be touched upon by the three consultants, such minor, unintentional deviations do not pose structural integrity problems.

(Slide.)

In summary, for the history, erection and welding was performed in the '77 - '81 time period and the welding program was in accordance with AWS D-1.1, the 1975 edition.

As Mr. Rudolph stated, in response to CAR-19 KG&E prepared a management plan to address the CAR-19 findings.

The findings were five in total, and 51 actions were delineated to address those findings in the management plan.

The plan was structured to address the issues in two areas: program issues and hardware issues. The review of each of these areas was not limited to AWS structural steel. The reviews also included other applications of AWS welding such as heating, ventilation and air conditioning supports, electrical raceway supports and pipe whip restraints.

Although not in the scope of CAR-19, non-welding related quality programs were reviewed for comparable programmatic deviations. In accomplishing this, KG&E and Daniel conducted a program assessment of the piping, hanger, mechanical, electrical and other civil disciplines.

In the assessment, the attributes of those programs were examined and found to be different from the structural steel weld program, including post-inspection walk-downs by a combined group of both KG&E and Daniel, the use of unique component identification such as pump numbers, hanger numbers and valve numbers, and component testing performed on those components, and a document review by a combined review group of Daniel and KG&E.

To elaborate on those, the importance of unique component identification is the majority of other components, other than welds, are identified on the design document documents such as a pump number, a hanger number, a cable number, a termination number, et cetera.

Now what that means is when you start out, you know you have 103 widgets and all 103 of those widgets have a name. So when you have completed your widget installation and inspection, you anticipate to have 103 widget inspection references. And if you come up with 102, you know right off the bat you are missing one. It is inherent in those programs.

Those programs use-- The majority of them use travelers which is a package that is put together by field engineering and given to the craftsman to do his installation, a package for a hanger. When that hanger is complete it is inspected by the craftsman, his foreman, the field engineer.

It is then submitted to quality control for inspection. Quality control has performed in-process inspections during the erection of that hanger and witnessed hold points as specified in that traveler package.

When it is all complete it is submitted for final review by quality control. After quality control reviews that -- that is Daniel quality control -- it was then submitted to a combined walk-down group. The combined walk-down group consisted of a separate group of Daniel inspectors and KG&E quality personnel, construction quality personnel who walk together out and look at that hanger. That was called the combined walk-down.

After that inspection it was then submitted to a documentation review group. Now the documentation package itself when complete in the field was reviewed by the people that put it together. I'm talking a separate group. Once the combined walk-down group looked at that hardware, it was passed on to a combined review group that was staffed by both KG&E and Daniel.

That review group looked at that package for that hanger. Of course this crosses over to other components but I'm using the hanger as a specific example. They looked at that. They did a procurement review. The procurement review was to see that the material identification numbers that were listed in that particular traveler were good heat numbers by looking at the heat number logs.

The checked that the inspectors signed it off and that the welders had done so correctly from a documentation standpoint. Then they reviewed the total package for content as far as documentation content.

From that point, the hanger would go to the vault after it stopped at the ANI, the authorized nuclear inspector who, during this whole process, had the opportunity to insert hold points or in-process inspection, but at the end he signed off on that hanger traveler.

That hanger traveler then went to the vault where it rested until all the other hangers in that system were completed and processed the same way, and accumulated into a package called an N-5 as part of the ASME program. And at that time everyone involved had a chance to look at it again if they so wished. Those were generally sampling points at that time. Then the N-5 was signed off.

Now that program as compared to the welding program, the structural steel welding program, was quite different. When you go out there into the plant you don't know how many welds you are going to make. Now it is true that the welds are identified on a drawing, but when you get out there we also document temporary welds.

You may wish to put a beam seat up, like I was having a hard time holding that plate up. You're going to swing a beam in, make a weld to the embed. You might attach some temporary device to that embed, drop the beam on it to rest there while you make the weld, and then you later wash that off.

You may put spannel beam to beam for a temporary installation. You may put a lifting device to pick the beam or component up. You may affix an erection convenience device and later remove that. Those type of activities were all documented on MSSWRs.

When you started out you don't know how many of those you are going to generate. You also don't know the sequence that you are actually going to follow through the thousands of installations.

Now a joint may have, as I showed before, many welds on it. Now the inspector may have inspected all those welds at one time; he may not have. He may have inspected the beam seat and created a document, an NSSWR. He may later have inspected the clip angle weld and created another document, so you don't have the finite scope identification in structural steel welding that you do in other components.

There's a computerized list of every cable in the plant. It has an ID number. There's a computerized list of instruments. There's a computerized list of terminations, raceways, cable tray and conduits. They all have names. Welds didn't have names. Some of them have got names today, but they didn't have names when they were installed.

(Laughter.)

So it was very important.

They also-- During the structural steel installation period, the combined review group that I mentioned that did the walk-down after the inspection did not exist at the site. That group is not required by any code, regulation, law or anything. That group was installed in 1983 I believe.

Now although some of these other programs had installations performed prior to that, the documentation process to the combined review groups occurred after the implementation of that program. So there was an enhancement in the program that covered the other areas.

That enhancement did not exist in the structural steel program. That enhancement will exist if we are fortunate enough to build another unit at Wolf Creek. That enhancement is there now and, as Mr. Rudolph stated, we don't have to make another one, that one is there and that one would cover this issue in a future plant installation. It would also cover the issue of any construction we do out there in coming months.

Now from this whole assessment we concluded -- I wanted to point out another particular issue.

In the slide that Mr. Rudolph presented we discussed finding things, et cetera. One of the things, when we look at CAR-19 and we look at other welding programs and we ask the question, well, 'could CAR-19 spill over into those other welding programs,' all those activities you noticed were reinspection.

The reinspection that we performed on CAR-19 had already been performed on socket welds, electrical raceway supports, HVAC supports -- and some other programs that don't come to mind right now -- had already been reinspected. So when we looked at CAR-19 for spillover into those other programs, because of the programs themselves that I explained, and in addition the reinspections that were occurring subsequent to initial installations, were all weighed to give us a high level of confidence to assure compliance of those programs to 10 CFR 50, the FSAR, ANSI and design and procedural requirements.

Now getting back specifically to AWS, I mentioned we did a programmatic review and we did a partner review.

(Slide.)

The programmatic review was to cover that the welders were qualified -- these are the welders that performed the installation in the 1977 to '81 time frame --

actually we covered all the welders -- that they were qualified in accordance with AWS.

We did that by looking at their qualification records, the testing methodology used -- because AWS does address the methodology of qualification of welders -- and we looked at the retrievable weld records we had to verify that the welders, i.e. -- the welder is giving a number -- like D023 becomes your name if you are a welder and you apply a weld; you don't put your name down, you put D023. Well, we keep records on what that welder is qualified -- what procedures he is qualified to do, what rod he is qualified to -- that determines what rod he is qualified to draw.

Now we looked at our program for qualifying welders, and then we had an overview by Bechtel Corporation, by their material and quality services group -- M&QSS -- and to also verify that it complied to AWS.

The welding procedures, the procedures that you use to make a weld, many of them are prequalified in the AWS code. In addition to that, there is methodology to qualify other joints. We reviewed that, as did Bechtel, to say that we did it in accordance with AWS.

The filler material purchasing control again was reviewed for compliance to AWS by Daniel and Bechtel. The control, although not specifically -- tells you to control to AWS but not specifically how. It tells you to keep it dry, et cetera.

This qualified welder, once assigned a number, we keep records as to what he is qualified to weld. He puts his -- that number on his rod withdrawal slip, and he is only allowed to pull rods that he is qualified to use. In this case the rod was all E-7018. If this welder tried to pull stainless steel rod to weld that structural steel out there, the rod room would not issue it to him.

If you look at the inspection critera -- we previously showed that -- you see that it complied to AWS. We looked at the -- to see that our inspectors were certified to ANSI.

AWS does not mention certification of inspectors. And that's confusing to some people because there are AWS-certified welding inspectors. They do now have a program to certify people, but they do not require the people that you use be certified to that. There is an AWS-certified welding inspector.

In the primary inspection the ANSI -- the welding inspector is certified to ANSI. In the secondary they use certified -- AWS-certified welding inspectors who we then qualify to ANSI.

And in that case I could answer specifically the question did we look at their background. Yes, we did. One gentleman, in the time frame we had we couldn't document his background and we re-reinspected his. And that's documented in the CAR-19 report.

Our documentation was in accordance with AWS -- which only refers to procedures and welder qualification records -- and ANSI, which I previously explained that that governs our documentation.

In the programmatic review we also looked, as Mr. Ruldolph mentioned, at the surveillance report; KG&E surveillance report S3-72 which was in question. And we provided evidence that that had been addressed previously, and there was documentation to demonstrate that.

Now the documentation review centered on the retrievability of MSSWRs. As I stated before, we could not retrieve the records.

Our review concluded that inadequate implementation of our welding documentation procedures was a contributing factor to having less than 100 percent retrievability. It was poor implementation of those procedures.

Other programs have better procedures. The procedures for AWS could have generated 100 percent retrievability but poor implementation of them didn't.

Having established a root cause for those, we looked at other programs for potential similar problems.

And I addressed how we looked at some of the other programs.

Yes.

MR. DENISE: Mr. Berra, you said you identified the root cause. Who was responsible? You said it was inadequate implementation. Who inadequately implemented those procedures?

MR. BERRA: From a corporate standpoint, Daniel.

MR. DENISE: Well, from a --

MR. BERRA: From an individual standpoint?

MR. DENISE: Yes.

MR. BERRA: All right. The welders -- The procedures addressed the welder completing his weld, putting some of the information -- the rod withdrawal slips, et cetera, information relative to that -- on the MSSWR; and then an inspector coming up, looking at the weld, the records, and signing the record.

The procedure was not -- well, when I said it could have resulted in -- it was not clear in the methodology for documenting that or keeping that record.

Now one of the things that happened during the life of AWS welding, it started out -- and those who are familiar with keeping track of a lot of paper and passing through hands -- it started out as being one piece of paper, one copy. That one copy out there where the guy was up at the weld was a one-part form. It later turned into a three-part form.

But the one-part form was stored in the field, in the building. It wasn't, as weld records were later, where when you did it you sent that weld record off. In other words, the traveler, as I mentioned, for instance, on let's say, a hangar, there's a -- one of those parts already went up to the document review group; the other part stayed out there with the traveler. When the traveler comes up it's got that weld record in it. If you lost the weld record there's also a copy of the weld record already in existence in the office.

In the structural steel there was not -- the record was to be kept in the field. It was not kept. And, you know, we couldn't retrieve it. So I can only say it wasn't kept.

MR. DENISE: Well, it sounds as though you're saying that they found a little slot to stick it in out there in the field -- say in a beam -- and they left it there.

MR. BERRA: No, they kept it at their work station.

They have what's called in many cases a headache shack where it would be something like this: Have a door underneath it and something -- I don't know why they call it that; maybe it gave them headaches to do their paperwork. But that's where they did their paperwork. But they kept it in those in the headache shacks, in the gang box if they were working on an elevation, working that area off.

MR. DENISE: Well, my -- You stated that the system broke down, and that in approximately 75 percent of the cases the system worked and the MSSWRs were retrievable. You have identified by building where there were missing or irretrievable MSSWRs.

MR. BERRA: Yes.

MR. DENISE: And you said the root cause was the improper implementation of procedures and had it been implemented it could have or would have resulted in retrievable MSSWRs.

My question was who went wrong. You said Daniels on a corporate basis. And then I think we got off on another track.

MR. BERRA: All right.

The procedures -- First, we later learned, with other documentation programs, we should have duplicated the forms initially to protect it, and filed it in two separate places.

The other is that the -- we allowed the filing or storage of those to go along for some time before reestablishing that they were being kept correctly, because we didn't have them in a controlled environment. That was -- our control of where we stored them --

MR. DENISE: I'm trying to find out who "we" is. We did this; we did that. I'm trying to find out: Was it everybody, nobody, or --

MR. BERRA: Okay. The construction part of Daniel was to keep the records and then turn them over at a later period.

MR. DENISE: Now this wasn't necessarily the welder himself?

MR. BERRA: No.

MR. DENISE: It's the construction crew?

MR. BERRA: Yes. It was the non-manual portion, the field engineers, inspectors, who worked out of that box and should have kept the records so that when you got there later and said, 'Okay, Joe, give me the records,' he would give you the records and they would be there 100 percent. But when Joe gave us the records Joe only had 75 percent of them.

MR. DENISE: Do you have Joe's and Jack's and Jill's names written down somewhere, the ones who didn't have their records in order?

MR. BERRA: No, because that function -- although the function stayed there through the life of the plant, some people are not as zealous as some of us and don't stay there during the entire time of the plant, and we had quite a turnover, not only of people leaving the site but people that were inspectors originally but became inspectors ultimately -- or became something other than an inspector. That was changing -- the guardian of that record there was a changing Joe.

MR. DENISE: So is it your statement then that you don't know the people by name --

MR. BERRA: That's correct.

MR. DENISE: -- who did not execute their part of the procedure that would have resulted or could have resulted in an MSSWR being put in the records? You don't know their names?

MR. BERRA: I don't know because the custodian of, let's say, the box changed over the six years. And at what time during the six years did that record go from retrievable to not retrievable I cannot say. I only know that it wasn't retrievable when we went to touch it in 1983. I can't tell whether it became unretrievable in '77 or '78 or '80.

MR. DENISE: I have one other question. You mentioned you went from a one-part to a multiple-part record.

Do you have information on the percentage of retrievables that came from the multiple-part versus the single-part? Did you notice any difference?

MR. BERRA: It's in the -- The multiple-part were used in the latter part of the steel. They were used primarily in the piping and hangar portion, although there was some, you know -- the multiple part -- They didn't use MSSWRs in piping, but they used multiple weld records.

MR. DENISE: I thought I understood you to say that you had a multiple-part MSSWR that flowed from D-1.1, from AWS D-1.1 -- to meet AWS D-1.1 inspection.

MR. BERRA: I misstated if I said that.

MR. DENISE: Okay.

MR. BERRA: AWS does not ask for any form to record the welding inspection. ANSI requires you to have records; it doesn't tell you to have one or three.

But we just -- As the project evolved we got into a multi-part weld record -- others and MSSWRs. But unfortunately at that time MSSWR structural steel was essentially complete.

MR. DENISE: Okay.

I asked the question: Did you notice any difference in percentage retrievability between the single-part and the multiple-part MSSWRs which applied to AWS D-1.1 welding?

Now is your answer that you really didn't have multiple-part MSSWRs which applied to AWS D-1.1 welding?

MR. BERRA: We didn't have that for any significant time period.

MR. DENISE: So no significant fraction then?

MR. BERRA: Yes, no significant fraction. We had it for the tail-end. But it was an insignificant usage of them for us.

MR. DENISE: Were you missing any of those?

MR. BERRA: Yes. I say that, but I don't know that specifically because I didn't break the two out.

But we don't have that problem in the multi-part weld records for other programs. And that's why I'm postulating the good retrievability in the other areas. I'm postulating that we had the three-part in the welding of structural steel we would have got the same results as we did in the --

MR. DENISE: I heard you say that, and I was just really trying to find out if you had any objective evidence to support that, and -- like you had applied the multi-part form at the latter stages of AWS D-1.1 inspection and if as a consequence of that you found 100 percent retention or 75 percent retention --

MR. BERRA: I don't know.

MR. DENISE: -- or it was not assessed.

MR. BERRA: We had virtually 100 percent. And I've noticed that others have had to call up for missing weld records somewhere in these other programs.

MR. DENTON: Were there any audits made of the original weld inspection programs -- audits by the utility of Daniels -- and, if so, what did they find?

MR. BERRA: There were audits by both Daniel quality assurance and KG&E quality assurance. And there were some actions taken. I believe one of the -- and I'm going to have to -- I may be stating this incorrectly, but my memory says that one of the audits did recommend multi-part, one of the latter audits, in the retention of weld records.

Where we did find S-372 audit surveillance that was referred to earlier, it did come up with some missing records. But they were predominantly -- by "predominantly," 90-some percent of the group that they looked at; not of the total program, but of the group they looked at -- 90-something percent of the records that they found missing had to do with another program: electrical supports. And there were actions taken on that that ceased that from being a continuing problem.

MR. O'CONNELL: These structural steel erection programs appear to have been completed in the '79, '80, '81 time frame; each one of these time lines shows when it was completed. And inspection was completed at the same time?

MR. BERRA: Yes.

MR. O'CONNELL: And I assume the collection of the MSSWRs from the work stations occurred back in that time frame?

MR. BERRA: No, sir.

MR. O'CONNELL: The job was --

MR. BERRA: The job was essentially -- I'm using the term essentially complete. Take for instance in some buildings there was some steel left out for construction convenience that were installed much later than the rest of the steel in that building. There were still iron-workers who did the installation, that craft that did the installation.

The iron-workers resided in the building a lot longer than -- the iron-workers stayed in the building longer than these periods. And why they were there is they were in there putting in non-structurally-significant steel such as toe-plates, hand-rails, fixing gratings. So they still had their gang boxes, their work stations still in the building.

MR. O'CONNELL: What triggered the collection of the welding records?

MR. BERRA: The accumulation for turnover to KG&E.

MR. O'CONNELL: Nothing was turned over until three years later?

MR. BERRA: Yes, sir. Now this is the typical -- This is typical methodology -- it is not typical that you lose the records, but it is typical methodology. We turn over -- We make turn-overs of systems, mechanical or electrical systems, when we complete a system. We turn over a system, all the piping in that system. We make a package; we turn that over.

Likewise in a building, we do the civil package for that building at one time, and we accumulate all the records and turn over the entire building at a time. And when those records were put together it was obvious we didn't have all the records for the structural steel welding.

MR. MARTIN: I want to go back to the prior slide on the welding history summary. I want to make sure we understand -- or at least for KG&E we clarify a statement.

You say the welding program, your conclusion is that your summary for KG&E is that the welding program was in accordance with AWS D-1.1.

MR. BERRA: Yes, sir.

MR. MARTIN: All right.

To clarify, I presume you are meaning by the program elements the skill of the craft, the workmen, the inspector qualifications, programmatically all of the right pieces were there. You are not arguing by that statement that the implementation was fully complete. I would presume one would argue -- at least a reasonable reader would say that AWS D-1.1 says if a weld was to be made you go out and you make it.

MR. BERRA: Yes, sir.

MR. MARTIN: Therefore I would argue that perhaps under that argument there are to be no missing welds that ought to be there.

So I'm just trying to make sure that I understand just what you mean by this in light of the fact that there were identified welds that should have been made that did not get made.

MR. BERRA: Yes, sir, that's true.

MR. MARTIN: Or they were too long or they had defects of some nature in them. Now how do I understand that statement being offered in light of those known defects?

MR. BERRA: If I can go back to the reasonable person you mentioned earlier that would say that, you know, you've got a missing weld, that's true, we had missing welds.

The AWS tells you to make the welds. Obviously the first check you made is if the weld is there. We did; some of those welds -- they weren't there. Later I will discuss how we feel that they weren't there. But that is not in accordance with AWS.

But the code itself -- AWS -- if, for instance, all of you -- if you think of the gross error, the missing weld, if all we found was something -- a millimeter -- exceeding size by a millimeter, that also is not in accordance with AWS.

So you could also say any project that has that does not meet AWS. It's true that it does not meet the strict letter of AWS because it tells you nothing should be, you know, too short, and there are no tolerances given.

Now we did exceed that in shortness and we did exceed it in missing.

So our implementation of the program -- the program was totally installed and existed in compliance with AWS, and we had some unintentional errors in the application and execution of that program.

The main thing here is did we have a program for precluding us meeting AWS. Did we leave out -- did we not train inspectors, did we not train welders, were our procedures wrong, did we forget AWS criterion, et cetera?

No, we didn't. We applied all those aspects but in the execution of it we created some errors.

MR. THOMPSON: One question: You said it was not general practice to have the number of MSSWR's missing at other construction sites.

MR. BERRA: Pardon me. I was being a little facetious there, I meant that you do lose records, MSSWR's are not a typical methodology used for documenting welds like that. For instance, in many projects -- in some projects, okay -- some installations would take a planned view of that structural -- the planned view that Mr. Ivany showed. And when he got through with inspection of the whole plant and signed that off they would have inspected all the welds at that plant, that meets the requirements of ANSI.

As I said before, AWS doesn't even require you to do that but that would meet the requirements of ANSI. Therefore they would have one record for each floor. Take it you had four floors in the building or six floors, that's 24 records approximately. 24 records, 24 drawings is a lot easier to count to keep track of, et cetera.

In our case we had tens of thousands of MSSWR's relative to some other facilities only having 20 or 30.

MR. THOMPSON: I guess what I was interested in, Daniel -- you know this is not the only project Daniel has been on. Was there something different between Daniel here and Daniel elsewhere or, if we looked at Daniel elsewhere, would it be saying the same thing?

MR. BERRA: You would not see a common use of MSSWR's.

MR. THOMPSON: Would you see it anywhere else?

MR. BERRA: I really don't know where else you would see it. It was not common use with Daniel.

MR. DENISE: Mr. Berra, I understand that Mr. Martin asked you a number of questions about that statement. Perhaps I might ask you also: Is that a Daniels statement; is that a Bechtel statement; is that a KG&E statement or is that everybody's statement?

MR. BERRA: That's everybody's statement. I believe Mr. KOester will correct me if I'm wrong that the presentation I am giving you is not a Daniel presentation.

MR. KOESTER: It's a project statement.

MR. RATHBUN: He already made the statement in his introduction remarks.

MR. DENISE: I just like to check the details from time to time to be sure I'm straight.

MR. BERRA: My whole presentation is a project presentation.

As stated earlier the CAR-19 management plant not only addressed programmatic aspects but also the welding hardware rather than utilizing a sampling plan or limiting the hardware scope to only that without primary inspection documentation. It was decided that the CAR-19 inspection verification would address 100 percent of the structurally significant AWS field-welded joints.

That's important because the industry -- not only the nuclear industry but industry in general that performs inspections -- recognizes the existence of sampling plans. Mill Standard 105, I believe it is, gives you -- that is recognized as a methodology to determine the credibility of some occurrence, the statistical credibility.

But it was decided that we would not do that. It was also decided that the question was concerning the retrievability of records and therefore the question could come up about the welding on those joints that we could not retrieve records for.

One approach would have been to look at those only in the re-inspection plan. But in the continuation of the conservativeness that Mr. Ivany mentioned in the design, our re-inspection program continued with that conservativeness and we addressed 100 percent of the structurally significant welds.

Now recognizing that this inspection verification was a secondary inspection and that AWS does not address such inspection, it was necessary to develop secondary inspection procedures and a plan to evaluate the results of that re-inspection.

That statement I just made is an interpretation of the project that was confirmed by AWS. That is, AWS is not intended for re-inspections over the life of the structure during the initial erection welding program.

At the time of the secondary inspection -- which was the last half of 1984 -- approximately 60 percent of the steel was painted and some of the steel was fire-proofed. In addition, construction activities subsequent to the primary inspection made some of the joints inaccessible for secondary inspection.

This was either because they became encased in concrete or other structures were erected after them that precluded you to get in and do a complete measurement of the initial weld.

Aware of the objective of CAR-19 and the then as-built status of the project, KG&E, Bechtel and Daniel developed a program for the secondary inspection.

A question was also asked of AWS that since they do not address secondary inspections, who would they recommend determine the methodology used for secondary inspections in evaluating the results of that inspection.

AWS answered: the owner and/or the engineer as the owner's representative and the contractor, and that's who developed it for the CAR-19 secondary inspection.

(Slide.)

The program used for that included the development of the secondary inspection procedures and the incorporation of those procedures into the site quality control procedures.

Specifically, I believe our procedure QCP-200 was amended to have an attachment specifically addressing the CAR-19 re-inspection program. That attachment was approved by Daniel, Bechtel as the designer and KG&E as the owner. We then performed a certification of inspectors.

When we initiated some re-inspections prior to the issuance of CAR-19 but subsequent to the occurrences in August, as shown on Mr. Rudolph's slide, we started doing some inspections. It was decided at that time to utilize inspectors that were certified to AWS -- AWS certified welding inspectors. They were some that existed on the site that were at that time in Daniel employ and also there were some at the site that were in Bechtel employ that were working for KG&E.

We used those inspectors to go out and do some inspections of the welds.

At that point in time that's where we came up with the missing welds and we kept going and issued CAR-19. Those inspectors had performed several inspections, maybe 2000 of the 11,000 welds eventually looked at.

We decided that although they were certified to AWS, we also decided to certify them to ANSI 45.26. So we did certify those inspectors and some additional certified welding inspectors we brought on the site to those requirements.

As I mentioned earlier, one of the inspectors, we couldn't verify his previous employment history and rather than continue along that vein, we just reperformed secondary inspections on the scope of work he handled.

The identification of structurally significant joints by the engineer, Mr. Brown will cover how that was performed. That resulted in approximately 2670 joints.

The validity of inspection in the presence of paint -- because we recognized that although the AWS code says inspect before you paint, if you do re-inspections on painted structures, you have to come up with a criteria for that. That validity again Mr. Brown will discuss as will the consultants.

We had to remove fireproofing from the joints that were fireproofed.

We did an inspection of the structurally significant joints.

We did an investigation of missing welds with primary records. During the secondary inspection plan, we looked at over 11,000 structurally significant welds. We found two welds on separate joints in separate buildings that -- inspected by two different inspectors -- we found those two welds missing and we found a record that said the weld was there.

Now we did an extensive investigation into that, including interview of one of the inspectors -- both inspectors were no longer at the site at the time of the secondary inspection and neither were employed by Daniel at the time of the secondary inspection. We did a hardware review and a documentation review and found the occurrence limited to these two cases out of the in excess of 11,000 welds looked at.

Our conclusion is that this was an error, an unintentional human error. I could go into more detail but there was an extensive review of that that backed that up and it is included in CAR-19 in the summary.

During our inspection -- our re-inspection, we documented the construction configuration of the joints. Previously I stated in the primary inspection you went out and inspected the weld -- maybe all the welds in a joint, maybe not. At that particular time you may have come back later and inspected more welds in a joint.

This time we approached it because this was a secondary inspection plan and we knew that we were going to come up with deviations in that inspection plan. We anticipated that.

In anticipation of that we knew we would go to the architect-engineer for his evaluation of those deviations. That was part of our plan.

So when we found the deviation, we documented the constructed configuration of that joint, the whole joint, so that when the engineer evaluated that joint if, in the case of a missing weld, for instance, he knew what the surrounding existing welds actual size was and length rather than just what was on the drawing so he knew what he could use for strength in that joint, what was really there. So we looked at it as a joint rather than as a weld. Obviously a missing weld has no strength but the joint is what he examined. That was the evaluation.

We reworked the joints. The numbers will be discussed by Mr. Brown. We reworked and he will discuss the specifics and then we issued a summary report of CAR-19.

Now Mr. Jerry Brown will discuss the technical aspect of inspection in the presence of paint, and I will touch on some of the related statistics with that.

The previously stated approximately 60 percent of the joints were painted at the time of the secondary inspection, which means 40 percent were not painted.

In addition, 125 of the joints that were painted during the secondary inspection happened to be joints that fell into that 10 percent of specific weld detail requirement to perform magnetic particle examination.

So we had magnetic particle examination records for 125 of the joints, and those Mt's were performed back during the initial installation so they were primary inspection records, although when we went out for the secondary, they were painted.

So with those 125 records and the 40 percent of the joints that were unpainted, together they represent 44 percent of the total of structurally significant field welded joints.

Now although we didn't use statistics in our program and that wasn't the intent of our program, I can't help that 44 percent happens to be a statistic and it is a rather large statistic and had we chosen to use the sampling plan to demonstrate painting -- the acceptability of the painted welds, we would have chosen a much smaller number. Nonetheless, we had 44 percent covered by MT's and secondary inspections without the presence of paint.

For the characteristics that are considered more difficult to evaluate and measure in the presence of paint -- that is, incomplete fusion, undercut, cracking and porosity -- we found no unacceptable welds for those characteristics. That large sample size supports a high confidence factor that the same results would be obtained in the painted joints.

The actual results of the secondary inspection of the painted joints -- because we did in fact inspect those -- yielded identical results: that is, no unacceptable joints for those characteristics were found.

Although the evaluation of secondary inspection did not identify even one significantly deficient joint -- and by that I mean one that would have failed in service -- the inspectors did indicate a large number of minor deviations from design drawings.

An evaluation was made to determine the root cause for these minor deviations not being identified in the primary inspection. The most common deviations were undersize, underrun and overrun. I am using the terminology "underrun" to mean a weld that is shorter than that specified on the drawing. It is a common terminology, although AWS uses "underrun" when it references size and I use "size" when I reference size and "underrun" to mean less than the specified length.

Now some of the joints contained more than one deviation, such as the weld may be too long and the part that's too long might also be less than the size shown on the drawing, which would be a double deviation on that one weld. So some of these joints have more than one deviation in them, therefore you cannot sum the numbers used to get the total joint, they give you more than total.

Now for undersize, the most common deviation, 765 joints contained one or more undersized welds.

(Slide.)

The majority of those exceeded AWS allowable by less than 1/16th of an inch. AWS allows you to have -- for size, it allows you to be undersized up to a 16th of an inch and that undersized condition that has to be less than a 16th of an inch undersize cannot exceed 10 percent of the weld length.

Now if you exceed 1/16th inch undersize, for no matter how small a piece of the weld you exceed that, you are outside AWS code. We had 765 joints outside that code. The majority of those, like I say, exceeded the 1/16th inch allowable by less than a 16th of an inch.

Now these profiles here show theoretically what the weld size as the designer determined it is. It shows two acceptable profiles. You can see concavity and convexity, you can see they are within the bounds -- that they meet or exceed the bounds shown by what was termed by the engineer.

This is an undersize, where one leg of it is less than the size called for by the designer. And if you do have an undersize, it is typically here rather than here (indicating), because the weld will tend to lay down as you are making the weld and give you some overlength here and could give you some undersize here (indicating).

Now the methodology used in the 1984 inspection timeframe was again the visual look for the weld for size. Now these welds are all supposed to be 5/16ths of an inch. One of them is less than 5/16ths of an inch, this one here (indicating).

The way the welder -- the way the inspector looked at it -- it took him some time to make this thing -- but there is one small spot right here (indicating) that is the low point in that weld.

Now since it is supposed to be a 5/16th inch weld, to see if that one spot exceeded the 1/16th undersize allowable, you take a quarter-inch, which is 1/16th less gauge, and apply it to that point and make a determination -- as I just did -- that that weld is somewhere between a quarter-inch and five-sixteenth-inch in size and therefore it's okay as far as the code goes.

MR. KNIGHT: Should I interpret your presentation as being what you are exhibiting is the typical type of problem found? Clearly your presentation minimizes the differences.

MR. BERRA: Two things: I don't have the exact percentage, it's in the -- 90 percent of the undersized welds were less than a sixteenth-inch more than what the code allows you. When you hear Mr. Reedy explain how they teach you to inspect welds, AWS teaches you, you'll see the significance of that -- the insignificance of that.

MR. KNIGHT: I am rather familiar with that, I just wanted to know --

MR. BERRA: I also wanted to show that in the initial inspection you look for high and low points. You check that and you may very well have stopped your inspection at that point, once having found that that one met that criteria. But the methodology, using these gauges, that you are familiar with -- I don't know if everyone -- that in the early 1970's it was virtually non-existing in using the gauge to determine the size of that weld. It increased, so that by in the Eighties you had extensive use of the gauge.

Now the inspectors we sent out for the secondary inspection took the gauge and did 100 percent measurement of all the legs they could get to with this gauge of the weld on that joint.

Now this joint -- I said that measurement meets

AWS. This joint does not meet AWS for size, the reason being that when I run this gauge down it is less than a sixteenth undersize but more than 10 percent of the four-inch weld. Therefore not only is this weld rejectable in the secondary inspection, but I reject the whole joint because I dealt in joints.

MR. KNIGHT: I believe a statement was made earlier, throughout all of this, in recognizing that sometimes excruciating thoroughness of running a gauge over a weld, there were no structurally deficient welds found, is that correct?

MR. BERRA: "Structurally significant" meaning that no weld that would have failed.

MR. KNIGHT: How about the worst service condition would have exceeded allowable stresses?

MR. BERRA: Yes, but not because of the undersize situation that I just explained here.

None of these that I just mentioned, these 765, exceeded design allowables because of the sixteenth-inch undersize.

MR. KNIGHT: That's what I would have anticipated.

MR. BERRA: Although with the large number there was a deviation but none of them that came....

Now the second most common deviation was overrun. Now overrun, as I stated, the "overrun" that I am using for overlength, that is a weld that is longer than specified. 754 joints were found to have deviation and therefore rejectable in the secondary inspection because they had welds longer than that specified. 658 of those 754 were in the return portion of the clip angle to embed, that's the clip angle weld to the embed.

(Slide.)

Although AWS does not specify specifically what is a rejectable overlength weld, they say that welds that are substantially in excess with no definition of what "substantially" means, those that are substantially in excess should be evaluated.

For the design detail for the clip angle, the return weld on clip angle to embedded plate, keeping in mind that this clip angle is generally 18 inches long so you have something like this (indicating). The return weld, these portions of the weld are to be twice the weld size as a minimum but not to exceed one-inch maximum.

So this should be 5/8ths as a minimum and not exceed one inch max.

This is an example of an overrun (indicating). This is an example of an underrun (indicating).

On this you have two acceptable return welds on this side -- actually it is one weld. The top and bottom are acceptable for return. On this side you have an example of an undersize -- underrun, rather, and an overrun. Either one of those conditions is rejectable and 658 were too long. None of that caused the design stress allowables to be exceeded.

MR. KNIGHT: Will someone refresh my memory? Is the return considered in the structural calculation?

MR. IVANY: No, not in terms of the weld area.

MR. BERRA: So the important part there again is significance. A large number, 700-and-something joints, and a weld that is not included even in the design calculation.

The next most common deviation is underrun. That is a weld less than that shown in the design going as far as length it goes. AWS does not give a tolerance on the length of a weld. It tells you if one is substantially in excess you should evaluate it, but it doesn't tell you anything about one less in length so it tells you it has got to be that length.

We had 233 joints that contained one or more welds that were underlength by a fraction of an inch. Now again keeping in mind that this is a scaled-down version, basically a foot and a half would be this side of the weld typically. We had 233 that were short by a fraction of an inch. They were all rejected by the secondary inspectors and all evaluated as acceptable by the designer.

The evaluation for root cause concluded that the major reason for differences between the primary and secondary inspection was the different inspection methodology. The same criteria, different methodology in inspecting for that criteria, referring to the 100 percent measurement of the weld as one of the primary instances and the no tolerance philosophy that had evolved by that time.

Both the primary and secondary visual inspections were for the AWS attributes that appear on the slide. However, the methodology and philosophy of inspection in the secondary inspection actually exceeded that required by AWS.

Mr. Roger Reedy will also address the subject of the methodology changes in inspection. Mr. Jerry Brown will discuss the results of the overall engineering evaluation of the results of the inspection.

(Slide.)

And, in summary, the KG&E management plan addressed all of the CAR-19 findings and some additional such as the look into other programmatic areas that arrived at the following conclusions:

Quality assurance program deficiencies were confined to CAR-19 issues.

Presence of weld inspection documentation without presence of welding was caused by human error.

Weld record retrievability problems did not carry over to other programs.

The welding program is in accordance with AWS D1.1, 1975 edition.

All quality criteria as specified in the related design documents are met and all structural steel erection commitments in the Wolf Creek FSAR are satisfied.

Are there questions, gentlemen?

MR. BARTON: Let me ask one question:

You previously -- and you have focused appropriately on the 21 percent of the joints for which weld -- for which field welding was done and for which issues were raised. In the review -- and then you described the records program used in other activity areas, covering a broad scope as really being not subject to the deficiencies inherent in the field erection as welded.

That general statement you made, is it applicable -- was there a different record system applicable to the 20 or 30 percent which were bolted connection such that you did not have a record retrievability difficulty relative to all the bolted connections that are field erected pipe connections and were the shop records in good shape?

MR. BERRA: The answer is yes but I'll go through the reasons why.

The shop welded welds were not only performed in the shop, they were inspected in the shop by the fabricators' inspection program. The records are retained in the shop.

That program was overviewed by resident inspectors from Bechtel Corporation. Those records were audited by KG&E. I don't know whether --

MR. MYERS: By Bechtel and KG&E.

MR. BERRA: Also, as you well can imagine, that shop is in a permanent place and their recordkeeping -- that's all they do -- is for fabricating and they have their records as compared to building a plant. So that addresses the shop.

In addition to that, there was receipt inspection above what is required by any code, and that did include a receipt examination inspection of the welds. That was an extra program initiated at Wolf Creek.

The bolted connections again are different in the sense that you know of the bolted connections whereas we didn't keep track of temporary bolting like we do welding. The obvious difference being the use of a temporary bolt doesn't give you a heat related area that putting a temporary weld does, you don't affect the characteristics of the base material.

The bolted connection program was re-audited itself because of another reason, a question had come up about bolting -- I don't remember which caused it -- which caused a relook at the bolting and the bolting records and the methodology used for bolting so that was covered.

It was looked at by CAR-19. CAR-19 didn't have to spend a lot of time looking at it because of the information that was available about the bolting.

MR. MARTIN: Thank you.

MR. THOMPSON: One quick question:

You talked about the human errors in the missing welds that were not identified. Could you either give us a little bit of why you concluded it was human error or show us on your diagram which welds were missing? Or is that something that is going to be covered elsewhere?

MR. BERRA: There are several areas. One, we had some missing welds -- actually it was missing material associated with beam seats, a beam seat -- which this doesn't have (indicating), what you would have in here is another member to support this beam during the erection process. Typically that's a temporary installation. We had, I believe, in the neighborhood of 30 joints where that beam seat wasn't there that showed on a drawing.

Several of those -- and I don't have the number -- you can see that the beam seat had been there and what is called washed off because you can see the indication that the welds had been there previously. This substantiated that the carbon practice for erecting a structural steel beam would be to put a temporary device there to set this beam down, tack the other on, weld it out and wash off the beam seat. Some of the drawing details did require that you leave that there. So that was missing.

There were 69 missing -- 69 joints that had missing -- 66, pardon me, joints that had missing welds that we feel were caused by misinterpretation of a detail. One detail was used for 60 radial stops that were installed on the polar train; on all 60 of those radial stops the same missing welds occurred. We can only postulate that they were all done by the same welder, that that detail confused the welder and therefore the inspector.

It has somewhat also confirmed that when we did the re-inspection and the different inspectors looked at those 60 not all of them raised the question; one of them raised the question, his initial answer from the field engineer in looking at the joint was there's nothing wrong with it, that they would check further with the design engineer and, yes, there was supposed to be a weld there. So that one detail should only have been used 60 times and wasn't used.

The pressurizer welds. There are six supports for the pressurizer. There was a missing weld on each of those six supports, the exact same weld missing on all six. Looking at that detail, you can see where the human error could arise during the erection process.

Typically if you did put a beam seat it was down here (indicating). That particular support had a beam seat here -- and it wasn't a beam seat but it looks like a beam seat sitting on top of it also -- and there was a weld up there (indicating).

Now although that involved 66 joints, that was only two details and in all our review we did not come up with any other details that we found misinterpreted so it was not a pervasive problem, it was a limited problem. But it just so happened that it involved those. That's where we get human error.

The other two -- in detail I could go through -- we looked at what the inspector did, how he kept his records and from that we projected the human error. We found no malicious intent.

MR. THOMPSON: That was for just one particular weld?

MR. BERRA: Yes, that inspector was one weld, both of them was one weld. One was a clip angle and I don't see any reason for misinterpretation there. He wrote down A,B,C,D, and when I talked to him on the phone, he said sometimes you would write this down and you would go look at it and if it wasn't all there you would take the D off or whatever and he didn't. I forget which one of the four was not present. He couldn't remember ever having done that. That's where the human error comes in.

Any other questions?

MR. DENISE: I have a couple. On the generation of MSSWR's, would an MSSWR be produced for a weld which the inspector called deficient?

MR. BERRA: No.

MR. DENISE: He only produces MSSWR's on acceptable welds, is that correct?

MR. BERRA: There would have been an incomplete MSSWR generated. That is, the welder would have entered his portion of the data on the weld record, he would have asked the -- he would have put the weld on the list, an inspector would have gone up to inspect it, the inspector would have noted -- if he noted a deficiency he would not have signed it, he would just tell the welder of the deficiency and the welder would repair it and fill out another MSSWR. The inspector would come up, if that was acceptable he would sign that and that would be the MSSWR record.

MR. DENISE: Okay.

MR. MARTIN: In your document requirements, would they have required in this program for both of those records to have been retained or only the completed one?

MR. BERRA: Only the completed one.

MR. MARTIN: So if the craftsman or anybody tossed it in a GI can or took it home and threw it away or anything, that would be no violation of record control for the Wolf Creek facility? That is, the incomplete one?

MR. BERRA: That was an in-process inspection. You keep a record of the in-process inspections.

Now the welding program was overviewed by welding engineers, Bechtel welding engineers, the welding engineering department. They do not report to the production department and they are not the inspectors.

But they overview the program of both the qualifications of the welders and the maintaining of those qualifications. We use them to maybe look in a difficult joint, meaning that the welder is doing his weld, the welding engineer puts his hood on and looks at the joint. He keeps the equipment up that they use to do the welds and he is very cognizant of the various welders' abilities.

You and I may both be qualified to do a weld, you're just more qualified than I am, you're just better. So if we had a very difficult weld he might use you instead of me.

He also overlooks what is happening with your rejects. If one of us is getting a little too much in the reject area, he will take us back to the booth and weld up some coupons for practice and then put us back out. If we can't cut it that way, then he'll lift your stamp identification number and you cannot weld.

MR. DENISE: You talked earlier about building turnover, I think it was your first slide. I noticed that you had building turnover on the fuel building and the ESWS buildings early in 1984. Those buildings had a significant fraction -- or at least one of them did -- of missing MSSWR's.

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The pump house, I believe, had the largest number, although there are only 36 joints in the pump house and we had a large number.

(Slide.)

MR. BERRA: We forgot to take credit for these in the numbers I was giving you earlier. We went back and did the 100 percent re-inspection of the pump house but I didn't count those in the ones we had.

MR. DENISE: The thrust of my question really was though you had a lot of missing records in early turnovers. This didn't seem to surface until the last quarter or the end of the third quarter of '84.

MR. BERRA: The records was here in 1984 and I believe CAR-31 was October of '83. CAR-31 is the CAR that addresses the miscellaneous records, that was in October of '83.

MR. REEDY: That was written in August of '83.

MR. BERRA: So it was in August of '83 that the CAR was written and earlier than that you started noticing -- in fact when the CAR was written, CAR-29, it already says on the CAR how many of the records are missing for several of the buildings. So we knew we had a records problem. We counted them up, saw the percentage, issued a CAR that showed a large percentage for several buildings and all of this was in '83.

MR. DENISE: The statement in response to CAR-19 which says that neither CAR-29 nor CAR-31 required matching of MSSWR's to ESWS welds or welded connections, if this had been required, corrective action for either CAR, the problems identified in portions of KG&E CAR-19 would have been realized.

You're saying had there been a matching back then --

MR. BERRA: In CAR-29 it was a look at hardware only. It was a look at the physical attributes of several joints. At that time the primary inspection records were not retrieved to see if they existed at that moment in time. That statement says that perhaps if we had pulled those records and none of them were there we would have solved the problem earlier as far as record retrievability.

The CAR-31 approach was not for the pump house, which was 36 joints, and a couple of buildings here, one other building, to arrive at those numbers there was a match joint-for-joint of the weld record to a joint in the building.

In CAR-31 -- since, at that time, no previous inspection was performed by anybody -- there was a sample NRC inspection in the summer of '83 and no deficiencies were found in the weld. There were some of the sampling plans done and no deficient welds were found.

CAR-31 postulated that it wasn't a hardware problem, that there was a problem in retrievability, that the hardware was good. As CAR-19 verification demonstrated, the hardware was good. So it really didn't make any difference if you were missing weld record B, out of A through Z, or weld record D, the fact is you're missing a record.

So what we went through was a counting of "Y ou should have 100 records, you've got 82, you don't have them all."

Had we matched them, I don't think that would have made any difference in CAR-31 because we still wouldn't have had them all. We knew that and that's why we issued CAR-31.

MR. THOMPSON: Any other questions? I know everybody is dying to eat. I would recommend a kind of a recess to 2:15, about an hour from now. The Staff needs to caucus at 1:45 in Darryl Eisenhut's office, I&E Region 4, Engineering, my staff should meet with Darryl at 1:45.

We will reconvene here at 2:15. At the latest, it will be about 2:20.

(Whereupon, at 1:10 p.m., the conference in the above-entitled matter was recessed, to reconvene at 2:15 p.m., this same day.)

AFTERNOON SESSION

(2:30 p.m.)

MR. MARTIN: Let me suggest that we begin the afternoon session. Mr. Eisenhut is detained. Mr. Hugh Thompson, the Director of the Division of Licensing, will be down imminently. However, the Staff is assembled. I believe at the end of the prior session we had just met the point between two of the KG&E presenters. So Mr. Koester, if you will continue with the remainder of the presentation and start your next presenter, please.

MR. KOESTER: I will let him introduce himself.

MR. BROWN: My name is Jerry Brown. I am the Civil Engineering group leader with Bechtel Power Corporation. I will now describe the selection of the structurally significant joints and the engineer evaluation that was performed with regard to the re-inspection of AWS structural steel welding at Wolf Creek.

(Slide.)

The joints included in the re-inspection and evaluation program were all structurally significant AWS steel welded joints which support or potentially support safety-related equipment and building components. The joints are, of course, located in the safety-related buildings. They were installed by Daniel and other structural steel and miscellaneous steel contractors. And they were inspected originally under the miscellaneous structural steel weld records inspection program.

A total of 2670 structurally significant joints, representing over 11,000 welds were identified for this re-inspection program.

(Slide.)

This viewgraph included in your handout provides some pertinent information on the re-inspection of these joints which were identified by a review of the structural and miscellaneous steel erection and field work drawings, Bechtel detailed drawings and project non-conformance reports, field change requests and field fabrication requests.

The structurally significant joints were re-inspected by AWS certified weld inspectors to the existing project acceptance criteria contained in AWS D1.1 and the structural steel and miscellaneous steel specifications of the 2670 structurally significant joints. 2551 joints were re-inspected. All welds were inspected by the inspectors on 1292 of these joints. 119 joints were not accessible for re-inspection. 2670 joints, 2551 joints were re-inspected, 1292 joints, all welds were accepted by the inspectors. 119 joints were inaccessible.

Approximately 40 percent of the re-inspected joints were not painted. These joints were not required to be painted because they are encased in fireproofing materials which were removed for this re-inspection. The welds of the remaining joints were inspected through the paint.

The re-inspection of welds through paint was evaluated during the early stages of this re-inspection program because it was recognized that all weld attributes could not be visible through the paint. This evaluation was performed by Bechtel materials and quality services group and it was reviewed by both Region 4 and NRR personnel. Evaluation established that an adequate re-inspection program could be performed through the paint. This position was established on the following bases:

The weld attributes that could be inspected in accordance with AWS are listed on this next viewgraph.

(Slide.)

Of these, the attributes that were judged potentially masked by the paint are cracks, fine porosity, tight undercut and lack of fusion. Although tight undercut is included as one of the items, it is our opinion that rejectable undercuts -- that which is greater than 1/32 of an inch -- would be visible through the paint. In fact, roughly half of the cases of undercut identified during the re-inspection were on painted joints.

The re-inspection of the other three attributes was determined not to be critical to this program for the following reasons:

AWS D1.1 is applicable to a variety of welding procedures and materials. Some of these procedures and materials are susceptible to cracking, porosity and fusion problems as a result of variabilities in the welding procedures and material properties.

In these cases, the inspection for these attributes is, of course, critical. However for the E7018 electrodes, the sealed metal arc welding procedures and the mild carbon steels used for the erection of structural steel at Wolf Creek, this is not the case. These are the most commonly used procedures in highly weldable materials available to the steel construction industry.

Years of experience on all types of steel construction have demonstrated that with reasonable precautions and controls these procedures and materials are not susceptible to cracking, porosity and fusion problems.

The re-inspection of over 1000 unpainted joints, the magnetic particle examinations performed by Daniel during construction, and the magnetic particle examination performed by Region 1 personnel all support this position.

In the re-inspection program, only three cracks and welds were identified, all in beam-to-beam seat welds. The engineering evaluation was that these cracks were the result of load-induced stress conditions, that none of the cracks in any way reduced the capacity or impaired the integrity of the joint, that they were not the result of variation in welding procedures or material properties and that they probably were not present during the period of the original inspection.

Only five cases of minor porosity and 91 instances of minor lack of fusion were identified during the re-inspection. None of these welding deviations resulted in allowable stresses being exceeded in the joints.

As explained in the NRC exit meeting at Wolf Creek on February 9 of this year, the magnetic particle examination of welds by Region IV personnel found no indications of cracks, porosity or lack of fusion. We believe all these results support our conclusion that the weld attributes critical to this re-inspection can be inspected through the paint.

The re-inspection reports were all forwarded to Bechtel for evaluation. For any joint with a noted welded deviation, the complete as-built condition of all welds on that joint were described on a re-inspection report.

Each joint was evaluated for the as-built condition using conservative engineering assumptions to determine if the allowable stresses committed to in the design criteria and the FSAR were satisfied.

As an example of these conservative assumptions, in those cases where some portion of a weld was noted as undersized in our evaluation we typically assumed the entire weld length was undersized or, for deviations, identified as lack of fusion or partial fusion for some length of the weld, that entire portion of the weld was considered as missing in our evaluation.

In those cases where the allowable stresses were not satisfied, an additional evaluation was performed to determine the ultimate capacity of the joint.

82 joints were determined to exceed the allowable stresses in as-built condition. These included six similar pressurizer supports and 60 identical reactor building polar crane radial stops.

Analysis demonstrated that none of the 82 joints would have failed in the as-built condition for the most critical loading combination. All of these joints were repaired in order to assure that the allowable stresses would not be exceeded in the completed facility.

The re-inspection identified 130 which had a missing weld or welds and 20 joints that had missing material. They were typically beam seats that were missing in the joints.

These missing welds and materials resulted in allowable stresses being exceeded in 69 of the 82 previously noted joints. Allowable stresses were not exceeded for the remaining 81 joints that had missing welds and material.

KG&E management directed that all accessible welds in this group be repaired in order to restore their original design condition. 67 of these joints were repaired. The remaining 14 were evaluated by Bechtel and approved to use as is.

119 joints were totally inaccessible in the re-inspection program, while an addition 165 joints had some portion of the weld length inaccessible. Sufficient information or alternate load paths were available to allow a case-by-case evaluation of 201 of these joints. Therefore there were only 83 of the 2670 joints which were not evaluated on a case-by-case basis in this program.

However based on the very large sample of joints which have been evaluated on a case-by-case basis, we have very high confidence, in the order of 99 percent, that the reliability of these 83 joints is the same as the joints which have been evaluated.

Therefore we would not expect to find a joint in this group that would fail under the design loading conditions as a result of welding deviations.

In summary, the 100 percent re-inspection of AWS structural steel welding was initiated primarily as the result of the identification of missing welds during the sample re-inspection performed in September of 1984.

The conclusion of the re-inspection evaluation program has determined that missing welds and material -- primarily missing welds -- were the significant welding deviation which resulted in the violation of design allowable stresses.

Additionally, the incidences of missing welds were generally limited to three specific areas of construction: pressurizer supports, polar crane radial stops and beam seats and are not attributable to an across-the-board breakdown of the welding quality program. The results of the engineering evaluation of AWS structural steel welding at Wolf Creek have not identified any significant deficiencies in the welding which would have impaired the health and safety of the public.

If there are any questions, I would be happy to address them.

MR. DENISE: I have questions. Just a little housekeeping.

How is the paint thickness determined on those welds?

MR. BROWN: KG&E used the appropriate equipment for measuring paint thickness. They actually went back and measured the paint thickness on almost all of the joints of paint, if not all, certainly in excess of 1000. The paint thickness varied from two to 15 mils but I believe greater than 90 percent of the joints had less than 10 mils of paint.

MR. DENISE: Can someone tell me -- maybe someone else can tell me if it was measured with a standard scratch gauge or what model or whatever?

MR. RUDOLPH: It was measured with a scratch gauge.

MR. FOUTS: The thickness of the paint was measured with a dry film thickness gauge, I believe made by Keene Taylor.

MR. DENISE: On the re-inspection, where are the re-inspection records kept?

MR. BERRA: Which records, the --

MR. DENISE: I'm on to joints now, the secondary inspection of miscellaneous structural steel welds. Where are the re-inspection records kept?

MR. BERRA: Currently now they are in KG&E construction, specifically in John Fletcher's area.

MR. DENISE: What are they called? Do they have a name?

MR. BERRA: I think it's called the inspection verification plan.

MR. DENISE: Will these be part of the permanent plant records or are they going to go to some --

MR. BERRA: Yes.

MR. DENISE: They are not going to go in one of these boxes?

MR. BROWN: Dick, with regard to that, all the deviations, all the inspection reports of deviations are attached to non-conformance reports which were forwarded to Bechtel Power Corporation. Those non-conformance reports have been dispositioned. For all joints with noted deviations there is a copy of the re-inspection record attached to the NCR.

MR. DENISE: I'm asking KG&E. Are these re-inspection -- is re-inspection documentation now part of the permanent plant records?

MR. REEDY: They will be retained as quality records.

MR. KOESTER: I wanted my quality assurance man to tell you that.

MR. DENISE: I understood you to say, Mr. Brown, that the 83 inaccessible joints were assessed as being acceptable based on the statistics associated with the other inspections. Is that correct or incorrect?

MR. BROWN: That's correct.

MR. DENISE: I want to talk about some of those joints for just a minute.

I am making reference to Mr. Koester's letter to me dated February 22nd, 1985 and I make reference to joints R-175 first called beam-to-beam. It supports the pressurizer, full penetration welds between the pressurizer support members.

MR. BROWN: Yes, I'm familiar with it.

MR. DENISE: Okay.

Is that an inaccessible weld?

MR. BROWN: That weld was inaccessible at the period of the re-inspection under this program.

MR. DENISE: Has it been re-inspected since?

MR. BROWN: It has not.

MR. DENISE: Is it physically accessible now?

MR. BROWN: I would think it is now or would be shortly.

MR. DENISE: Does anyone from KG&E either have a different or a confirming opinion of that? That's joint R-175, 2029 foot level, the reactor.

I was asking about 175. I'm just asking are they physically accessible in the sense that someone could go down inside the containment and --

MR. BROWN: 175 can be made accessible.

MR. DENISE: What does "made accessible" mean?

MR. BROWN: I'm sure that access can be reached to that particular joint.

Of the 83 joints that are classified as

inaccessible -- in other words, sufficient information was not available at the time of the engineering evaluation for Bechtel to do a case-by-case evaluation at that point in time joint R-17, due to ILRT and SRT in the reactor building was not accessible. We could not gain sufficient information at the time of our evaluation to do a case-by-case evaluation of that joint. There are a few joints that fall into that category of the 83.

MR. DENISE: Could you tell me what the other ones --

MR. MARTIN: How many?

MR. BROWN: I would say -- I'm guessing, I haven't counted them -- I would speculate it's in the range of 10 to 15.

MR. DENISE: 10 to 15?

MR. BROWN: Yes.

MR. MARTIN: Help me to understand, I may oversimplify this. Because of very termporary field conditions, that is, an ILRT/SRT test, which is finite in time frame, there were a number of joints I presume inside the reactor building that therefore were temporarily inaccessible.

MR. BROWN: That's correct.

MR. MARTIN: And the owner, architect/engineer and constructor decided to treat them as though they were forever inaccessible and treat them in the same fashion as a joint bearing in concrete?

MR. BROWN: There is more than adequate information --

MR. MARTIN: I'm thinking in terms of the corrective action program that has been identified and how one would go about handling the re-inspection program and the total corrective action program. I don't think conceptually it was our belief that if something was inaccessible on Tuesday but available on Wednesday that you would write it off as inaccessible but rather you would come back on Wednesday and inspect it as part of the corrective action program.

Now one might be one issue. More than one becomes a different issue.

It seems to me that those could be inspected if they are now reasonably accessible because temporary field conditions have changed and those analyses could be done.

MR. BROWN: Certainly during the entire period of the inspection we worked around accessibility to gain access to all of the joints as appropriate, with the exception of these few that were inaccessible for a period of time right at the point in time when this issue was wrapping up and we were required to submit a report.

All the data available for those joints did not pose a problem to the engineering evaluation. It would not change the results of our evaluation.

MR. DENISE: Let me try it from a slightly different direction by asking, first of all, for Mr. Rudolph a question:

Was the thrust of CAR-19 to require the inspection of all accessible joints, painted or unpainted, where inaccessible was defined as being buried in concrete or another physical impediment to access?

MR. RUDOLPH: CAR-19 recommended that inspections be performed on those joints that would be accessible. Now what that means basically are those joints that would always be inaccessible would be evaluated for a usability or serviceability for use. But for those that in the process of this inspection, due to constraints of a temporary nature, those joints, when they became then accessible after those temporary field conditions were removed, those would then be inspected generating MSSWR's and those records be retained as quality records.

For those joints that were inaccessible permanently, like you mentioned, there is no need in my opinion to recommend corrective actions that would involve doing anything beyond documenting a suitability for use evaluation, which has been done.

The answer to the question is that for those joints that are accessible as a result of removing the temporary constraints that were in place at the time of the inspection, those joints would be inspected and the documentation will be retained as quality documentation consistent with the recommendation of the CAR.

MR. DENISE: Okay.

MR. MYERS: Could I make one more comment on the subject?

The analysis that Mr. Brown gave you was an analysis of the data available to him at the time that he did the analysis. He is telling you that he did not analyze 83 joints; he did analyze approximately 2500 or 2600 and, from an engineering point of view, the extrapolation from a sample to 2500 -- from 2500 to 83 is certainly valid.

It is a separate question other than the engineering evaluation as to the records of those joints that he did not analyze.

MR. DENISE: We've crossed that threshold. We are now on a different tack.

MR. RUDOLPH: From a quality context, which I believe is the perspective that you're asking this question, by going ahead and inspecting and documenting those inspections, that will not impair or degrade any of the engineering analysis or drop any of the numbers down. That will only enhance the numbers and make them stronger.

From that perspective though that you asked the question, will inspections be performed, will those inspections be documented, and will those inspection records be retained as quality assurance records, I believe the answer is yes.

MR. DENISE: I don't think I've asked that question yet.

(Laughter.)

MR. RUDOLPH: I'm sorry, that's the way I understood it.

MR. DENISE: I asked the question earlier for those ones which have been inspected, are the records of those inspections part of the permanent plant record,--

MR. RUDOLPH: The answer is yes.

MR. DENISE: -- and I got the answer Yes.

Now where I was headed with Mr. Brown, since I understand that he made an analysis -- I guess you made the analysis -- that Mr. Koester reports on February the 22nd that identifies joints R-175 and 176 as inaccessible, and I understand now that this means at the time you gathered data to determine the acceptability of joints, this particular joint was inaccessible because the containment was pressurized and it is hard for folks to inspect joints under 60 pounds of pressure or so. All right.

Now I'm going to follow this point one second. Is CAR-19 closed?

MR. RUDOLPH: Yes. And the action is verified.

MR. DENISE: All right.

Now does the non-inspection of joint R-175, which is accessible, fulfill the requirements of CAR-19?

MR. RUDOLPH: Yes.

MR. DENISE: Okay. So you are defining inaccessible to include temporarily inaccessible?

MR. RUDOLPH: When the CAR was written, as quality assurance manager, I have no foresight as to predict what joints may or may not be inaccessible. I do not know. That is not-- That's the engineer's determination at that point in time, so you attack that kind of problem from two perspectives.

You basically say for those that are accessible, do the inspection, document inspection, retain the records. For those that are inaccessible, provide an engineering evaluation for suitability of use.

When the inspections were performed, there were a number of joints that were inaccessible as a result of some testing activities.

I think the commitment we've made here, which wasn't expanded upon in the CAR, not purposely because it was never defined as to what accessibility represented, the commitment we have made here is for those that are not -- for those that are accessible and those for which the testing restraints have been removed, those will be inspected.

The numbers like were quoted as 82 will only get smaller. There are a definite number or a finite number that will never be accessible, and that was never the intent of the CAR, other than to evaluate those for suitability of service. And that's the perspective that I had when I wrote the corrective action request.

MR. DENISE: I probably have that corrective action request with me, and I can't lay my hands on it. But having reviewed it at least once, it seemed to me that I recall the definition, and we just talked about this, that the definition of "inaccessible" was buried in concrete or otherwise physically inaccessible by physical interferences.

I think I heard you say that you would not consider a temporary inaccessibility due to on-going work as meeting the CAR-19 definition of inaccessibility. Did I misquote you, or misunderstand you? I just want to go step by step.

MR. RUDOLPH: Because there is obviously-- What we're talking about is a definition in my CAR. I'm going to look at the CAR and I am going to find out what we were thinking at the time that we wrote it before I answer the question.

But I understand where you are coming from, and I think the answer is -- I know the answer is the numbers will only improve as a result of the inspections.

The accessibility that in the frame of mind that, you know, I had when the CAR was written is basically that it was in concrete or permanently inaccessible. There would be no inspections necessary and that's why we went one step further and said for those situations we need a serviceability-for-use evaluation from Engineering, which is exactly what happened.

MR. DENISE: Do you want to talk about the words now, or we can wait until later?

MR. RUDOLPH: I can address them later.

MR. DENISE: Because I want to go on to more important things. I don't want to argue over the words.

MR. BERRA: It is easy to address because the CAR itself does not give a definition of inaccessible, nor does it even mention inaccessible. Those were in subsequent meetings that were held.

MR. DENISE: All right.

So on the letter which I have -- and I don't know that you're familiar with it, KML NRC 85-065 -- it lists the 83 inaccessible joints. I would like to know if those joints are inaccessible by the definition of inaccessible meaning buried in concrete or otherwise having a physical -- permanent physical impediment.

Now secondly I understood you to say that those joints which don't meet that category but which are presently or will be shortly accessible because of plant conditions, you would inspect them to be inspected. Did I hear you correctly?

MR. RUDOLPH: Yes.

MR. DENISE: Okay.

So I will tell you that I particularly would like to know about joints 195 and 175 and 176-- I'm sorry.

I am particularly interested in the early answers on joint R-195, R-175 and R-176.

MR. BROWN: I think we can provide information right now on those particular joints.

MR. IVANY: I believe we provided some additional information to Region IV this morning before the meeting that indicates that we have documentary evidence of the existence of the welds in those joints and of inspection on those joints, not part of the secondary inspection but from other sources.

For example, on joint R-175 we have a magnetic particle test report Number 4322. We also have a--

MR. DENISE: What is the date of that, Mr. Ivany?

MR. IVANY: 11/5/84.

MR. DENISE: Okay.

MR. IVANY: We also have our reinspection identified as Item 5 on non-conformance report 1SN-20616-CW, which was submitted to Bechtel for evaluation.

MR. DENISE: And the date on that NCR?

MR. IVANY: The disposition date was 10/8/84.

MR. DENISE: I understand the disposition date, and I don't know about the opening date, but in the first case it sounds as though the MT on 175 was done during the period of reinspection, so it must have been accessible.

Is that a reasonably true statement? I think you said November--

MR. BROWN: That was at the point in time when reinspection was attempted.

MR. DENISE: November '84 I think you said, Jim, MT? Okay.

MR. BERRA: The NCR I believe was-- As I've mentioned earlier, at the time we issued the CAR, approximately 2,000 welds had previously been inspected. I believe the NCR was written and part of that is the missing documentation NCR which precipitated that specific inspection of one of the joints. Although that inspection was made and we have the record for it, all the numbers we got were out of the specific CAR-19 records relative to those joints.

I don't know why the MT -- what caused the MT to be performed. I do know what caused the NCR to be generated.

MR. DENISE: I am not in any way trying to ignore or undercut the statistical position that Mr. Brown has put forth, that one can reach certain conclusions with confidence levels that are based on the inspection of the other welds.

My line of questioning is simply are these joints which are called inaccessible inaccessible really, and if they are not, ought not they be inspected to determine location and size and length and those other attributes which go into your reinspection program in order that your reinspection program meets the full intent that it was designed for?

MR. BROWN: There was sufficient data in the record that would indicate that those joints would not have to be reinspected.

MR. DENISE: Where did the data come from?

MR. BROWN: The reinspection of the 2551 joints that were reinspected. There is sufficient confidence and reliability developed from that sample that provides us with adequate confidence of the reliability of those 83 joints, whether they be in concrete or for those few that were temporarily -- temporarily at the very end of the reevaluation period where we could not get that data in time to complete the report.

There is no difference whether those joints are accessible now or whether they are embedded in concrete, we have adequate confidence in the reliability of those joints.

MR. MARTIN: May I suggest -- and I think I understand from the exchanges that (a), I believe that was the correspondence from Mr. Koester to me satisfying the enforcement issue, or is that the last report?

MR. DENISE: The one I was referring to?

MR. MARTIN: Yes.

MR. DENISE: This was a letter from Mr. Koester to me.

MR. MARTIN: Let us look for a supplement to that to address the commitments that have just been made in terms of changing those numbers, changing the identification and resolving the remaining classification of joints.

MR. BERRA: Dick, I did a quick count. There's nine of them were only inaccessible due to the IART. The remaining were either embedded in concrete or were inaccessible due to the installation of other structural steel, or inside a wall, a dry wall.

MR. DENISE: How does joint R-195 look?

MR. BERRA: 195 is inaccessible for one of the welds in that joint. It is enclosed in a metal enclosure in the reactor building.

MR. DENISE: Is it partially accessible?

MR. BROWN: Yes, it is. The accessible part of the joint has been inspected.

MR. DENISE: I hope you see that the cause of my question is because of two attributes in the report. One was the equipment that the structural steel supported, and the other was the calculated factor of safety.

But again I will repeat, I'm not arguing with your statistical basis.

MR. KOESTER: Mr. Martin, we will review this list that was transmitted to Mr. Denise on February 22nd, and any that are accessible today, a supplement will be reported to you and any problem welding, that will be documented.

MR. DENISE: I don't have any other questions.

MR. MARTIN: Well, let's move on. In terms of trying to schedule the remainder of the afternoon,--

MR. KOESTER: Mine is very short.

MR. MARTIN: Then it is up to you as to whether or not we might want to give everyone a break or not. And I will leave that to you.

MR. KOESTER: I would just as soon go and finish this up if it is all right with you.

I am Glen Koester from KG&E.

From what you have heard so far I think it is obvious we have put together a very comprehensive program to determine and evaluate the welding at the Wolf Creek Generating Station. However, all the people who have talked to today have been associated with the project in some manner during the design and the construction phases of the plant.

Therefore, I think it is important that you hear from individuals not directly involved with our project, and who we have asked to take an independent look at various aspects of our corrective action program.

We retained the services of three independent groups to do this look at our program: The Reedy Associates, Lehigh University, and ABTECH.

Mr. Roger Reedy was asked to take an independent look at our overall corrective action program.

Dr. John Fisher and Dr. Roger Slaughter, both professors of civil engineering at Lehigh University, were asked to take an independent look at certain aspects of our program, primarily relating to the engineering evaluation performed by Bechtel, the significance of the deviations, and the inspection through paint. Dr. Fisher will review with you their look at the Bechtel program.

Dr. Jeffrey Egan from ABTECH was also asked to make his independent review of our corrective action program.

In your packet you will find all of these gentlemen's credentials.

I will now ask Mr. Reedy, Dr. Fisher and Dr. Egan, in that order, to discuss with you their independent reviews, and I will allow them to introduce themselves.

Mr. Reedy.

MR. REEDY: My name is Roger Reedy. I reviewed the KG&E program with regard to CAR-19 at the Wolf Creek site. In my review I looked at inspection procedures, interviewed personnel from Bechtel, from Kansas Gas and Electric, and from Daniel Construction. I reviewed the KG&E program, reviewed the work that was performed by the Region I inspectors, read the notes from the exit interview, and I visited the plant to look at some of the structural welds that had been inspected, both by -- in the program and by the NRC inspectors.

In order to have some better idea of my approach to this I would like to review for a minute some of the ideas of code philosophy and code hierarchy.

In the structures that we're talking about, we are talking about structures that were designed to the AISC code, and it is through the AISC code that we get into the AWS D-1.1 welding document.

As a commentary I would like to read from an AISC publication with regard to the use of the AWS specifications. It is called "Quality Criteria and Inspection Standards," Second Edition. It starts out as an introduction:

> "The human element is involved in all phases of structural design and fabrication. Therefore, it is not surprising that an unintentional deviation from a drawing or a specification can occur. Not all errors or deviations need to be altered or repaired. Many could be accepted without change with no penalty to the structure or its end use. There are times when repair work creates higher residual stresses and does more harm than good. In general, it should be the engineer's decision whether or not the deviation is harmful to the end use of the product."

I would also like to read from the commentary document to the AWS D-1.1 code, and this is with regard to the paragraph on "Application."

> "This code was specifically written for use in the construction of buildings, bridges, and 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."

With that as an introduction, I would like to get into a little bit of the background of inspection philosophy.

This morning Mr. Berra got up and showed a model of the welded joint or connection. And he's right, it is heavy.

In order to inspect this type of structure it is necessary to use good judgment, in other words, common sense. In order to evaluate a weld, or review it, it is ordinary for an inspector to take a look at the weld, to take a metal gauge of some kind, even a ruler, and measure the length by eye, look at the number of passes, the weld passes in the structure and, by looking at size at several points and the number of passes, he can make a very good judgment as to whether or not that weld is adequate. And I say "adequate."

He cannot tell every nook and cranny by looking at it as to whether or not you might be slightly undersized.

To take a weld fillet gauge and run it the length of a weld to see every little point that may occur from time to time and declaring a weld to be undersized because of one point undersized is not required, and it reaches the point of being ridiculous.

Engineers design welds to the nearest 16th of an inch. If you read drawings you will see that the welds are described as a 1/4-inch weld, a 3/8th-inch weld, a 1-1/4-inch weld, a 5/16th weld. To judge a weld as being adequate on the basis of a 32nd-inch undersize is not even practical.

If we look at undercut you would judge undercut to the closest 32nd of an inch because that is the practical measurement for seeing if you might have a problem.

If you're looking at weld length you might choose to measure to the nearest 1/4th of an inch.

So inspectors who have been taught how to measure welds and how to judge welds through the AWS training course are taught to use this type of judgment. They are taught to round off their measurement.

Now there is an ANSI standard, C25.1, which describes how to round off measurements. All of us who are engineers and who have an engineering college degree ran across that standard or the application of that standard when we were in college. I hope we haven't forgotten it.

When we look at the requirements of AWS D-1.1, they give a number of weld attributes to be reviewed. One of the defects or indications that is not allowed is cracks, and cracks can cause a problem. But not all crack-like indications are rejectable.

If you have a requirement to visually look for cracks and you look and you cannot find them -- and this is possible -- another method but more severe and more restrictive would be to use mag particle examination. That test might show the indication of a small, minor crack.

The fact that you found it would probably mean, by common sense, you would want to fix it, but it is not required to be looked at with that more sensitive tool.

The AWS philosophy for inspection, therefore, is based on good judgment, common sense, and rounding off of measurements in order to determine the adequacy of welds.

From my review of the program down at Wolf Creek, the welds that we are talking about were all judged on that basis for the first time. When those welds were inspected, before they were accepted, they were judged on the basis of what is adequate and what is taught by AWS as to how to inspect welds.

It is the same philosophy that is used today for the inspection of bridges, tubular structures, and steel buildings, skyscrapers.

However, the secondary inspection that was performed was a no-tolerance philosophy. Using that philosophy, a liberal interpretation of the code was used. If a weld size was called minimum, if it deviated even in a small speck a 32nd of an inch long, it was called to be inadequate. Even if the weld were a 64th of an inch below size, it could be described as inadequate.

All minor indications that may occur could be cause for rejection. That philosophy is contrary to the philosophy of AWS.

Any secondary inspection that uses that philosophy is going to find many deviations because you are going from common sense into a no-judgment criteria.

The inspectors are taught always to use judgment. Judgment has to be used in determining whether lighting is adequate, when to use, what type of gauges, when rounding off is allowed, and all of this is taught in the AWS courses.

Now, we saw John Berra up here this morning describing running a fillet gauge over the length of the weld, and I just can't emphasize enough that that has caused many unnecessary reairs, and I read the commentary on what that can cause.

There is nothing in the AWS specification that says a fillet weld gauge has to be used. The specification calls for the use of appropriate gauges. Obviously, there may be conditions when a fillet weld gauge might be appropriate and might be the easiest tool, but then the inspectors should compensate that for the tolerances that are allowed.

I talked to people who are responsible for the writing of both AWS and AISC, and they gave me a pretty good quote. They stated that codes and standards are generally written by reasonable men to be interpreted by reasonable men. I am not trying to slight women. And when we don't have that reasonableness in the interpretation, the problems arise.

I would like to talk for a minute about painted welds.

AWS D-1.1, the 1984 edition, paragraph 3.11.2, states:

Welded joints should not be painted until after welding has been completed and the weld accepted.

The welds were completed and the welds had been inspected and accepted before they were painted. So the requirement of AWS was met.

There is concern, and the reason for requiring the weld to be accepted before it is painted, is that the paint might mask some weld discontinuities. Let's think for a minute what weld discontinuities could be masked by paint.

It is possible that some cracks could be masked by paint. Some very tight lack of fusion, minor porosity, and some minor undercuts all could be masked.

However, minor porosity is not a structural problem, and neither is minor undercuts, and any tight lack of fusion could be examined further, the same as cracks, with a mag particle examination if you were concerned that that condition might exist.

The other attributes of welds which are mentioned in D-1.1 are size, location, existence, concavity, and arc strikes. Obviously, all those could be judged through paint.

The MT examination is a far more critical examination than visual. This is recognized by all the codes and standards which treat visual examination as a rather minor examination, as compared to the more restrictive ones of MT, which is magnetic particle examination, liquid penetrant examination, radiography, or ultrasonic examination.

The magnetic particle examination and dye penetrant are surface type examinations, but the radiographic and ultrasonic exams are volumetric type examinations.

None of the welds that we are talking about were required to be examined by any of those methods.

When the NRC team from Region I was here, some evaluation was made about the magnetic particle examination through paint. In order to create a crack in the weld, it was necessary to use proper wire to weld over it to generate the crack. The sample was painted with up to 11 mils of paint, and magnetic particle examination found every crack that was visually identified before the paint was applied. In fact, the MT examination found things that weren't found visually.

When the NRC went out to the field and reviewed the plant weld joints, there were about 64 reviewed, both with and without paint. They were reviewed both by magnetic particle and by visual with no indication. All other criteria for location, existence, size met the requirements.

10 CFR 50, Appendix B requires that the utility provide adequate confidence -- and I want to underline "adequate confidence" -- that the structure will perform satisfactorily in service. The welds that we are referring to were inspected and accepted to AWS criteria. They were then inspected and accepted to a no-tolerance type of criteria. In addition, 64 random joints or more were inspected and met the criteria with no rejections.

If we were to apply the standard, the Mil Standard 105(d), we would find that this was satisfactorily for a 95 percent confidence and reliability level for the rest of the paint.

Obviously, because the welded joints met the code criteria, the no-tolerance criteria, and an adequate random sample, or a sample randomly selected met the more strict examination requirements of MT, I believe that KG&E has demonstrated with adequate confidence that the design, construction, and code inspection requirements have been met.

It is my feeling, in summary, that the structural welds meet the requirements of AWS D-1.1.

MR. DENISE: I see that you are a member of AWS.

MR. REEDY: That is correct.

MR. DENISE: Is there anything you have said today that we ought to interpret as speaking for AWS?

MR. REEDY: There is nothing that I said that should be interpreted as me speaking for AWS, ASME, or any other society. I am speaking for myself as a consultant.

MR. DENISE: So are you interpreting AWS in any way?

MR. REEDY: I think everyone of us who reads the document interprets it. I interpret it, and I also read from documents that are said to be interpretations of requirements, yes, both the commentary of AWS and the criteria document from AISC.

MR. DENISE: When you went up to the Wolf Creek site, do you recall what welds you looked at or what buildings they were in or approximately how many there were?

I think you included that in your litany of things that you did.

MR. REEDY: I included that I looked at a number of welds. I really didn't count them. I was in the reactor building, I was in the turbine building, I believe, the auxiliary building, and I just randomly looked around, stopped, examined the welds to see what I could see, and what I saw was similar to what I have seen in other nuclear plants of similar configuration.

MR. DENISE: Did they look pretty good?

MR. REEDY: Yes, they did.

MR. DENISE: Are you a qualified inspector?

MR. REEDY: I am qualified to ANSI N-25.2.23. I am not a CWI inspector, or a certified welding inspector.

MR. DENISE: You made some comments about the code. In our application of it; that is, as it is applied and committed to by KG&E, is it sufficiently clear -- the code?

MR. REEDY: There is no code that is sufficiently clear, in my mind, and I am speaking from my experience as a code member for some 20 years.

MR. DENISE: You also said some things about practicality.

MR. REEDY: Yes.

MR. DENISE: Is the code practical, or are you concerned about the impracticality of application by what you call reassonable men and women?

MR. REEDY: Let me say for the minute that I am speaking for myself, but I am chairman of the ASME Section 3 Code Committee, and one of my biggest concerns in replying to questions that have been written in to the Code Committee has been to how unreasonable we have gotten in our application of codes and standards.

I talked to Dr. William E. Cooper, who is a well-known code authority of some stature, both literally and figuratively, and he raised the question to me the other day about how crazy we have gotten in our interpretations with regard to comparison of the as-build conditions with the design conditions, for example, and he just couldn't believe that, having been out of the Code Committee for some four years, that we had gone off our rockers and come around to what people are doing today with regard to that.

So I think it is a general concern with everyone on the committee that I know that we do have too many people who are becoming impractical in the use and application of the code, yes.

MR. DENISE: You did make a statement -- I think I recall it correctly -- that it is your opinion that KG&E did comply with AWS D-1.1?

MR. REEDY: Yes.

MR. DENISE: Is that opinion based on the things that you stated previously about the review of the records and your review of the welding and the corrective action program?

MR. REEDY: Yes, it is. In other words, everything that I reviewed gave me a data point that was similar to other situations and other places, and when I saw the no-tolerance type inspection philosophy being used and implemented and compared to what they had as deviations it seemed to me obvious that requirements really had been met to begin with.

MR. DENISE: To begin with is very important, not subsequent?

MR. REEDY: No, to begin with.

MR. DENISE: Okay, to begin with.

My last question really has to do with many of the things that you said about the activity that Region I conducted for Region IV. I interpreted those as kind remarks.

Did that add anything significant to your understanding of the confidence in being able to inspect painted welds?

MR. REEDY: It certainly added to my confidence on the question of inspection of painted welds because the number of inspections that were performed were larger than before.

In my experience in that type of situation that I had when I was with a manufacturer of heavy welded equipment, we did do some tests like that, but not to the extent that the inspectors from Region I performed down there on the site.

So it just added to my confidence.

MR. DENISE: Thank you.

MR. LIAO: I have one comment and one question, the comment being:

You said earlier that other MT method was not required. I would like you to elaborate on that.

Instead of not required, you should say that was not practical for fillet welds. Well, that would take RT, and obviously, you are not thinking about the UT, and the only other possibility is portable RT. And I think that the variation in intensity and also where you can place the film or the source, you know, would almost make the RT impossible to do it.

MR. REEDY: In general, the radiographic technique --

MR. LIAO: I am talking about fillet welds now.

MR. REEDY: The radiographic technique for fillet welds is impractical. I have never seen a good radiograph made, but I have seen radiographs being tried to be made that caused more problems than they helped.

MR. LIAO: Tried to be made.

My question really asks for your comment.

You see how the staff selected 50 welds and stripped paint, and used MT and did not find any rejectable welds in terms of Mil-105(d) standard. What does it tell me?

I have my answer, but I want to know what you think of that.

MR. REEDY: I have been working for the last two months on a statistical sampling basis type of document that we are trying to put together. In the Mil standard, I believe the number is around 58.

A random sample of 58 with no rejections would give you a 95-95 confidence on an unlimited population of welds.

MR. LIAO: Unlimited population?

MR. REEDY: Yes.

MR. LIAO: But for this case I think we talk about 1200 or 1300 welds?

MR. REEDY: Yes.

MR. LIAO: And not -- you put in that batch. As I recall, it is something like 1200 or 3000 in one batch, and you said that was 50, and that gives you -- everything comes out clean. It gives you something like not more than .25 percent welds that might be rejectable. Except I couldn't find the confidence level associated with that.

Maybe you can help me.

MR. REEDY: I am a little off my track. It is just that I have been working pretty heavily on this within the last month and a half or two months.

My understanding of a population requirement of 1200 is 3000, that something like 58 joint assemblies of the same type would give me a confidence level of 95 percent, so less than 5 percent would be rejectable.

MR. LIAO: But 105 says .25 percent.

MR. REEDY: It could. I can't answer you that because --

MR. LIAO: Let me pursue that point a little further.

When one finds the weld rejectable in my mind, or what do you think? Is it fair to say that a joint is rejectable?

MR. REEDY: I am not sure that I understand your question.

MR. LIAO: When you say the weld is rejectable by any reasonable inspection, when you say that that weld is rejectable, isn't it equivalent to saying that the joint is rejectable?

MR. REEDY: If I found a weld that was rejectable, then that joint would be rejectable, if I understand your question, yes.

MR. LIAO: Why is that?

MR. REEDY: Well, someone would have to do something to that joint. Now, I don't believe that any other joints that were in question were really originally rejectable. I say that there were some things that aren't on them.

MR. KNIGHT: Would you say someone has to do something to the joint or someone has to at least evaluate that joint?

MR. REEDY: That is correct.

MR. MYERS: The joint -- if a weld is rejectable, as John Berra says, from an inspection point of view, the joint is rejectable if a weld is rejectable.

However, the joint is not rejectable from a structural or owner acceptance point of view until we evaluate it. Once we evaluate it, it is either then classified as accessible or rejectable based on whether the allowable stresses are exceeded for the total joint.

The statistics that Mr. Brown gave you indicated that very few of the joints that had rejectable welds were in fact rejectable from an allowable stress consideration. So most probably the joint was not rejectable.

MR. LIAO: So rejectable is not really rejectable at all.

MR. MYERS: Absolutely, sir.

MR. THOMPSON: One question. You talked about some reinspection programs I guess you have been familiar with in other organizations.

How many reinspection activities of this magnitude have you been involved with or overseen or taken a look at?

MR. REEDY: I have not been involved in the oversight inspection on any of them. I have been more as a consultant on the side looking at what other people were doing and commenting on what other people were doing.

MR. THOMPSON: In that context, how many have you been involved in?

MR. REEDY: Probably about four.

MR. THOMPSON: Of this magnitude?

MR. REEDY: Well, it is hard to say magnitude.

MR. THOMPSON: Thousands of joints?

MR. REEDY: Yes, 17,000's, because one weld four inches long is a 32nd inch undersized, that type of thing.

MR. THOMPSON: And how would you compare this program to the other programs that you have seen?

MR. REEDY: The other program went in and really work 17,000 supports. That is how silly it was. This program didn't go in and rework all those.

If we are talking about the no-judgment inspection or no-judgment type of criteria, that has been the problem in the site that caused the 17,000 supports to be reexamined, yes, and at every site that I have seen where reexamining of welds was performed and undertaken to rework and redo, it was all on the basis of a no-judgment philosophy.

What I see here is no different from what I saw at the other places.

MR. MARTIN: Thank you.

MR. KEOSTER: Dr. Fisher is next.

DR. FISHER: Good afternoon, ladies and gentlemen. I'm John Fisher, professor of civil engineering at Lehigh University.

I have been experienced in dealing with joints since 1956. I have worked actively on welded joints for probably the past fifteen years. Most of the work has been devoted to the behavior under repeated loading and fracture aspect of welds.

I'm a member of the AISC Specification Committee, and have served as the Chairman of the Connections Subcommittee, more recently in the preparation of resistance factor design specification that AISC is just now offering.

In 1967 I served as secretary to the joint AWS-AISC committee that developed the data base upon which the current specifications are based insofar as the strength and resistance values for fillet welds. That committee was under the chairmanship of Dr. Amerikan, and the results of that were incorporated in the 1969 edition of the AISC specifications.

In October I was contacted by KG&E and Bechtel Power Corporation with regard to whether or not KG&E should retain myself and my colleagues, as needed, to review the significance of the discontinuities and deviations that had been observed in the inspection, as well as to examine the analysis techniques of the connection capacities and the dispositions that were being undertaken by Bechtel.

Dr. Slaughter, one of my colleagues visited the site on November 1st and 2nd to examine and photograph a number of the typical conditions. And he and I have sat down and discussed those several times.

We prepared letter reports on those which were submitted to KG&E.

Let me address first the issue of the significance of the deviations.

First of all, there is the issue of missing welds; and the significance of whether a weld is missing and has a consequence on the capacity of the joint is something that is not going to be determined by the inspection but is a process that the engineer responsible for making that assessment is only capable of doing.

So the question of missing welds depends on an analytical assessment, and in the case of a number of the joints that were in existence,-- For example, there were missing welds that had to do with end returns; there were missing welds in conjunction with the fact that beam seats were used for erection purposes that, in a number of cases, were apparently not used at all and, in other cases, were removed.

Now, generally the design of most the connections, as was pointed out by Mr. Brown, did not utilize the beam seat for the carrying capacity in the design resistance values, it was primarily there for erection sequence in making it easier to build the structure. And, hence, the fact that some of the beam seats were missing on subsequent inspections, because they were shown on the joints, has no practical significance on the capacity and behavior that would be expected of those joints.

The other factor is, there were a lot of difficulties associated with end returns. End returns were, on this job, called for in both the tension and compression side of the angles; and that's not the usual case in construction in America.

For example, there's absolutely no reason or rationale to place an end return on the compression side of those joints, and it was a complete waste of time to even be concerned with inspecting them.

So the fact that welds were missing, I believe should be looked at in the sense that some of them had some consequence, and that could be analytically assessed. Others had no consequence at all, and, in fact, probably should never have been called for for inspection, particularly on end returns on the compression side. And returns on the tension side are there to increase ductility.

If one read the original article that was published Johnson and Green-- These studies were carried out at Lehigh University in 1939 and 1940, and from those tests evolved the requirements for end returns. They did not change the capacity of joints, they merely increased the ductility.

In fact, in Johnson's article he discusses the fact that there are pros and cons as to whether one should even use end returns.

Now let us return the issue of undersized welds or oversized welds, as the case may be.

(Slide.)

In the 1967-68 study upon which the AISC's specifications were based, this is the distribution of the weld sizes that were measured in that study.

Three measurements were made on each of the welds that form that basic data base. You can see that there is substantial deviation; in fact, significantly, if one takes two standard deviations for the quarter-inch, the three-eighths and the half-inch welds, you can see that it is well in excess of a sixteenth of an inch. That is the expected -- and, in fact, was -- the deviation that existed in the basic data base upon which the specification is based.

It is, as is also apparent here, not unusual for welds on the mean to be slightly larger than the specified weld geometry that is called for; and that is quite apparent if one looks at the mean values for each of the three weld sizes documents.

(Slide.)

If we look at the non-dimensional plot of this distribution curve, we see that the variability that I have cited -- it is apparent that the smaller the weld there generally tends to be a greater degree of oversize, but the degree of underrun, or undersize welds, as well as the oversize of the weld, is about the same for all welds in terms of the percentage of the basic leg size. That is the expected deviation that is inherent in the design values that are in the specification today, that are in the AISC specifications.

In the lower plot -- this is a study that was carried out in 1964 by Caterpillar Tractor. Caldwell, at the time, was the assistant chief engineer. And this was compared with the 1967-68 study by Omar Blodgett. And you can see that comparable deviations existed. And, hence, one should not expect production welds at any site to be significantly different than this.

This is the type of deviation that is inherent in the specification and the criteria that have been developed for it.

Now let me move on to the issue of overlength end returns, or underlength, [illegible] the case may be.

Overlength end returns basically produce more restraint. The basic end return is to provide for protection for the root of the weld.

If you look at Johnson's original study you will find that the beam capacity, the end shear capacity, did not differ whether there were end returns or not. And he pointed out that the primary effect of this was to provide greater rotational capacity.

Hence, we provide end returns because it shields the upper end of the weld which is subjected to the highest tensile stress; it shields it and provides more ductility.

Now, that end weld, the longer it is, will start to crack during the load deformation process. The fact that it cracks is not going to deteriorate the shear capacity of the beam. That has been demonstrated in every test that has ever been carried out on end welds.

So the main effect of end welds, or the end returns is the fact that if it's excessive it will tend to stiffen the joint, and provides more end rotation restraints.

We obviously have moved today to eliminating angle type connections on many of the steel structures to the simple end plate connection that has substantially less ductility insofar as the distortion of the outstanding leg of angles.

Hence, the issue here of restraint at the end in the type of structure that we're dealing with is a trivial one because if anything unloads the beam, provides end restraint, it has no significant impact.

This is not the case in a cyclically loaded structure. If we have significant end restraint we get deep cracking, which we do experience in bridges and other types of components that are subjected to repeated loads.

Now, cracks are obviously a problem. And the only cracks that were observed in the inspections were those cracks that were associated by the welded joint that was placed between the beam seat -- or the beam and the beam seat. This was an expected crack, in retrospect, because the end rotation would subject that weld which tends to be undersized because the end of a rolled section is rounded and one places a weld along that, and it will tend to be smaller than desired.

Hence, when the beam end rotation occurs the weld beam is subjected to transverse shear and has less ductility than one subjected to longitudinal shear forces, and, hence, cracking occurs.

Those welds should never have been placed there to begin with. You had an end seat and you had web angle connections. It's absolutely irrational to place welds along a beam seat when you have held it in place with either a clip angle or a end reactioning.

So those are a trivial condition having nothing to do with the resistance of the structural components and the connections that were placed there.

Undercut has also been discussed. And most of the undercut was down the edges of angles, or associated with angles. Undercut is a problem where we have repeated loads such as in bridge and elsewhere. And the criteria that's in AWS specification is primarily there because of repeated load applications.

In the case where stresses are developed in the primary component that will be normal to the undercut, that is a problem.

When you have an undercut along the edge of an angle it's an insignificant factor, it has nothing to do with the performance of the structural component because it will never be subjected to any significant tensile perpendicular to it. And hence it acts-- It doesn't even act as a notch because the forces that are being placed upon it are not ones to which it would be sensitive.

There has been raised the issue of lack of fusion. There was no evidence of any significant lack of fusion that we could see from any of the inspection reports.

The porosity. Porosity has to be gross to have any significance on a statically loaded structure. We carried out tests on some embedments that were removed from the Hope Creek plant and found that 25 to 50 percent porosity had an insignificant effect upon the shear capacity of the welded joints.

When you get to porosities of that size, that is easily visible through paint, as was the case in the Hope Creek plant.

Now, the beam seats I have already addressed, the fact that that can be rationally addressed.

(Slide.)

We looked at the distribution of the size of welds that form the basis for the specification. Now let's look at the resistance.

If you look at the shear capacity of that same data base where we have plotted here the frequency of occurrence versus the fillet weld shear capacity, non-dimensionalized by the electrode capacity, you will note that the mean value is .84. That means that the design value that we use today, which is 30 percent of the tensile strength, has a factor of safety that is nearly 3 against the mean.

Now, this is for longitudinal welds. That is, in a fillet weld, the force that you subject it to has a significance upon its failure mode and resistance value. The longitudinal force that we're talking about here is a lower bound resistance provided by fillet welds.

(Slide.)

Let's look next at the basic data as it is plotted for individual tests.

This is the longitudinal weld. It was a test series that covered 836, 8441 and 8514, which are quenched and tempered steels. These have been non-dimensionalized. The effects of dilution of the electrode are apparent here.

So you can see that when based upon the scatter that has been plotted, as again reflected here by the variability of each of these different samples that were provided.

MR. THOMPSON: Let me ask one question.

You talk about the sample, and you showed us, I guess, the standard deviation from the welds. That would account for part of that?

MR. FISHER: That is correct.

This is just showing us individual data as was acquired in the 1967-68 study.

What we have done, then, is taken the 133 samples associated -- that was for E-70 electrodes. This includes 110 E-70 and E-60 electrodes. We only considered the E-70 electrode, as shown in the previous slide.

Then we see that is the statistical distribution of the samples.

There have been subsequent studies that have been carried out to demonstrate that this distribution really does not change with time.

Now, as I pointed out, this is the design allowable shear value. This was adopted in 1969. Now, at the time there was also an arbitrary limit state which was not relevant, which never should have been imposed. It was .4 the yield point on the leg.

That was done because in the AISC specification the shear was permitted on the web of a girder to be .4 the yield point. Someone thought that since this was the shear on the base metal that same limit state should have been imposed. We now realize that that was an absolute mistake, that it has no relevance to the capacity. Because not a single weld ever failed on the leg.

In fact, the only way we can make a weld fail upon the leg is to have excessive convexity, which is not unusual today, and therefore the weld will not shear through the throat but will shear through the leg.

Aside from that, most weld failures will be through the throat.

Because of that, then, the leg limit state that is in the current AISC specification is not correct, and we are in the process of changing them. That has already appeared in the load resistance factor design, and we met this past November and are taking steps to change it in the other allowable stress design provisions.

(Slide.)

If we look, though, at the weld that is subjected to transverse loading, we see that the resistance is significantly increased. In other words, the orientation of the weld has an impact upon the resistance. And in the American specifications we have elected to take the lower bound associated with the longitudinal weld. So that the .3 that has been used by Bechtel is a very conservative application.

As I move into the analysis aspect of what they have done, one should bear in mind that they are applying allowable stress which is based essentially on longitudinal welds, and that because most of these welds have more than one orientation -- we're talking about a C-shaped weld or a vertical weld -- hence the load is not always applied longitudinally. In fact, when we get to bending, which the vertical welds on the outstanding leg are, they are subjected to two force vectors, one vertically to take the shear, and the other from the moment. And those are vectorially added; which is an extremely conservative application.

If one just looked at the difference between the 7th edition of the AISC manual, which is essentially the technique that was used by Bechtel, and compared it to the 8th edition which was published in '82, you would find that they could have increased the capacity of all the welds by 8 to 25 percent, depending upon which of the welds was critical. Because in the current manual we are making use of the capacity under different angles, because there are obviously mathematical models: we have gone from one extreme of simplification, and now that we have mathematical tools, computers that are available, we can better estimate the resistance values.

(Slide.)

Longitudinal welds obviously have a great deal of ductility. This is a photograph from one of the specifications that we used in '67 and '68.

You can see from the distortion between the saw cut which was used to limit the weld length, that there was substantial distortion capability.

One of the reasons, in the American code that we -- when we do combine longitudinal and transverse welds, we must consider the compatibility conditions. And we have gone to the lower bound because under most conditions, unless one does an elaborate analysis and takes into account the distortion, we have felt that the use of the longitudinal weld picture here is the one that is conservative and most appropriate.

(Slide.)

Now, in their analysis Bechtel has used, as I have indicated, an allowable stress design approach which was in the 7th edition of the AISC manual. In the 7th edition, the direct vector addition, which is what I show here by Q, the square root of the sum of the squares of the end original force vectors, that is extremely conservative, as you can see.

So if we had a single vector on a weld, we would be-- This is stress. The stress that you'd be comparing this with would be 21, because it's non-dimensionalized in terms of the tensile strength of the weld metal. This happens to be a study on E-60 electrodes.

You will note that the factor of safety increases substantially when we introduce bending because of the direct vector addition. It was in recognition of this that AISC changed the criteria that is in the 8th edition of the manual.

We have known for years that when we subject welds to combined load vectors that we have factors of safety that are in the order of 4 or more. And hence, we have tried to reduce that degree of factor of safety.

Now, welds have had a factor of safety applied on the basic limit state. That is a little over 2. And that's because they are subjected to some of the vagaries that I have discussed here this afternoon.

It is inherently known and understood that the kind of deviations that were in existence at the Wolf Creek plant are normal to all structural steel fabrication. We cannot build a structural steel component as though it were a Swiss watch, and we should not have imposed the kind of quality control that they did on their subsequent inspection. I think that was a basic mistake, to try to examine each inch of these welds as though they were critical. And that would be contrary to the intent of the specifications.

So inherent in the specifications, in the sizing and the variability that's going to be associated with the size of the weldment, and there is inherently built into, in recognition of that variability, the reason we have used the lower bound. The design shear value recognizes that that variability is a reality.

Now turning, finally, to the question of inspection: I have a lot of experience with inspection through paint because I do a lot of my work on bridges, and on bridges most of the inspections that we carry out are, in fact, on painted structures. In those locations, actually

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paint we find in the presence of cracks. If a structure is going to be subjected to load is a good indicator, we find more cracks in bridges because of the paint film cracking than in any other type of inspection.

Magnetic particle has been demonstrated to be an adequate method to apply to discover discontinuities that are subsurface or not propagated through the paint film. But it is seldom that we have to rely upon that.

I think there's no question that the types of discontinuities that one would have expected here, visual inspection of the paint would have revealed a significant number of them. In fact, I think when one looks at the fact that there were a significant number of these joints painted, you should always put that in context of the number that were not painted that were inspected, that have not demonstrated anything of significance.

Most of the deviations that we have looked at I would consider to be trivial and should never have been recorded as major deviations to begin with. I think that was a mistake. This was contrary to the intent of the specifications.

I believe that that sample should easily be applied to the samples that were painted. And the fact is that you've gone beyond that. I think that you have inspected -- or they have inspected a much greater sample than is rationally called for.

A statistical sample could have been selected, as is the case when we build a structure from scratch. And if that is violated then we go further steps to see if there is justification for exploring that.

That's all I would have to say, gentlemen.

MR. LIAO: Dr. Fisher, a couple of times you made quite a few strong statements like 'You should not impose watchmaking accuracy to the structural welding; it should not be required in the first place.'

I'm looking through your resume and I notice that you have been consulting with other industries quite often. And my question to you is:

What would you think if you were to take ten inspectors, bridge inspectors, for D-1.1 type welding to a nuclear power plant? What would be your judgment on what kind of rejection rate those guys might call? Better or worse than --

MR. FISHER: Well, bridge has different requirements. If we're dealing with a power plant the structure we're talking about is not subjected to repeated loading.

I think we must differentiate structures when we talk about inspection on those subjected to repeated loads where there is a fatigue crack propogation problem or initiation and those that are subjected to essentially static behavior. And I would put earthquake response in the static behavior.

MR. LIAO: Okay. So it's equivalent?

MR. FISHER: I would say that from the -- where I have looked at some welds in the field -- and I did not go into the field to look at the welds here because I didn't have time; I sent one of my colleagues. But I have looked at welds in other power plants such as Limerick and elsewhere. And in general I find that the quality of the work is what I would consider to be comparable to other building type structures.

First of all, you make use of drew welds where you don't for the most part remove the reinforcement. In bridges we would not do that because we have a fatigue potential problem. The fillet welds are for attachments, and I would say the use of a fillet weld and the welds that I have seen in the field are not unlike those that I would see in a bridge.

Now you must understand, though, that in fillet welds we don't use simple end connections that are fillet welded because that is not a weld that will function under repeated loads; it will crack. And we've had experiences with that in the field. So our field end connections on components tend to be mechanically fastened or bolted.

MR. LIAO: Okay. Let me rephrase my question then.

D-1.1 inspectors, weld inspectors in industries other than the nuclear industry, if you take some --

MR. FISHER: I can give you an opinion here.

MR. LIAO: Yes. That's all I'm asking; opinion or stipulation --

MR. FISHER: I personally think that the

inspectors associated with power plants have been instructed incorrectly.

I think there has been a failure to instruct them on what are the probable tolerable deviations. And I do not think there has been enough focus put on that. And that's why I say when you look at someone who is putting a weld gauge down the entire length of every inch of a weld, something is wrong, because that is not the intent; it never was the intent of the specification, whether it be AWS or AISC.

And so I think you must go back to when the specifications were derived. Those gauges didn't even exist, as Mr. Reedy has pointed out. And you effectively have been digging yourself into a hole by imposing things that were in my opinion unnecessary.

MR. LIAO: Okay. I have another question.

On one of your charts you show an additional factor of safety when you have combined axial and bending loads.

Would you care to say in most situations in the real structures you hardly ever see the joints. That is designed for strictly one type of loading only?

MR. FISHER: Yes. In sheer splices.

If you have a direct -- For example, if your wind bracing is framing into a column, and you put a sheer splice, that's a single vector. There is not significant bending being introduced.

MR. LIAO: How about rigid frames?

MR. FISHER: Of course in rigid frames it depends on the particular weld and what kind of connection.

If you have a beam column connection you're going to have either groove welds on the flanges and a connection plate that may be -- and it is not unusual to have a bolted connection plate on the web today because you could then field-erect it with the bolts and put the groove bolts on. So you would have -- Well, it would be a pseudo-combination joint. So I think you could have a variety of these types.

But for the type of weld that we're talking about in the force vector that I showed you in that last slide where you have either a simple, a very simple weld -- This is the simplest one that you could go to, where you have an eccentricity and you have -- those welds are then subjected to two force vectors, the direct sheer and the bending component.

Now once you start imposing bending the capacity -- because you are using elastically you extrapolate to the extreme fiber. Only one little component is what controls the weld. THere is no account made of the redistribution. So in the procedure that Bechtel used, that's what they assumed.

If you look at the AISC manual today, the Red Book 82, you will find that they have used an ultimate strength approach for a number of welds. And those are now in the manual, whether they be the vertical weld group, the box type of connection or the C-shaped welds that you would have around angles. And they have applied a factor of safety of 3.33 to that model resistence. There is -- in other words, to account for other uncertainties, as I've already pointed out.

We expect the kind of deviations in the weld that we are seeing at Wolf Creek. And that is inherently built into the limit statements.

MR. LIAO: So are you saying that most of the joints we are talking about today in Wolf Creek are basically rigid frame joints; that you don't see much of the single vector type of joint?

MR. FISHER: Yes -- Well, they are not rigid frames. I would say they are C-connections because they've used angled beams.

MR. LIAO: Okay.

MR. FISHER: That theoretically is a zero-moment connection, okay? The designer assumed there was zero end-moment. But that's not to say that's what it is.

MR. LIAO: I see. Okay.

My last question: I understand you used to be a consultant to the lawyers in the case of the Kansas City Regency Hotel where it collapsed.

MR. FISHER: Not to the lawyers; to an insurance company.

MR. LIAO: Oh, I see. So you are on the defendants' side.

MR. FISHER: That's right.

MR. LIAO: All right.

MR. DENISE: Just for the record, you are a member of the AISC?

MR. FISHER: Yes, sir, I am.

MR. DENISE: But you're not speaking for them officially here today?

MR. FISHER: No, sir.

MR. DENISE: Nothing that you're saying is an interpretation of the code except your personal comments on what's already been interpreted?

MR. FISHER: Well, I would only say this: That since I served as chairman of the committee I am telling you what essentially I told the committee when they adopted the provisions that are there now.

MR. DENISE: Which KG&E argument are you supporting?

MR. FISHER: I was asked to look at the

significance of the discontinuities and the analytical technique used by Bechtel in assessing those, and then the significance of inspecting deviations through paint. Those are the three issues they asked me to look at.

MR. DENISE: Okay.

Since I'm a simple-minded engineer, let me try it this way:

Bechtel has dispositioned -- used as-is -- many of the drawings in which there were deviations identified in the reinspection program of KG&E. Are you agreeing that those dispositions specifically or generally are conservative?

MR. FISHER: That's correct. That's what I tried to get across.

If I had been pushed to the wall to say -- I would probably have used a more liberal analysis because I think it's justified. It's what we would use today. And maybe some that they have said had to be repaired I would have said were all right.

I think they have used an extremely conservative disposition procedure.

MR. DENISE: Okay. That's Bechtel's structural analysis.

I was just curious, on my last question. You seem to be very concerned about the degree of inspection that KG&E implemented in their corrective action program. Is your main message that they just went overboard more than is necessary; not that they did a poor job but they went overboard?

MR. FISHER: That's correct. I think -- I do not think from the discontinuities and deviations that I see reported that there was justification to go to the degree of inspection that was done.

In my opinion it would have been logical to look at this from a more rational basis and either select a statistical sample of some substantially reduced size rather than trying to inspect 100 percent. Then if that statistical sample revealed something, take it the next step and increase the sample.

But that's what exactly we do on bridges, and I'm sure that's what you do in many parts of -- in other parts of the power plant that may be associated with the ASME code. I can't believe that you do things that much differently.

MR. DENISE: But the thrust of that kind of comment is that, I think, that they were extremely conservative and probably could have justified much less.

MR. FISHER: That's correct. That's the point I tried to make.

MR. DENISE: Thank you.

MR. FISHER: Maybe inadequately.

MR. THOMPSON: So that I understand what you're telling us, as I understand there were 82 joints that required rework because the allowable stresses were exceeded in the as-built condition, and that there were 67 additional joints reworked to install missing and under-linked welds unless prohibited by field conditions. And you're saying you agree that at least all of that 82 and 67 were done conservatively.

And you're saying but you don't believe that they needed to repair all 82 and 67 of those welds?

MR. FISHER: That is correct. I would have to look at all 82 of those numbers.

But from what I have just indicated, that the capacities are going to be up to 25 percent based on the current code. If you were designing that plant today the analysis they have done could be liberalized for many of those welds. 25 percent depends on which weld we're talking about, whether it is the C-shaped weld on the web or the outstanding leg weld.

And I would have not even considered as being rational to look at this end return on the compression side. That should have been wiped off; it should not even have been considered.

MR. THOMPSON: Now for the 60-some-odd polar crane stops that were missing welds, or the six pressurizer supports that were missing welds, you think those there -- that repairing those welds was the appropriate thing to do?

MR. FISHER: That's correct.

MR. DENISE: I have one more question.

You spoke to the cracking of overrun on the return.

MR. FISHER: Yes.

MR. DENISE: Is the thrust of your statement there was that you agree with Bechtel that the overrun would crack and relieve, and that crack would therefore transfer the load to the -- .

MR. FISHER: That's correct. That's what would happen even if it were a standard overrun.

If you subjected the beam end to sufficient rotation and distort the angle you will crack that weld.

And if you go back to Bruce Johnson's original paper in 1940 -- it was published in the -- I have a copy right here -- it was in October 1940 -- you would find that in Bruce's study that he had observed the same thing in the laboratory, that that has to be the failure mode. And you delay, you increase the rotational capacity.

So end returns are primarily put on there for rotational capacity, not strength, because the mode of failure is to crack that weld and then eventually you either

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fracture the plate, it tears the plate in a tearing mode, or you will sheer the weld off.

MR. DENISE: I take it when you get to that point you're way past any design conditions.

MR. FISHER: Oh, you're at the limit state. We're not talking about design distortions. In design distortions you're not going to see anything of any consequence.

MR. DENISE: I wasn't carrying it that far; I was merely trying to deal with one point made by a Bechtel analyst that said the overrun on the return could be dispositioned because it would crack under stress without affecting the strength of the weld.

MR. FISHER: Well, I would qualify it to say it may crack it.

MR. DENISE: If it did anything it would crack and relieve.

MR. FISHER: That's correct. That's what happened at the beam seat. You see the rotation there cracked the weld.

MR. DENISE: But that wasn't the return.

MR. FISHER: No.

MR. KOESTER: Dr. Egan.

MR. EGAN: My name is Jeffrey Egan. I'm the president and technical director of APTEC Engineering

Services. And I believe I am the last technical speaker in today's presentation.

There are certain advantages to being last. The first of those is that you get to have the last word. The disadvantages are, of course, that everybody has told you everything that I'm going to tell you. And the second disadvantage is that I've already missed my plane to California.

(Laughter.)

Just for the record, I am a member of ASME. I am a member of the American Welding Society and the American Society for Non-Destructive Testing. I am a member of the International Institute of Welding, I am a member of the Pressure Vessel Research Committee of the Welding Research Council, and of their committee on the Significance of Defects.

I have been vice chairman of the Materials and Fabrication Committee of the Pressure Vessels and Piping Division of ASME.

As you may have guessed, I am also a member of the British Welding Institute where I spent seven years as a research engineer studying the significance of weld imperfections. I am also a member of the Institution of Mechanical Engineers.

In regard to previous questions on who it is I represent, I represent the welds.

(Laughter.)

May I have the first slide, please.

(Slide.)

I was asked by Kansas Gas & Electric to do an independent evaluation of their approach to the resolution of CAR-19, the object of today's discussion, and to make some recommendations for a timely close-out of CAR-19, about which I have some emotional feelings.

To do that there were certain activities that I undertook which are illustrated on the next slide.

(Slide.)

Basically I reviewed the final report that was put together by KG&E. And I want you all to see this and to hold it and to weigh it. This is the final report. In addition to that, in the back of this there is a list of some 47 supporting documents. I speak for the welds.

I undertook a site visit, and at that site visit I reviewed most of the supporting documents that I thought were necessary for me to come to conclusions with regard to the quality of the welds in this particular issue. I reviewed the weld procedures, filler metal specs, inspection criteria that was used by Daniel throughout the period of this construction, and I also looked at the validation of the reinspection and particularly of the elements of the structure that were painted.

Incidentally, we have done nine of these things, and that is probably nine too many. That is, a re-verification weld exercise.

I also examined some of the welds in the auxiliary and reactor buildings. I interviewed KG&E, Daniel and Bechtel personnel, and I prepared a report which I believe you all have a copy of which has some focus questions related to the impact of this CAR-19.

(Slide.)

This basically summarizes the results of my review of all those documents and of the welding at the Wolf Creek Generating Station.

The first point is that the related welding activities -- other than the records-retention problem that we heard about -- are sound and well documented. And my conclusion from that is the welding is not out of control, as we have seen in other situations where it is pretty obvious from a review of welding activities that are related that something has gone wrong not only with the documentation but with the welding.

So our first conclusion is that the welding is not out of control, and that ought to be an indication that we are going in the right direction.

The reinspection program has been extensive, properly performed and documented. And if you go back in the history of this thing and look at actually what happened you'll find that there was, where the records were collected together, something like 70 percent of the original weld records available.

And a lot of the work we do in the disposition of non-conforming situations you do not have the luxury of such a huge sample. Any statistician at APTEC that I gave a 70 percent sample to would jump in glee for that number of records of the original inspection.

The validation of inspection with paint has been completed and in my opinion is an entirely appropriate thing to do in this particular example. Remember, something like 40 percent of these reinspections were done without paint on the structure.

Again, 40 percent is a very generous sample for anybody that's going to take those numbers and manipulate them statistically.

We have heard at length I think from Bechtel and others that the imperfections that were noted in the reinspection are typical for carbon manganese structural steel welding, and we have seen nothing that would cause us anxiety or concern with regard to the structural significance. And again, speaking for the welds, we find no safety significance of the imperfections.

I have used the word "imperfections" purposely in this slide, and have heard words this afternoon that relate defect, discontinuity, all sorts of other words. Let me just tell you what the AWS structure welding code defines a defect as.

Let me first define a discontinuity.

An interruption of the typical structure of a weldment such as a lack of homogeneity in the mechanical or metallurgical or physical characteristics of material or weldment. A discontinuity is not necessarily a defect. And I think therein lies one of our problems. You find all these things which are generally brought about by a more intense scrutiny of the welds, and we start classifying them as defects.

The definition of a defect, according to AWS, is a discontinuity or discontinuities which by nature of accumulated effect render a part or product unable to meet minimum applicable acceptance standards or specifications. And these under this code are set by the owner or his agent, the architect-engineer.

This term designates rejectability. In my review of this program that has been undertaken at Wolf Creek there are no welds that fall into that category. The architect-engineer has analyzed the situation and determined that what we're talking about are discontinuities under the terms of AWS.

Within the International Institute of Welding we have a committee -- it's Commission Five -- which deals specifically with nomenclature. It's one of those things that everybody wants to attend. They use the word "imperfections" instead of "discontinuity."

(Slide.)

Based on my review of all the available information, my knowledge of the code, and so on, I would conclude that the reinspection program is sound and effective and ensures that you have got quality when you want D-1.1 quality welds in the structure.

As far as the important aspect of structural integrity is concerned, it is assured because we have seen the analysis. I have reviewed the analysis that Bechtel has done, and in my opinion, it is in accord with Dr. Fischer's. I would concur that the analysis is conservative.

We will perform, I suspect, a far more racy analysis than the ones that Bechtel have done.

Let me just conclude with a couple of simple comments. You have basically heard what happened, and you have heard what caused it. You also have heard this morning how it has been corrected and how we can prevent it happening again. And as I said earlier, if I could presume to speak for the welds, they are healthy.

Let me just tell you something that has been written down in the foreword for the commentary on our AWS Structural Welding Code. 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.

In this case on CAR-19, that is exactly what has been done.

Thank you.

MR. MARTIN: I have one question perhaps.

I am inferring from some of your opening remarks that had the problem been yours to address in its totality when it originated, had you come upon inspection records which represented 70 percent of the weld population, that on that basis alone you could have -- I inferred from what you were saying that you could have gone forward and statistically ascertained that you probably didn't have to do anything beyond the statistical analysis.

Did you mean to infer that?

MR. EGAN: That is a correct inference from what I said.

I believe, however, there are good reasons for the program that was adopted by Kansas Gas & Electric. They are close to the license application. The development of a statistical analysis requires not only that it is done properly but those that will review it understand exactly what the implications of it are.

In the programs that we have developed, both with the nuclear industry and in other industries that use structural welding, to disposition similar situations using statistics, we have spent a lot of time explaining to people our approach and how it happens.

In many cases you could use up more time doing that than to get on with the damn job and either inspect it or replace it. So I think that was their decision.

But your inference is correct.

MR. MARTIN: And I am not trying to infer from that that represented a criticism of the decision of KG&E.

MR. EGAN: No. I have hindsight, of course. I have this 20/20 hindsight.

MR. MARTIN: We are both blessed with 20/20 hindsight.

(Laughter.)

MR. EGAN: What I am saying to you is everything is baked, and all I can say now about the 70 percent sample, why didn't I use it?

MR. MARTIN: Given that there is not a fallacy in that argument, in light of those joints, albeit small in number, for which there were not welds that should have been there, would you still have been able to make the same inference justifiably, given that the later inspection did in fact ascertain that welds were not where they should have been?

MR. EGAN: Yes, I believe you could do that. I referred to at least an approach in the letter report that I prepared for Kansas Gas & Electric.

The real question is -- and I think somebody asked it earlier -- the whole of the approach that Kansas Gas & Electric has developed is based on the joint basis. If you look at it on the weld basis, as Mr. Berra explained, maybe we have got six welds per connection or joint.

The real question is what is the likelihood -- and I use "likelihood" synonymously with "probability" because it is easier for us to understand what the likelihood of one weld having discontinuities and being calculated as being defective, and then what is the likelihood of two welds in this connection, and what is the likelihood of three welds in this connection, and what is the likelihood of four welds in this connection having discontinuities which subsequently when we analyzed to be demonstrated to be defective.

That is a relatively simple calculation. I did it in the letter. It turns out that if you have a connection with six welds the likelihood of four of those being -- have discontinuities and being defective at a 3 percent reject rate is greater than 1 by 10 to the minus 6, which is the standard, which we will now accept for the integrity of the reactor pressure vessel from WASH-1318 and the Rasmussen study and subsequent reports. 1 by 10 to the minus 6 is a number that is agreed with in the industry for the disruptive failure of a nuclear pressure vessel.

What we are saying is that that is the likelihood of having four out of six of these joints defective.

I think the follow-on from that is that maybe we are expecting an unreasonable standard in the particular activities that went on.

MR. MARTIN: Thank you.

MR. DENISE: In your last statement, I am trying to understand the timeframe in which you make those calculations. It seems to me -- and you can tell me if this is true -- it seems to me that you reach that conclusion on probability after seeing the reinspection program, is that true?

MR. EGAN: After I have done the review I make a full ledger report, that is correct, yes.

MR. DENISE: Now, after the reinspection programs, what numbers would you have calculated?

MR. EGAN: I would have used 2 percent as a typical reject rate for the reinspection of already accepted structural steel welds.

We have been involved in several reinspection programs in both this industry and in the hydroelectric power industry for tunnel liners, and so on, and in the reinspection of those types of steel, structural steels, with AWS D-1.1 welds, even when the steel has been signed off you bring in a new team of inspectors, you get a reject rate on the reinspection of about 2 percent, and there are good reasons for that. The guys you bring in know you are reinspecting it. So they have also got some hindsight to wondering why they are doing it, so they had better do a good job.

There are military programs that have been conducted on just that thing. What is a typical reject rate for reinspection of an already inspected part? And because of human error they run about 2 percent. So I would have used 2 percent, and I would have got similar numbers.

MR. DENISE: But you would have assumed that 2 percent as representative of an industry, loosely used industry standard and expectation?

MR. EGAN: That is correct.

MR. DENISE: And therefore, your calculations based on the industry standard, if I understand what you did, would have been at a lower probability than you calculated based on the 3 percent reject rate?

MR. EGAN: It is still in the same order of magnitude, 10 to the minus 6. You know, you can go from about -- we did some Class 1 piping recently at 1.5 percent, and if you look at 1.5 through to 3, it is still 10 to the minus 6 by the time you look at four out of six.

MR. DENISE: Okay. Did you personally inspect any welds?

MR. EGAN: Yes, I did.

MR. DENISE: Are you a qualified inspector?

MR. EGAN: I have been a CSWI, which is a certification scheme for weld inspection personnel of the British Welding Institute. I am currently not a qualified weld inspector.

MR. DENISE: But you are not speaking for AWS or AISC today?

MR. EGAN: I am speaking for myself and the welds. In fact, the welds are wondering why they are involved.

(Laughter.)

MR. THOMPSON: Mr. Koester.

MR. KOESTER: I would like to take this opportunity to thank the three independent consultants, and I am sorry you missed your airplane to California, Dr. Egan, but I missed mine to Wichita, Kansas, too.

(Laughter.)

And it is harder to get to Wichita, Kansas than it is to California.

I would also like to thank the members of my staff who have made presentations here today, and we are still available to answer any questions you folks might have.

But KG&E has always had and continues to have a very firm commitment to protect the health and safety of the public as well as our own employees. That is why we undertook such an extensive program to evaluate the acceptability of the structural steel welding at Wolf Creek.

As you have heard earlier, our reinspection efforts found several minor deviations that gave the appearance of a higher than expected reject rate. However, the primary reason for these rejects resulted from the no-tolerance inspection philosophy discussed by Mr. Reedy. The vast majority of these deviations would not be rejected by a qualified AWS inspector at another facility unless they were making the same type secondary inspection that we made.

The fact that KG&E took a more conservative approach during the reinspection effort does not in any way invalidate the initial weld inspection.

As discussed earlier, the reinspections did identify a few joints in which some welds had not been made. These primarily resulted from a misinterpretation of the weld detail and not from gross inadequacies in the inspection program.

While we strive for perfection, we must all recognize that human errors can and do occur. That is one of the reasons why we do design and build these plants with so much conservatism.

This is demonstrated by the fact that none of the joints with missing welds would have failed. A point that needs to be emphasized is that we mean it would not have failed under the worst postulated loading conditions. This would include normal loading plus any loads resulting from a postulated worst case accident.

Our primary objective in the overall corrective action program discussed earlier was to assure that Wolf Creek is structurally sound and will not fail under the worst postulated accident conditions.

We have done that. In doing so, we also reaffirmed that the AWS welding was done in accordance with the applicable codes, and we did not limit our review of this matter to welding alone. We also looked at other areas to assure that they were completed in accordance with applicable requirements and in a manner that provides adequate protection of the health and safety of the public.

We also had three of the leading authorities on structural steel welding independently review our program to assure that we were not taking a biased look at ourselves.

As you have heard from their discussions today, from their reviews of the various aspects of our programs, we did a very thorough, conservative assessment of our AWS welding program, and they found nothing to question or invalidate the conclusions that we have made.

I sincerely believe that anyone knowledgeable in engineering and construction practices would have to agree that KG&E's corrective action program verified that the structural steel at Wolf Creek generating station is safe and sound.

This completes our presentation on AWS structural steel welding at Wolf Creek. We firmly believe that the record is clear, and we are ready to receive our operating license, commence loading fuel, and proceed through power ascension.

Thank you very much, and we are available for any other questions.

MR. THOMPSON: Thank you for that presentation.

I think I would like to turn now to make sure that if the staff has any questions of KG&E now concerning this reinspection program that we identify while we have all the people here who can answer any of the questions or at least identify any particular area of concern that we still have.

Has everybody asked all the questions they have?

We do have some members of the public here. I would like to know if any member of the public would like to make any comments at this time.

Identify yourself.

MR. SMITH: I would like to ask Mr. Martin if he has seen CAR-19.

MR. MARTIN: I am quite sure that I have, yes.

MR. SMITH: And when can we expect to see that in the public document room in light of the extensive discussion on the document?

MR. MARTIN: I expect --

MR. O'CONNELL: It is in the public document room. CAR-19 is part of the December 31 letter that is in the public document room as an attachment.

MR. THOMPSON: Are you sure that it is in the public document room because you have seen it there or because you know the system would have normally put it there?

MR. O'CONNELL: I called the local public document room branch yesterday. They verified that it was sent to Emporia Local Public Document Room by their contractor.

MR. THOMPSON: It is publicly available if that is the question. We can work with you after this meeting if you have problems getting that particular document.

Does anyone else have any particular questions?

MS. STEPHENS: I have some comments.

MR. THOMPSON: I guess what we are looking for is comments rather than questions. If you have some particular comment about the program, I guess I would prefer it that way, and then if it is questions let's take a look at those.

Do you have any general comments first?

MS. STEPHENS: No.

MR. THOMPSON: Do you have a question?

MS. STEPHENS: I have two questions.

I am Ms. Stephens, with the Nuclear Awareness Network.

I would like to know -- Mr. Berra, excuse me if any of this is redundant. We had problems hearing in the back of the room earlier -- what percentage of the MSSWs were in triplicate form.

MR. KOESTER: I believe Mr. Berra said that he did not know the answer to that when he was making his presentation this morning when Mr. Denise asked the same question.

MR. THOMPSON: It is a very, very small percentage, and I don't believe he had a number at all.

MS. STEPHENS: We couldn't hear it.

MR. THOMPSON: I would say it is very small.

MS. STEPHENS: Of the percentage of those that were in triplicate of the missing documentations, were all three documents missing?

MR. MARTIN: We did pursue that. It may be that you did not hear. Mr. Denise did pursue that, and Mr. Berra does not know the answer.

In any event, the number of forms in triplicate were a very small fraction of the total. But in any event, he didn't know the numbers. We did ask those similar questions.

MS. STEPHENS: Okay. I couldn't hear back there.

MS. VARRICCINO: On that same topic I would like to ask what the procedure was at the Calloway plant as to the duplication of documents or keeping them in a controlled environment.

MR. THOMPSON: You can ask us that question, I guess, the staff, later on. I don't know the answer to that, but I will be more than happy to find out.

MS. VARRICCINO: Would Mr. Berra know that?

MR. THOMPSON: I don't know whether he would know that or not.

MS. VARRICCINO: He is the only one not shaking his head.

MR. BERRA: I don't know the answer to that.

MS. STEPHENS: I would like to know when the first NCR was generated on MSSW. In other words, was it simultaneous with CAR-19?

MR. THOMPSON: Is that the one with the structural welds, with the structural supports?

MS. STEPHENS: I am asking, in essence, was there an NCR generated around the same timeframe as CAR-19?

MR. MYERS: Is your question related to the NCRs written because the MSSWs were missing, or is your question related to the fact that once an inspection is done and there is an imperfection found an NCR is written? What is your question? Which of those two?

MR. THOMPSON: You mean the answer is different?

MS. STEPHENS: In normal procedure, does a CAR generate an NCR, or vice versa?

MR. MYERS: If the CAR involves inspection of equipment and the inspection results --

MS. STEPHENS: By equipment, you mean --

MR. MYERS: Anything. If the CAR includes inspection of hardware and the inspection indicates that the hardware has an imperfection, the system requires an NCR. A CAR does not necessarily have to involve inspection.

MS. STEPHENS: But in this instance, did it involve inspection, in CAR-19 in March of '83?

MR. MYERS: I am sorry?

MS. STEPHENS: But CAR-19 in March of '83 did involve inspection?

MR. REEDY: You might be confusing terminology. CAR-19, KG&E's CAR-19 was initiated in October of 1984.

MS. STEPHENS: October of 1984?

MR. REEDY: KG&E's corrective action request associated --

MS. STEPHENS: Okay. Then the initial one was DIC CAR-29 and -30, is that right?

MR. REEDY: 31 for documentation.

MS. STEPHENS: Okay. 31. Now I have that straight.

Was there an NCR initiated or generated as a result of that CAR?

MR. REEDY: Yes. If you are asking the question is Daniel CAR-31, NCRs were generated, yes, and the time period was August of '83.

MS. STEPHENS: So they weren't generated until August of '83. They were not generated in March, when the initial CAR came out, right? Isn't that right? Weren't they in March of '83?

MR. THOMPSON: Do you want time to caucus for a minute?

MR. KOESTER: I am not sure where we are going.

MR. THOMPSON: We are having a public meeting, and we are just asking members of the public here do they have any comments or questions they wanted to ask, and we have a few moments. If they are issues that we can resolve here, I think it is important to resolve them.

MS. STEPHENS: Thank you.

MR. THOMPSON: Mr. Denise.

MR. DENISE: I might say that in the questions that are being asked there is not an understanding on the other side of the table as to what the question is.

MS. STEPHENS: It is my understanding that CAR-29 and -31, DIC, were initiated on March 22 of 1983.

Is that correct?

MR. KOESTER: No.

MS. STEPHENS: Okay. Well --

MR. THOMPSON: Let me ask you again where you are going so maybe we can understand.

MS. STEPHENS: What I am asking is when the first NCR was initiated, and it is my understanding that it was initiated, as they are saying here, in the timeframe of, well, August 30, I guess.

What I am trying to determine --

MR. KOESTER: We showed it on the board this morning. It is in the record. CAR-29 was issued March 1983. CAR-31 -- this is DIC CAR. Let's keep them straight -- August '83.

MS. STEPHENS: Okay.

MR. MARTIN: The information, by the way, that was presented on the slides that had to do with reports will be in the public document room. So if it is a question of accessibility to review those documents to clarify some questions you have, they will be available shortly in the public document room, for those that are not already there.

So if I may ask, if the purpose of your questioning is to ascertain clarification of dates, I wonder if having the documents available shortly would not satisfy that as well, as opposed to if you are looking for making a comment on a more fundamental aspect rather than a series of dates.

MS. STEPHENS: I am trying to understand when the first NCR was issued. I am trying to understand if it was generated at the time the problem was initially recognized or if August 30 represents the first issuance.

MR. REEDY: I will explain that.

MR. KOESTER: I think one of the questions, you can write an NCR on a lot of things. What kind of an NCR are you talking about? We can write an NCR for numerous things, and we have written NCRs on that project that were there for a long, long time.

There's been all kinds of NCR's written. That's the purpose of them. We just did not develop an NCR terminology in August or March of 1983.

MR. REEDY: I'm talking about in reference to miscellaneous structural steel welds that are referenced in CAR 29, DIC CAR 29.

MR. DANIEL: CAR 29, as we've just established, was issued in March of 1983 consistent with the issuance of that car non-conformance reports were generated.

MR. REEDY: At that time?

MR. DANIEL: At that time.

MR. REEDY: Okay. That was my question.

MR. DANIEL: Okay.

Car 31, which we have established was initiated August 1983, at that time non-conformance reports were written.

MR. REEDY: Okay. Thank you.

MR. DAVID SMITH: I do have one other comment.

MR. THOMPSON: Be brief.

MR. DAVID SMITH: In light of the close of this meeting and Mr. Koester's comment that Wolf Creek is ready for licensing I would like to know what KG&E has to offer, as well as the NRC, as far as the issues brought up in the James Wells case as to harrassment and intimidation of workers?

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

MR. THOMPSON: I'm really sorry but that's an issue that is entirely beyond the scope of this particular meeting. And it is one in which NRC has an ongoing review which is not appropriate at this particular meeting.

Let's see. I would like to kind of be able to focus on some type of closure on the issue. As I understand it we have to complete two inspection reports which are currently in the final stages of preparation, both by Region IV and Region I.

And once those are done then I think based on the information and any resolution of any significant issues that are identified as a result of those particular inspection reports then we'll be prepared to provide our final conclusion with respect to this issue.

We anticipate doing that early next month; hopefully within a week.

Is that generally the right time frame?

MR. MARTIN: That's right.

MR. THOMPSON: Any other particular issues, questions of where we are or where we are going?

(No response.)

MR. THOMPSON: Okay. Thank you very much.

The two hour meeting is now over.

(Whereupon, at 4:50 p.m., the conference in the above-entitled matter was concluded.)

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

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 <u>painted</u> welds can reviewed to provide <u>adequate</u> 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:

- Weld location (including existence)
- Length
- Size
- Undercut
- Cracks
- Craters
- Fusion
- Concavity
- Convexity
- Overlap
- Porosity
- Arc Strikes (with regard to cracks)
- 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.

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

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 weld. 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
Co-Chairman, Fritz Engineering Laboratory

Roger G. Slutter
Professor of Civil Engineering
Director - Operations Division

JWF:RGS:rag

Enclosures

cc: J. A. Bailey

Table J2.3
Design Strength of Welds

Types of Weld and Stress[a]	Material	Resistance Factor φ	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				Weld metal with a strength level equal to or less than "matching" may be used
Tension or compression parallel to axis of weld				
Shear on effective area	Base Weld elect.	0.90 0.80	$0.60F_y$ $0.60F_{EXX}$	
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				
Shear parallel to axis of weld	Base[e] Weld elect.	0.75 0.75	$0.60F_u$ $0.60F_{EXX}$	
Tension normal to effective area	Base[e] weld Electrode	0.90 0.80	F_y $0.60F_{EXX}$	
Fillet Welds				
Stress on effective area	Base[e] Weld elect.	0.75 0.75	$0.60F_u$ $0.60F_{EXX}$	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] Weld elect.	0.75 0.75	$0.60F_u$ $0.60F_{EXX}$	Weld metal with a strength level equal to or less than "matching" weld metal may be used

Note: [illegible] in ASD [illegible]

---

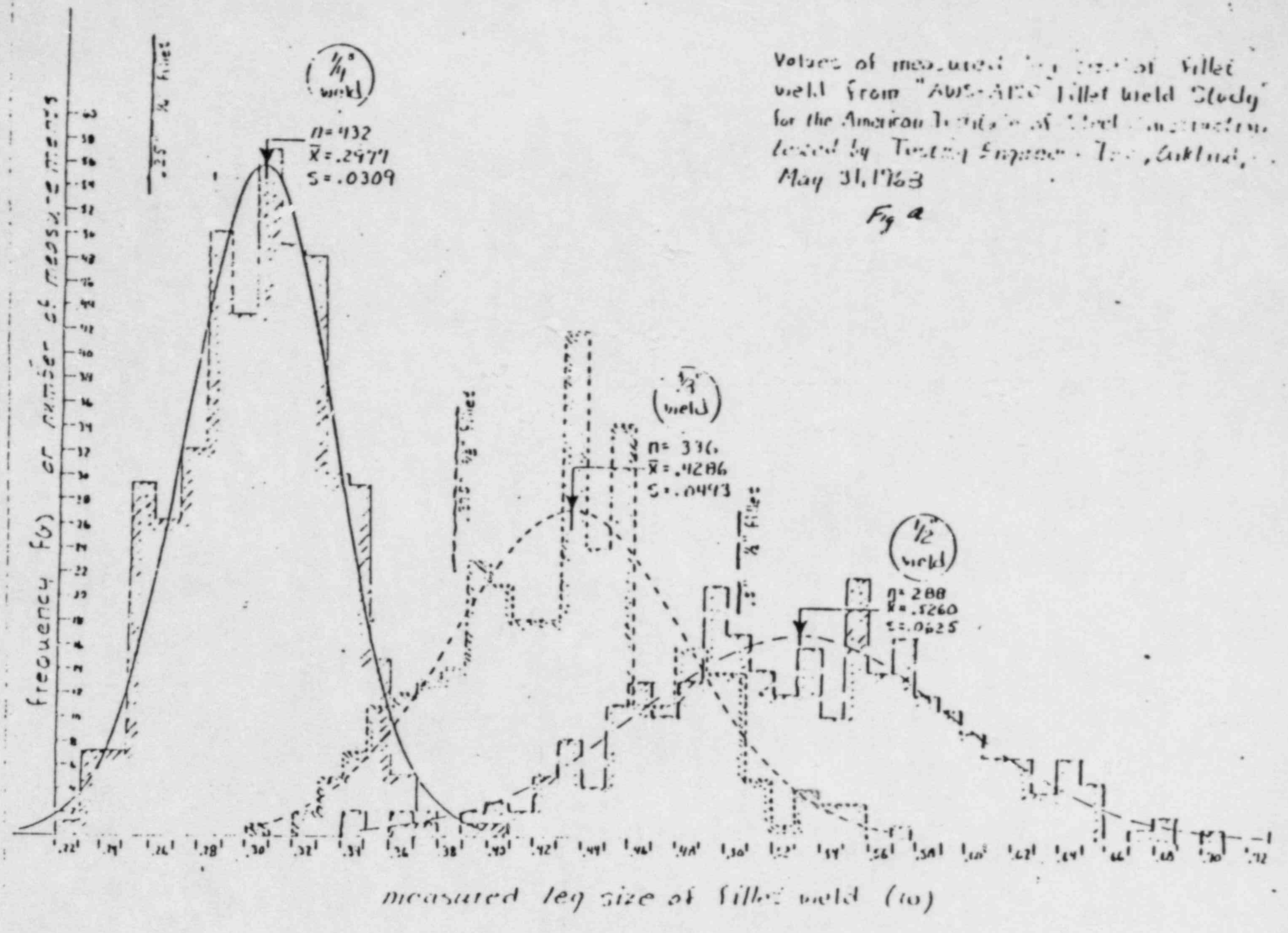
or definition of effective area, see Section J2.

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, 1963

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

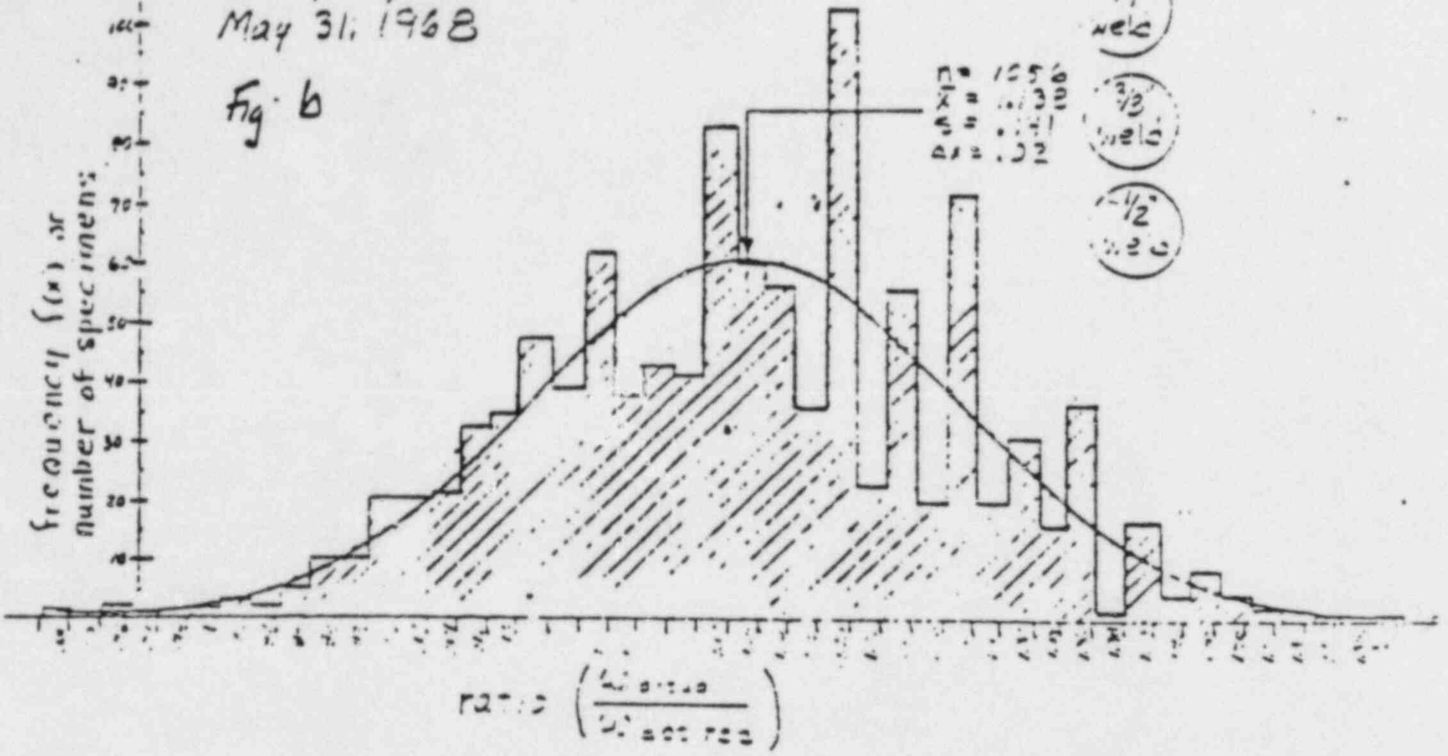


Fig. c

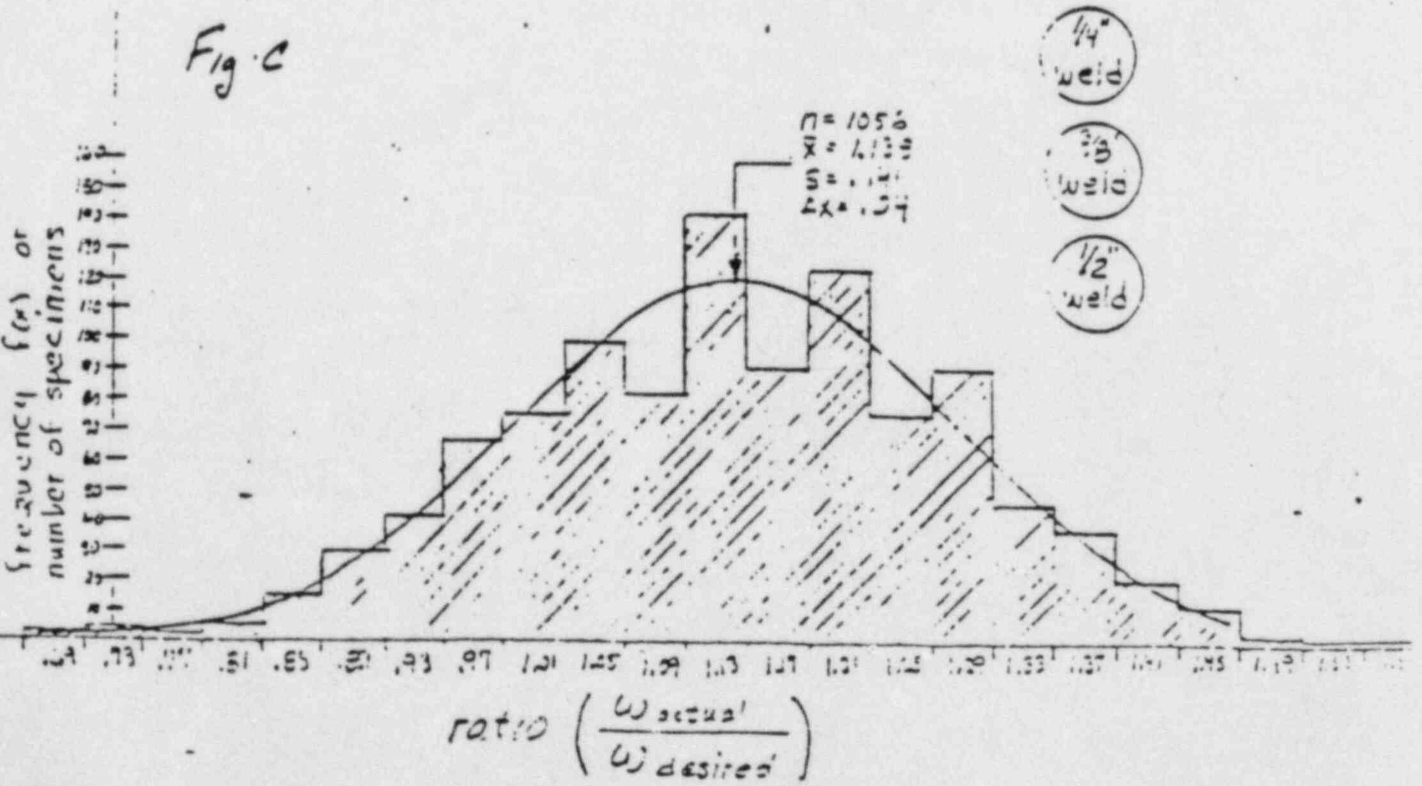
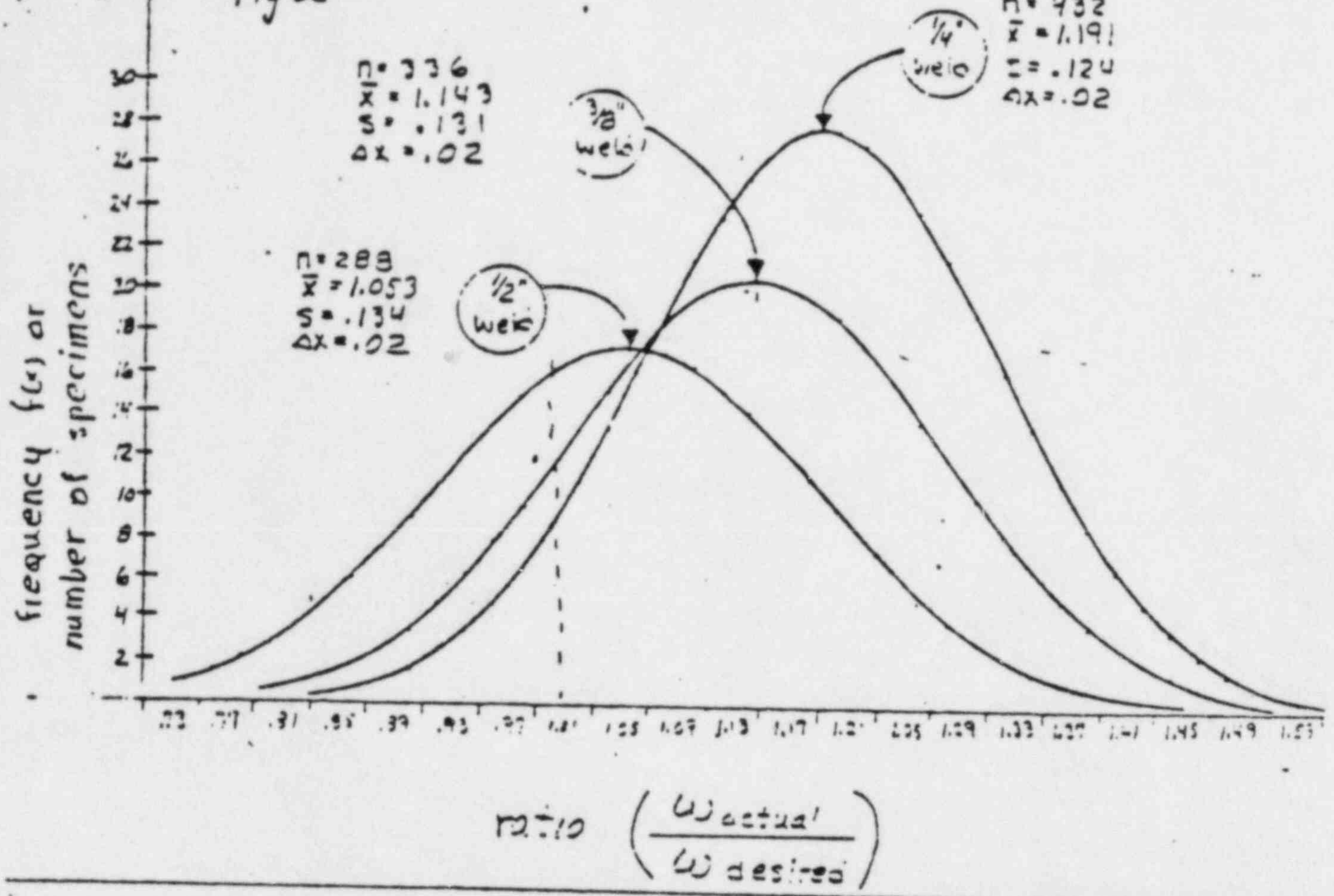


Fig d



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"

682 (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 1.0
10.4% corresponds to 1.2

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

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

and

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

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

from this we get $\bar{x} = 1.11$
$s = .10$

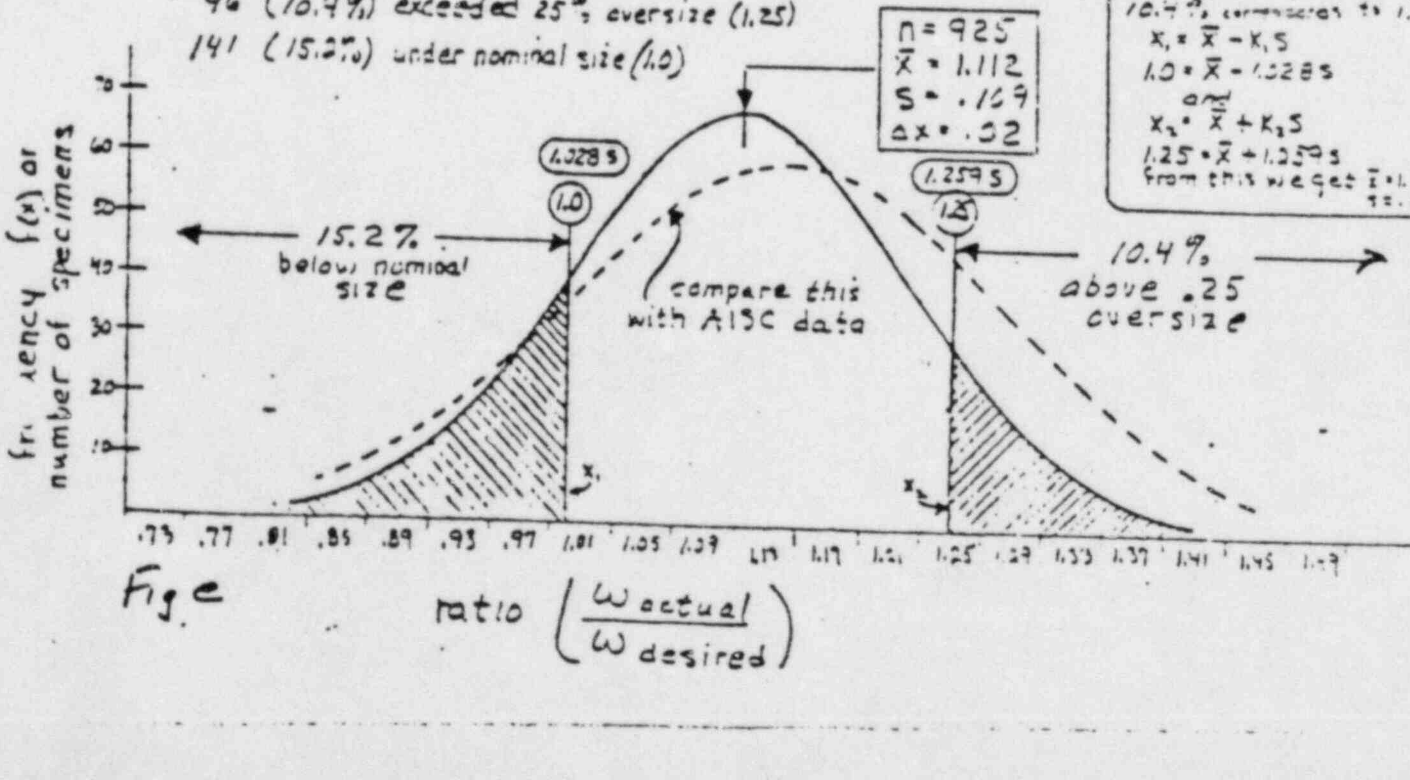


Fig. e

7. Johnson, R. P., "Research on Steel-Concrete Composite Beams," *Journal of the Structural Division*, ASCE, Vol. 96, No. ST3, Proc. Paper 7122, Mar., 1970, pp. 445-459.
8. Johnson, R. P., and Hope-Gill, M. C., "Application of Simple Plastic Theory to Continuous Composite Beams," *Proceedings of the Institution of Civil Engineers*, Part 2, Vol. 61, Mar., 1976, pp. 127-193.
9. Ollgaard, J. G., Slutter, R. G., and Fisher, J. W., "Shear Strength of Stud Connectors in Lightweight and Normal Weight Concrete," *American Institute of Steel Construction Engineering Journal*, Vol. 8, No. 2, Apr., 1971, pp. 55-64.
10. Ravindra, M. K., and Galambos, T. V., "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, Proc. Paper 14006, Sept., 1978, pp. 1443-1457.
11. Slutter, R. G., and Driscoll, G. C., Jr., "Flexural Strength of Steel-Concrete Composite Beams," *Journal of the Structural Division*, ASCE, Vol. 91, No. ST2, Proc. Paper 4294, Apr., 1965, pp. 71-99.
12. *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, 1978.
13. "Steel Structures for Buildings—Limit States Design," *CSA Standard S16.1-1974*, Canadian Standards Association, Rexdale, Ontario, Canada, Dec., 1974.

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LOAD AND RESISTANCE FACTOR DESIGN CRITERIA FOR CONNECTORS[a]

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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 ϕ 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 γ_D and γ_L are the corresponding load factors. The principal purpose of this paper is to develop expressions for the parameters ϕ 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.

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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, γ_k, and the resistance factor, ϕ, in Eq. 1 depend upon a "safety index," β, 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 β = 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 β 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 β is defined (11) as

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

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, σ_{τ_u}, was 2.56 ksi, and the coefficient of variation, V_{τ_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, σ_{τ_u} = 2.67 ksi, and V_{τ_u} = 0.036. Unpublished studies by Nash and Holtz for the same category gave $(\tau_u)_m$ = 86.8 ksi, σ_{τ_u} = 9.88 ksi, and V_{τ_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, σ_{τ_u} = 4.77 ksi, and V_{τ_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, τ_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, $(\sigma_u)_m$ (4)	Standard deviation, σ_{σ_u} (5)	Coefficient of variation, V_{σ_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

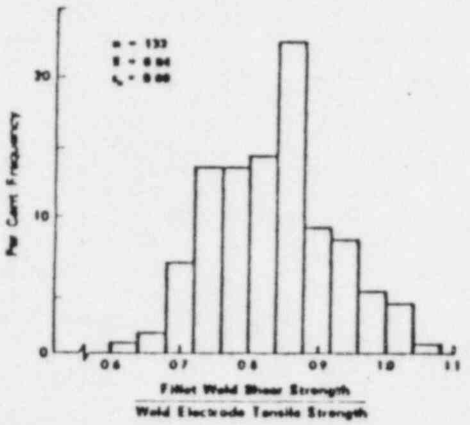

FIG. 1.—Relationship of Weld Shear Strength to Electrode Tensile Strength

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

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

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

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

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

$$V_M^2 = \frac{V^2_{\tau_w}}{\sigma_u} + \frac{V^2_{\sigma_u}}{F_{EXX}} = (0.10)^2 + (0.04)^2 = 0.0116 \qquad (6)$$

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

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

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

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

and the corresponding coefficient of variation is

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

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.

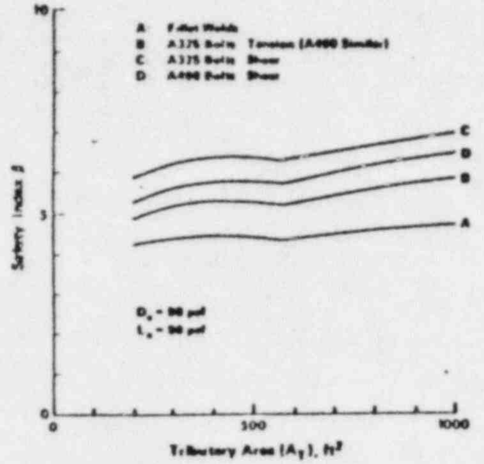

FIG. 2.—Safety Index for Various Connectors

for the safety index β (Eq. 3) can now be performed a variety of dead-load 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

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

in which σ_u = the ultimate tensile strength of the bolts; F_u = the specified minimum tensile strength; and A_t = 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_{σ_u}/F_u = 0.07 for A325 bolts and 0.02 for A490 bolts.

It will be assumed that V_F = 0 (as for fillet welds) and that V_P = 0.05 (reflecting the good control characteristics of bolt manufacturing). In addition, the area of the bolt A_g, 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:

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

$$\text{for A490 bolts:} \quad R_m = 0.80\,A_g F_u; \quad V_R = 0.05 \quad \text{. . . . . (11b)}$$

The term A_g can be obtained from the 1978 AISC Specification where $1.7(A_g F_t) = 1.7\,c\,(D_c + L_m)$ or

$$A_g = \frac{c}{F_t}(D_c + L_m) \quad \text{. . . . . (12)}$$

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:

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

In general terms, Eq. 13 can be expressed as

$$R_m = \left(\frac{\sigma_u}{F_u}\right)_m \frac{A_s}{A_b}\frac{F_u}{F_t} c\,(D_n + L_{rn}) \qquad (14)$$

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 $D_n = L_r = 50$ psf. For A325 bolts, the safety index varies from 4.81 to 6.42 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

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

in which τ_u = the shear strength; σ_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:

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

and for A490 bolts:

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

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

$$A_b = \frac{c}{F_v m}(D_n + L_{rn}) \qquad (17)$$

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:

$$\left.\begin{array}{l} R_m = 0.75 \dfrac{F_u}{F_v} c(D_n + L_{rn}) \\ \text{or for A490 bolts:} \quad R_m = 0.67 \dfrac{F_u}{F_v} c\,(D_n + L_{rn}) \end{array}\right\} \qquad (1$$

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

$$R_m = \left(\frac{\tau_u}{\sigma_u}\right)_m \left(\frac{\sigma_u}{F_u}\right)_m \left(\frac{F_u}{F_v}\right) c\,(D_n + L_{rn}) \qquad (19)$$

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 $D_n = L_{rn} = 50$ 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

$$P_s = m(k_s)_m (T_i)_m \qquad (20)$$

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):

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

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)

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

with a coefficient of variation equal to 0.10.

The slip coe nt obtained from a sample of 312 specimens of A7, A3 , A440, and FE   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):

$$F_v A_b m = c\,(D_n + L_n) \qquad (24)$$

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:

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

In general terms, Eq. 25 can be written as

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

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 $D_n = L_n$ = 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 [illegible]s: Combined Shear and Tension.—For a fastener subjected to both tension and shear, the following relationship has been recommended (5):

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

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, ϕ, can be established from

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

and $$V_R^2 = \frac{V^2_{R_{exp}}}{R_n} + \frac{V^2_{\tau_u}}{F_u} + V_P^2 + V_F^2 \quad \text{(34)}$$

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, ϕ 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

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

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:

$$\phi R_n \ge \gamma_E(c_D\gamma_D D_m + c_L\gamma_L L_m) \quad \text{(36)}$$

in which γ_E = the load factor representing uncertainties in the analysis. From Ref. 11:

$$\gamma_E = \exp(\alpha\beta V_E) \quad \text{(37)}$$

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

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

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 γ can be established for the three values of β. These are tabulated in Table 3.

For beams, columns, and other main structural components ($\beta = 3$), 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 β factors can be imposed on the value of ϕ to be used. For the category described in Table 3 as "Connections—All Others," this means that

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

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, ϕ, 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		
	γ_E (2)	γ_D (3)	γ_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 $\bar{\phi} =$ [illegible].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

$\beta = 4.5$ was cons ive 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)

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

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, $\phi = 0.88 \times 0.73 = 0.64$.

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. Gaylord, 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

1. Butler, L. J., and Kulak, G. L., "Strength of Fillet Welds as a Function of Direction of Load," *Welding Journal*, Welding Research Council, Vol. 36, No. 5, May, 1971, pp. 231s–234s.
2. Butler, L. J., Pal, S., and Kulak, G. L., "Eccentrically Loaded Welded Connections," *Journal of the Structural Division*, ASCE, Vol. 98, No. ST5, Proc. Paper 8874, May, 1972, pp. 989–1005.
3. Dawe, J. L., and Kulak, G. L., "Welded Connections under Combined Shear and Moment," *Journal of the Structural Division*, ASCE, Vol. 100, No. ST4, Proc. Paper 10457, Apr., 1974, pp. 727–741.
4. Denara, L. F., "Survey of Existing Published Information, Appendix D," Report of Weld Panel of the Steel Structures Research Committee, Department of Science and Industrial Research, London, England, 1938.
5. Fisher, J. W., and Struik, J. H. A., *Guide to Design Criteria for Bolted and Riveted Joints*, John Wiley and Sons, Inc., New York, N.Y., 1974.
6. Freeman, F. R., "The Strength of Arc-Welded Joints," *Proceedings*, Institution of Civil Engineers, London, England, Vol. 231, 1930, pp. 322–325.
7. Higgins, T. R., and Preece, F. R., "Proposed Stresses for Fillet Welds in Building Construction," *Welding Journal*, Vol. 47, No. 10, Oct., 1968, p. 429-S.
8. Holtz, N. M., and Kulak, G. L., "High-Strength Bolts and Welds in Load-Sharing Systems," Studies in Structural Engineering, No. 8, Nova Scotia Technical College, Halifax, Canada, Sept., 1970.
9. Khanna, C. K., "Strength of Long Fillet Welds," thesis presented to Nova Scotia Technical College, at Halifax, Canada, in 1969, in partial fulfillment of the requirements for the degree of Master of Engineering.
10. Ravindra, M. K., and Galambos, T. V., "Tentative Load and Resistance Factor Design for Connections," *Research Report No.* 33, Department of Civil Engineering, Washington University, St. Louis, Mo., May, 1975.
11. Ravindra, M. K., and Galambos, T. V., "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, Proc. Paper 14008, Sept., 1978, pp. 1337–1353.
12. "Report of Structural Steel Welding Committee," American Welding Bureau, 1931.
13. "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," American Institute of Steel Construction, New York, N.Y., 1978.
14. "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, 1976.

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

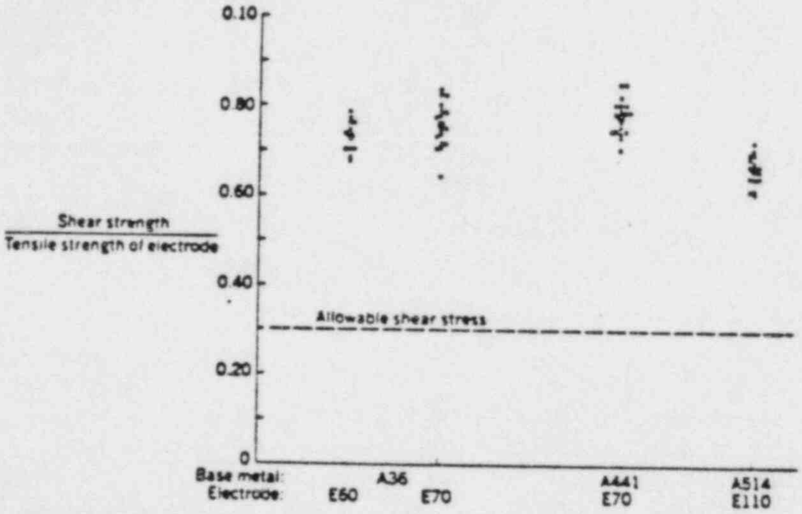

Fig. 19.3 Shear strength of longitudinal fillet welds with matched base metal.

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

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Fritz Engineering Laboratory
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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

Research in Civil Engineering and Related Fields

APTECH IS APPLIED TECHNOLOGY

February 17, 1985

Mr. John Bailey
Kansas Gas and Electric Company
Wolf Creek Generating Station
Post Office Box 309
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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. <u>The integrity of a structural detail is not affected by the imperfections detected in the reinspection program.</u> 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	$3.17X10^{-2}$	$8.7X10^{-3}$
2	$1.0X10^{-3}$	$7.6X10^{-5}$
3	$3.2 X 10^{-5}$	$6.6X10^{-7}$
4	$1.0X10^{-6}$	$5.7X10^{-9}$

* 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 <u>not</u> 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.

ROGER F. REEDY, P.E.

Mr. Reedy has worked in the pressure vessel and nuclear power industries since 1956. His experience includes the design, analysis, fabrication, and erection of nuclear power plant components and implementation of the applicable quality systems. His background encompasses boiling water, pressurized water, and HTGR nuclear power plants, as well as pressure vessels and storage tanks for petroleum, chemical, and other energy industries. Mr. Reedy is an acknowledged expert in the design of pressure vessels and nuclear components meeting the requirements of the ASME Boiler and Pressure Vessel Code.

He has been involved in licensing, engineering review, project coordination, and training of personnel. He has testified as an expert witness in litigations and before regulatory groups, including USNRC, ASLB and ACRS on topics such as design criteria, applications, fabrication techniques, and material applications.

Mr. Reedy has been an active participant for the past 15 years as a member and as chairman of major nuclear Codes and Standards Committees in the development of design, construction and quality criteria for nuclear power plant components. He has served utilities, architect/engineers, and manufacturers as a consultant on all aspects of nuclear power plant licensing, design, quality considerations, and construction.

Roger F. Reedy is currently chairman of the ASME Section III Code for Nuclear Power Plant Components. He is also a member of the N 626.3 Committee which developed the rules concerning duties and responsibilities of engineers designing ASME Code components for nuclear plants. This standard specifies minimum qualifications and details the engineer's responsibilities with regard to coordinating material application, fabrication details, quality assurance and non-destructive examinations of the component.

He has worked with the Republic of China Atomic Energy Council to set up an independent quality assurance and inspection program for all nuclear components installed in Taiwan. In addition, for about the past ten years, Mr. Reedy has given lectures on the ASME Code and quality assurance to NRC I & E inspectors in each of the Regions.

Mr. Reedy was one of the initial members of the Pressure Vessel and Piping Division of ASME and helped start the ASME Training Programs for engineers. The program was so successful that other engineering groups have developed similar programs.

Professional Background

- American Society of Mechanical Engineers
  - Boiler and Pressure Vessel Committee
  - Chairman, Subcommittee on Nuclear Power (Section III)
  - Executive Committee, member

  In 1980, he was awarded the 1980 ASME Centennial Medal by the Policy Board for Codes and Standards in recognition of his decades-long contribution to the development of the Boiler and Pressure Vessel Code.

  - Subgroup on Containment, past chairman
  - Subgroup on Fabrication and Examination, former member
  - ASME Pressure Vessel and Piping Division
  - Past Chairman
  - Nuclear Codes and Standards Committee, member
  - ANSI/ASME N626.3 Specialized Professional Engineers Committee, member

Professional Registration

Professional Structural Engineer - Illinois
Professional Civil Engineer - California, Illinois, Indiana, Michigan, Wisconsin

Professional Experience

1981 - present REEDY ASSOCIATES
Los Gatos, California
President

Currently consulting with utilities, manufacturers and architect/engineers.

1976 - 1981 NUCLEAR TECHNOLOGY, INCORPORATED
San Jose, California
Successively Manager, Special Projects and Chief Consultant

As Manager, Special Projects, he was responsible for coordinating Nutech's quality assurance program and their role as Monitor of the Mark I Containment Modification Project.

His CBI experience and ASME Code (Section III) expertise was a key element in working with the utilitites and General Electric to define and execute a modification program acceptable to the U.S. Nuclear Regulatory Commission.

Was then advanced to Chief Consultant, serving as ex-officio advisor to all in-house projects and all clients on design, quality and construction questions concerning application of the ASME Code.

During his term at NUTECH, Mr. Reedy developed and wrote Code Capsule, a biennial commentary on the changes to the ASME Boiler and Pressure Vessel Code.

1956 - 1976 CHICAGO BRIDGE AND IRON COMPANY
Oak Brook, Illinois
Successively Designer, Staff Engineer, Project Engineer, Design Manager and Senior Engineer.

Duties included design of pressure vessels and storage tanks, including cryogenic vessels, vacuum chambers, multi-layer vessels, environmental chambers, and high-pressure chambers. His duties required close liaison with shop and field personnel, providing Mr. Reedy with an intimate knowledge of practical shop and field construction techniques including the applicable quality requirements.

He has designed more than 50 containment vessels and was the responsible Design Manager for most of the nuclear containment vessels fabricated by CBI. He also designed the first field-erected nuclear reactor.

As Senior Engineer, he consulted with the design staff and other departments concerning ASME Code requirements and special projects.

Education

B. S., Civil Engineering, Illinois Institute of Technology, 1956

Qualified Lead Auditor, ANSI N 45.2.23

Dr. John W. Fisher has been a member of the Lehigh University faculty since 1964 and was promoted to Professor of Civil Engineering in 1969. In August 1971, he was named Associate Director of Fritz Engineering Laboratory at Lehigh University. Prior to joining the Lehigh staff, he was Assistant Bridge Research Engineer with the National Academy of Sciences at the AASHO Road Test for three years.

A native of Scott City, Missouri, Dr. Fisher graduated from Washington University in St. Louis, Missouri, in 1956 with the Bachelor of Science degree in Civil Engineering, and received his Master of Science and Doctor of Philosophy degrees from Lehigh University in 1958 and 1964, respectively.

A structural engineer, he is a specialist in structural connections, fatigue and fracture resistance of riveted, bolted and welded connections, and the behavior and design of composite steel-concrete members. He has been engaged in some forty research projects in these areas since 1961, supervising about fifty (50) graduate research assistants on these projects. He is currently director of the following on-going research projects:

1. Steel Bridge Members under Variable Amplitude Long Life Loading, National Academy of Sciences.
2. Fatigue Studies of Sudan Railroad Bridges, Sudan Railways Corporation.
3. Development of Guidelines for Investigation of Localized Failures, U. S. Department of Transportation, Federal Highway Administration (FHWA).
4. Corrosion Fatigue Characteristics of Bridge Steels, U. S. Department of Transportation (FHWA).
5. Determination of Cracking in Electroslag Welds at Meadville, Pennsylvania Department of Transportation - FHWA.
6. Study of Blue Route Bridge Defects and Structural Response, Pennsylvania Department of Transportation - FHWA.
7. Evaluation of the Electrogas Weldments in Kittanning Bridge, Pennsylvania Department of Transportation.

Dr. Fisher received the Walter L. Huber Research Prize from American Society of Civil Engineers in 1969 for research on high strength bolts, composite design of continuous beams, fatigue behavior of welded steel beams, field performance of bridges and behavior of rigid frame connections.

Dr. Fisher received the American Welding Society Adams Memorial Membership Award in 1974 for recognition of advancing the knowledge of welding for undergraduate and graduate students.

In 1977, Dr. Fisher received the T. R. Higgins Lectureship Award for outstanding contribution to engineering knowledge of fabricated structural steel as author of the paper "Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments".

In 1979, Dr. Fisher received the American Society of Civil Engineering Ernest E. Howard Award for outstanding contributions to structural engineering through added understanding and design criteria in the area of fatigue, connections and composite action.

In February 1980, Dr. Fisher was named Engineer of the Year by the Lehigh Valley Chapter of the Pennsylvania Society of Professional Engineers, receiving his award from thirteen different Lehigh Valley engineering and technical societies.

In October 1981, Dr. Fisher received the American Society of Civil Engineers Raymond C. Reese Research Prize for the paper "Fatigue Strength of Fillet Welded Cruciform Joints" co-authored with K. H. Frank.

Dr. Fisher was featured in Highway Research Profiles of Transportation Research News in 1975 for his outstanding contributions to highway research.

In 1974, Dr. Fisher assisted with rewriting the Swiss Steel Specifications in Zurich, Switzerland, from May through August, for Basler and Hofmann Consulting Engineers, Zurich, Switzerland.

Dr. Fisher is the author of the American Institute of Steel Construction Booklet Bridge Fatigue Guide - Design of Details, 1977. He is the principal author of the Guide for Design Criteria for Bolted and Riveted Joints, published by Wiley Interscience in 1974, and is a co-author of the book, Structural Steel Design, published by Ronald Press Company in 1965 and 1974. He has published over one hundred (100) reports and articles which have appeared in scientific journals.

Dr. Fisher, upon their request, developed and presented short courses on fatigue and fracture of bridge structures and inspection of bridges for the Federal Highway Administration, U. S. Department of Transportation, and Pennsylvania Department of Transportation.

Dr. Fisher has given over one hundred (100) lextures and talks on the design and behavior of welded and bolted connections, composite members and fatigue and fracture of steel structures. This includes special seminars in Chicago (1975), (1980), New York (1976) and Pittsburgh (1981) on the design of connections and fatigue resistance of structures for the local chapters of ASCE.

Dr. Fisher is listed in Who's Who in the East, Who's Who in America, Who's Who in American Education, American Men and Women of Science, and Engineers of Distinction. He is a member of Tau Beta Pi, National Engineering Honor Society' Chi Epsilon National Civil Engineering Honor Society' and Society of Sigma Xi, Honorary Scientific Research Society.

He is a member of the International Association of Bridge and Structural Engineers; member of Commission II, International Association of Bridge and Structural Engineers; American Society of Civil Engineers; the Pennsylvania Society of Professional Engineers; the National Society of Professional Engineers; the American Society for Engineering Education; American Railroad Engineering Association; and the American Welding Society.

He was Chairman of the Steel Bridge Committee of the Transportation Research Board, National Academy of Sciences (1974-1980). He is a member of the American Society of Civil Engineers Task Committee on Bridge Safety. He is a member of the Research Council on Structural Joints; American Railway Engineering Association Committee 15 - Steel Structures; and the American Institute of Steel Construction Specification Committee.

Dr. Fisher was licensed to practice by the State of Illinois, Department of Registration and Education, February 6, 1961.

Since 1965, Dr. Fisher has been a consultant to many companies and organizations, including the following:

1. Nelson Stud Welding Company, A United-Carr Division of TRW, (1965-); Structural Consultant on miscellaneous composite design problems.
2. Bethlehem Steel Corporation (1965, 1967, 1968, 1971, 1976); preparation of article and design examples on high-strength bolted connections; development of design procedure for composite beams with slabform.
3. CAVA Industries (1967); evaluation of bolted crane rail system.
4. American Iron and Steel Institute (1966, 1967, 1970); develop load factor design criteria for welded and bolted connections; subsequently adopted as interim specifications by AASHO Committee on Bridges and Structures.
5. Air Products and Chemical Corporation (1968); review of bolted field connections for lifting ring for Esso Heat Exchangers in Libya. Recommended modifications for a fail-safe erection.

6. Delaware River Port Authority (1968, 1969, 1970); determination of cause of fatigue cracking in floor beam stringers of Welt Whitman Bridge, and development of corrective measures to prevent further cracking and failure.

7. Hewitt-Robbins Division of Litton Industries (1969); review of bolted and welded shear connection design for rotary elevator of large ore carrier for Great Lakes.

8. Galloway and Guthrey, Architects and Engineers, Knoxville (1970-71); determination of cause of collapse of high school gymnasiums.

9. Connecticut Department of Transportation (1970-71); determine cause of fatigue cracking of bridge stringers on Connecticut Turnpike and recommend corrective measures as necessary.

10. Texas Department of Highways (January, 1971); provided instruction on the design and behavior of welded connections under static and cyclic load conditions to bridge design personnel in Austin, Texas.

11. DiStasio and Van Buren, Inc., Consulting Engineers, (February-April, 1971); assisted with field testing of composite steel-concrete building in New York City to determine adequacy with understrength concrete slab.

12. Other miscellaneous consultations during the period 1966 to 1971 on the behavior and design of welded and bolted joints for such firms as R. C. Reese and Associates, Consulting Engineers; Parsons, Brinkerhoff, Quade and Douglas, Consulting Engineers; Zorah Vosganian and Associates.

13. Modjeski and Masters Consulting Engineers (January-June, 1972); assisted with the evaluation of dynamic deformations of the steel support bents of the Summit Bridge and the determination of whether or not the induced vibrations would lead to fatigue crack growth.

14. Page Communication Engineers, Inc. (February-March, 1972); developed installation procedure for galvanized high strength bolts for use in microwave towers in Iran.

15. Paul Weidlinger and Associates (September, 1972-76); field studies on vibrations of rock crushing plant and fracture evaluation of cracked girders for Con Edison Astoria Plant.

16. Chicago Heights Steel Company, Allied Structural Steel Company, C. E. Morris Company, and Fort Pitt Bridge Works (March-July, 1973); determination of the cause of cracking of welded built-up girders at end of cut-short transverse stiffeners, during handling and transportation to site.

17. Ohio Department of Transportation (Spring, 1973); determined cause of cracking in cut-short stiffeners, developed repair procedures and design modifications.

18. Delaware River Port Authority (March, 1973-76); to provide an evaluation of the causes for the cracks forming in the vertical members of the Chester-Bridgeport Bridge; determine whether or not other vertical members are susceptible to cracking; and to provide recommendations for correcting the existing undesirable conditions and preventing their occurrence elsewhere.

19. Basler and Hofmann Consulting Engineers, Zurich, Switzerland (May-August, 1974)1 assisting with rewriting of Swiss Steel Specifications.

20. Ammann and Whitney, Consulting Engineers (1974-76); assisting with evaluation of Jamaica Elevated for fatigue and fracture damage.

21. American Institute of Steel Construction (1974); preparation of Design Guide on AASHTO Fatigue Specifications.

21a. Lukens Steel Corporation (1974-76); consultation on the failure of Bryte Bend Bridge.

22. Ontario Ministry of Transportation and Communications (1974); consultation on fatigue damage.

23. Tippetts, Abbett, McCarthy and Stratton, Consulting Engineers (1975); evaluation of cause of fracture of Tehran Airport Structure.

24. Paul Weidlinger and Associates, Consulting Engineers (1975); assisted with evaluation of the fatigue and fracture resistance of a welded crane girder.

25. Hansen, Holley and Biggs, Consulting Engineers (1975); assisted with evaluation of the performance of various bolted and welded joints.

26. Canadian National Railways (1975-76); investigated the estimated fatigue damage in components of the Fraser River Bridge, New Westminister, B. D.

27. Minnesota Department of Highways (1975); investigated the causes of cracking of the Lafayette Street Bridge in St. Paul and recommended repair procedures.

28. H. C. Lochner, Inc., Consulting Engineers (1975); investigated the causes of cracking of the Poplar Street Complex approach ramps in East St. Louis and recommended repair and retrofit procedures.

29. Richardson, Gordon and Associates, Consulting Engineers (1975); evaluated effect of fire damaged material in steel bridge structure.

30. Acres Consulting Services, Ltd., Niagara Falls, Canada (1975); investigated connections used on Whitelake Bridge to ascertain fatigue strength.

31. Ontario Department of Transportation (1976); assisting with preparation of Specification for Design of Highway Bridges.

32. Bethlehem Steel Corporation (1976); revised article in Bolt Booklet.

33. Washington State Highway Commission (1976); investigated the fatigue and fracture resistance of Broadway Interchange Bridge in Everett, Washington.

34. Richardson, Gordon and Associates, Consulting Engineers (1976-77); evaluation of fracture resistance of the Sewickley Bridge eyebars.

35. Scott Paper Company (1975-76); assisted with assessment of fatigue failure of welded machinery.

36. American Institute of Steel Construction (1976-77); preparation of booklet on "Bridge Fatigue Guide".

37. Esso Research Corporation (1977); assessment of fatigue and fracture resistance of welded details for offshore platform.

38. The Lummus Company, Division-Combustion Engineering (1977); assessed strength of welded beam-to-column connections.

39. Louisville and Nashville Railroad (1977); evaluation of fatigue and fracture resistance of electroslag weldments in railroad bridge.

40. Fuller Company - GATX (1977); assessed fatigue strength of welded connections on large dryers.

41. Bethlehem Steel Corporation (1978-); failure of Hartford Coliseum.

42. Regional Transit Authority - Chicago (A. Tedesko) (1978); assisted in assessment of failure of Dan Ryan Elevated Structure.

43. CONRAIL (1978); investigated the cause of the failure of a Hulett Walking Beam at Astabula, Ohio.

44. Allied Structural Steel (1977-); Consultant on the defects found on New Silver Bridge at Point Pleasant, West Virginia.

45. Ontario Hydro (1978-); Consultant on fatigue failures in bolts and weldment of intake cover structure in Lake Ontario.

46. Vermont Public Service Board (1978-); Consultant on cracks that formed in torus of Vermont Yankee Nuclear Reactor.

47. Kaiser Transit Group (1978); developed loading spectrum for laboratory fatigue test of prestressed concrete T-beam for Dade County, Florida Rapid Transit System.

48. Bechtel Power Corporation (1978); supervised and evaluated influence of bent anchors on capacity of anchor plates.

49. Buckland and Taylor (1978-); assisted with fatigue design criteria for Lions Gate Bridge, Vancouver, British Columbia, Canada.

50. Aetna Insurance (1978-); evaluation of causes of failure of Cargill Grain Elevator - Shiloh Tank Company.

51. Sealand Services (1978-); assisted with lawsuit on failure of SL7 type crane; provided consultation on retrofitting fatigue damaged crane structures.

52. Wiss, Janney & Elstner (1979-81); assisted with evaluation of crack problems on Fremont Bridge.

53. Illinois Central Gulf Railroad (1979); evaluated potential fatigue damage in southern pine stringers of Bluford District.

54. Louisiana Department of Transportation (1979-81); evaluated cracking in Gulf Outlet Bridge, New Orleans.

55. Deleuw, Cather and Company (1979-); evaluation of fatigue critical details on aerial structures of Washington, D. C. Metro System.

56. Illinois Department of Transportation (1979); provided instruction on the fatigue and fracture concepts and their application to bridge design.

57. Bethlehem Steel Corporation (1979—); failure of Kemper Arena.

58. Iowa Department of Transportation (1979); assisted with recommendations for retrofitting fatigue damaged structures.

59. Illinois Department of Transportation (1979-); providing services on significance of cracking in I24 Bridge at Paducah, Kentucky.

60. General Electric Company (1977-78); provided consultation on the fatigue design of the MOD-1 1500 KW Wind Turbine Generator.

61. DiStasio and Van Buren Inc. (1980); provided consultation on the capacity of Type 3 semirigid connections.

62. Maryland Department of Transportation (1980); provided evaluation on causes of failure of aerial inspection crane and the cracking of curved box girder bridge.

63. Lukens Steel Corporation (1977-80); consultant on failure and litigation of Raccoon Mountain Stay Rings.

64. Richardson, Gordon and Associates (1980-); assisting with evaluation of Susquehanna River Bridge, Northeast Corridor.

65. Cumberland Bridge Company (1974-80); assisted with litigation on I274 and I75 Bridges in Kenton County, Kentucky.

66. Burlington Northern (1980-81); evaluated the reasons for failure of Sandpoint, Idaho Bridge.

67. Dravo Corporation (1980-); assisting with arbitration of failure of unloaded at Solmar, France.

68. Catapillar Tractor Company (1980); provided consultation on fatigue design of welded details.

69. Zaladastani Associates (1980); provided consultation on improperly installed bolts at Worcester Civic Center.

70. Envirodyne Engineers (1980); provided consultation on fatigue damage in Illinois Toll Road structures.

71. Bechtel Power Corporation (1980); assisted with evaluation of embedded anchor plates at the Calloway site.

72. Modjeski and Masters Consulting Engineers (1980-); assisted with evaluation of cracking in girder webs of Luling Bridge near New Orleans.

73. Canadian National Railways (1979-); providing consultation on fatigue cracking in steel pier caps and in riveted bridges.

74. New Jersey Transit (1981-); providing consultation on fatigue design of frame of Grumman Buses that they ordered.

75. Modjeski and Masters Consulting Engineers (1981); assisted with evaluation of cracking of I470 hanger cables in West Virginia.

76. United States Steel (1981-); consultant on cracking and litigarion of the Praire due Chien Bridge, Wisconsin, Iowa.

77. Bechtel Power Corporation (1981); assisted with evaluation of welded joint capacity of embedded plates with porosity in welds at Salem site; assisted with evaluation of weld penetration at the Limerick site.

78. Ammann and Whitney Consulting Engineers (1981-); assisting with evaluation of the Willets Point Elevated structures.

79. Sherman and Jackson, Attorneys at Law (1981-); Hyatt Regency Pedestrian Walkway Failure in Kansas City.

1. Fisher, J. W., Driscoll, G. C., Jr. and Schutz, F. W., Jr.
BEHAVIOR OF WELDED CORNER CONNECTIONS, Welding Journal, Vol. 37, No. 5, p. 217-s, 1958

2. Fisher, J. W., Driscoll, G. C., Jr. and Beedle, L. S.
PLASTIC ANALYSIS AND DESIGN OF SQUARE RIGID FRAME KNEES, Welding Research Council Bulletin Series No. 39, April 1958

3. Fisher, J. W. and Driscoll, G. C., Jr.
CORNER CONNECTIONS LOADED IN TENSION, Welding Journal, Vol. 38, No. 11, p. 425-s, 1959

4. Kingham, I., Fisher, J. W. and Viest, I. M.
CREEP AND SHRINKAGE OF CONCRETE IN OUTDOOR EXPOSURE AND RELAXATION OF PRESTRESSING STEEL, Highway Research Board Special Report No. 66, 1961

5. Fisher, J. W. and Viest, I. M.
FATIGUE TEST OF BRIDGE MATERIALS OF THE AASHO ROAD TEST, Highway Research Board Special Report No. 66, 1961

6. Fisher, J. W. and Viest, I. M.
Discussion, BASIC COLUMN STRENGTH, by L. S. Beedle and L. Tall, Journal of the Structural Division, ASCE, Vol. 87, No. ST2, Proc. Paper 2555, 1961; Transactions, ASCE, Vol. 127, Part II, 1962

7. Fisher, J. W.
AN INVESTIGATION OF HIGHWAY BRIDGES SUBJECTED TO CONTROLLED TRAFFIC, Proceedings, Second Annual Bridge Engineering Conference, Colorado State University, 1961

8. Fisher, J. W.
STRUCTURAL STEEL DESIGN, Chapter 6 "Tension Members", Chapter 19 "Welded Connections", Ronald Press Co., New York, 1964; Second Edition, 1974

9. Fisher, J. W.
BEHAVIOR OF AASHO ROAD TEST PRESTRESSED CONCRETE BRIDGE STRUCTURES, PCI Journal, Vol. 8, No. 1, p. 14, February 1963

10. Fisher, J. W. and Viest, I. M.
BRUCHENFORSCHUNG BEIM AASHO-STRASSENVERSUCH, Schweiz Bauzeitung 81, Jahrgang Heft, Vol. 18, Nos. 2 and 9, May 1963

11. Fisher, J. W. and Viest, I. M.
BEHAVIOR OF AASHO ROAD TEST BRIDGE STRUCTURES UNDER REPEATED OVERSTRESS, Highway Research Board Special Report No. 73, 1963

12. Fisher, J. W. and Huckins, H. C.
MEASURING DYNAMIC VEHICLE LOADS, Highway Research Board Special Report No. 73, 1963

13. Fisher, J. W.
Discussion, ANALYSIS OF MULTISPAN BRIDGES SUBJECTED TO MOVING LOADS, by R. X. Wen and T. Toridis, Proceedings, Symposium on the Use of Computers in Civil Engineering, Vol. 1, Lisboa, p. 93, 1962

14. Fisher, J. W., Lee, G. C., Yura, J. A. and Driscoll, G. C., Jr.
PLASTIC ANALYSIS AND TESTS OF HAUNCHED CORNER CONNECTIONS, Welding Research Council Bulletin No. 91, October 1963

15. Fisher, J. W. and Viest, I. M.
ULTIMATE STRENGTH CONCEPTS FOR DESIGN OF STEEL BRIDGES, Proceedings, AASHO, 1963

16. Staff, AASHO Road Test
BRIDGE RESEARCH, AASHO Road Test Report No. 4, Highway Research Board Special Report No. 61D, 1962

17. Rumpf, J. L. and Fisher, J. W.
CALIBRATION OF A325 BOLTS, Journal of the Structural Division, ASCE, Vol. 89, No. ST6, December 1963

18. Fisher, J. W., Ramseier, P. O. and Beedle, L. S.
STRENGTH OF A440 STEEL JOINTS FASTENED WITH A325 BOLTS, Publications, IABSE, Vol. 23, 1963

19. Fisher, J. W. and Viest, I. M.
FATIGUE LIFE OF BRIDGE BEAMS SUBJECTED TO CONTROLLED TRUCK TRAFFIC, Preliminary Publications, Seventh Congress, IABSE, pp. 614-627, 1964

20. Fisher, J. W. and Yen, B. T.
Discussion, FLEXURAL FATIGUE TESTS OF PRESTRESSED STEEL I-BEAMS, by W. D. Reneker and C. E. Ekberg, Journal of the Structural Division, ASCE, Vol. 90, No. ST4, August 1964

21. Wallaert, J. J. and Fisher, J. W.
SHEAR STRENGTH OF HIGH STRENGTH BOLTS, Journal of the Structural Division, ASCE, Vol. 91, No. ST3, June 1965, pp. 99-125

22. Fisher, J. W. and Beedle, L. S.
CRITERIA FOR DESIGNING BEARING-TYPE BOLTED JOINTS, Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October 1965, pp. 129-154

23. Fisher, J. W. and Rumpf, J. L.
ANALYSIS OF BOLTED BUTT JOINTS, Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October 1965, pp. 181-203

24. Sterling, G. H., Troup, E. W. J., Chesson, E. and Fisher, J. W.
CALIBRATION TESTS OF A490 HIGH-STRENGTH BOLTS, Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October 1965, pp. 279-298

25. Fisher, J. W.
BEHAVIOR OF FASTENERS AND PLATES WITH HOLES, Journal of the Structural Division, ASCE, Vol. 91, No. ST6, December 1965, pp. 265-286

26. Wallaert, J. J., Sterling, G. H. and Fisher, J. W.
WHAT HAPPENS TO BOLT TENSION IN LARGE JOINTS?, Fasteners, Vol. 20, No. 3, Winter 1965

27. Christopher, R. J., Kulak, G. L. and Fisher, J. W.
CALIBRATION OF ALLOY STEEL BOLTS, Journal of the Structural Division, ASCE, Vol. 92, No. ST2, April 1966

28. Fisher, J. W.
HIGH-STRENGTH BOLTING FOR STRUCTURAL JOINTS, Bethlehem Steel Corporation Booklet No. 2190, 1965, 1967, 1969 and 1972

29. Fisher, J. W. and Beedle, L. S.
HIGH STRENGTH BOLTING IN THE U.S.A., Final Report, Seventh Congress, IABSE, 1966

30. Fisher, J. W. and Viest, I. M.
FATIGUE LIFE OF STRUCTURAL MEMBERS, Final Report, Seventh Congress, IABSE, 1966

31. Fisher, J. W., Kulak, G. L. and Beedle, L. S.
BEHAVIOR OF LARGE BOLTED JOINTS, Highway Research Record No. 147, Highway Research Board, pp. 52-64, 1966

32. Slutter, R. G. and Fisher, J. W.
FATIGUE STRENGTH OF SHEAR CONNECTORS, Highway Research Record No. 147, Highway Research Board, pp. 65-88, 1966

33. Fisher, J. W. and Kulak, G. L.
Discussion, BRITTLENESS IN HIGHER STRENGTH BOLTS, by F. L. Gill and R. M. Hansen, Journal of the Structural Division, ASCE, Vol. 92, No. ST4, August 1966

34. Kulak, G. L. and Fisher, J. W.
Discussion, PLASTIC DESIGN OF ECCENTRICALLY LOADED FASTENERS, by A. Abolitz, Engineering Journal, AISC, Vol. 4, No. 2, April 1967

35. Fisher, J. W., Chairman, Task Committee on Flexural Members
COMMENTARY ON WELDED COVERPLATED BEAMS, Journal of the Structural Division, ASCE, Vol. 93, No. ST4, August 1967

36. Kormanik, R. and Fisher, J. W.
BEARING-TYPE BOLTED HYBRID JOINTS, Journal of the Structural Division, ASCE, Vol. 93, No. ST5, October 1967

37. Slutter, R. G. and Fisher, J. W.
FATIGUE STRENGTH OF SHEAR CONNECTORS, AISI Bulletin No. 5, October 1967

38. Fisher, J. W. and Beedle, L. S.
BIBLIOGRAPHY ON BOLTED AND RIVETED JOINTS, ASCE Manual No. 48, 1967

39. Reemsnyder, H. S. and Fisher, J. W.
SERVICE HISTORIES AND LABORATORY TESTING, Journal of the Structural Division, ASCE, Vol. 94, No. ST12, December 1968

40. Allan, R. N. and Fisher, J. W.
BOLTED JOINTS WITH OVERSIZE OR SLOTTED HOLES, Journal of the Structural Division, ASCE, Vol. 94, No. ST9, September 1968

41. Kulak, G. L. and Fisher, J. W.
A514 STEEL JOINTS FASTENED BY A490 BOLTS, Journal of the Structural Division, ASCE, Vol. 94, No. ST10, October 1968

42. Fisher, J. W. and Kulak, G. L.
TESTS OF BOLTED BUTT SPLICES, Journal of the Structural Division, ASCE, Vol. 94, No. ST11, November 1968

43. Daniels, J. H. and Fisher, J. W.
FATIGUE BEHAVIOR OF CONTINUOUS COMPOSITE BEAMS, Highway Research Record No. 253, Highway Research Board, 1968

44. Fisher, J. W.
EFFECT OF WELDMENTS ON THE FATIGUE STRENGTH OF STEEL BEAMS, EXPERIMENT DESIGN AND PRELIMINARY RESULTS, Appendix to Proceedings, Third Conference on Dimensioning and Strength Calculations, Hungarian Academy of Sciences, Budapest, 1969

45. Kulak, G. L. and Fisher, J. W.
BEHAVIOR OF LARGE A514 STEEL BOLTED JOINTS, Journal of the Structural Division, ASCE, Vol. 95, No. ST9, September 1969

46. Lee, J. H., O'Connor, C. and Fisher, J. W.
EFFECT OF SURFACE COATINGS AND EXPOSURE ON SLIP, Journal of the Structural Division, ASCE, Vol. 95, No. ST11, November 1969

47. Daniels, J. H., Kroll, G. D. and Fisher, J. W.
BEHAVIOR OF COMPOSITE-BEAM TO COLUMN JOINTS, Journal of the Structural Division, ASCE, Vol. 96, No. ST3, March 1970

48. Fisher, J. W. and Yoshida, N.
LARGE BOLTED AND RIVETED SHINGLE SPLICES, Journal of the Structural Division, ASCE, Vol. 96, No. ST9, September 1970

49. Fisher, J. W.
FATIGUE STRENGTH OF WELDED A514 STEEL BEAMS, Preliminary Publication, Conference on Fatigue of Welded Structures, Brighton, England, July 1970

50. Fisher, J. W.
DESIGN OF COMPOSITE BEAMS WITH FORMED METAL DECK, Engineering Journal, AISC, Vol. 7, No. 3, July 1970

51. Fisher, J. W., Frank, K. H., Hirt, M. A. and McNamee, B. M.
EFFECT OF WELDMENTS ON THE FATIGUE STRENGTH OF STEEL BEAMS, NCHRP Report 102, Highway Research Board, National Academy of Sciences, 1970

52. Ollgaard, J. G., Slutter, R. G. and Fisher, J. W.
SHEAR STRENGTH OF STUD CONNECTORS IN LIGHTWEIGHT AND NORMAL-WEIGHT CONCRETE, Engineering Journal, AISC, Vol. 8, No. 2, April 1971

53. Hirt, M. A., Yen, B. T. and Fisher, J. W.
FATIGUE STRENGTH OF ROLLED AND WELDED STEEL BEAMS, Journal of the Structural Division, ASCE, Vol. 97, No. ST7, July 1971

54. Subcommittee on Inspection of Steel Bridges for Fatigue
INSPECTION OF STEEL BRIDGES FOR FATIGUE DAMAGE, Journal of the Structural Division, ASCE, Vol. 97, No. ST8, August 1971

55. Fisher, J. W.
PLASTIC DESIGN IN STEEL, Chapter 8, "Connections", ASCE Manual 41, 1971

56. Fisher, J. W.
FATIGUE STRENGTH OF WELDED STEEL BEAM DETAILS AND DESIGN CONSIDERATIONS, Proceedings, Canadian Structural Engineering Conference, March 1972

57. Fisher, J. W., Daniels, J. H. and Slutter, R. G.
CONTINUOUS COMPOSITE BEAMS FOR BRIDGES, Preliminary Report, Ninth Congress, IABSE, May 1972

58. Hirt, M. A. and Fisher, J. W.
FATIGUE BEHAVIOR OF WELDED STEEL BEAMS, Highway Research Record No. 400, 1972

59. Power, E. H. and Fisher, J. W.
BEHAVIOR AND DESIGN OF SHINGLE JOINTS, Journal of the Structural Division, ASCE, Vol. 98, No. ST9, September 1972

60. Fisher, J. W. and Gurney, T. R.
HIGH-CYCLE FATIGUE OF CONNECTIONS AND DETAILS, SOA Report 4, Technical Committee 18, Proceedings, ASCE-IABSE International Conference on Tall Buildings, June 1973.

61. Fisher, J. W. and Yen, B. T.
DESIGN, STRUCTURAL DETAILS, AND DISCONTINUITIES IN STEEL, Proceedings, ASCE Specialty Conference on the Safety and Reliability of Metal Structures, November 1972

62. Struik, J. H. A., Oyeledum, A. O. and Fisher, J. W.
BOLT TENSION CONTROL WITH A DIRECT TENSION INDICATOR, AISC Journal, Vol. 10, No. 1, 1973

63. Frank, K. H. and Fisher, J. W.
Discussion, FATIGUE OF COVER PLATED BEAMS, by Murad and Heins, Journal of the Structural Division, ASCE, Vol. 99, No. ST1, Proc. Paper 8855, January 1973

64. McMackin, P. J., Slutter, R. G. and Fisher, J. W.
HEADED STEEL ANCHORS UNDER COMBINED LOADING, Engineering Journal, AISC, Vol. 10, No. 2, 1973

65. Hirt, M. A. and Fisher, J. W.
FATIGUE CRACK GROWTH IN WELDED BEAMS, Journal of Engineering Fracture Mechanics, Vol. 5, 1973

66. Fisher, J. W.
CLASSIFICATION OF TYPICAL WELDED BRIDGE DETAILS FOR FATIGUE LOADING, Research Results Digest No. 59, Transportation Research Board, 1974

67. Fisher, J. W. and Struik, J. H. A.
GUIDE TO DESIGN CRITERIA FOR BOLTED AND RIVETED JOINTS, Wiley Interscience, 1974

68. Fisher, J. W., Albrecht, P. A., Yen, B. T., Klingerman, D. J. and McNamee, B. M.
FATIGUE STRENGTH OF STEEL BEAMS WITH TRANSVERSE STIFFENERS AND ATTACHMENTS, NCHRP Report No. 147, Transportation Research Board, 1974

69. Fisher, J. W. and Irwin, G. R.
FRACTURE ANALYSIS OF FLAWS IN WELDED BRIDGE STRUCTURES, Proceedings, United States-Japan Seminar on Significance of Defects in Welded Structures, University of Tokyo Press, 1974

70. Fisher, J. W.
GUIDE TO 1974 AASHTO FATIGUE SPECIFICATION, American Institute of Steel Construction, 1974

71. Albrecht, P. A. and Fisher, J. W.
AN ENGINEERING ANALYSIS OF CRACK GROWTH AT TRANSVERSE STIFFENERS, Publications, IABSE, Vol. 35-I, 1975

72. Fisher, J. W.
FATIGUE PROPERTIES OF WELDED STRUCTURAL JOINTS, Section 5.4, Welding Handbook, AWS, 1976

73. Grant, J. A., Fisher, J. W. and Slutter, R. G.
HIGH STRENGTH STEEL COMPOSITE BEAMS WITH FORMED STEEL DECK, Preliminary Report, Tenth Congress, IABSE, 1976

74. Demo, D. A. and Fisher, J. W.
ANALYSIS OF FATIGUE OF WELDED CRANE RUNWAY GIRDERS, Journal of the Structural Division, ASCE, Vol. 102, No. ST5, May 1976

75. King, J. P. C., Csagoly, P. F. and Fisher, J. W
FIELD TESTING OF AGUASABON RIVER BRIDGE IN ONTARIO, Transportation Research Record 579, TRB, 1976.

76. Fisher, J. W. and Daniels, J. H.
AN INVESTIGATION OF THE ESTIMATED FATIGUE DAMAGE IN MEMBERS OF THE 380 FT. MAIN SPAN, FRASER RIVER BRIDGE, AREA Bulletin 658, Proceedings Vol. 77, June-July 1976.

77. Fisher, J. W.
CONNECTIONS FOR STEEL STRUCTURES, Preprint 2748, ASCE Annual Convention, Philadelphia, 1976.

78. Fisher, J. W. and Grant, J. A.
FURTHER STUDIES ON COMPOSITE BEAMS WITH FORMED STEEL DECK, IABSE 10th Congress, Final Report, Tokyo, Japan, 1977.

79. Grant, J. A., Fisher, J. W. and Slutter, R. G.
COMPOSITE BEAMS WITH FORMED STEEL DECK, AISC Engineering Journal, Vol. 14, No. 1, 1977.

80. Fisher, J. W., Pense, A. W. and Roberts, R.
EVALUATION OF FRACTURE OF LAFAYETTE STREET BRIDGE, Journal of the Structural Division, ASCE, Vol. 103, No. ST7, July 1977.

81. Daniels, J. H., Yen, B. T. and Fisher, J. W.
STRESSES IN ORTHOTROPIC DECK OF RIO-NITEROI BRIDGE UNDER TRAFFIC, Transportation Research Record 607, Transportation Research Board, 1977.

82. Fisher, J. W., Yen, B. T. and Daniels, J. H.
FATIGUE DAMAGE IN THE LEHIGH CANAL BRIDGE FROM DISPLACEMENT-INDUCED SECONDARY STRESSES, Transportation Research Record 607, Transportation Research Board, 1977.

83. Fisher, J. W. and Pense, A. W.
FATIGUE REQUIREMENTS FOR BRIDGES AND METHODS TO IMPROVE FATIGUE STRENGTH, Proceedings, Ninth National SAMPE Technical Conference, Society for the Advancement of Material and Process Engineering, 1977.

84. Fisher, J. W.
BRIDGE FATIGUE GUIDE - DESIGN AND DETAILS, American Institute of Steel Construction, 1977.

85. Fisher, J. W. and Yen, B. T.
FATIGUE STRENGTH OF STEEL MEMBERS WITH WELDED DETAILS, Engineering Journal, AISC, Vol. 14, No. 4, 1977.

86. Zettlemoyer, N. and Fisher, J. W.
STRESS GRADIENT CORRECTION FACTOR FOR STRESS INTENSITY AT WELDED STIFFENERS AND COVER PLATES, Welding Journal, Vol. 56, No. 12, December 1977, pp. 393s-398s.

87. Zettlemoyer, N. and Fisher, J. W.
STRESS GRADIENT CORRECTION FACTOR FOR STRESS INTENSITY AT WELDED GUSSET PLATES, Welding Journal, Vol. 57, No. 2, February 1978, pp. 57s-62s

88. Fisher, J. W., Pense, A. W., Slockbower, R. E. and Hausammann, H.
RETROFITTING FATIGUE DAMAGED BRIDGES, Transportation Research Record No. 664, 1978, pp. 102-109

89. Fisher, J. W., Galambos, T. V., Kulak, G. L., Ravindra, M. K.
LOAD AND RESISTANCE FACTOR DESIGN CRITERIA FOR CONNECTORS, Journal of the Structural Division, ASCE, Vol. 104, No. ST9, September 1978, pp. 1427-1441

90. Zettlemoyer, N. and Fisher, J. W.
STRESS GRADIENT AND CRACK SHAPE EFFECTS ON STRESS INTENSITY AT WELDED DETAILS, Welding Journal, Vol 57, No. 8, August 1978, pp. 246s-250s

91. Fisher, J. W.
FATIGUE CRACKING IN BRIDGES FROM OUT-OF-PLANE DISPLACEMENTS, The Canadian Journal of Civil Engineering, Vol. 5, No. 4, 1978

92. Fisher, J. W., Hanson, J. M., Lally, A., Scheffey, C. and Salgo, M. N.
FRACTURES - PROBLEM FOR WELDED STEEL BRIDGES, Civil Engineering - ASCE, Vol. 48, No. 4, April 1978, p. 70-73

93. Fisher, J. W.
RETROFITTING PROCEDURES FOR FATIGUE-DAMAGED FULL-SCALE WELDED BRIDGE BEAMS, Research Results Digest No. 101, Transportation Research Board, 1978.

94. Fisher, J. W., Pense, A. W. and Roberts, R.
THE FAILURE OF THE LAFAYETTE STREET BRIDGE - INFLUENCE OF DESIGN DETAILS, Proceeding of Forum on Prevention of Structural Failures, American Society for Metals, 1978

95. Fisher, J. W., Kato, B., Woodward, H. M. and Frank, K. H.
FIELD INSTALLATION OF HIGH-STRENGTH BOLTS IN NORTH AMERICA AND JAPAN, IABSE Surveys S-8/79, International Association for Bridge and Structural Engineering, Periodical 1/1979, February 1979

96. Fisher, J. W., Hausammann, H., Sullivan, M. D. and Pense, A. W.
DETECTION AND REPAIR OF FATIGUE DAMAGE IN WELDED HIGHWAY BRIDGES, NCHRP Report 206, Transportation Research Board, June 1979

97. Fisher, J. W., Fisher, T. A. and Kostem, C.N.
DISPLACEMENT INDUCED FATIGUE CRACKS, Engineering Structures, Vol. 1, No. 5, October 1979, pp. 252-257

98. Frank, K. H. and Fisher, J. W.
FATIGUE STRENGTH OF FILLET WELDED CRUCIFORM JOINTS, Journal of the Structural Division, ASCE, Vol. 105, No. ST9, September 1979, pp. 1727-1740

99. Fisher, J. W., Yen, B. T. and Frank, K. H. MINIMIZING FATIGUE AND FRACTURE IN STEEL BRIDGES, Journal of Engineering Materials and Technology, Transactions ASME, Vol. 102, January 1980, pp. 20-25.

100. Fisher, J. W., Pense, A. W., Hausammann, H. and Irwin, G. R. QUINNIPIAC RIVER BRIDGE CRACKING, Journal of the Structural Division, ASCE, Vol. 106, No. ST4, April 1980, pp. 773-789.

101. Fisher, J. W. and Mertz, D. R. FATIGUE CRACKING IN LONGER SPAN BRIDGES, The Transactions of the New York Academy of Sciences, Vol. 352, pp. 193-219.

102. Fisher, J. W., Barthelemy, B. M., Mertz, D. R. and Edinger, J. A. FATIGUE BEHAVIOR OF FULL SCALE WELDED BRIDGE ATTACHMENTS, NCHRP Report 227, December 1980.

103. Fisher, J. W., Hanson, J. M., Hausammann, H. and Osborn, A. E. N., FRACTURE AND RETROFIT OF DAN RYAN RAPID TRANSIT STRUCTURE, Final Report, 11th Congress of IABSE, Vienna, 1980.

104. Roberts, R., Fisher, J. W., Irwin, G. R., Boyer, D. D., Hausammann, H., Krishna, G. V., Morf, U., Slockbower, R. E. and Nishanian, J. FATIGUE AND FRACTURE RESISTANCE OF A WELDED BRIDGE DETAIL, Advances in Fracture Research, Pergamon Press, Oxford and New York, 1980, pp. 2101-2108.

105. Kostem, C. N., Fisher, J. W., Reese, R. T. and Kealey, T. R. VIBRATION OF A STEEL TRUSS HIGHWAY BRIDGE, Proceedings 2nd Specialty Conf., Dynamic Response of Structures, ASCE, January 15-16, 1981, pp. 61-71.

106. Norris, S. N. and Fisher, J. W. THE FATIGUE BEHAVIOUR OF WELDED WEB ATTACHMENTS, Journal of Constructional Steel Research, Vol. No. 2, January 1981, pp. 27-38.

107. Fisher, J. W. and Mertz, D. R. DESIGNING STEEL BRIDGES TO PREVENT FATIGUE DAMAGE, The Metallurgical Society of AIME, TMS Paper A81-33, 1981.

108. Fisher, J. W., Slockbower, R. E., Hausammann, H. and Pense, A. W. LONG TIME OBSERVATION OF A FATIGUE DAMAGED BRIDGE, Proceedings ASCE, Vol. 107, No. TC1, April 1981, pp. 55-71.

109. Fisher, J. W., Mertz, D. and Edinger, J. FATIGUE RESISTANCE AND REPAIR OF FULL-SCALE WELDED WEB ATTACHMENTS, International Conf. on Inspection, Maintenance, and Repair of Road and Railway Bridges, Brussels-Paris, April 13-17, 1981, Vol. II, pp. 435-440.

Dr. Roger G. Slutter has been a member of the Lehigh University faculty since 1966 and was promoted to Professor of Civil Engineering in 1975. Prior to joining the staff of Lehigh University he was Design Engineer for the Concrete Products of Martin Marietta Company.

Dr. Slutter graduated from Lehigh University in 1953 with a Bachelor of Science degree in Civil Engineering and received his Master of Science and Doctor of Philosophy degrees from Lehigh University in 1956 and 1966, respectively.

A structural engineer with extensive experience in materials and testing, he is currently Chairman of the Operations Division of Fritz Engineering Laboratory. This position entails general supervision of all laboratory facilities and testing equipment.

Prior to joining the staff of Lehigh University, Dr. Slutter eas involved in the design and manufacture of precast and prestressed concrete products ranging from concrete pipe to bridge beams. Since joining the Lehigh staff his experience has been largely in the area of experimental research and testing of structural components and materials. The research projects on which he has worked include prestressed concrete members, composite steel and concrete members, concrete anchors, relaxation of prestressing materials, fatigue of structures and materials, connections and polymer concrete systems. He has assisted in most structural research programs and supervised all industrial testing programs at Fritz Engineering Laboratory since 1962.

He is a member of American Society of Engineers, American Concrete Institute, American Society for Testing and Materials, Society for Experimental Stress Analysis and American Society for Metals. He has participated in Code writing efforts of American Society of Mechanical Engineers, American Institute of Steel Construction, Association of American Railroads and American Association of State Highway and Transportation Officials.

A list of papers and publications is provided as follows:

Publications of Dr. Roger G. Slutter

1. Slutter, R. G., Ekberg, C. E., Jr. and Wather, R. E. FATIGUE RESISTANCE OF PRESTRESSED CONCRETE BEAMS IN BENDING, Proceedings, American Society of Civil Engineers, Journal of the Structural Division, Vol. 123, No. ST4, July 1958.
2. Slutter, R. G., et al STRUCTURAL STEEL DESIGN, Chapter 13, The Ronald Press, 1964.
3. Slutter, R. G. and Driscoll, G. C. COMPOSITE DESIGN FOR BUILDINGS, Progress Report No. 3, Report No. 279.10, Lehigh University, Bethlehem, PA., January 1962.
4. Slutter, R. G. and Driscoll, G. C. THE FLEXURAL STRENGTH OF STEEL AND CONCRETE COMPOSITE BEAMS, Report No. 279.15, Lehigh University, Bethlehem, PA., March 1963.
5. Slutter, R. G. PUSHOUT TESTS OF WELDED STUD SHEAR CONNECTIONS IN SOLITE CONCRETE, Report No. 200.63.409.1, Lehigh University, Bethlehem, PA., June 1963.
6. Slutter, R. G., Driscoll, G. C. and King, D. C. FATIGUE OF COMPOSITE BEAMS: FATIGUE STRENGTH OF 1/2 INCH DIAMETER STUD SHEAR CONNECTORS, Report No. 285.6, Lehigh University, Bethlehem, PA., March 1964.
7. Slutter, R. G. and Adams, R. G. TESTS OF COMPOSITE BEAMS WITH HOLORIB COMPOSITE SLABS, Report No. 200.63.408.2, Lehigh University, Bethlehem, PA., July 1964.
8. Slutter, R. G. and Fisher, J. W. PROPOSAL FOR STUDIES OF THE STATIC AND FATIGUE BEHAVIOR OF SIMPLE AND CONTINUOUS COMPOSITE BEAMS USING STUD CONNECTORS, Report No. R6-96, Lehigh University, Bethlehem, PA., July 1965.
9. Slutter, R. G. and Fisher, J. W. TENTATIVE DESIGN PROCEDURE FOR SHEAR CONNECTIONS IN COMPOSITE BEAMS, Report No. 316.1, Lehigh University, Bethlehem, PA., March 1965.
10. Slutter, R. G. and Driscoll, G. C. FLEXURAL STRENGTH OF STEEL CONCRETE COMPOSITE BEAMS, Journal of the Structural Division, ASCE, Vol. 91, ST2, Proceedings Paper 4924, April 1965.

11. Slutter, R. G. and Fisher, J. W. FATIGUE STRENGTH OF SHEAR CONNECTORS, Highway Research Board Record No. 127, 1966.

12. Slutter, R. G. and Driscoll, G. C. CLOSURE: FLEXURAL STRENGTH OF STEEL CONCRETE COMPOSITE BEAMS, Journal of Structural Division, ASCE, Vol. 92, ST5, October 1965.

13. Slutter, R. G. and Fisher, J. W. TESTS OF LIGHTWEIGHT CONCRETE PUSHOUT SPECIMENS CONTAINING STUD SHEAR CONNECTORS AND METAL DECK FORMS, Report No. 200.66.438.1, Lehigh University, Bethlehem, PA., March 1966.

14. Slutter, R. G. and Fisher, J. W. FINAL REPORT - STEEL RESEARCH FOR CONSTRUCTION BULLETIN No. 5, AISI, October 1967.

15. Slutter, R. G. TESTS OF CINCINNATI CENTER COMPOSITE BEAMS, Report No. 200.67.458.2, Lehigh University, Bethlehem, PA., January 1968.

16. Slutter, R. G. and Rivera, U. C. TESTS OF LIGHTWEIGHT CONCRETE COMPOSITE BEAMS WITH METAL DECKING, Report No. 200.67.458.1, Lehigh University, Bethlehem, PA., January 1968 and Report No. 200.69.458.2, Lehigh University, Bethlehem, PA., August 1969.

17. Slutter, R. G., Fisher, J. W. and Ollgaard, J. G. SHEAR STRENGTH OF STUD CONNECTORS IN LIGHTWEIGHT AND NORMAL WEIGHT CONCRETE, AISC, Vol. 8, No. 2, April 1971.

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Recent work includes elastic-plastic finite element analysis, the effect of imperfections on structural integrity, significance and effect of residual and restraint stresses on structural performance, measurement of residual stresses; selection of welding procedures for avoiding hydrogen cracking; analyses of defects in containments, repair welds and procedures; analyses of reheat treatment cracking; prediction of stress corrosion crack growth in BWR piping; analyses of safe end failures in BWR vessels; evaluation of corrosion fatigue performance of deep water platforms: fracture analyses of steam generator support components; evaluation of defects in main steam piping; fracture controls for chilled natural gas pipelines.

BACKGROUND AND PROFESSIONAL HONORS

- B.E. (Mech.), University of Canterbury, New Zealand
- DIC, Imperial College of Science and Technology
- Ph.D., London University
- Member, American Society of Mechanical Engineers
- Member, American Welding Society
- Member, Institution of Mechanical Engineers (Chartered Engineer)
- Member, Welding Institute
- EPRI Corrosion Advisory Committee
- EPRI Pressure Vessel Study Group
- Chairman, Condenser Availability and Integrity Workshop, Miami, Florida, 1979
- Member, Fatigue Program Advisory Committee of the Maritime Transportation Research Board

SELECTED REPORTS, PUBLICATIONS, AND INVITED LECTURES

A Fracture Control Procedure for Nuclear Pressure Vessels, Conference on Practical Application of Fracture Mechanics to Pressure Vessel Technology, I. Mech. E., London, England (May 1971).

Designing to Prevent Fracture in Tall Buildings, ASCE/IABSE Joint Committee, Technical Committee 18, State-of-the-Art Report (January 1972) (with S. T. Rolfe).

The Significance of Defects in Butt Welds in C/Mn Steels With Special Reference to Fitness for Purpose, Welding Research Abroad (March 1972).

J - A Path Independent Integral for Characterizing Fracture Behavior, Welding Institute Research Bulletin (March 1973).

Compatibility of Linear Elastic (K_{Ic}) and General Yielding (COD) Fracture Mechanics, Engineering Fracture Mechanics, Vol. 5 (1973), Pp. 167.

SELECTED REPORTS, PUBLICATIONS, AND INVITED LECTURES (continued)

- *A Comparison of Deformation Parameters for Work Hardening and Non-Work Hardening Behavior,* International Journal of Fracture (1973).

Techniques for Assessing Fracture Toughness, Conference on Mechanics and Physics of Fracture, Cambridge University, England (1975).

Repair Welds Without Post-Weld Heat Treatment, International Institute of Welding, Sydney, Australia (1976).

Stress Corrosion Crack Growth and Fracture Predictions for BWR Piping, 1978 ASME/CSME Pressure Vessels and Piping Conference, Montreal, Canada (1978) (with R. C. Cipolla).

NATO Lecture Tour: Belgium, The Netherlands, Germany, Denmark, Norway, Sweden, Portugal (1971).

Third International Congress on Fracture, Munich, Germany (April 1973).

Finite Element Techniques in Fracture Mechanics, Stuttgart University, Germany (April 1973).

Damage Tolerance Requirements for Pressure Vessels, ASM Structural Design Forum, San Francisco, California (1975).

Failures in Welded Structures, ASME, WAM, Atlanta, Georgia (1977).

Residual Stresses in Welded Construction and Their Effects, Welding Institute, London, England (1977).

The First US/Japan Joint Symposium on Corrosion Problems in Light Water Reactors, Japan (1978).

The Application of Fracture Toughness Data to the Assessment of Pressure Vessel Integrity, Second International Conference on Pressure Vessel Technology, San Antonio, Texas (October 1973).

Steel Castings for Structural Use, Proceedings of Offshore Technology Conference, Newcastle, England (February 1974) (with S. J. H. Still).

Techniques for Assessing Fracture Toughness, Conference on the Mechanics and Physics of Fracture, Churchill College, Cambridge, England (January 1975).

Damage Tolerance Requirements for Heavy Wall Pressure Vessels, Third Annual ASM Materials/Design Forum Prevention of Structural Failure Through Quantitative NDE and Fracture Mechanics (July 1975).

The Application of Elastic-Plastic Fracture Mechanics in Fracture Safe Design, Nuclear Engineering and Design, Vol. 45, No. 1 (January 1978).

The Application of Small Scale Tests to the Prediction of Structural Integrity,
Seminar on Small Scale Testing, Milan, Italy (May 1979).

The Significance of Defects in Welded Long-Span Bridge Structures,
New York Academy of Sciences, O.H. Amman Centennial Conference, New York (November 1979)

On-Line Monitoring of Critical Components to Improve Reliability,
Symposium on Critical Materials and Fabrication Issues, ASME, San Francisco (August 1980)

STRUCTURAL STEEL WELDING
PRESENTATION

KANSAS GAS AND ELECTRIC COMPANY
WOLF CREEK GENERATING STATION

FEBRUARY 27, 1985

KG&E/NRC MEETING
AWS STRUCTURAL STEEL WELDING
PHILLIPS BUILDING • BETHESDA, MARYLAND • FEBRUARY 27, 1985

INTRODUCTION
- NRC
- KG&E - Gene Rathbun; Manager Licensing and Radiological Services

GENERAL DESIGN PHILOSOPHY
James Ivany; Civil Engineering Supervisor, Bechtel

QUALITY ASSURANCE PROGRAM AND HISTORY OF CORRECTIVE ACTION REPORT NO. 19
William Rudolph; Manager Quality Assurance (WCGS)

WELDING HISTORY AND MANAGEMENT PLAN
John Berra; Vice President - Operations, Daniel International Corporation

ENGINEERING EVALUATION
Jerry Brown; Civil Engineering Group Leader, Bechtel

INDEPENDENT REVIEWS
Glenn L. Koester; Vice President - Nuclear
- Roger Reedy; Professional Engineer, Reedy Associates
- Dr. John Fisher; Professor of Civil Engineering, Lehigh University
- Dr. Geoffrey Egan; President, APTECH

SUMMARY
Glenn L. Koester

- STRUCTURAL STEEL WELDING IS DONE TO AWS D1.1 — 1975
- AWS IS NOT CODIFIED
- CODE APPLICATION BY OWNER — ARCHITECT/ENGINEER

KG&E SUBMITTALS TO NRC CONCERNING AWS STRUCTURAL STEEL WELDING

10CFR50.55(e) REPORTS

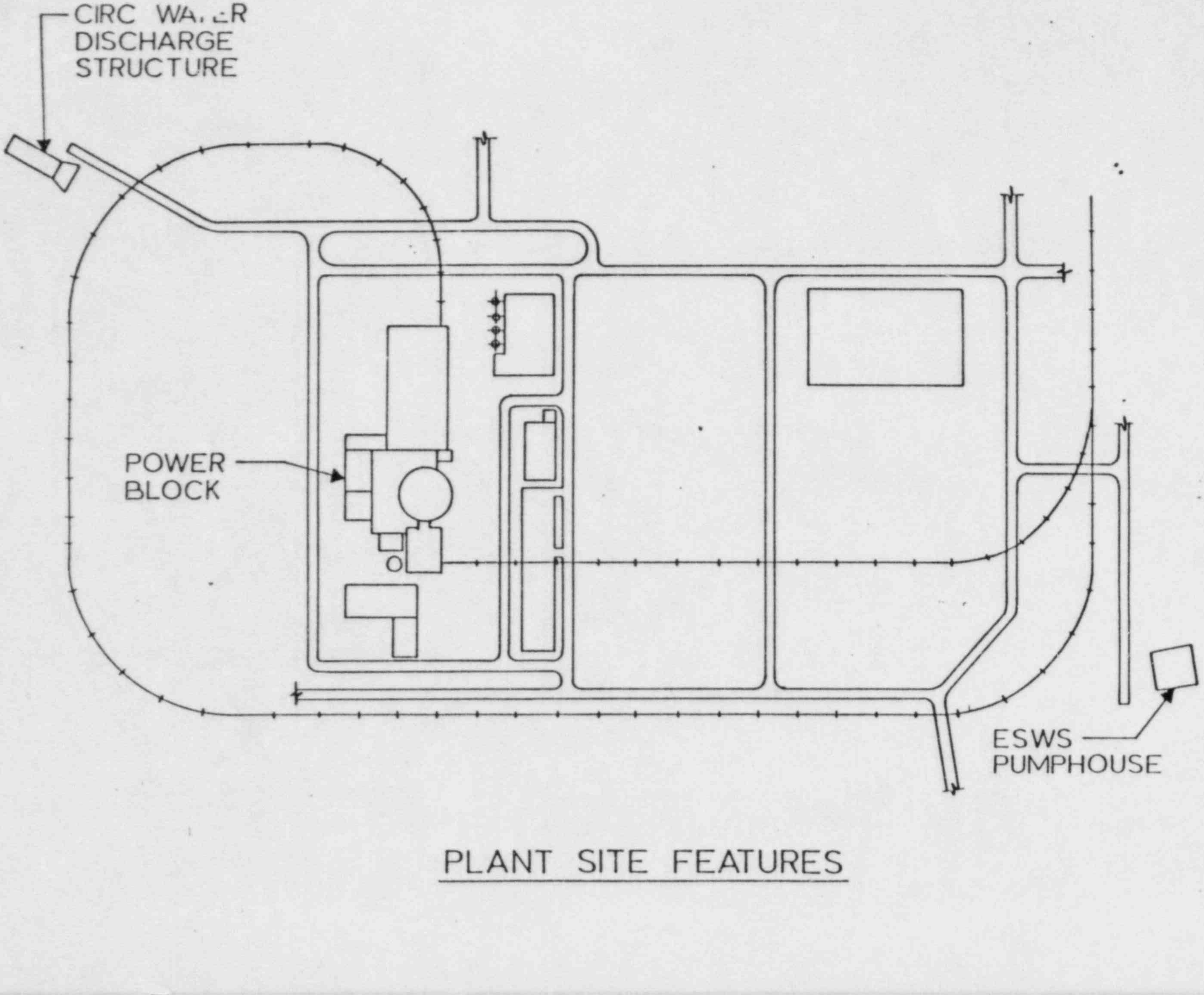
- October 17, 1984 - KMLNRC 84-187
- January 18, 1985 - KMLNRC 85-025

FINAL REPORT

- December 31, 1984 - KMLNRC 84-238
- January 21, 1985 - KMLNRC 85-037

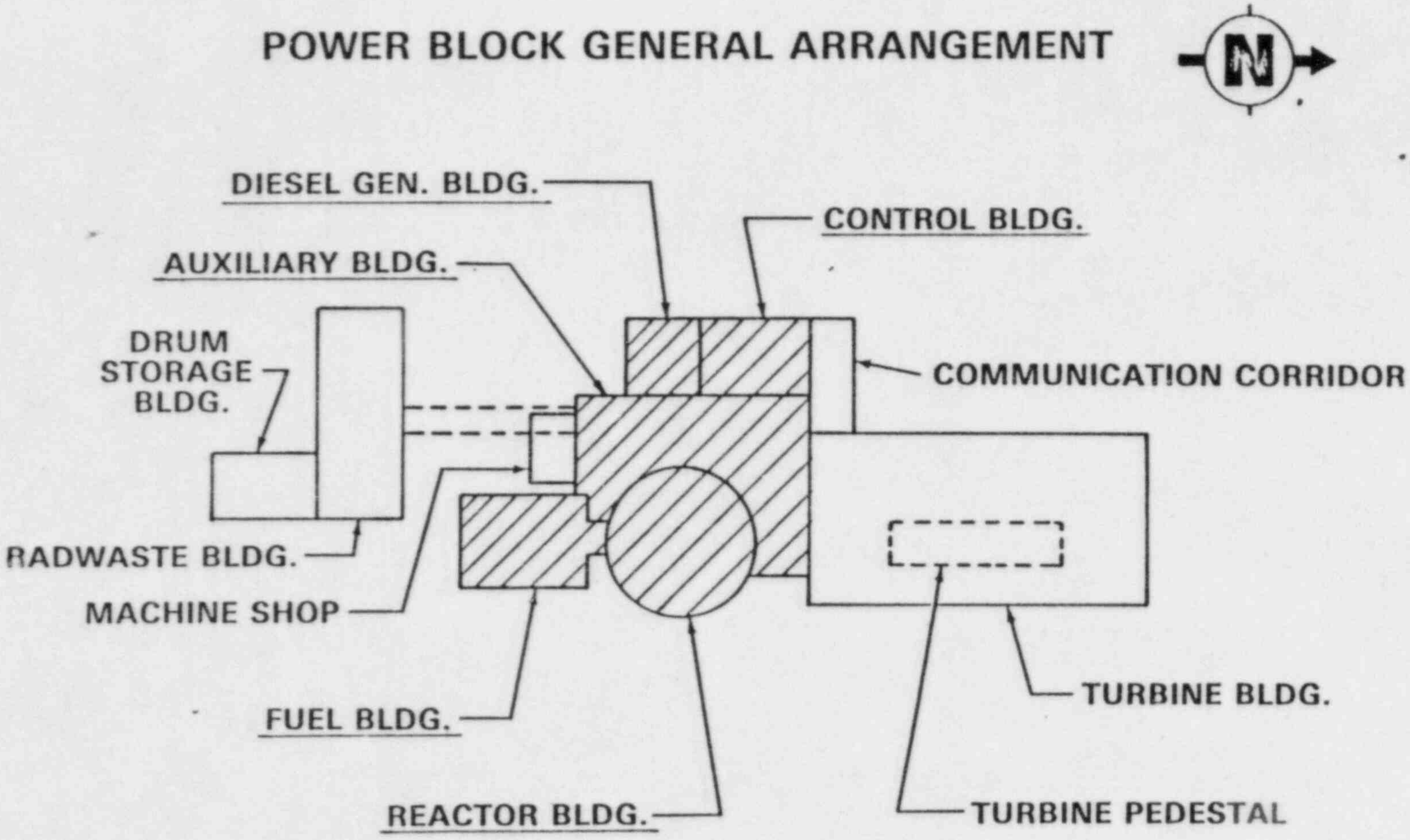
SUPPLEMENTAL INFORMATION

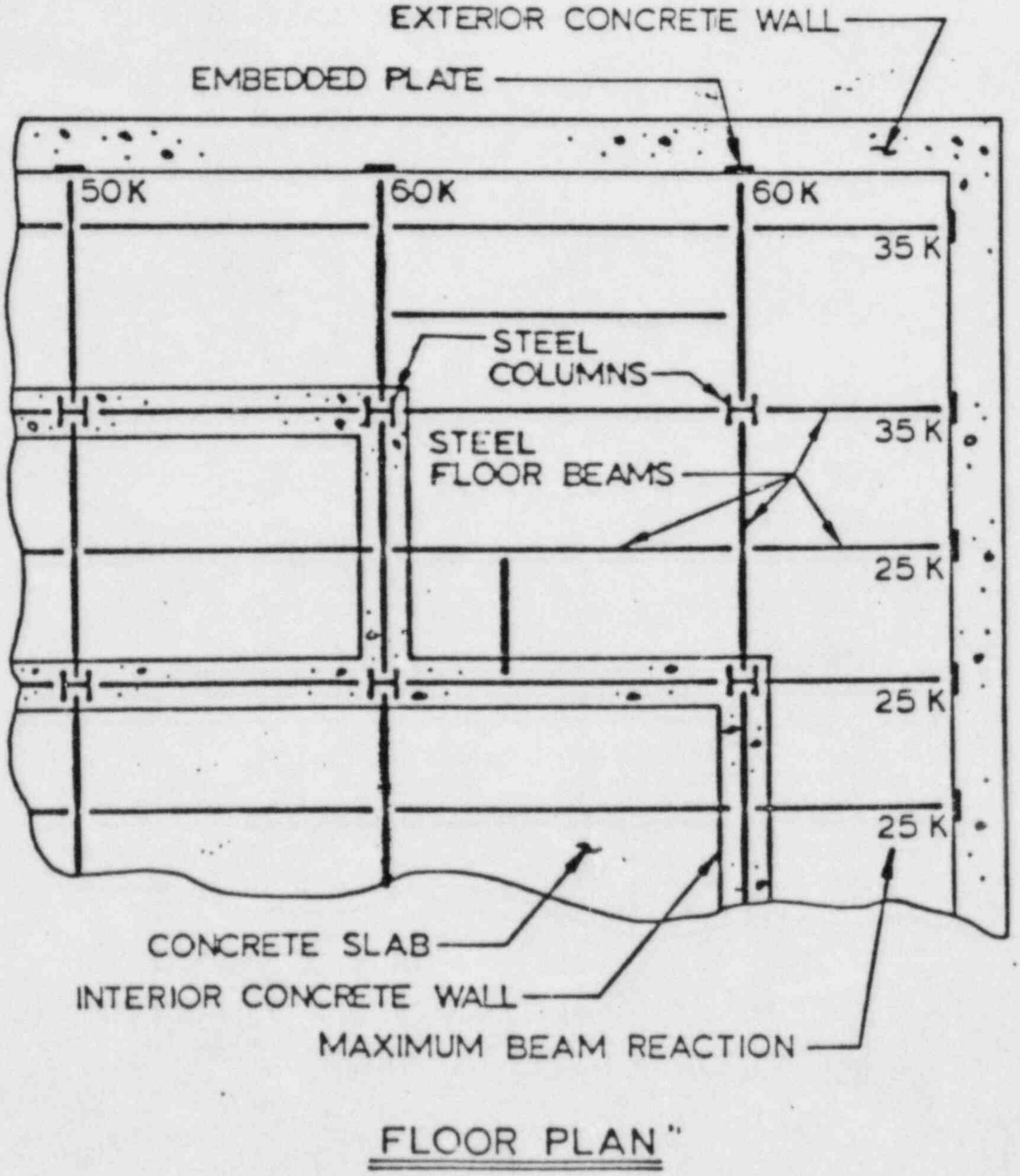
- February 14, 1985 - KMLNRC 85-054
- February 15, 1985 - KMLNRC 85-057
- February 18, 1985 - KMLNRC 85-058



PLANT SITE FEATURES

POWER BLOCK GENERAL ARRANGEMENT



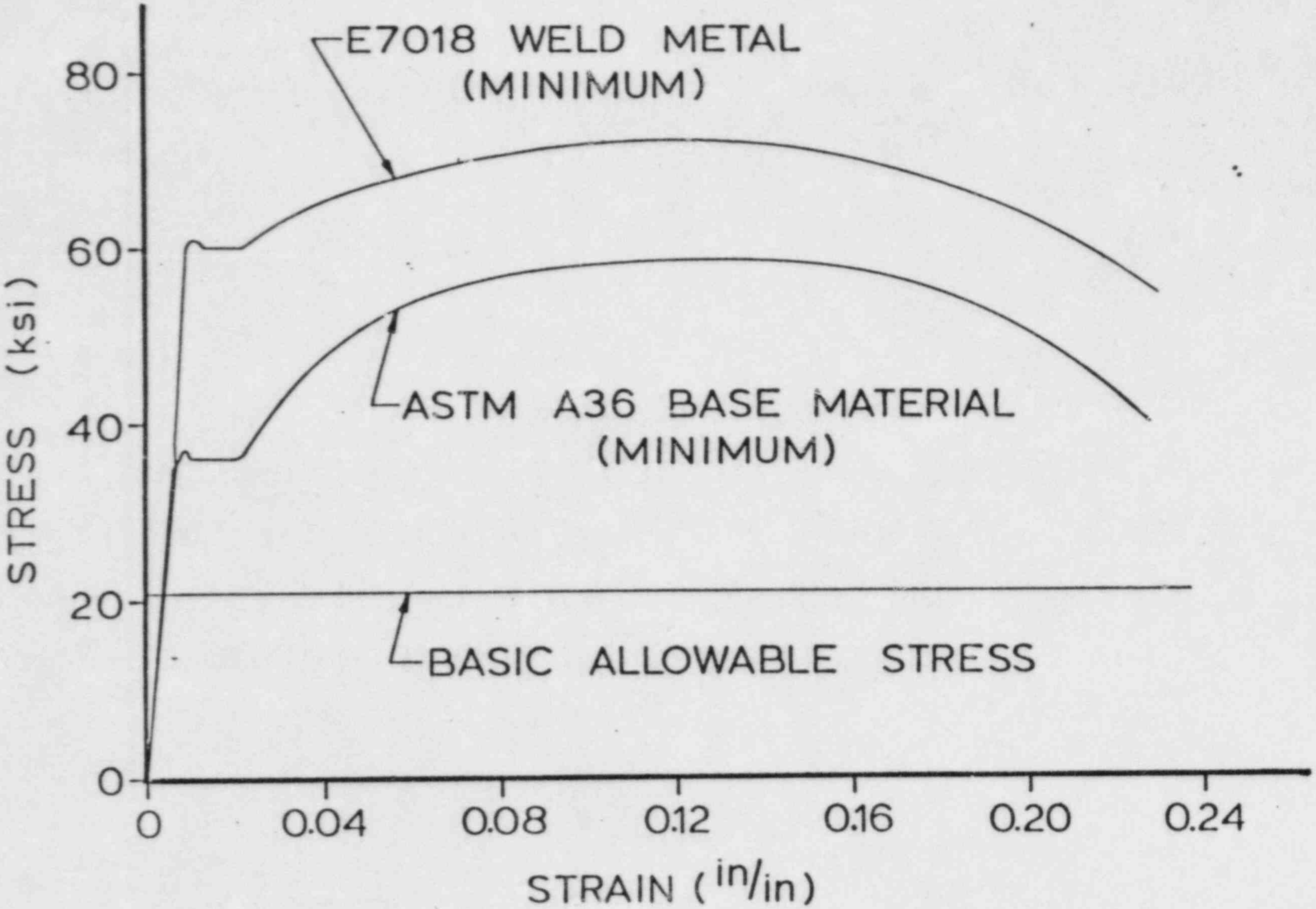

EXTERIOR CONCRETE WALL
EMBEDDED PLATE
50K
60K
60K
35 K
STEEL
COLUMNS
35 K
STEEL
FLOOR BEAMS
25 K
25 K
25 K
CONCRETE SLAB
INTERIOR CONCRETE WALL
MAXIMUM BEAM REACTION

FLOOR PLAN"

The design, fabrication, erection, and inspection of welded connections in structural steel for buildings are governed by the following standards:

- Structural Welding Code AWS D1.1, developed by the Structural Welding Committee of the American Welding Society (AWS)
- Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, developed by the American Institute of Steel Construction (AISC)

Allowable shear stresses for fillet welds are set at 30 percent of the weld metal ultimate tensile strength, whereas the ultimate shear strength is in the range of 65 to 75 percent of ultimate tensile strength.


E7018 WELD METAL
(MINIMUM)
ASTM A36 BASE MATERIAL
(MINIMUM)
BASIC ALLOWABLE STRESS
STRESS (ksi)
80
60
40
20
0
0
0.04
0.08
0.12
0.16
0.20
0.24
STRAIN (in/in)

Allowable stresses are specified at a level *below* ultimate capacity for several reasons, including the following:

- Load Definition
- Variations in Materials and Construction

SUMMARY
BASIC DESIGN MARGINS
STRUCTURAL STEEL
WELDED CONNECTIONS

- CONSERVATIVE CODE ALLOWABLES
- CONSERVATIVE DEFINITION OF LOADS
- CONSERVATIVE USE OF *MINIMUM MATERIAL* STRENGTHS
- MINIMIZED VARIATIONS IN MATERIALS AND CONSTRUCTION

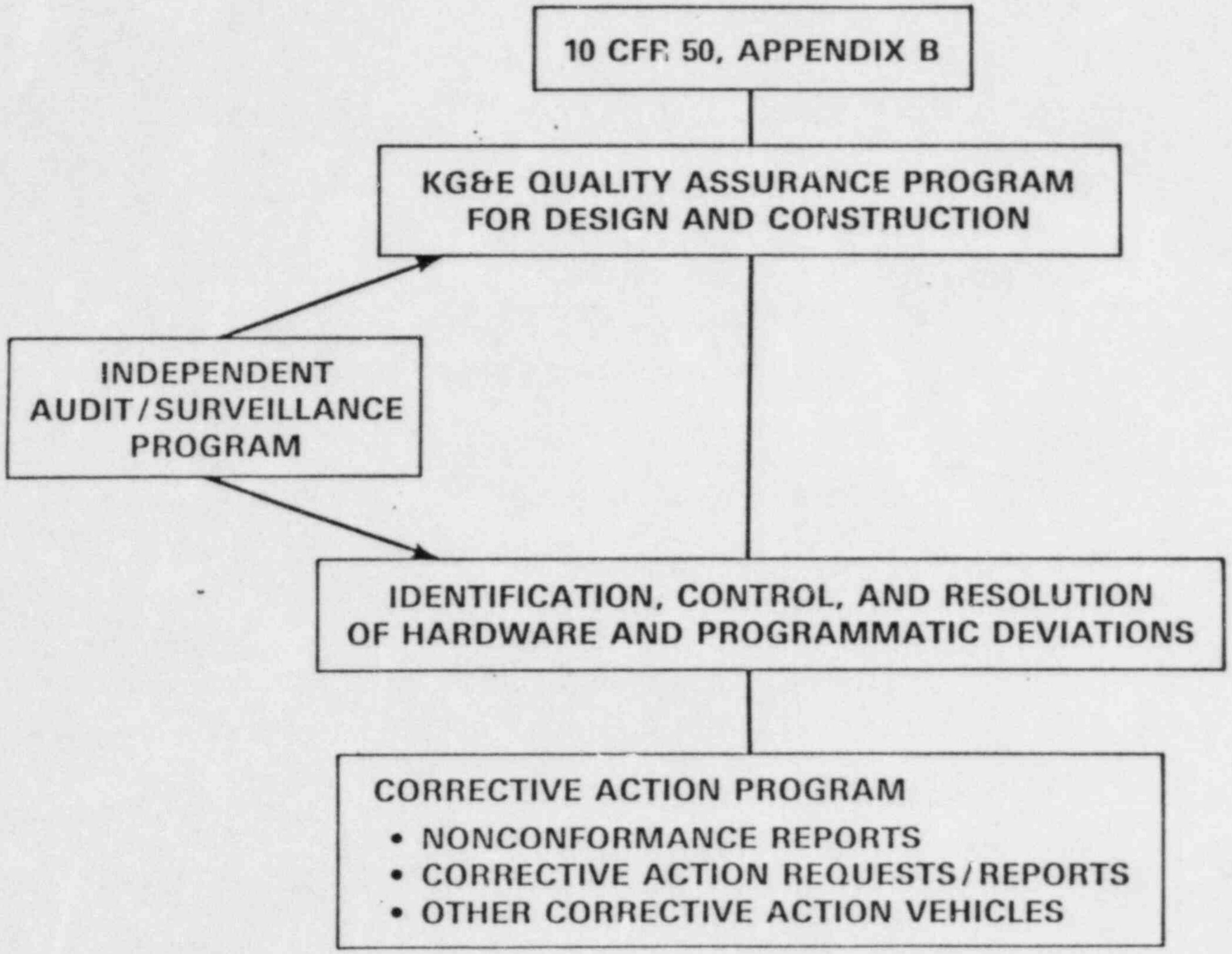
PLUS

- CONSERVATIVE ENVELOPING OF MULTISITE EARTHQUAKES
- CONSERVATIVE DESIGN METHODOLOGY
- CONSEQUENCE CONSIDERATIONS

EQUALS

LARGE FACTORS OF SAFETY AGAINST FAILURE

KG&E QUALITY ASSURANCE PROGRAM OVERVIEW



10 CFR 50, APPENDIX B

KG&E QUALITY ASSURANCE PROGRAM FOR DESIGN AND CONSTRUCTION

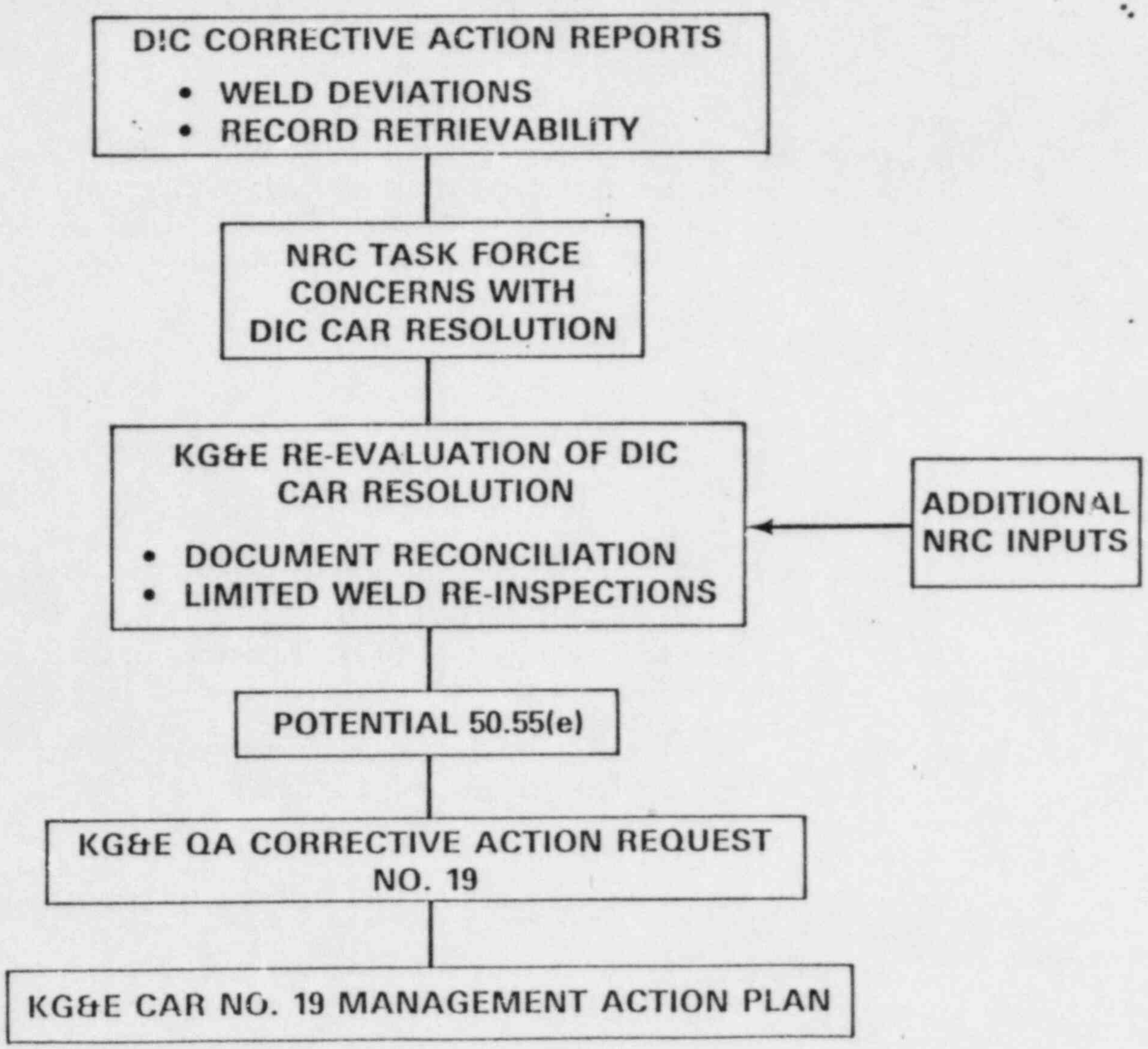
INDEPENDENT AUDIT/SURVEILLANCE PROGRAM

IDENTIFICATION, CONTROL, AND RESOLUTION OF HARDWARE AND PROGRAMMATIC DEVIATIONS

CORRECTIVE ACTION PROGRAM

- NONCONFORMANCE REPORTS
- CORRECTIVE ACTION REQUESTS/REPORTS
- OTHER CORRECTIVE ACTION VEHICLES

AWS D1.1 STRUCTURAL STEEL WELDING CONCERNS
BACKGROUND INFORMATION



DIC CORRECTIVE ACTION REPORTS

- WELD DEVIATIONS
- RECORD RETRIEVABILITY

NRC TASK FORCE CONCERNS WITH DIC CAR RESOLUTION

KG&E RE-EVALUATION OF DIC CAR RESOLUTION

- DOCUMENT RECONCILIATION
- LIMITED WELD RE-INSPECTIONS

ADDITIONAL NRC INPUTS

POTENTIAL 50.55(e)

KG&E QA CORRECTIVE ACTION REQUEST NO. 19

KG&E CAR NO. 19 MANAGEMENT ACTION PLAN

KG&E QA CORRECTIVE ACTION REQUEST NO. 19 PROGRAM OBJECTIVES

- DOCUMENT A CONSOLIDATED PROJECT PLAN
- ASSURE BY OBJECTIVE EVIDENCE THAT AWS D1.1 SAFETY-RELATED STRUCTURAL STEEL WELDING COMPLIES WITH ALL QUALITY CRITERIA
- ASSURE THAT INSPECTION DOCUMENTATION REFLECTS APPROPRIATE INFORMATION AND IS:
  - AVAILABLE
  - COMPLETE
  - TRACEABLE
- EVALUATE OTHER AWS D1.1 SAFETY-RELATED WELDING ACTIVITIES

KG&E QA CORRECTIVE ACTION REQUEST NO. 19 FINDINGS - OVERVIEW

- MISSING WELD RECORD DOCUMENTATION
- WELD DEVIATIONS
- WELDS NOT MADE/MISSING MATERIAL
- PRESENCE OF WELD INSPECTION DOCUMENTATION WITHOUT PRESENCE OF WELD
- VERIFICATION OF COMPLETED CORRECTIVE ACTION TO KG&E SURVEILLANCE REPORT S-372

KG&E CAR NO. 19 MANAGEMENT ACTION PLAN

QA VERIFICATION PROCESS

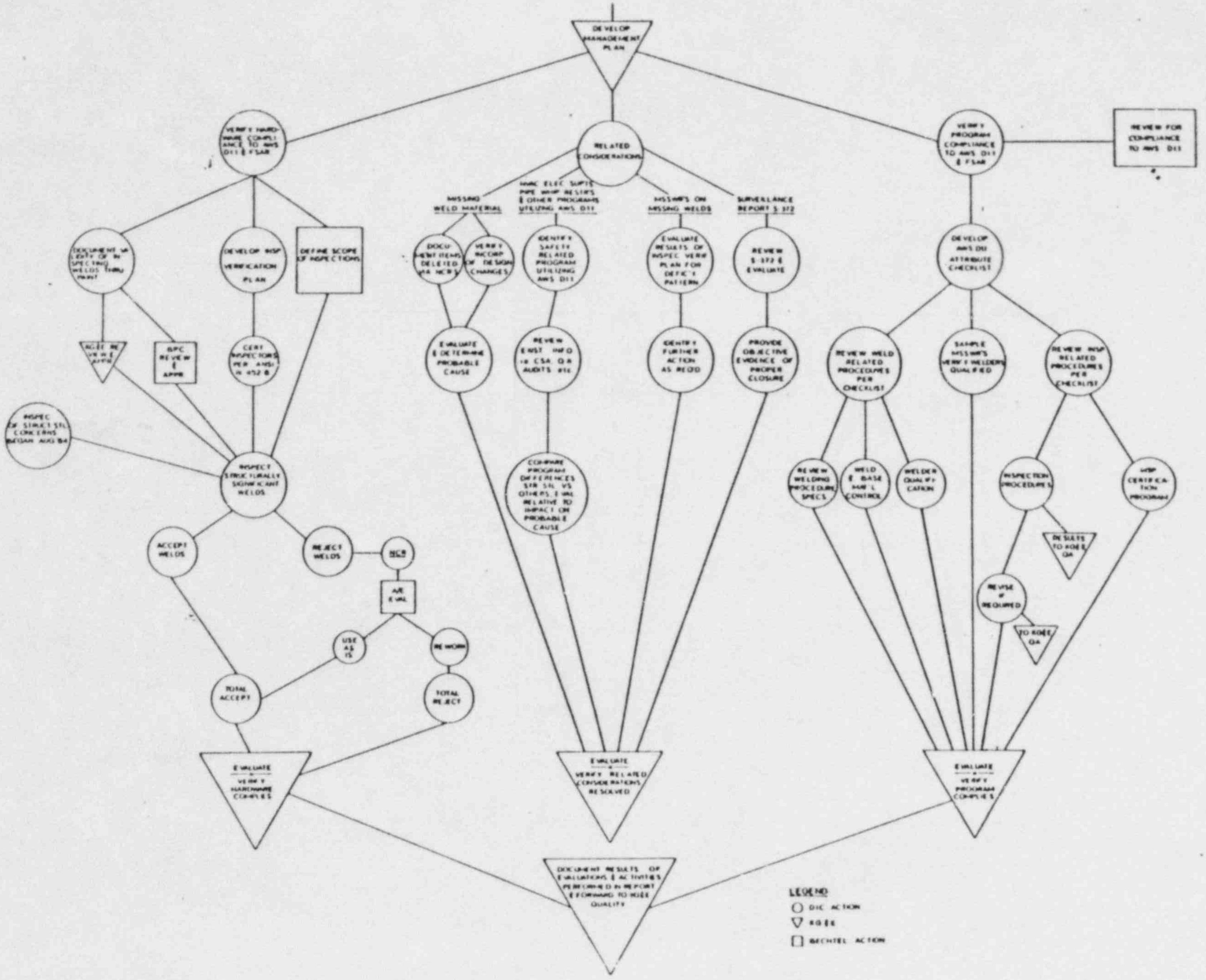
- TWO EXPERIENCED QA AUDITORS ASSIGNED ON A FULL-TIME BASIS
- IN-PROCESS SURVEILLANCES WERE PERFORMED
- A THOROUGH AUDIT OF EACH CORRECTIVE ACTION STEP WAS PERFORMED
- RESULTS OF THE AUDIT AND SURVEILLANCES:
  - CAR No. 19 Management Action Plan was Effective
  - CAR No. 19 Findings were Satisfactorily Resolved

KG&E QA CORRECTIVE ACTION REQUEST NO. 19

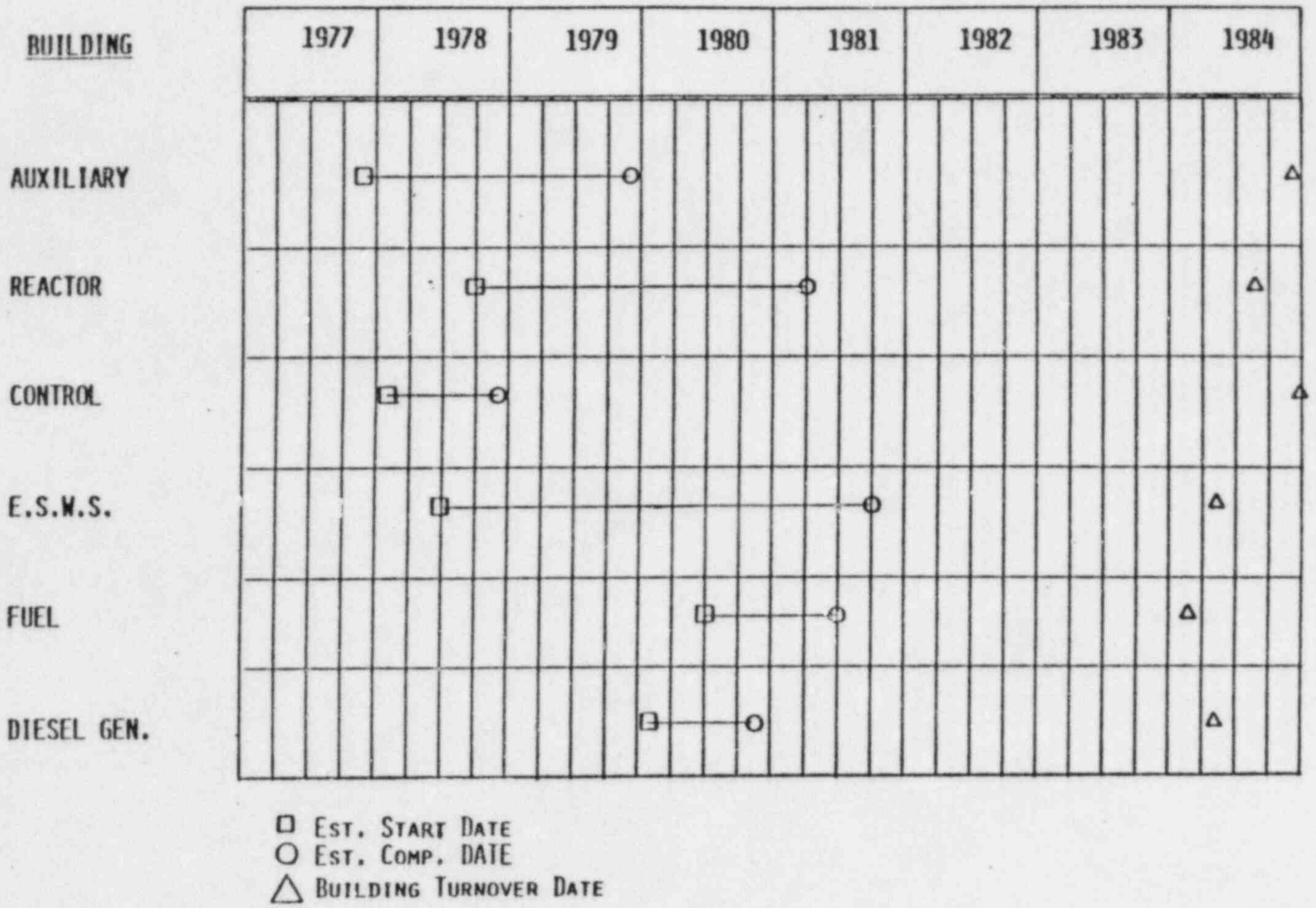
SUMMARY

- KG&E QA CAR 19 RECOMMENDED CORRECTIVE ACTIONS - READILY ADOPTED
- KG&E MANAGEMENT ACTION PLAN - EXCEEDED CAR 19 RECOMMENDATIONS THUS PROVIDING A MORE COMPREHENSIVE TREATMENT OF AWS D1.1 WELDING CONCERNS
- RE-INSPECTION OF VIRTUALLY *ALL* SIGNIFICANT SAFETY-RELATED STRUCTURAL STEEL WELDING - WITH AND WITHOUT RECORDS
- EVALUATION OF OTHER AWS D1.1 SAFETY-RELATED WELDING PROGRAMS
- EVALUATION OF OTHER SAFETY-RELATED PROGRAMS BEYOND AWS D1.1

LOGIC CHART FOR RESOLUTION CAR 19



SUMMARY OF STRUCTI STEEL ERECTION



AWS D.1.1-75

- DESIGN OF WELDED CONNECTIONS
- WORKMANSHIP
- FILLER METAL REQUIREMENTS
- WELD PROCEDURE QUALIFICATION
- WELDER QUALIFICATIONS
- INSPECTION

MISCELLANEOUS STRUCTURAL STEEL WELD RECORDS

MSSWR

- DRAWING NUMBER
- JOINT NUMBER
- AREA / LOCATION
- BASE MATERIAL PIECE OR HEAT NUMBER
- ROD WITHDRAWAL DATA
- FILLER MATERIAL HEAT NUMBER / LOT NUMBER
- WELD PROCEDURE
- WELDER IDENTIFICATION NUMBER
- QUALITY INSPECTOR

WELD ATTRIBUTES TO BE INSPECTED PER AWS D1.1-75

- PRESENCE
- LOCATION
- LENGTH
- SIZE
- UNDERCUT
- CRACKS
- CRATERS
- FUSION
- PROFILE
- OVERLAP
- POROSITY
- ARC STRIKES
- SLAG
- SPATTER

WELDING HISTORY SUMMARY

- ERECTION/WELDING PERFORMED IN 1977-1981
- WELDING PROGRAM WAS IN ACCORDANCE WITH AWS D.1.1-1975

CAR 19 MANAGEMENT PLAN PROGRAMMATIC REVIEW

- WELDERS QUALIFIED IN ACCORDANCE WITH AWS D.1.1-75
- WELDING PROCEDURES IN ACCORDANCE WITH AWS D.1.1-75
- FILLER MATERIAL PURCHASE AND CONTROL IN ACCORDANCE WITH AWS D.1.1-75
- INSPECTION CRITERIA COMPLIED WITH AWS D.1.1-75
- INSPECTORS CERTIFIED TO ANSI 45.2.6
- DOCUMENTATION IN ACCORDANCE WITH AWS D.1.1 AND ANSI 45.2
- KG&E SURVEILLANCE REPORT S-372 CLOSURE VERIFICATION

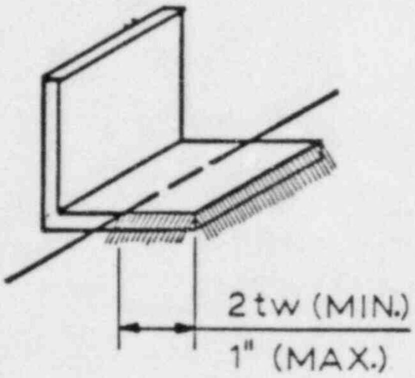
CAR 19 MANAGEMENT PLAN WELDING HARDWARE REVIEW

- DEVELOPMENT OF SECONDARY INSPECTION PROCEDURES
- CERTIFICATION OF INSPECTORS
- IDENTIFICATION OF STRUCTURALLY SIGNIFICANT JOINTS BY ENGINEER
- VALIDITY OF INSPECTION IN PRESENCE OF PAINT
- FIREPROOFING REMOVAL
- INSPECTION OF STRUCTURALLY SIGNIFICANT JOINTS

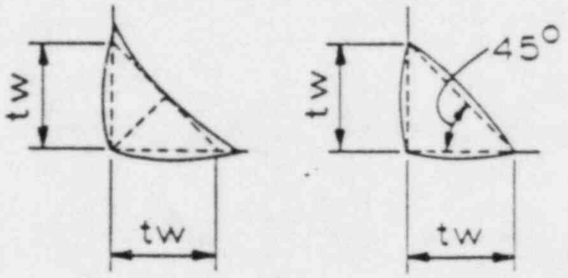
CAR 19 MANAGEMENT PLAN WELDING HARDWARE REVIEW

(Continued)

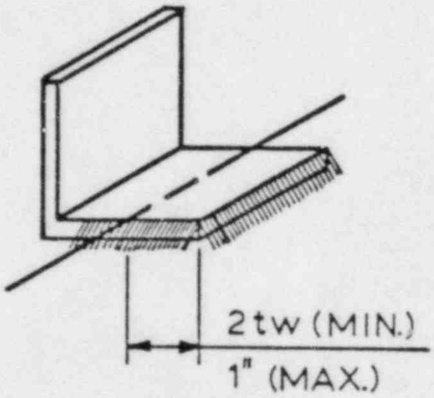
- INVESTIGATION OF MISSING WELDS WITH PRIMARY RECORDS
- DOCUMENTING CONSTRUCTED CONFIGURATION OF JOINTS
- EVALUATION OF CONSTRUCTED CONFIGURATION BY THE ENGINEER
- REWORKING JOINTS
- ISSUANCE OF SUMMARY REPORT



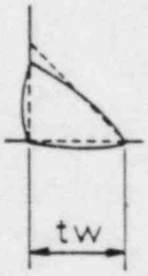
ACCEPTABLE RETURN WELD



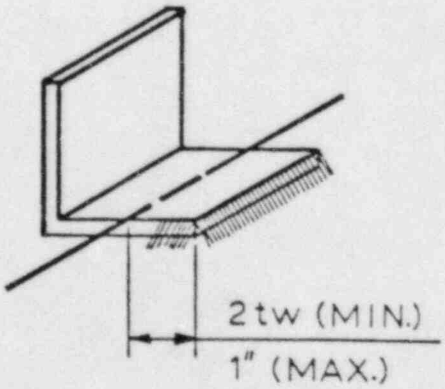
ACCEPTABLE PROFILES



OVERRUN



UNDERSIZE



UNDERRUN

tw = REQUIRED WELD LEG

CAR 19 MANAGEMENT PLAN CONCLUSIONS

- QA PROGRAM DEFICIENCIES CONFINED TO CAR 19 ISSUES
- PRESENCE OF WELD INSPECTION DOCUMENTATION WITHOUT PRESENCE OF WELDING WAS CAUSED BY HUMAN ERROR
- WELD RECORD RETRIEVABILITY PROBLEMS DID NOT CARRY OVER TO OTHER PROGRAMS
- WELDING PROGRAM IS IN ACCORDANCE WITH AWS D.1.1-75
- ALL QUALITY CRITERIA AS SPECIFIED IN THE RELATED DESIGN DOCUMENTS ARE MET
- ALL STRUCTURAL STEEL ERECTION COMMITMENTS IN THE WOLF CREEK FSAR ARE SATISFIED

Structurally significant AWS field welded joints are joints which:

1) support or potentially support safety-related equipment and building components,
2) are located in the Reactor Building, Auxiliary Building, Control Building, Diesel Generator Building, Fuel Building, or Essential Service Water System Pumphouse,
3) were installed under the structural steel erection contract (Bechtel Specification 10466-C122) or the miscellaneous steel erection contract (Bechtel Specification 10466-C132), and
4) were originally inspected under the Daniel International Corporation (DIC) "Miscellaneous/ Structural Steel Weld Records" (MSSWR) Inspection Program.

WELD ATTRIBUTES TO BE INSPECTED PER AWS D1.1-75

- PRESENCE
- LOCATION
- LENGTH
- SIZE
- UNDERCUT
- CRACKS
- CRATERS
- FUSION
- PROFILE
- OVERLAP
- POROSITY
- ARC STRIKES
- SLAG
- SPATTER

REINSPECTION DATA AWS STRUCTURAL STEEL WELDING AT WOLF CREEK

Structurally Significant Joints	2,670
Totally Inaccessible Joints	119
Reinspected Joints	2,551
Unpainted Joints	1,043
Joints Requiring Rework[1]	82
Additional Joints Reworked[2]	67
Significantly Deficient Joints (10CFR50.55(e))	0

(1) DESIGN ALLOWABLE STRESSES ARE EXCEEDED IN THE AS-BUILT CONDITION.

(2) DESIGN ALLOWABLE STRESSES ARE NOT EXCEEDED IN THE AS-BUILT CONDITION. THESE JOINTS ARE BEING REWORKED PER KG&E MANAGEMENT DIRECTION TO INSTALL MISSING AND UNDERLENGTH WELDS UNLESS PROHIBITED BY FIELD CONDITIONS.

2/27/85

Figure A

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

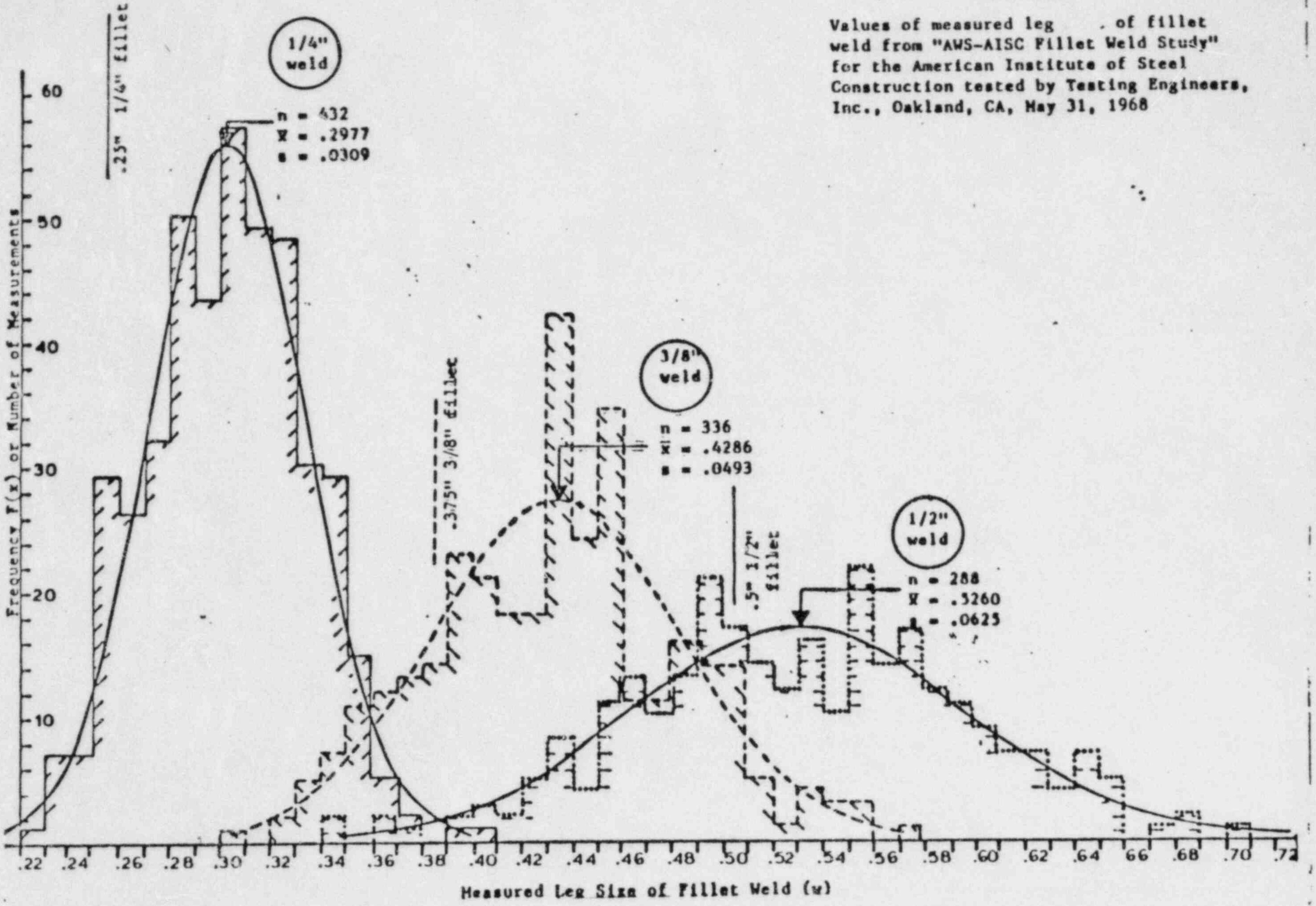


FIGURE D

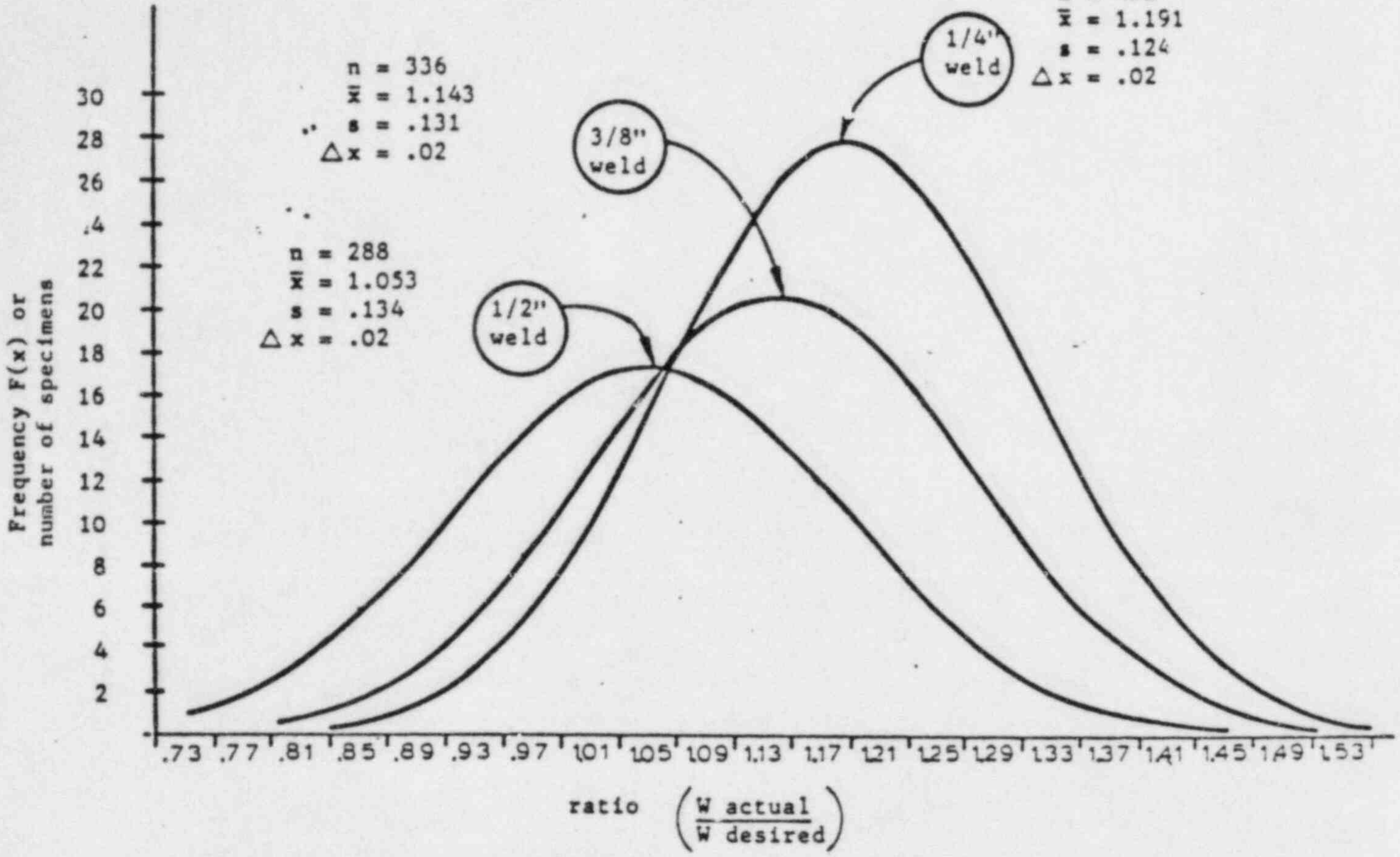
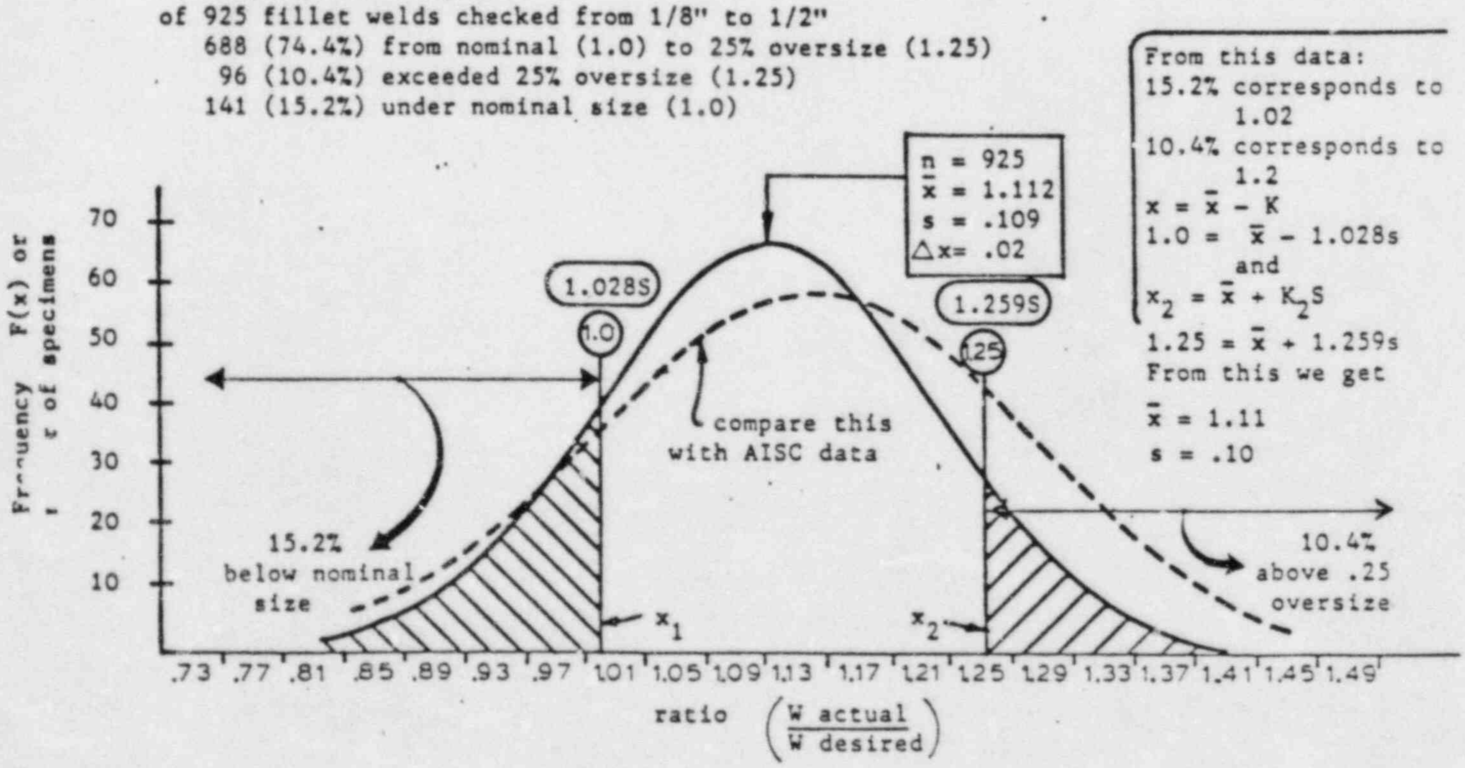
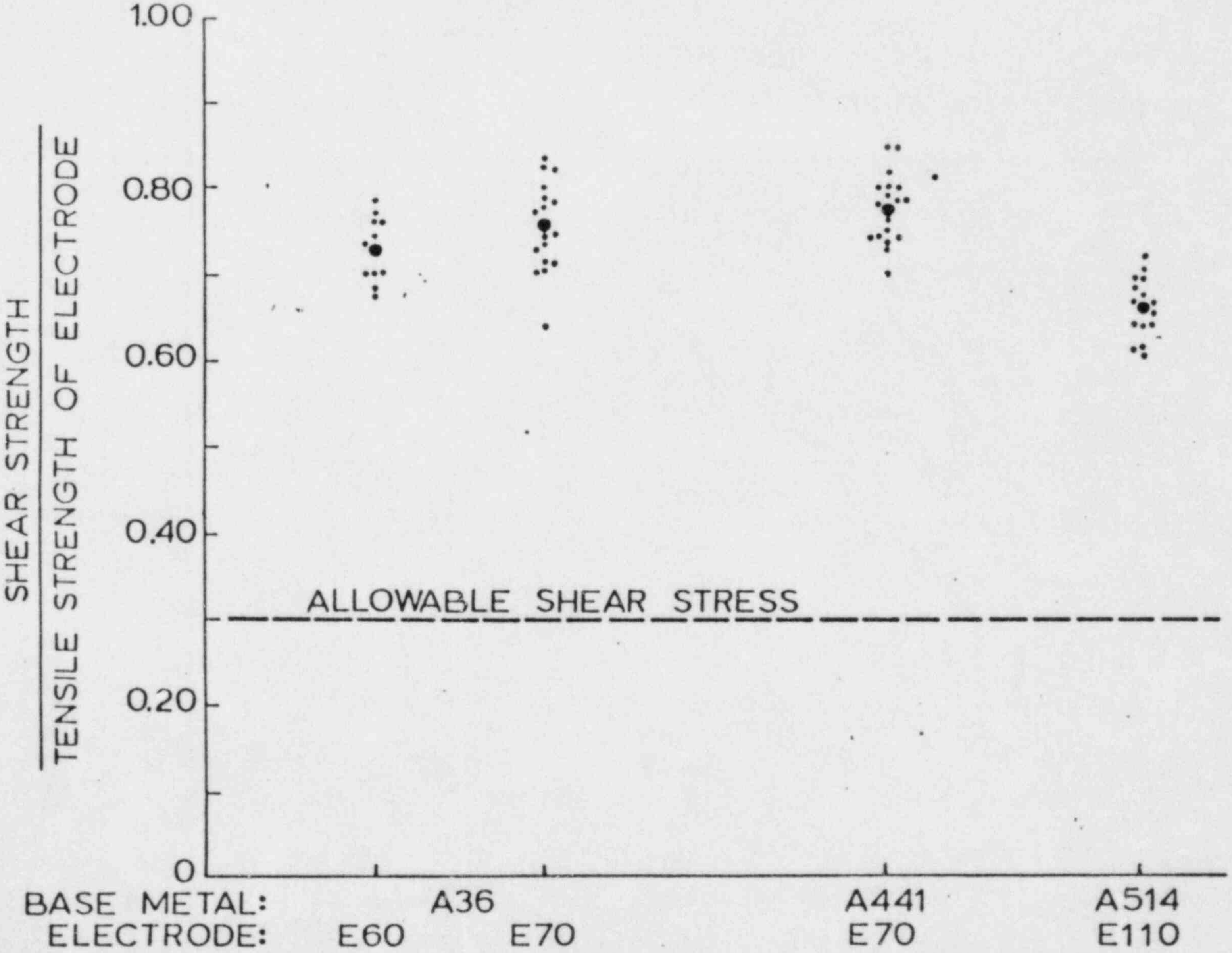


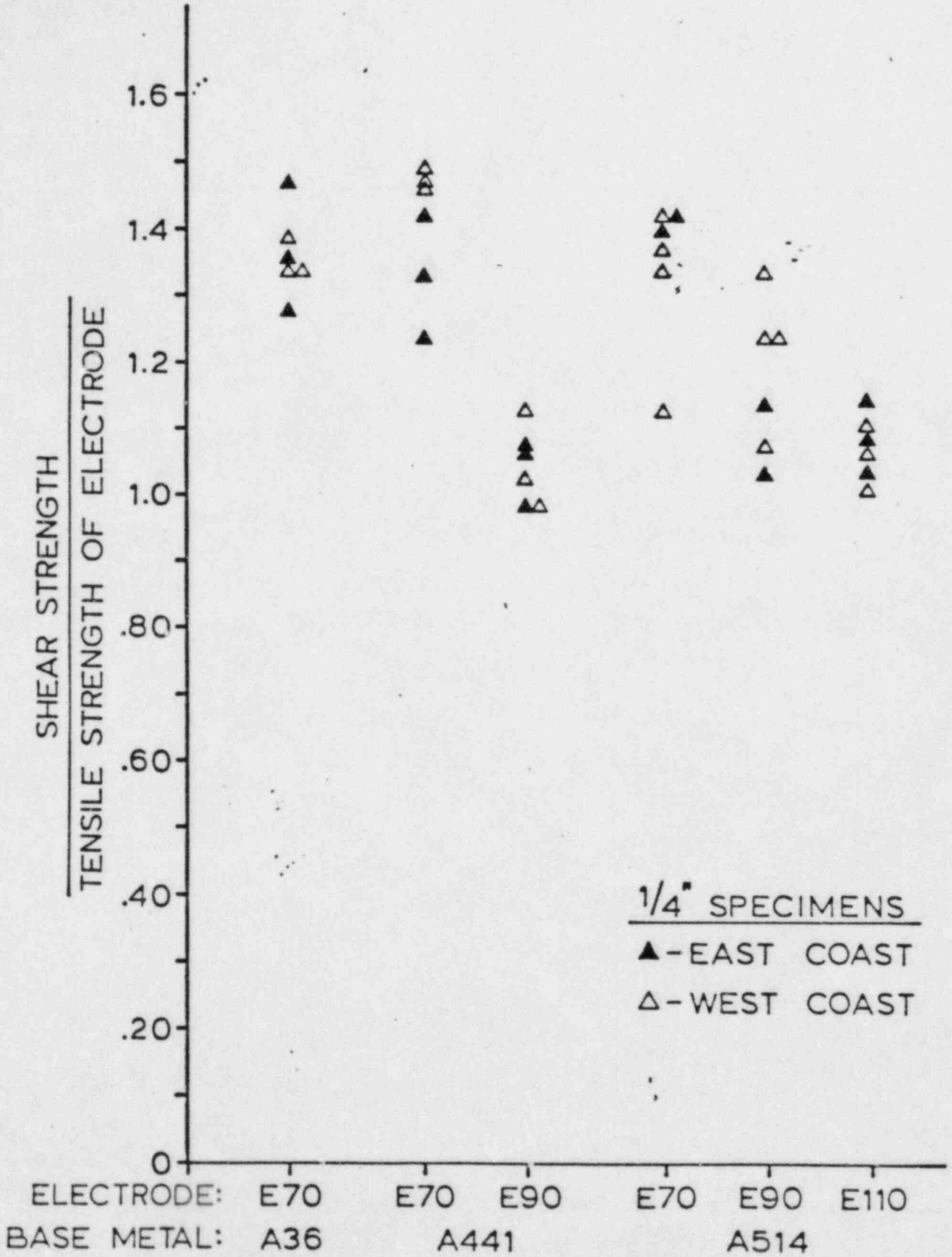
FIGURE E

Correspondence from Mr. W. C. Cadwell, Asst. Ch. Eng. of Caterpillar Tractor Co.
Peoria, IL 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 1.02
10.4% corresponds to 1.2
$x = \bar{x} - K$
$1.0 = \bar{x} - 1.028s$
and
$x_2 = \bar{x} + K_2S$
$1.25 = \bar{x} + 1.259s$
From this we get
$\bar{x} = 1.11$
$s = .10$



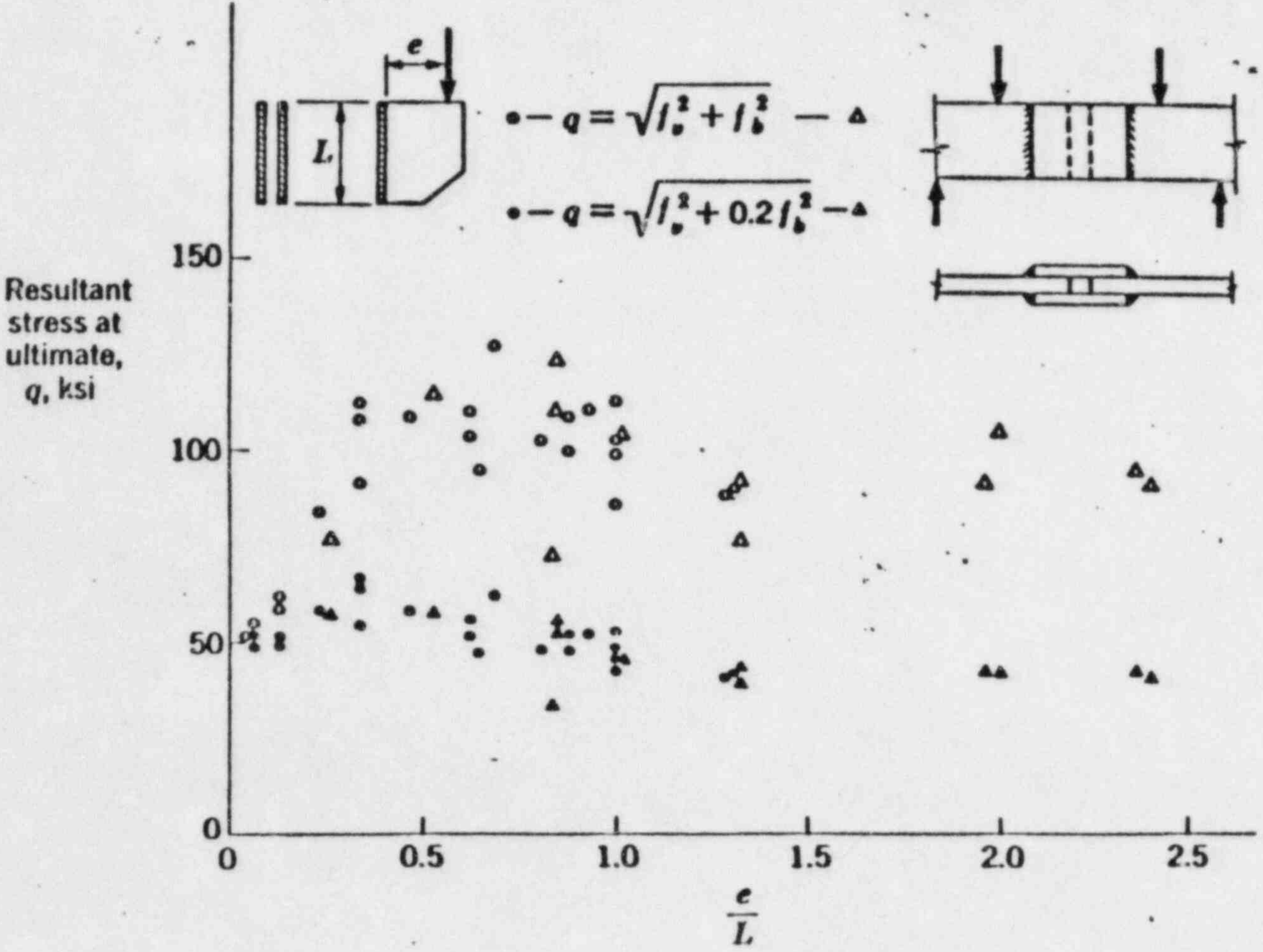
SHEAR STRENGTH OF LONGITUDINAL FILLET WELDS WITH MATCHED·BASE MATERIAL

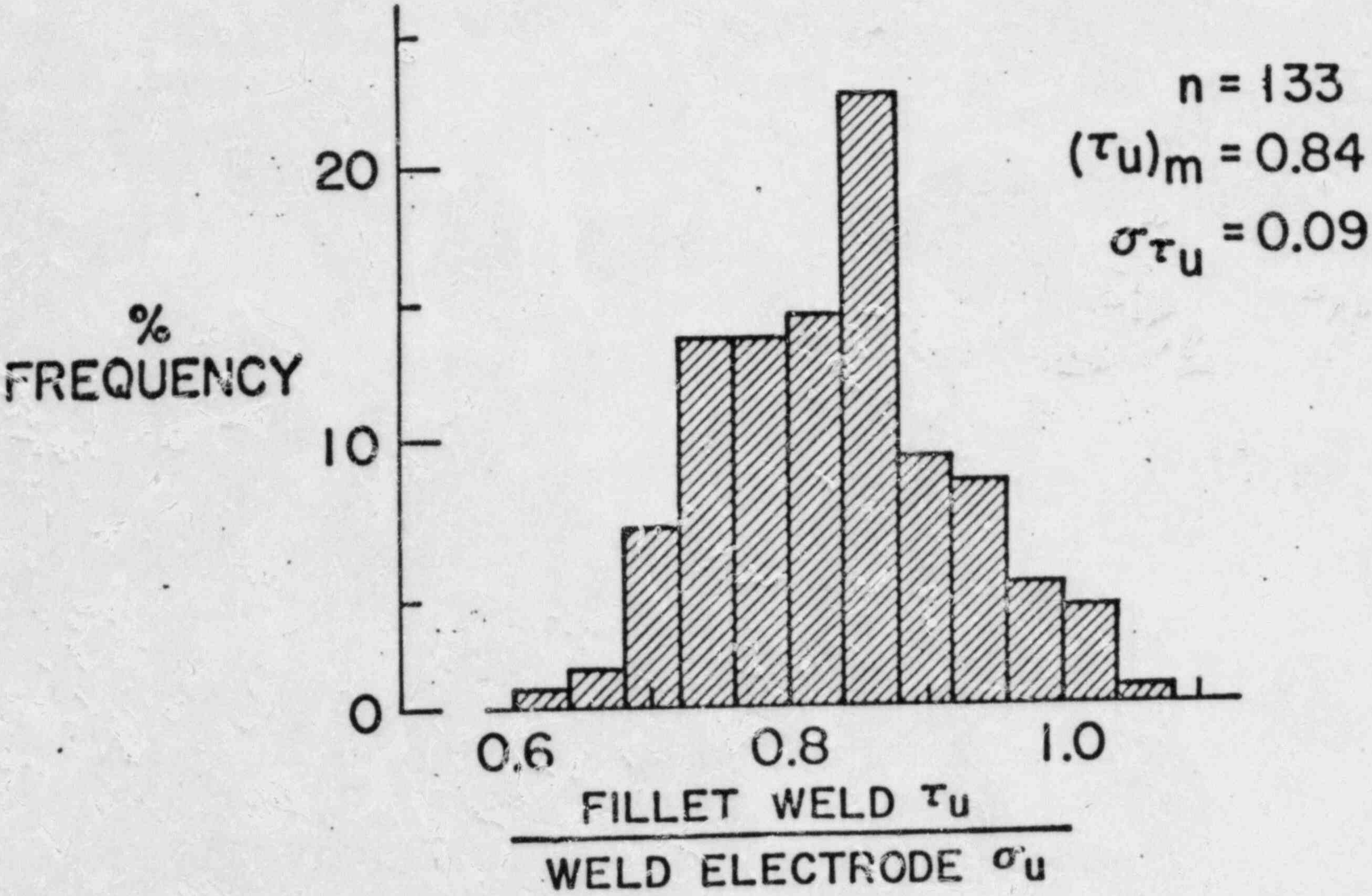


SHEAR STRENGTH OF TRANSVERSE FILLET WELDS WITH MATCHED BASE METAL

WELDS SUBJECTED TO BENDING AND SHEAR

(COURTESY CIVIL ENGINEERING AND PUBLIC WORKS REVIEW)




n = 133
$(\tau_u)_m = 0.84$
$\sigma_{\tau_u} = 0.09$
% FREQUENCY
20
10
0
0.6
0.8
1.0
$\dfrac{\text{FILLET WELD } \tau_u}{\text{WELD ELECTRODE } \sigma_u}$

OBJECTIVE

TO INDEPENDENTLY EVALUATE KG&E's APPROACH TO THE RESOLUTION OF CORRECTIVE ACTION REQUEST (CAR) NUMBER 19 AND MAKE RECOMMENDATIONS FOR A TIMELY CLOSEOUT OF CAR 19

ACTIVITIES

1) FINAL REPORT REVIEW (KG&E REPORT)
2) SITE VISIT (FEBRUARY 15-17)
3) REVIEW OF SUPPORTING DOCUMENTS
    - Weld Procedures
    - Filler Metal
    - DIC Inspection Criteria
    - Reinspection Validation (Painted)
4) WELD INSPECTION OF PAINTED AND UNPAINTED WELDS IN THE AUXILIARY AND REACTOR BUILDINGS
5) DISCUSSIONS WITH KG&E, DIC, AND BECHTEL PERSONNEL
6) PREPARATION OF REPORT

RESULTS

- RELATED WELDING ACTIVITIES ARE SOUND AND DOCUMENTED
- REINSPECTION PROGRAM HAS BEEN EXTENSIVE, PROPERLY PERFORMED, AND DOCUMENTED
- VALIDATION OF INSPECTION WITH PAINT HAS BEEN COMPLETED
- IMPERFECTIONS NOTED IN REINSPECTION ARE TYPICAL FOR C/Mn STRUCTURAL WELDING
- *NO SAFETY SIGNIFICANCE OF THE IMPERFECTIONS*

CONCLUSIONS

- REINSPECTION PROGRAM IS SOUND AND EFFECTIVE, AND ENSURES AWS D1.1 QUALITY WELDS
- IMPERFECTIONS ARE MINOR AND STRUCTURAL INTEGRITY IS ASSURED

SUMMARY BY GLENN L. KOESTER - 2/27/85

KG&E HAS ALWAYS HAD, AND CONTINUES TO HAVE A FIRM COMMITMENT TO PROTECT THE HEALTH AND SAFETY OF THE PUBLIC AS WELL AS OUR OWN EMPLOYEES. THAT IS WHY WE UNDERTOOK SUCH AN EXTENSIVE PROGRAM TO EVALUATE THE ACCEPTABILITY OF THE STRUCTURAL STEEL WELDING AT WOLF CREEK. AS YOU HEARD EARLIER, OUR REINSPECTION EFFORTS FOUND SEVERAL MINOR DEVIATIONS THAT GAVE THE APPEARANCE OF A HIGHER THAN EXPECTED REJECT RATE. HOWEVER, THE PRIMARY REASON FOR THESE REJECTS RESULTED FROM THE "NO TOLERANCE" INSPECTION PHILOSOPHY DISCUSSED BY MR. REEDY. THE VAST MAJORITY OF THESE DEVIATIONS WOULD NOT BE REJECTED BY A QUALIFIED AWS INSPECTOR AT ANOTHER FACILITY UNLESS THEY WERE MAKING THE SAME TYPE SECONDARY INSPECTION THAT WE MADE. THE FACT THAT KG&E TOOK A MORE CONSERVATIVE APPROACH DURING THE REINSPECTION EFFORTS DOES NOT IN ANY WAY INVALIDATE THE INITIAL WELD INSPECTIONS.

AS DISCUSSED EARLIER, THE REINSPECTIONS DID IDENTIFY A FEW JOINTS IN WHICH SOME WELDS HAD NOT BEEN MADE. THESE PRIMARILY RESULTED FROM A MISINTERPRETATION OF THE WELD DETAIL AND NOT FROM GROSS INADEQUACIES IN THE INSPECTION PROGRAM. WHILE WE STRIVE FOR PERFECTION, WE MUST RECOGNIZE THAT HUMAN ERRORS CAN AND DO OCCUR. THAT IS ONE REASON WHY WE DESIGN AND BUILD THESE PLANTS WITH SO MUCH CONSERVATISM. THIS IS DEMONSTRATED BY THE FACT THAT NONE OF THE JOINTS WITH MISSING WELDS WOULD HAVE FAILED. A POINT THAT NEEDS TO BE EMPHASIZED IS THAT WE MEAN IT WOULD NOT HAVE FAILED UNDER THE WORST POSTULATED LOADING CONDITIONS. THIS WOULD INCLUDE NORMAL LOADING PLUS ANY LOADS RESULTING FROM A POSTULATED WORST CASE ACCIDENT.

OUR PRIMARY OBJECTIVE IN THE OVERALL CORRECTIVE ACTION PROGRAM DISCUSSED EARLIER WAS TO ASSURE THAT WOLF CREEK IS STRUCTURALLY SOUND AND WILL NOT FAIL UNDER THE WORST POSTULATED ACCIDENT CONDITIONS.

WE HAVE DONE THAT.

IN DOING SO, WE ALSO REAFFIRMED THAT THE AWS WELDING WAS DONE IN ACCORDANCE WITH THE APPLICABLE CODES.

WE DID NOT LIMIT OUR REVIEW OF THIS MATTER TO WELDING ALONE. WE ALSO LOOKED AT OTHER AREAS TO ASSURE THEY WERE COMPLETED IN ACCORDANCE WITH APPLICABLE REQUIREMENTS AND IN A MANNER THAT PROVIDES ADEQUATE PROTECTION OF THE HEALTH AND SAFETY OF THE PUBLIC.

WE ALSO HAD THREE OF THE LEADING AUTHORITIES IN STRUCTURAL STEEL WELDING INDEPENDENTLY REVIEW OUR PROGRAM TO ASSURE THAT WE WERE NOT TAKING A BIASED LOOK AT OURSELVES. AS YOU HEARD FROM THEIR DISCUSSIONS TODAY, FROM THEIR REVIEW OF THE VARIOUS ASPECTS OF OUR PROGRAM, WE DID A VERY THOROUGH, CONSERVATIVE, ASSESSMENT OF OUR AWS WELDING PROGRAM AND THEY FOUND NOTHING TO QUESTION OR INVALIDATE THE CONCLUSIONS WE HAVE MADE.

I SINCERELY BELIEVE THAT ANYONE KNOWLEDGEABLE IN ENGINEERING AND CONSTRUCTION PRACTICES WOULD HAVE TO AGREE THAT KG&E'S CORRECTIVE ACTION PROGRAM VERIFIED THAT THE STRUCTURAL STEEL AT WOLF CREEK GENERATING STATION IS SAFE AND SOUND.

THIS COMPLETES OUR PRESENTATION ON AWS STRUCTURAL STEEL WELDING AT WOLF CREEK. WE FIRMLY BELIEVE THE RECORD IS CLEAR AND <u>WE</u> ARE READY TO RECEIVE OUR OPERATING LICENSE AND COMMENCE LOADING FUEL AND PROCEED THROUGH POWER ASCENSION.

ENCLOSURE 3

AWS D1.1 STRUCTURAL STEEL WELDING CONCERNS

BACKGROUND INFORMATION

DATE	EVENT	COMMENTS
Sept. 1980	UNDERSIZED SOCKET WELDS	. Concern identified on small bore piping at another project. Performed sample inspection.
Sept. 1980	DIC CAR #7	. 100% reinspection of socket welds on small bore piping made prior to 6/80.
Mar. 1980	MECHANICAL/STRUCTURAL/ ELECTRICAL DEFICIENCY REPORTS	. Mechanical/Structural closed in May . Electrical addressed by CAR #9
Sept. 1981	KG&E QA Surv. Rpt. S-372	. Adverse trend associated with missing electrical support weld inspection documentation.
Sept. 1981	DIC CAR #9	. Provided corrective actions for KG&E Surv. Report S-372.
Aug. 1982 Feb. 1984	DIC CAR #19	. 100% Reinspection of fillet welds made prior to 4/1/81 on ASME and Special Scope PIPE hangers.
Feb. 1983	Random Reinspection Structural Steel Fillet Welds	. Inspection performed in all Q-building Unacceptable percentage of welds are deficient in the Auxiliary, Control & Fuel Buildings.
Mar. 1983	DIC CAR #29	. Obtain corrective actions of deficient welds noted above.
Aug. 1983	DIC CAR #31	. Not all MSSWR's can be located for "Q" welds in the Fuel, Reactor & ESWS Pumphouse. . NCR's generated for each safety related building or area with missing MSSWR's.

DIC CAR #29 → Potential 50.55(e) (Mar. 1983) → Withdraw Potential 50.55(e) (Oct. 1983)

DOCUMENT RECONCILIATION TASK 8/13/84

INSPECTION VERIFICATION PLAN 8-17-84

POTENTIAL 50.55(e) 9-18-84
KG&E QA CAR #19

ENCLOSURE 4

KG&E LETTERS

&

SURVIELLENCE

KG&E LETTERS

Mr. Rathbun

FORM 110

KGE
THE ELECTRIC COMPANY

KANSAS GAS AND ELECTRIC COMPANY
P. O. Box 208 Wichita, Kansas 67201

OCT 1984
RECEIVED
COMPLIANCE

OCT 1984
NUCLEAR
DEPT.

[illegible]	
[illegible]	X
[illegible]-A64	X
1E40675-K152	X
1E[illegible]410-[illegible]04	X
[illegible]	
Cher[illegible]	X
Green	
[illegible]ll	
[illegible]h	
Maynard	
Rathbun	
W[illegible]holz	
Chronological (2)	X
Creel	X

October 17, 1984

Mr. D.R. Hunter, Chief
Reactor Project Branch 2
U.S. Nuclear Regulatory Commission
Region IV
611 Ryan Plaza Drive, Suite 1000
Arlington, Texas 76011

COPY

KMLNRC 84-187
Re: Docket No. STN 50-482
Subj: Interim 10CFR50.55(e) Report - Inspection
of Structural Steel Welds

Dear Mr. Hunter:

This letter provides an interim 10CFR50.55(e) report concerning inspections of structural steel welding at Wolf Creek Generating Station. This matter was initially reported by Mr. Otto Maynard of Kansas Gas and Electric Company (KG&E) to Mr. Lawrence Martin of the Nuclear Regulatory Commission, Region IV, on September 18, 1984.

During a review of safety-related structural steel weld inspection records, it was determined that 22% of the Miscellaneous Structural Steel Weld Records (MSSWR) could not be retrieved. Initially it was believed that the problem was limited to the retrievability of weld records and did not represent a hardware concern. In an attempt to confirm that hardware was not affected, a sample of structural steel welds were reinspected. This inspection verification effort identified some welds that did not meet current acceptance criteria for AWS D1.1 welding. The deficiencies were categorized as follows:

- Undersized welds
- Weld defects
- Incorrect configuration
- Weld underrun
- Weld undercut

A small number of safety-related structural steel welds were missing or had missing material. However, only one missing weld was identified for which an inspection record existed. This weld had been inadvertently included on an inspection record that included several other welds.

8411060571 2pp.

Due to the above findings, KG&E is initiating an extensive corrective action program to address the above findings and to take the actions necessary to assure the adequacy of the structural steel welding at Wolf Creek. This plan includes a significant reinspection effort to identify (and rework if necessary) nonconforming structural steel welds. Details of the corrective action program will be provided to the on-Site NRC Task Force.

A final report on this issue will be submitted upon completion of reinspection effort and the full scope of the concern is known. Until that time, the status will be carried on KG&E's monthly report as TE 53564-K152. In the interim, please direct any questions concerning this subject to me or to Mr. Otto Maynard of my staff.

Yours very truly,

Original Signed GLENN L. KOESTER

Glenn L. Koester
Vice President - Nuclear

GLK:bb
xc: RCDeYoung
PO'Connor (2)
HBundy

bxc:1 Cy to 1)BRuddick; 2)HMacklin; 3)RTerrill-GO
GLFouts/FDuddy
NAPetrick
JMEvans
AMee
JMHarvey/JBerra
RGreen/LWMcGriff
INPO Record Center
KRBrown/WCadman-GO
RLRives-620 GO
MLJohnson-MS3-01
WGEales-MS6-03
EWCreel-MS7-02
CCMason
FTRhodes
RJGlover
RMGrant
WJRudolph
OMaynard
GRathbun (2) - MS6-02
EDProthro/IDFile-202 GO

COPY

KG&E THE ELECTRIC COMPANY

KANSAS GAS AND ELECTRIC COMPANY
P. O. Box 208 Wichita, Kansas 67201

December 31, 1984

bxc:
NAPetrick
JMEvans
AMee
JSilberg
KRBrown/WCadman-GO
RLRives-620 GO
RTerrill-702 GO
JABailey-MS7-01
MLJohnson-MS3-01
WGEales-MS6-03
EWCreel-ME7-02
GLFouts
CCMason
RJGlover
RMGrant
FTRhodes
WJRudolph
OMaynard
GRathbun (2)
BRuddick/HMacklin
EDProthro/IDF-202

JAN 1985
RECEIVED
NUCLEAR
DEPT.

Mr. Richard C. DeYoung, Director
Office of Inspection and Enforcement
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

KMLNRC 84-239
Re: Docket No. STN 50-482
Ref: 1) Letter dated 11/21/84 from RDMartin, NRC to GLKoester, KG&E
2) Letter KMLNRC 84-234 dated 12/21/84 from GLKoester, KG&E, to RCDeYoung, NRC.
Subj: Enforcement Action 84-107

Dear Mr. DeYoung:

Enclosed is Kansas Gas and Electric Company's (KG&E) response to Violation I (Violation Assessed Civil Penalty) as documented in Reference 1. Pursuant to 10CFR2.201, the following five items are addressed in the Enclosure for the alleged violation:

1. Admission or denial of the alleged violation;
2. The reasons for the violation, if admitted;
3. The corrective steps that have been taken and the results achieved;
4. The corrective steps that will be taken to avoid further violations; and
5. The date when full compliance will be achieved.

KG&E's response to Violation II (Violation Not Assessed Civil Penalty) was transmitted by Reference 2 which also documented the agreed upon extension for the response to Violation I.

Please contact me or Mr. Otto Maynard of my staff if you have any questions concerning KG&E's response to the alleged violation.

Yours very truly,

Kent R Brown
for Glenn L. Koester
Vice President - Nuclear

GLK:kc

xc: RDMartin w/a
RPDenise w/a
PO'Connor w/a

LICENSING ROUTING
NRC/LK
Chronological (2)
IE 40V50-G84
Greel
Coordinator

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OATH OF AFFIRMATION

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

I, Glenn L. Koester, of lawful age, being duly sworn upon oath, do depose, state and affirm that I am Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, that I have signed the foregoing letter of transmittal, know the contents thereof, and that all statements contained therein are true.

KANSAS GAS AND ELECTRIC COMPANY

ATTEST:

By Glenn L Koester
Glenn L. Koester
Vice President - Nuclear

ED Prothro
E.D. Prothro, Assistant Secretary

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

BE IT REMEMBERED that on this 21st day of December, 1984, before me, Evelyn L. Fry, a Notary, personally appeared Glenn L. Koester, Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, who is personally known to me and who executed the foregoing instrument, and he duly acknowledged the execution of the same for and on behalf of and as the act and deed of said corporation.

IN WITNESS WHEREOF, I have hereunto set my hand and affixed my seal the date and year above written.

Evelyn L. Fry
Evelyn L. Fry, Notary

My Commission expires on August 15, 1985.

Enclosure to KMLNRC 84-238

VIOLATION ASSESSED CIVIL PENALTY

FINDING

Criterion X f 10CFR Part 50, Appendix B, requires that a program for inspection of activities affecting quality be estabished and executed by or for the organization performing the activity to verify conformance with the documented instructions, procedures, and drawings for accomplishing the activity.

Criterion XVI of Appendix B further requires that measures be estabished to assure that nonconformances are promptly identified and corected.

Criterion XVII requires that sufficient records be maintained to furnish evidence of activities affecting quality.

Daniel International Corporation (DIC) Construction Procedure No. QCP-VII-200 describes the requirements for performance and inspection of safety-related structural steel welds with respect to committed conformance to the American Welding Society (AWS) D1.1-75. Appendix I in Revision 4 of this procedure invokes a prohibition with respect to lack of fusion, overlap, slag, arc strikes, and weld splatter. Paragraph 6.5.1 of AWS D1.1-75 requires inspector verification that the size and length of welds conform to the drawing requirements and that no specified welds are omitted.

Contrary to the above, the inspection program for safety-related structural steel welds was not adequately executed to assure conformance to the requirements of Construction Procedure QCP-VII-200 Revision 4 and the AWS D1.1-75 Code nor were adequate records kept to document the quality of the welds. Furthermore, once deficient welds were identified, no actions were taken to correct the deficiencies. This inadequate inspection program and the failure to take corrective actions is evidenced by the following:

1. A random reinspection of 241 structural steel safety-related welds, which was made in accordance with Revision 4 of QCP-VII-200, was performed by DIC and documented in Corrective Action Report (CAR) No. 1-W-0029 dated March 22, 1983. Sixty-two percent of the inspected welds were found by the DIC inspectors to not conform to the requirements of Revision 4 of QCP-VII-200. The reported defects that resulted in rejection by the DIC inspectors included arc strikes, slag, lack of fusion, overlap, and weld splatter.

2. Another reinspection of a sample of structural members with lowest design safety margins was initiated on September 14, 1984. The results of the licensee reinspection activities (verified by NRC inspectors) as of September 28, 1984, were as follows:

a. A missing weld was found at the same location in each of six pressurizer support connections. In addition, five of 14 fillet welds in one pressurizer support connection were undersized by 1/8-inch to 1/4 inch with respect to the drawing-required size of 5/8-inch, and two of these welds were also under the required length; i.e. 3-inch and 5-inch lengths, respectively, versus drawing-required length of 8 inches. The weld dimensions of the remaining five pressurizer support connections were not included in the NRC verification activity.

b. Reinspection of nine structural steel connections in the auxiliary building identified two missing welds in one connection. In addition, weld size and length discrepancies were identified in each of the nine connections. Of the total of 106 welds in the connections, eight were found to be undersized by 1/16-inch to 3/16-inch with respect to drawing-required width. Two of the undersized welds were also under the required length; i.e., 2 1/4-inch and 2 1/2-inch lengths, respectively, versus a drawing required length of 3 inches. An additional nine welds were also under the drawing-required length of 3 inches by 1/2-inch to 1-inch. Examination of 54 weld returns in the nine connections found 26 to be undersized by 1/16-inch to 3/16-inch with respect to drawing-required widths. One of the undersized weld returns was also under the required length; i.e., 2 inches versus a drawing-required size of 3 inches. In addition, 36 weld returns exceeded the drawing-required maximum length of 5/8-inch by 1 5/8-inches to 3 5/8-inches. An additional eight weld returns exceeded the drawing-required maximum length of 3/4-inch by 1/2-inch to 2 1/8-inches.

3. The absence of required Miscellanious Structural Steel Weld Records (MSSWRs) for documenting welding and inspection of safety-related structural steel welded connections was identified by KG&E in CAR No. 1-C-0031. As a result of this identification it has been established that approximately 16 percent of MSSWRs could not be located, which precludes positive verification of control of welding and performance of required inspections. Approximately 80 percent of the MSSWRs applicable to the activities described in paragraph 2 above could not be located. Records were not available to indicate that an initial inspection was performed of either the pressurizer support connections or the auxiliary building strutural connection which was identified to be missing two welds.

MSSWRs were located for certain welds in four structural connections which indicated acceptable welds. However, reinspection of these four connections showed one undersized weld in one connection and undersized and overlength weld returns in the four connections.

RESPONSE

1. Admission or denial of alleged violation;

Kansas Gas and Electric Company (KG&E) does not dispute that problems existed in the inspection and documentation of safety-related structural steel welds. However, KG&E does take exception with one of the statements documented in the NRC's Notice of Violation and Proposed Imposition of Civil Penalty.

The statement with which KG&E takes exception is:

> "Furthermore, once deficient welds were identified, no actions were taken to correct the deficiencies."

All identified weld deficiencies were documented on Nonconformance Reports and either reworked to correct the deficiency or evaluated by the Architect/Engineer for a use-as-is-disposition.

2. The reasons for the violation, if admitted;

The reasons for the violation are discussed in the attached final report.

3. The corrective steps that have been taken and the results achieved:

The corrective steps that have been taken and the results achieved are documented in the attached final report.

4. The corrective steps that will be taken to avoid further violations:

The corrective steps taken to avoid further violations are discussed in the attached final report.

5. The date when full compliance will be achieved:

Corrective steps to resolve the safety-related structural steel welding concern will be completed by January 15, 1985.

THE ATTACHMENT

TO KMLNRC 84-238

WAS REVISED.

SEE KMLNRC 85-037

KG&E
THE ELECTRIC COMPANY
KANSAS GAS AND ELECTRIC COMPANY

GLENN L KOESTER
VICE PRESIDENT NUCLEAR

January 18, 1985

Mr. R.P. Denise, Director
Wolf Creek Task Force
U.S. Nuclear Regulatory Commission
Region IV
611 Ryan Plaza Drive, Suite 1000
Arlington, Texas 76011

KMLNRC 85-025
Re: Docket No. STN 50-482
Ref: 1) Interim Report KMLNRC 84-187, dated 10/17/84
From GLKoester, KG&E, to DRHunter, NRC
2) Letter KMLNRC 84-238, dated 12/31/84
From GLKoester, KG&E, to RCDeYoung, NRC
Subj: Final 10CFR50.55(e) Report - Inspection of Welds

Dear Mr. Denise:

This letter provides the final report submitted pursuant to 10CFR50.55(e) concerning inspections of structrual steel welding at Wolf Creek Generating Station. This matter was initially reported by Kansas Gas and Electric Company (KG&E) on September 18, 1984, and supplemental information was provided in Reference 1.

Reference 2 provided a comprehensive report which described the corrective actions taken by KG&E to resolve this matter. As stated in the report KG&E Corrective Action Request #19 was the corrective action vehicle initiated to assure resolution of the concern and Corrective Action Request #19 has now been closed.

The report transmitted in Reference 2 also documented that part of the corrective actions associated with CAR #19 included a review of other AWS Welding to determine whether any similar concerns could exist in areas other than structural steel. Only one other area was identified in which additional investigation was required. This was in the area of electrical equipment installation where the method of permanent installation is by welding the equipment mounting frame to the foundations embeds. Daniel Corrective Action Report 1-EW-0046 was initiated to document and track the resolution of this concern.

The specific concern associated with CAR 1-EW-0046 is that not all shims less than 1/4 inch thick were flush with the mounting frame as required by the AWS code. The code requires that shim less than 1/4 inch thick be flush with the frame and the size of the weld increased by the thickness of the shim. Some equipment mounting frames were identified in which the shim was not flush. This resulted in a situation where the shim carried the shear load. A walkdown was performed to identify and document these nonconformances. The rework associated with CAR 1-EW-0046 is in progress and scheduled for completion prior to fuel load.

201 N. Market — Wichita, Kansas — Mail Address: P.O. Box 208 | Wichita, Kansas 67201 — Telephone: Area Code (316) 261-6451

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Please contact me or Mr. Otto Maynard of my staff if you have any questions concerning this subject.

Yours very truly,

Glenn L Koester

Glenn L. Koester
Vice President - Nuclear

GLK:dab

xc: PO'Connor
HBundy
RCDeYoung
WGuldemond

FORM 110

KGE
THE ELECTRIC COMPANY

KANSAS GAS AND ELECTRIC COMPANY
P. O. Box 208 Wichita, Kansas 67201

January 21, 1985

Mr. Richard C. DeYoung, Director
Office of Inspection and Enforcement
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

KMLNRC 85-037
Re: Docket No. STN 50-482
Ref: Letter KMLNRC 84-238 dated 12/31/84 from
CLKoester, KG&E, to RCDeYoung, NRC
Subj: Supplemental Response - EA 84-107

C
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Dear Mr. DeYoung:

The Reference transmitted Kansas Gas and Electric Company's (KG&E) response to the Notice of Violation concerning structural steel welding at Wolf Creek Generating Station. Subsequent to the issuance of that response, KG&E has determined that some of the corrective actions concerning the installation of a few specific welds identified in Appendix D of the response must be revised because the weld locations are inaccessible. The KG&E Management Plan has always been to install any missing weld (even if not required to meet design allowables) unless the weld location was inaccessible. In the event the location was inaccessible, the Architect/Engineer would evaluate the joints on a case-by-case basis and either disposition the joint for use-as-is (i.e., design allowables were met in the as-built condition) or make a design change to meet the design allowable stress. In any event, the design allowable stress would be accomodated and the design changed to reflect the "as installed" configuration. Therefore, in accordance with the KG&E Management Plan the disposition of some of the Nonconformance Reports associated with the missing welds identified in Appendix D of the Reference have been changed to use-as-is due to the inaccessibility of the weld location. Attachments A and B provide revised sections of the initial response to reflect the final disposition of the welds discussed in Appendix D.

The attached revisions include the following changes:

1. Editorial changes.

2. Revision of Section IV to reflect the closure of item 1a3 concerning Welder Qualification Procedures. This item was open when the initial response was submitted.

3. Revision of sections I through V and Appendix D to reflect the number of joints reworked as a result of revised dispositions as described above. It should also be noted that the total number of joints evaluated was changed from 2669 to 2670 due to a counting error in the initial report on this subject.

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4. Revision of Appendix D to reflect the final disposition of specific missing welds.

Please replace sections I through V of the original report transmitted by the Reference with Attachment A and replace Appendix D of the original report with Attachment B. Please contact me or Mr. Otto Maynard of my staff if you have any questions concerning this supplemental response.

Yours very truly,

Glenn L. Koester
Vice President - Nuclear

GLK/keh

cc: PO'Connor (2)
WGuldemond
HBundy
RDMartin

Attachment A to KMLNRC 85-037

ATTACHMENT A

KANSAS GAS AND ELECTRIC COMPANY

FINAL REPORT

CORRECTIVE ACTION REQUEST NO. 19

FINAL REPORT

KG&E CORRECTIVE ACTION REQUEST NO. 19

TABLE OF CONTENTS

I. CAR-19 EXECUTIVE SUMMARY

Because of deficiencies (i.e., undersize, undercut,...) previously found in fillet welds on ASME and Special Scope hangers, DIC performed a random reinspection of structural steel fillet welds in February, 1983 in all "Q" designated buildings in the Powerblock. This reinspection indicated that an unacceptable percentage of structural steel fillet welds were deficient in the Auxiliary, Control and Fuel Buildings. A Corrective Action Report (CAR 1-W-0029) was initiated by DIC to implement reinspection, and nonconformance reports were generated to document and disposition deficiencies noted.

Subsequent to the issuance of CAR 1-W-0029 it was determined, during the course of document reviews in the Building turnover process, that Miscellaneous Structural Steel Weld Records (MSSWR's) could not be located as procedurally required for all structural steel welds in "Q" designated buildings. These missing MSSWR's resulted in DIC issuance of CAR 1-C-0031.

The concerns addressed in CAR's 1-W-0029 and 1-C-0031 as well as other items listed in the "Introduction" section of this report caused KG&E Construction Quality Control to initiate a limited inspection verification program. Through this inspection program additional concerns were raised as a result of the inspection verification results. These results identified instances of missing welds which had no inspection records, two missing welds which had inspection records, and welds with inspection records that did not completely comply with project inspection and documentation criteria. The results of the verifications combined with the missing weld inspection records identified the need for a formalized action plan to fully investigate the concerns and formulate corrective action as necessary. To accomplish this KG&E QA initiated Corrective Action Request 19, describing the concerns and recommending corrective action on October 17, 1984. Based on the corrective actions recommended by CAR-19 and additional actions deemed warranted in support of the investigations, a Management Plan was developed to designate the nature and extent of the investigations.

The Management Plan covered three basic categories of investigation and evaluation. One category was a process of reinspection to identify and evaluate actual hardware conditions in the field. A second category addressed the programmatic aspects of Structural Steel erection through evaluation of both construction and quality program procedures. A third category addressed related considerations such as other AWS D1.1 applications, evaluation of missing welds identified during the reinspections, evaluation of acceptable inspection records completed for welds found to be missing, and review and evaluation of surveillances, audits, and reports pertinent to AWS welding. Although not initially in the scope of KG&E CAR-19, non-welding related quality programs were reviewed for comparable programmatic deficiencies. In accomplishing this KG&E and DIC conducted an extensive program assessment of the Piping, Hanger, Mechanical, Electrical and Civil disciplines to ascertain the adequacy of the construction and quality programs instituted. This program assessment was conducted by KG&E and DIC Management representatives, and concluded that a satisfactory level of confidence exists to assure compliance of these to 10CFR50, the FSAR, ANSI N45.2, and design and procedural requirements.

The intent of the program evaluation was to evaluate the various construction and quality programs/procedures to determine their compliance to the AWS D1.1 Welding Code and FSAR commitments. This evaluation included

IMAGE EVALUATION
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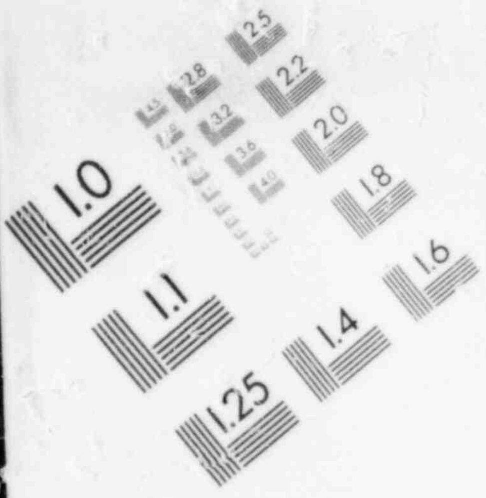

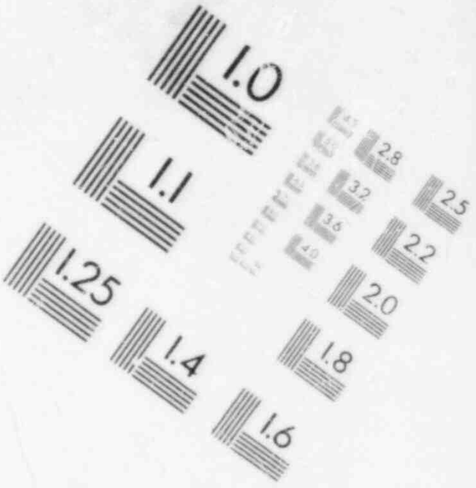

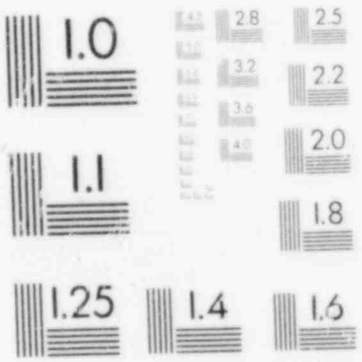

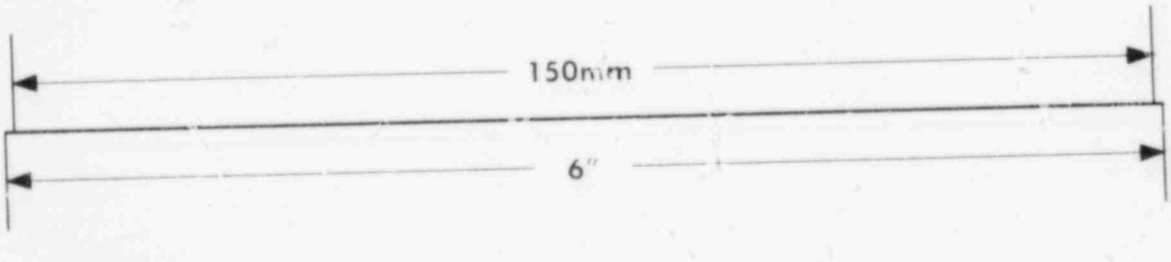

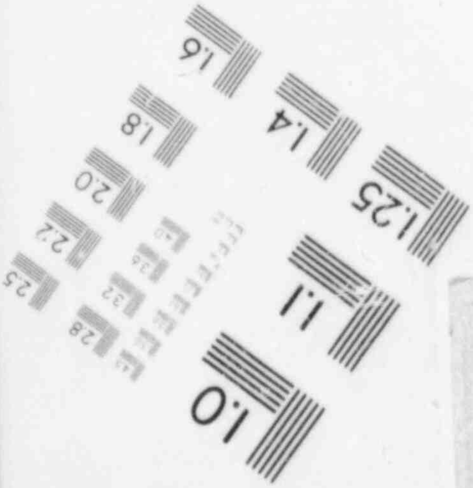

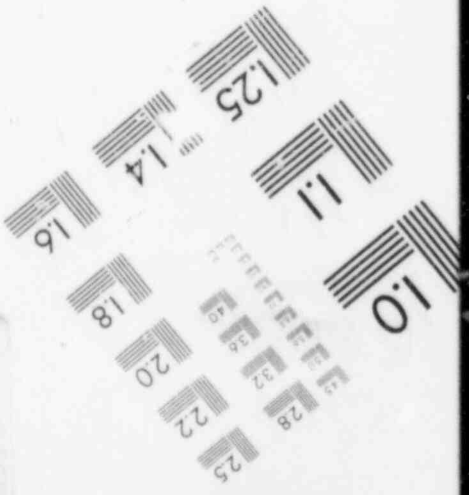

relevant aspects of the various related programs from the initiation of purchase orders for procurement of structural steel and welding materials, to final installation and quality acceptance. The procedures for receiving, storage and handling of materials were evaluated, as well as compliance of procedures for training and certification of inspectors to ANSI N45.2.6 and welder qualification to AWS requirements. The procedure reviews included a thorough evaluation from their origination through subsequent revisions, including an analysis to assure current conformance to design document requirements. No findings were noted that were determined to be contributing factors to inadequacies in AWS D1.1 applications, although some procedural inadequacies were discovered and reconciled.

All other safety-related programs utilizing AWS welding were analyzed to ensure that the root cause identified as the reason for previous acceptance of deficient structural steel welds was not inherent, or impactive, to these programs as well. The method of documenting weld inspections, control of this documentation, and accountability to assure all required documentation was retrievable was researched for AWS D1.1 welding applications in raceway supports, electrical equipment, mechanical equipment, fire dampers, safety-related HVAC ductwork and supports, miscellaneous steel and embed fabrication, and pipe whip restraints for assurance that problems similar to those encountered in structural steel did not exist. Previously compiled information including Construction Self Assessment Reports, KG&E QA Reports and Surveillances, DIC QA Reports, DIC Project Monitoring Program Audits, and DIC Corrective Action Reports were reviewed to determine if the results of previous investigations indicated other potential problem areas relevant to AWS D1.1 welding. No findings were noted that could be considered to be contributing factors to inadequacies in AWS D1.1 programmatic applications. An analysis of hardware installations for other project applications of AWS D1.1 welding identified one other area to be investigated for AWS welding problems. This is in the area of electrical equipment installations where the method of permanent installation is by welding the equipment mounting frame to foundation embeds. DIC is addressing this potential problem on Corrective Action Report No. 1-EW-0046.

Reinspection of field welds was conducted utilizing AWS Certified Welding Inspectors who were also certified to the DIC Quality Program requirements of ANSI N45.2.6. Inspections were performed in strict compliance to the Inspection Verification Plan which established inspection criteria and documentation requirements, and was incorporated into an existing DIC Quality Procedure, QCP-VII-200, and approved by DIC, Bechtel, and KG&E.

DIC and Bechtel research substantiated that DIC welders and welding procedures applicable to AWS D1.1-1975 welding of structural steel installations were satisfactorily qualified in accordance with AWS requirements. Research by DIC and Bechtel resulted in assurance that the programs and procedures for the purchase and control of weld filler materials used in AWS D1.1 applications were in compliance with AWS requirements, and were properly implemented on site.

The retrievability and control of Miscellaneous Structural Steel Weld Records was thoroughly researched, and a determination made that inadequate implementation of DIC Construction procedures was the primary contributing factor relative to retrievability and accountability problems in this area.

An evaluation of the DIC Quality inspection training program demonstrated that this program and related procedures were in compliance to ANSI N45.2.6. Further investigation concluded that Quality inspection training was appropriate and adequate during the structural steel installation time frame.

An evaluation of DIC Quality inspection procedures and criteria applicable to the original structural steel installation/inspection period revealed several procedural inadequacies. A thorough analysis of the omission of each inspection criterion of AWS D1.1 structural steel applications was accomplished, with the conclusion that no adverse impact had resulted from these procedural inadequacies relative to AWS D1.1 welding inspection.

Inspection criteria to be used in the structural steel reinspection activities was procedurally defined and training of all personnel completed prior to reinspection initiation. Sufficient technical justification was established by Bechtel to validate inspection of welds through a predetermined maximum thickness of paint. An analysis of reinspection results determined the root cause of the previous acceptance of deficient structural welds to be due to DIC inspection implementation differences relative to inspection vs. reinspection techniques, and inadequate implementation of applicable DIC procedures during original inspection efforts. These inspection implementation differences are discussed elsewhere in this report, referencing the Reedy, Herbert, Gibbons documentary included in the Appendix, section VI.G.

Two joints (each missing one weld) of the two thousand six hundred seventy (2,670) reinspected (representing more than 11,000 welds) had documentation reflecting the installation of these welds when in reality they were not installed. Research revealed no evidence to indicate that either was a case of deliberate falsification. Additional investigations did indicate that human error was the cause of incorrectly documenting these nonexistent installations.

Reinspection found that approximately two (2) percent of the inspected welds were not installed as required by design documents. These errors were primarily due to craft/engineering confusion relative to installation drawing details and requirements. Failure to install these welds and materials, although in some cases determined to be significant in impact to stress allowable calculations, would not have resulted in material or structural failure if left uncorrected.

The total number of joints subjected to the reinspection program was two thousand six hundred seventy (2,6709). These joints were selected by Bechtel as structurally significant (See Appendix IV D) with the distribution being: 694 in the Auxiliary Building, 1300 in the Reactor Building, 265 in the Control Building, 98 in the Diesel Generator Building, 36 in the ESWS Pumphouse, and 277 in the Fuel Building. The reinspection documented an as found condition regardless of the weld acceptability. All results were forwarded to Bechtel in the form of inspection data sheets for evaluation. This evaluation was based upon Bechtel's review of reinspection data accumulated and nonconformance reports (NCR's) generated. The evaluation for structural adequacy was made based upon this cumulative data that reflected the as-built condition of the structurally significant joints prior to any rework or repairs. No deficiencies were identified, which if left uncorrected, would have adversely affected the safe operation of the plant. The results of this evaluation provides assurance that Safety Related AWS D1.1 structural steel welding complies with all Quality criteria as specified in the related design documents, and is within the tolerances of acceptable deviation as determined by the Architect/Engineer

Joints that in the as-built condition were determined to exceed the design allowable stresses were all reworked unless prohibited by field conditions. In addition joints in which the design allowable stresses were not exceeded in the as-built condition but were missing welds, were also reworked unless prohibited by field conditions.

II. INTRODUCTION TO CAR-19

A series of activities as identified below pertaining to weld inspection at Wolf Creek ultimately led to the issuance of KG&E CAR-19 addressing AWS D1.1 Structural Steel welding concerns.

In September, 1980, DIC initiated Corrective Action Report 1-M-0007 due to improper inspection technique application, which required 100% reinspection of all socket welds on small bore piping installed prior to June, 1980. Subsequent to this reinspection effort, DIC generated Corrective Action Report 1-W-0019 on August 17, 1982, due to a significant quantity of fillet weld discrepancies being identified, which required 100% reinspection of all fillet welds on ASME and Special Scope piping hangers made prior to April 1, 1981. DIC performed a random reinspection of structural steel fillet welds in February, 1983, in all "Q" designated buildings in the Powerblock to determine whether structural steel welds may have been deficient as a result of the same root cause relative to CAR 1-W-0019. It was determined from these reinspection results that an unacceptable percentage of structural steel welds were deficient in the Auxiliary, Control, and Fuel Buildings. Thus CAR 1-W-0029 was initiated by DIC to implement reinspections, and nonconformance reports were generated to document and disposition the deficiencies noted.

As a result of documentation review prior to building turnovers DIC initiated CAR 1-C-0031 in August, 1983, to document that Miscellaneous Structural Steel Weld Records (MSSWR) could not be located as required by procedures for all structural steel welds in "Q" designated buildings. Nonconformance Reports were generated to document missing MSSWR's in each of these buildings.

KG&E and DIC site management held meetings in May, 1984, to further discuss retrievability of MSSWR's and the problems that had been identified to date. Concerns were expressed through KG&E Quality First to KG&E Construction Management regarding the acceptability of "Use-As-Is" dispositions given to NCR's written as part of CAR 1-C-0031's corrective action in July, 1984, and KG&E Management requested DIC to generate a revision to CAR 1-C-0031 in letter KWCLC 84-814 of July 30, 1984, in response to some concerns noted. Revision 6 to CAR 1-C-0031 was generated by DIC in response to KG&E's concerns.

KG&E Quality Assurance performed a detailed review of DIC CAR 1-W-0029 and 1-C-0031 in August, 1984, identifying numerous concerns to KG&E Construction. In response KG&E Construction began a documentation reconciliation task on August 13, 1984, to determine which safety-related structural steel welds did not have supportive MSSWR's.

On August 17, 1984, KG&E Construction Quality Control initiated an Inspection Verification Plan to provide an accurate assessment of the "as-built" conditions of safety-related structural steel welds without MSSWR's. DIC and KG&E Management discussed revision of this inspection program on August 30, 1984.

KG&E, DIC and Bechtel made a joint presentation to an NRC Task Force on September 10, 1984, which identified the belief at that time that the problem was one of document retrieval, and not a hardware problem. The NRC Task

Force discussed the problems with KG&E again on September 13, 1984, during which KG&E Management agreed to perform a sample hardware inspection of six (6) randomly selected structurally significant joints in the Reactor, Fuel, Control, Auxiliary, Essential Service Water, and Diesel Generator Buildings. This inspection resulted in the discovery of missing welds and missing structural members, which were reported to the NRC by KG&E under 10CFR50.55(e) on September 18, 1984. Subsequent meetings were held with NRC Representatives on September 25, 1984, and September 28, 1984, to status inspection efforts and provide information updates. An AWS Welding meeting was held with the NRC on October 19, 1984, on site relative to structural steel welding, with a follow-up meeting on October 22, 1984, in which KG&E Management discussed AWS structural steel welding concerns with the NRC.

On October 17, 1984, KG&E Quality Assurance issued CAR-19 to KG&E Construction to obtain corrective actions associated with AWS D1.1 structural steel welding. The findings addressed in CAR-19 included missing MSSWR's for safety-related structural steel welds; deficiencies being identified in previously accepted structural steel welds, missing structural welds or missing structural material; and documentation that a weld was inspected and accepted, but no weld was installed.

KG&E and DIC Management representatives subsequently developed a logic chart to organize resolutions relative to CAR-19's concerns, a Management Plan to implement corrective actions, and published a CAR-19 Corrective Action Schedule to provide a means for tracking corrective action progress.

In addition, KG&E Management contracted Lehigh University to review the problems associated with the structural welds in the structures at Wolf Creek Generating Station. The results of their review is included in Appendix VI.F of this report.

III. CAR-19 OBJECTIVES

- To document a consolidated project plan for the identification, evaluation and resolution of problems associated with Safety-Related AWS D1.1 Welding.

- To provide assurance, based on objective evidence, that AWS D1.1 Welding of Safety-Related Structural Steel complies with all Quality Criteria as specified in the related design documents and is within the tolerances of acceptable deviations as determined by the Architect/Engineer.

- To provide assurance that the documentation which supports the inspection of safety related structural steel welds is:

  - Available - Complete - Reflects appropriate information - Traceable to the item or activity

- To evaluate supporting elements of the DIC Quality Assurance Program to ensure that those elements were adequately and effectively implemented to demonstrate that the DIC welding of Safety Related Structural Steel, HVAC Supports, Electrical Supports, Pipe Whip Restraints and any other AWS D1.1 safety related welding activities were in compliance with the FSAR (i.e. AWS D1.1 - 1975) and the Design and Construction QA Program Manual, Section 17.1.B.

- To evaluate DIC Construction/Quality programs in areas other than AWS D1.1 welding to determine the potential of programmatic deficiencies.

IV. CAR-19, DISCUSSION OF FINDINGS AND CORRECTIVE ACTIONS

The KG&E Management Plan for the resolution of CAR-19 was developed by DIC and KG&E Management personnel to document a consolidated project plan for the identification, evaluation and resolution of problems associated with safety-related AWS D1.1 welding. The intent of this plan is to verify that both the hardware and programmatic aspects of all safety-related activities utilizing AWS D1.1 welding are in compliance with the FSAR and the Design and Construction Program Manual.

The logic chart for the resolution of CAR-19 was developed to illustrate the approach to be used in providing the verifications needed for implementation of satisfactory corrective action. The Corrective Actions as described in the KG&E Management Plan are identified in the flow of activities as designated on the logic chart. The logic chart is included as an attachment to this report in the Appendix, section VI.B.

Five (5) findings were included in CAR-19. The detailed activities and investigative actions required to implement each Corrective Action are delineated in the KG&E Management Plan. The process of corrective action for each finding generated by CAR-19 entails multiple activities. Each finding and it's respective corrective actions are discussed in detail in the following. Supportive and/or investigatory documentation for each finding as discussed in this section is delineated in the Appendix, section VI.I.

Finding #1 of KG&E CAR-19 stated, "The results of the Document Reconciliation Task Force indicated that 1509 of 6816 MSSWR's for Safety Related Structural Steel Welds are missing".

Six (6) corrective actions were prescribed as appropriate for the resolution of this finding and related concerns. These corrective actions were focused toward programmatic evaluations, procedural criteria evaluations, and a reinspection program utilizing certified inspectors. Following is each of the six (6) corrective actions for Finding #1 with an analysis of the investigative actions taken and a summarization of each corrective action's results in accordance with the KG&E Management Plan's directions.

Corrective Action 1a)

"Based on DIC program requirements assure that all of the welders and welding procedures were qualified to AWS D1.1."

This activity was subdivided into three elements of research. These elements included development of an AWS D1.1-75 Attribute Checklist analyzing individual attributes relative to the welding process. The checklist lists all AWS requirements and compares those requirements with DIC Construction Welding Procedure requirements, in each case citing explicitly how the corresponding DIC procedure addresses separate AWS criteria. This checklist is conclusive data that provides evidence of all AWS D1.1-75 criteria being adequately addressed by DIC Construction Welding Procedure, CWP-506, "Welding of Carbon Steel". An attachment to this checklist documents the procedure review cycle for CWP-506, showing that each revision from 09/14/78 through the current revision dated 05/21/81 was consistently reviewed and approved by the individuals designated that responsibility.

A second element of this activity was the statistical sampling of AWS Welder qualifications in accordance with MIL-STD-105D. The total quantity of retrievable Miscellaneous Structural Steel Weld Records (MSSWR) applicable to AWS welding was initially identified to define the total population to be used in selecting a sample size. A "Single Sampling Plan for Normal Inspection" was utilized, randomly selecting MSSWR's for review of welders' qualifications. This sample included a variety of welders, a variety of AWS welding procedures, a representative sample of welders during the 1978-1984 time frame, and sampling from welders working in all Powerblock buildings. Identification of welders was taken from the MSSWR's and welder qualification records (W-105). These were then reviewed to assure that each welder was qualified to the weld procedure entered on the MSSWR at the time of weld installation.

A sample size of two hundred (200) was selected as being most representative, given the previous considerations. Based upon Table II-A of MIL-STD-105D, DIC desired a ninety-six percent (96%) Acceptable Quality Level (AQL). This AQL accepts fourteen (14) rejectable units from a sample of two hundred (200), and rejects the entire population when the fifteenth (15) rejection of the sample is observed.

Research performed by DIC Welding Engineering revealed thirteen (13) incorrect entries on MSSWR's, with only four (4) of these considered "rejectable" due to the nature of the discrepancies. All thirteen discrepancies were due to incorrect entries being made on the MSSWR, with nine (9) of the thirteen having the weld technique entered as N-1-1-A-6 rather than N-1-1-A-6A. These two weld techniques were evaluated by DIC Welding Engineering by comparison of attributes and essential variables, and it was determined that no adverse impact existed. The four (4) entries considered rejectable were due to welders incorrectly entering a welding procedure number for which they were not qualified on an MSSWR.

A Nonconformance Report, ISN 20984CW, was generated to document all thirteen (13) discrepancies noted, and was recommended for a "Use-As-Is" disposition by DIC Welding Engineering. This Nonconformance Report has been reviewed and disposition concurrence received from Bechtel, closing the NCR.

The third element of this activity was a review by Bechtel of DIC Welder Qualification Procedure and the DIC Welding Procedure Specifications to assure compliance to AWS D1.1-75.

Bechtel reviewed DIC Construction Welding Procedure, CWP-302, "Qualification of Welders", all revisions up to and including Revision 19. This review indicated full compliance with the AWS D1.1-75 for revisions 1 through 18. However, Revision 19 does not strictly comply with AWS D1.1-75 in the following areas. Nonconformance Report 1SN21472MW was generated to document these deviations and was dispositioned "Use-As-Is" by DIC Welding Engineering. Support for this disposition was generated by DIC Corporate Welding Engineering as well as by AWS.

Bechtel randomly selected Welding Procedure Specifications (WPS) from MSSWR's applicable to structural welds in the 1978-1984 time frame. The review of the WPS' indicated full compliance with AWS D1.1-75 with one exception, undercut criteria, which was allowed by the Wolf Creek Final Safety Analysis Report, Revision 0, October, 1979. Three of the WPS' permitted undercut to be acceptable provided the depth did not exceed 1/32 inch, which is a relaxation of AWS D1.1-1975 undercut criteria.

The exception to the AWS D1.1-75 undercut criteria exists in Revision 0 of the Wolf Creek Final Safety Analysis Report, Section 3.8.3.6.3.3, dated October, 1979, and was also added by a revision to Bechtel Civil Specification C-122 and C-132, the design specifications applicable to the structural steel connections in the CAR-19 reinspection program. Based upon these facts the Bechtel Material and Quality Services Department (M&QS) determined that the WPS' used during erection/installation of structural steel members did comply with AWS D1.1-75. Paragraph 1.1.2 of AWS D1.1 defines the "Engineer" as the duly designated authority who acts for and in behalf of the Owner, and the exception to AWS undercut criteria was documented in the FSAR to comply with this paragraph.

It is Bechtel M&QS' conclusion that the review of the DIC WPS' and supportive documentation demonstrates that the welding procedures used by DIC during structural steel installation did comply with the AWS D1.1-1975 Structural Welding Code Edition when used concurrently with supportive design documents and the revisions to the FSAR.

In conclusion, the three elements of analysis included in the research performed on Activity 1a offer assurance that all DIC welding procedures were qualified in accordance with AWS D1.1-75 requirements.

Corrective Action 1b)

"Review the DIC Program for the purchase and control of filler material to ensure that only acceptable filler material was used in safety related welds. Assure that both safety related and non-safety related filler materials were properly controlled to preclude improper applications."

This activity was divided into two elements of research, those being; the DIC review of procedures for the purchase and control of filler and base materials, and Bechtel's review for the purchase and control of filler materials.

DIC Civil Engineering performed an in-depth review of the DIC Program for purchase of structural and miscellaneous steel and found the DIC Program to be in accordance with the requirements of Bechtel Specifications 10466-C-121 (Purchase of Structural Steel), and 10466-C-131 (Purchase of Miscellaneous Steel). These specifications and their respective DIC procedures were found to adequately address applicable requirements for assuring correct material specification, grade, marking, traceability and other Quality Assurance requirements. In addition these specifications and procedures provide for buyer verification of any or all of the established specification requirements.

The DIC procedures applicable to procurement activities are as follows:

AP-VII-01 Development and Approval of Bidders List

AP-VII-02 Requisitioning of Daniel Procured Materials, Equipment and Service

AP-VII-03 Bid Requests

AP-VII-04 Receiving and Processing Bid Proposals

AP-VII-05 Issuing Purchase Orders and Change Orders

During a self-initiated KG&E review of safety-related procurement records in January, 1984, several cases were identified in which DIC purchase orders did not comply with all A/E specification requirements. As a result of these findings, DIC initiated a Corrective Action Report (CAR) 1-G-0036, to perform a complete review of all purchase orders to verify compliance to specification requirements. This investigation encompassed the review of five hundred thirty-six (536) safety-related purchase orders to assure hardware and documentation to be in compliance with specifications. Any discrepancies identified during this review were documented on Nonconformance Reports for resolution by DIC, KG&E and the A/E. Those nonconformances identified relative to structural steel were determined to be all documentation related with no hardware impact. All corrective actions were completed, all Nonconformance Reports resolved and closed, and Corrective Action Report 1-G-0036 was closed on 05/24/84.

DIC Civil Engineering accomplished a detailed study of the control and issuance of base materials applicable to structural steel installations. This review was based upon a thorough analysis of material control requirements for this application in the following DIC procedures:

AP-VIII-02	Material and Equipment Receiving
AP-VIII-03	Identification, Marking and Inspection
AP-VIII-04	Receiving Discrepancies
AP-VIII-05	Material Storage and Control
AP-VIII-07	Material Issue
QCP-IV-111	Erection of Structural Steel and Pipe Whip Restraints
WP-IV-111	Structural Steel and Pipe Whip Restraint Erection

This review investigated such areas as the use of Structural Steel Fabrication Requests, requisitioning and issuance of the material to craft for erection, maintenance of traceability through heat number transfer for material that is divided, and documentation of this heat number on permanent plant records. DIC Civil Engineering's research concluded that acceptable control and utilization of base materials is maintained through DIC programs and procedures.

Bechtel's Materials and Quality Services Group furnished information based on their research to ensure that the DIC Procurement program had in fact resulted in the proper filler material being purchased and subsequently utilized in structural steel installation activities. This review was documented in attachments to a letter from B. W. Bain of Bechtel Materials and Quality Services to Gary Stanley on 10/16/84. This analysis entailed the following activities: (1) A review of purchase orders/certified material test reports for conformance to AWS D1.1 requirements to verify that all heat numbers for welding filler material are acceptable for structural steel installations, (2) A comparison of all E7018 weld rod heat numbers issued to the DIC Rod Room during the time frame of structural steel installation/erection to verify that correct filler material was used, (3) A review of the DIC weld filler material issuance control procedure/program to ascertain that welders were only issued filler material for the welding procedures to which they were qualified, and applicable to the work being performed.

The results of these investigations were positive, with no discrepancies being found. This effort further substantiates that correct weld filler material was utilized in structural steel erection. DIC Welding Engineering reviewed the procedural details relative to issue of weld filler materials, identifying the control of filler materials explicitly for field issue as well as test shop issue. This review indicates that control is adequate, with supportive documentation, thereby assuring proper filler material issue. DIC Welding Engineering also noted that Quality Inspection performed, as required by DIC Construction Procedure QCP-VII-200, Inspection of Welding Process, random surveillances of welding process attributes. Among the attributes covered by this surveillance are that filler material control is implemented according to applicable welding procedures, and that the welder is currently qualified to the weld technique to be employed.

DIC Welding Engineering performed a review of the specification and procedural requirements relative to the purchase, issue and control of filler materials. It was determined that only E7018 electrodes have been used in AWS D1.1 applications, as required by all site AWS D1.1 welding techniques. All E7018 electrodes purchased by DIC are required to conform to AWS A5.1 (Specification for Mild Steel Covered Arc Welding Electrodes). To substantiate this fact DIC Welding Engineering performed a review of all purchase orders that involved E7018 electrodes. All these purchase orders were proven to have adequate documentation to justify that the electrodes conform to AWS specification A5.1.

Based upon procedural requirements, weld filler material issue controls, and random Quality Inspection surveillances, assurance has been provided that only acceptable filler materials have been utilized and that control has been as required for all AWS D1.1 applications.

Corrective Action 1c).

"Evaluate the adequacy of the DIC inspection criteria and procedures to determine if these elements could have adversely impacted the inspection results. Document and provide this evaluation to KG&E QA for review prior to inspection implementation. Any changes in inspection criteria and procedures shall be provided to KG&E QA for review prior to implementation."

This activity was divided into two elements. The first element was a review of DIC weld inspection criteria contained in QCP-VII-200. The inspection criteria was reviewed to determine compliance with AWS D1.1-75 and Bechtel Specifications 10466-C-132. The second element was to evaluate the results and determine if these elements could have adversely impacted the inspection results.

An AWS D1.1-75 and Bechtel Specification attribute checklist was developed by DIC Quality Engineering. Inspection criteria defined in QCP-VII-200, Appendix II was reviewed in accordance with the checklist. The review indicated that currently QCP-VII-200, Revision 20, meets or exceeds the inspection criteria as delineated in AWS D1.1-75 and the Bechtel specifications. The review of the QCP-VII-200 procedural history revealed most criteria was presented verbatim from AWS or the Bechtel specification. Other criteria, although not verbatim, was interpreted as being in compliance with AWS and the Bechtel specification. The review did indicate four (4) areas of inadequacy. The following is a list of these areas and the time frame affected:

1) Oversized Welds - 4/18/78 - 5/2/84 (Revisions 2 - 19)

   Inspection criteria for oversized welds was not delineated in QCP-VII-200 during this time frame.

2) Convexity - 3/30/77 - 1/18/83 (Revisions 0 - 15)

   During the time frame 3/30/77 through 12/15/81, QCP-VII-200 required the Quality Inspector to utilize the Weld Technique Sheet for compliance. During the time frame 12/15/81 through 1/18/83, QCP-VII-200 required: "Fillet welds may be slightly convex/concave." During the entire period, the following criteria was not delineated in QCP-VII-200 or the Weld Technique Sheets. "Except at outside corner joints, the convexity shall not exceed the value of 0.1S plus (+) 0.03 inches where S is the actual size of the fillet weld in inches."

3) Cracks - 12/15/81 - 5/26/82 (Revisions 9 - 11)

   Inspection criteria for cracks was not delineated in QCP-VII-200 during this time frame.

4) Lack of Fusion - 12/15/81 - 09/22/83 (Revisions 9-16)

   Inspection criteria for lack of fusion was not delineated in QCP-VII-200 during this time frame.

An evaluation was performed to determine if these procedural inadequacies could have adversely impacted the inspection results. The following is the results of the evaluation:

1) Oversized welds: Bechtel Specifications 10466-C-122 and 10466-C-132 were revised 4/18/78. This revision required oversized welds not to exceed 100% or 3/8" greater than specified, whichever is less. During a civil retrofit review of Bechtel specifications and DIC procedures, this procedural inadequacy was identified. Nonconformance Report 1SN 16988CW documented this deficiency and resulted in a recommended disposition of "Use-As-Is". Based on Bechtel's concurrence with this disposition, the omission of this item is considered to have no adverse impact to inspection results.

2) Convexity - Bechtel specifications required welds to meet convexity limits as delineated by AWS D1.1 until 12/08/82. After this date, Bechtel specifications altered the convexity requirement by stating that fillet welds need not satisfy convexity limits of AWS D.1.1. DIC Procedures have delineated criteria as "welds may be slightly concave/convex". Based on procedural control and the relaxed specification criteria, this item is considered to have no adverse impact to inspection results.

3 & 4) Cracks and Lack of Fusion - Inspection criteria for cracks and lack of fusion were inadvertently omitted during general revision from DIC inspection procedures on 12/15/81. The criteria was reinstated in site procedures on 5/26/82 for cracks and 9/22/83 for lack of fusion. The absence of this criteria occurred after the completion of main frame structural steel erection (5/81). However, to establish that there was no impact in other AWS D1.1 applications due to the omission of these items, twenty-six (26) DIC welding inspectors were interviewed. These interviews were used to determine the following:

1) Procedures used for training and inspection.

2) Inspection attributes addressed during training.

3) Inspectors' awareness that cracks/lack of fusion criteria was omitted from procedures for a period of time.

4) Did inspectors inspect/reject welds for cracks and lack of fusion?

The inspectors interviewed had inspected structural steel welds as well as HVAC and electrical support welds during the time frame in which the procedural deficiencies occurred. In all cases inspectors indicated that they had inspected/rejected welds for cracks and lack of fusion. Inspectors were aware of the procedural deficiencies, however, they continued to inspect/reject for cracks and lack of fusion. This is further substantiated based on re-inspection results conducted on structural steel. The rejection rate for cracks and lack of fusion is minimal when compared to the total number of welds inspected.

In conclusion, the review of weld inspection criteria utilized during the history of this project did indicate areas of procedural deficiencies. However, based on the above information, it has been determined that these inadequacies did not result in generic inadequacies in AWS D1.1 welding.

Corrective Action 1d)

"Obtain a documented evaluation to determine the validity of inspections performed with the presence of paint on the weld."

This activity was divided into three elements: obtain information from other utility/AE's that have developed a validation plan, with a subsequent review by DIC Welding Engineering and Bechtel and the addition of site specific requirements and justification. and Bechtel's submittal of a 'position letter' to KG&E for approval.

DIC Management obtained information from Carolina Power & Light Co., and Ebasco Services Incorporated relative to the validity of inspections performed with paint on the welds. This information was utilized by Bechtel in conjunction with their additional research to establish an A/E's position to KG&E. In summary, this position, more explicitly defined in letter BLKES-1348 from C. M. Herbst to G. L. Fouts, is: "With the exception of a number of attributes, fillet welds which have been coated with up to four (4) mils of primer and in some cases, up to an additional ten (10) mils of topcoat can be visually inspected to the AWS D1.1 acceptance criteria. Those attributes which cannot be fully evaluated are of little or no concern on the structural steel at WCGS."

This letter was submitted to KG&E, and subsequently KG&E discussed the validity of inspections performed with paint on welds with NRC representatives. KG&E Nuclear Plant Engineering reviewed letter BLKES-1348, concurring with the position stated by Bechtel in their letter KNPLKWC 84-065 of November 13, 1984.

Corrective Action 1e)

"Utilize personnel certified to ANSI N45.2.6-1978 for the inspection of safety related structural steel welds. Inspections shall be performed in accordance with the DIC Quality Program and training shall be performed and documented to assure that inspectors are cognizant of the DIC Quality program requirements."

This activity was divided into three elements. The first element required incorporation of the CAR-19 Inspection Verification Plan into DIC Construction Procedure QCP-VII-200, "Inspection of Welding Process". The second element required inspection personnel to be certified in accordance with the DIC certification program and ANSI N45.2.6-1978. The third element defined that the inspectors' site specific qualifications would be limited to the reinspection of structural steel welds in accordance with QCP-VII-200.

The Inspection Verification Plan was developed through the combined efforts of DIC, KG&E, and BPC personnel. Revision 0 was reviewed and approved by KG&E Quality Assurance on 10/19/84. Although Revision 0 to the Inspection Verification Plan in QCP-VII-200 was not issued until 10/19/84, some inspections were performed prior to this date by personnel qualified to accomplish these inspections. The same inspection criteria was utilized in these efforts, and all personnel performing these inspection functions were evaluated to ascertain their qualifications to be concurrent with the later certification requirements for KG&E CAR-19. Further discussion of these personnel is included in this discussion of Corrective Action 1e) on the following pages. A meeting was held with the Quality Inspection personnel on 10/20/84 to discuss the impact of the Inspection Verification Plan on their activities and to ensure their understanding of the plan. As a result of this meeting, a new revision, Revision 1, was issued to incorporate inspector feedback and KG&E Quality Assurance comments. Revision 1 of the Inspection Verification Plan was then incorporated into DIC Quality Procedure QCP-VII-200 with Procedure Change Notice 014. On 11/2/84 KG&E Quality Assurance, DIC, and BPC personnel held a meeting to address KG&E Quality Assurance concerns on gouges. Subsequently Revision 1 to PCN-014 was issued to incorporate these concerns into the Inspection Verification Plan.

It was decided that all personnel performing inspection verifications under the CAR-19 Inspection Verification Plan should not only be AWS Certified Welding Inspectors, but also be site certified under ANSI N45.2.6-1978.

ANSI N45.2.6-1978, Section 3.5.2 makes the following recommendations for education and experience when certifying Level II personnel:

1. One year of satisfactory performance as a Level I in the corresponding inspection, examination or test category or class, or

2. High School graduation plus three years of related experience in equivalent inspection, examination, or testing activities, or

3. Completion of college level work leading to an Associate Degree in a related discipline plus one year related experience in equivalent inspection, examination, or testing activities, or

4. Four year college graduation plus six months of related experience in equivalent inspection, examination, or testing activities.

When considering the certifiability of candidates, DIC management ensured that all personnel met the recommendations of section 3.5.2, ANSI N45.2.6-1978.

A training program for inspectors was established on 10/17/84. The program consisted of self study material covering the following subjects:

1. Quality Orientation

2. DIC Administrative Procedure AP-VI-02, "Nonconformance Control and Reporting"

3. The KG&E CAR-19 Inspection Verification Plan (PCN-014 to QCP-VII-200)

Additionally, a meeting was held on 10/20/84 with the inspectors to explain the contents of the Inspection Verification Plan, and to answer any questions they might have about the program. In order to ensure the capability of each candidate, a Field Practical Examination was also administered.

Certification files were compiled on each inspection candidate and are available for review in DIC Quality Training. Each file contains a copy of the inspectors resume', a signed copy of the Education/Experience evaluation form, a copy of the inspector's eye examination, the document of certification, the field practical examination, and the letter of recommendation. Additionally there is a training summary documenting the completion of required training and the training conducted on DIC Quality Procedure QCP-VII-200, PCN-14, Revision 0 and Revision 1.

Each certification file was reviewed by the DIC Quality Training Supervisor to ensure all candidates met the recommendations of ANSI N45.2.6-1978. Each file was again reviewed by the DIC Project Quality Manager (DIC's Certifying Authority) prior to the signing of the Document of Certification. The completed certification files were audited by KG&E Quality Assurance with no findings.

Eleven (11) personnel (Inspectors A through K) were involved in Structural Steel Inspection Verification prior to the issuance of KG&E CAR-19. These personnel were attached to DIC Engineering and were qualified, but not certified prior to the issuance of KG&E CAR-19.

In addition to the eleven (11) personnel above, an additional eleven (11) personnel (Inspectors L through V) were involved in Structural Steel Inspection Verification after the issuance of KG&E CAR-19. The certification status is given below:

INSPECTOR	STATUS
(1) A	Certified
(2) B	Certified
(3) C	Certified
(4) D	Certified
(5) E	Certified
(6) F	Qualified*
(7) G	Qualified*
(8) H	Certified
(9) I	Certified
(10) J	Certified
(11) K	Certified
(12) L	Certified
(13) M	Certified
(14) N	Certified
(15) O	Certified
(16) P	Certified
(17) Q	Certified
(18) R	Certified
(19) S	Certified
(20) T	Not Qualified**
(21) U	Certified
(22) V	Certified

NOTES:

* Personnel who were involved in Structural Steel Inspection Verification prior to the issuance of KG&E CAR-19, but were not involved in Inspection Verifications after the issuance of KG&E CAR-19 were investigated and qualified, but were not certified as they had already left the site or were assigned to other non-inspection related activities.

** Several attempts were made to verify Inspector T's experience after he left site. DIC Quality Training was unable to verify enough experience to qualify Inspector T's to ANSI N45.2.6-1978. All of Inspector T's work was reinspected by certified personnel.

Corrective Action 1f)

"Perform a 100% reinspection of all structurally significant safety related structural steel welds. The identification of "structurally significant" welds shall be made by the Architect - Engineer."

"Structurally significant" joints were defined by Bechtel as all field welded joints which support or potentially support safety related equipment and building components for the purpose of this Corrective Action activity. This basically included all field welds on structural and miscellaneous steel with the exception of handrail, toeplates, grating, checkered plate, stairs, ladders and monorail supports. These are non-Q items which typically see significant service loads during the construction process. Some are designated as II/I, however, II/I seismic loads are considered to be less severe than service loads. Monorails have been load tested as part of startup procedures, and were therefore not included in the scope of structurally significant items requiring reinspection. The joints were selected by Bechtel based on a review of erection drawings prepared by the structural and miscellaneous steel fabricators and a review of Field Change Request (FCR's), Nonconformance Reports (NCR's), Construction Variance Requests (CVR's) and Structural Steel Fabrication Requests determined applicable.

The DIC Nonconformance program, as defined in DIC Construction Procedure AP-VI-02, "Nonconformance Control and Reporting", was utilized to obtain and document a suitability for service evaluation of welds that were inaccessible due to physical location or embedment in concrete. All deficiencies identified during reinspection activities performed in accordance with Procedure Change Notice - 014 to DIC Construction Procedure QCP-VII-200 were identified on nonconformance reports for further dispositioning and resolution.

Bechtel performed a case by case evaluation of each structurally significant joint inspected according to the data furnished on Inspection Data Sheets and nonconformance reports. Their evaluation provided a determination of whether each structurally significant joint's as-built condition met design allowables, whether the as-built condition was a significant deficiency in accordance with 10 CFR 50.55(e), and whether any rework or repair to each joint was required.

The following is a statistical summary of the evaluation completed by Bechtel on all structurally significant joints:

TOTAL AWS WELDING

INSPECTIONS AND ENGINEERING EVALUATIONS

BUILDING	TOTAL JOINTS	JOINTS INSPECTED	JOINTS EVALUATED	JOINTS REQUIRING REWORK (1)	ADDITIONAL JOINTS TO BE REWORKED (2)	SIGNIFICANTLY DEFICIENT JOINTS (10CFR50.55(e))
AUXILIARY	694	694	694	8	40	0
REACTOR	1300	1300	1300	69	10	0
CONTROL	265	265	265	3	14	0
DIESEL GENERATOR	98	98	98	2	1	0
FUEL	277	277	277	0	2	0
ESWS PUMPHOUSE	36	36	36	0	0	0
TOTAL	2670	2670	2670	82	67	0

(1) DESIGN ALLOWABLE STRESSES ARE EXCEEDED IN THE AS-BUILT CONDITION

(2) DESIGN ALLOWABLE STRESSES ARE NOT EXCEEDED IN THE AS-BUILT CONDITION. THESE JOINTS ARE BEING REWORKED PER KG&E MANAGEMENT DIRECTION TO INSTALL MISSING AND UNDERLENGTH WELDS UNLESS PROHIBITED BY FIELD CONDITIONS.

Finding #2 of KG&E CAR-19 stated, "An Inspection Verification effort of safety related structural steel welding, undertaken by AWS certified weld inspectors identified several areas of deficiencies. These deficiencies are categorized as: undersized welds, weld defects, incorrect configuration, weld underrun, and weld undercut."

One (1) corrective action was determined to be appropriate for resolution of this finding, although this primary corrective action was subdivided into seven (7) research/data accumulation activities.

Corrective Action 2a)

"Determine and document the "root cause" of the previous acceptance of deficient structural welds. Analyze the HVAC Support, Electrical Support, Pipe-Whip Restraint and any other safety-related program utilizing AWS D1.1 Welding to ensure that the same "root causes" inherent in the structural steel welding program were not generic to other programs."

This summary reviews activities 2a-1 through 2a-7 of CAR-19 to determine the root cause of the previous acceptance of deficient structural welds and analyzes those root causes to determine if they were inherent to other safety-related programs utilizing AWS D1.1 welding.

A review of DIC Quality procedures was performed by Quality Engineering to determine if any historical procedural inadequacies could have been a contributor to "root cause". Although some historical deficiencies in inspection criteria were found to have existed, research demonstrated that some of the procedural inadequacies occurred after the vast majority of structural steel erection activities had been completed. Interviews with a sample of Quality Inspectors revealed that inspectors were cognizant of the omission of two other criterion (lack of fusion and cracks) during an applicable time frame, but inspected for these deficiencies in spite of their omission. Based upon this cumulative research procedural weld inspection inadequacies are not considered to be contributors to "root cause" of previous acceptance of deficient structural welds.

DIC Inspection training and certification procedure AP-VI-01 was used to train and certify Quality inspection personnel during the structural steel erection time frame. This procedure was analyzed to verify compliance to ANSI N45.2.6-1978, and was found to be in accordance with ANSI requirements. An evaluation of ANSI N45.2.6 requirements revealed that DIC procedure AP-VI-01 was in full compliance to ANSI requirements for the structural steel erection time frame and through all subsequent revisions to date.

The "root cause" of the previous acceptance of deficient structural welds has been determined to be due to inspection implementation and inadequate implementation of related procedures. Each of these contributing factors has several facets that are considered to be partial reasons for "root cause".

Differences in inspection techniques and consideration of inspection attributes for the original inspection time frame vs. the CAR-19 reinspection time frame are definite root cause contributors. The differences indicated are common to the nuclear construction industry and have been recognized as prevalent at many projects. A white paper documentary prepared by recognized nuclear construction consultants Reedy, Herbert, Gibbons and Associates, Inc. dated August 11, 1983, clearly defines the subject differences during their in-depth analysis of weld inspection on nuclear sites. (See Appendix IV.G)

The differences cited, inspection technique and inspection attributes, are addressed in section I of this white paper, "Continuous Measurement of Fillet Welds". The paper states that until about 1980 accepted inspection practice did not entail 100% physical measurement of each inch of welding, but rather depended upon individual inspector's evaluation of the weld's acceptability. Around 1980 QA/QC Inspectors began using fillet weld gauges to measure each inch of fillet weld to verify that the specified minimum weld size was met for the continuous length of weld. This physical measurement gradually replaced the previous accepted practice of visual judgement. The paper concludes that there has been a progression of the practice of physically measuring each inch of weld to a serious extreme.

The documentary cites that there is no requirement either in the ASME Section III Code or AWS D1.1 Standard to continuously measure the full length of fillet welds. Both ASME and AWS permit deviations from minimum size fillets as documented in ASME NB/NC/ND - 4427 and paragraphs 8.15.1.7 and 9.25.1.7 of AWS D1.1. The paper further contends that inspections can and should be made on a random basis to determine nominal sizes with no detriment to safety. Additional sections of this documentary address "Undercut Provisions of AWS D1.1" and "Encroachment on Minimum Thickness" with similar conclusions.

DIC research has shown that the inspection technique implemented during erection/inspection of structural steel at Wolf Creek was in accordance with common industry practice as stated in the previously referenced documentary. Inspectors were of the understanding that visual judgement was acceptable as an inspection technique in checking for nominal weld size, and that visual evaluation rather than 100% physical measurement of fillet welds was acceptable for assuring that welds met visual inspection attributes.

Given these considerations, one should expect a reinspection program using current applicable techniques to find deficiencies in welds previously accepted. The reinspection technique is one of 100% physical measurement of all attributes applicable rather than the visual judgement initially employed as acceptable during the structural steel erection time frame.

With the previous considerations in mind, an examination of the weld deficiencies identified during reinspection and their relative significance to the overall integrity of the initial inspection effort is in order.

The scope of the CAR-19 reinspection effort identified two thousand six hundred seventy (2,670) joints requiring reinspection. Of the two thousand six hundred seventy (2,670) total joints, two thousand eight hundred seventy (2,870) welds exhibited discrepancies of the more than eleven thousand (11,000) welds reinspected according to procedure QCP-VII-200, Procedure Change Notice 14. Each weld reinspected could have potentially contained five (5) categories of deficiencies according to the method utilized for tracking during the CAR-19 program, those being: undersize, defects (cracks, lack of fusion, incomplete penetration, overlap, slag inclusions, porosity, craters), underrun, undercut and configuration. Of the two thousand six hundred seventy (2,670) structural joints inspected, the following quantities of weld deficiencies were noted by category: 1,061 undersize, 330 defects, 476 underrun, 107 undercut, and 1,562 configuration.

The quantities of deficiencies noted for the three categories following are minor based upon a percentage comparison to the total number of welds reinspected. The approximate percentages for each of these three categories are, underrun 4%, undercut 1%, defects 3%. These percentages are within expectations considering reinspection emphasis and the previously noted differences in inspection technique and accepted inspection practice. Further statistical analysis revealed a majority (more than 60%) of the welds rejected for undercut discrepancies to be in excess of the 1/32" allowable undercut criterion by less than 1/16". A majority (approximately 60%) of the welds found to be underrun were underrun by less than 1/2". An analysis of the attributes contained within the 'defect' category revealed only small quantities in each. Based on the above statistical analysis , the discrepancies identified in the categories of underrun, undercut and defects are not considered to be contributors to the root cause that previously accepted welds were found deficient upon reinspection.

The quantity of welds rejected that did not meet the minimum leg size as specified on the design document, or exceeded the code allowable 1/16 inch undersize for less than 10% of the length of the weld, represents a percentage of 9% deficiencies for the total welds inspected. Discussions with DIC inspection personnel and Quality Management aware of approved inspection practices utilized during the structural steel erection time frame indicated that inspection methods were similar for this period to those described in the previously addressed documentary by Reedy, Herbert, Gibbons and Associates, Inc. Of the welds identified as being undersize, more than 90% were undersize by less than 1/8", further substantiating that inspection methods were as previously described. Based on the above evaluation, the quantity of deficient welds identified as being undersize is considered an indicator that previously accepted inspection techniques was the root cause of previously accepted welds being found deficient upon reinspection.

The quantity of welds indentified during reinspection exhibiting configuration deficiencies represented 13% of all deficiencies for the total welds inspected. Of the total number of deficiencies, more than 80% were revealed by research to be directly attributable to one design change implemented in February, 1978. This Design Change Notice C0011, Rev. 7, dated February 23, 1978, changed detail 10 on drawing C0011 to limit the length of the return welds on beam clip angle to embed plate welds. The significant number of discrepancies identified in this category indicates that the design change was not given sufficient emphasis by DIC Engineering, craft, and Quality Inspection to enable deviations from this requirement to be adequately controlled. This category is the largest single contributor to "root cause" of previously accepted deficient structural welds. Bechtel, as the Architect Engineer, performed an evaluation of all welds reinspected to determine which welds were acceptable from a technical viewpoint relative to allowable stress calculations and which welds would require rework in order to meet this criterion. From this evaluation 2589 joints were determined to be technically acceptable whereas 82 required rework. These statistics, revealing that 97% of the joints reinspected were technically acceptable, are indicative that the relative degree of significance of the deficiencies identified due to reinspection is minor.

Those areas utilizing AWS D1.1 welding other than structural steel were identified as: Pipe whip restraints; miscellaneous steel and embedment fabrications; fire dampers and safety-related ductwork and supports; electrical raceway supports; electrical equipment installation; and stud welding.

Previously compiled information including Construction Self Assessment Reports, KG&E QA Reports and Surveillances, DIC QA Reports, DIC Project Monitoring Program audits, DIC Corrective Action Reports and correspondence was reviewed to determine results of previous investigations of AWS D1.1 welding. No findings were noted during this review that could be considered contributing factors to root cause. Electrical II/I support welds were reinspected by Bechtel (ELKC: 009) through the "Sampling and Inspection Program for Electrical Support Welds" (7/84). Three hundred nine (309) were inspected and found acceptable. Electrical Quality Welding Inspectors performed inspections on Class IE support welds raceway (8/82). Pipe whip restraint welds were 100% nondestructively tested. HVAC ductwork support welds were 100% reinspected through implementation of DIC Corrective Action Report CAR-1-M-0012 and a traveler system was initiated to maintain better control and accountability (3/82-1/83).

Programmatic elements utilized in the inspection and documentation of the various applications of AWS D1.1 welding differed depending upon the Quality discipline responsible for inspection activities. The following methods were utilized in the applications noted to provide inspection documentation:

a) Raceway Supports - Raceway Support Checklist

b) Electrical Equipment - Quality Equipment Mounting Checklist in addition to MSSWR's

c) Fire dampers and safety-related ductwork and supports - Mechanical Travelers

d) Miscellaneous steel and embed fabrication - MSSWR's

e) Stud welding to embeds - Surveillance Reports

f) Pipe Whip Restraints - MSSWR's in addition to Nondestructive Examination Reports

All the methods utilized above were effective in providing inspection assurance and documentation of the respective activities when properly implemented. The travelers utilized as well as the other checklists noted provided a closed loop system where individual accountability for a weld was required, controlled, and documentation verified accurate and complete by Quality personnel. Conversely Miscellaneous Structural Steel Weld Records (MSSWR's) were used in an open-ended system for Main Frame Structural Steel Installations where craft construction personnel were responsible for control, maintenance and processing of this record following its completion. This system proved less than satisfactory in some applications, resulting in document retrievability problems that have been addressed by DIC and KG&E Corrective Action Reports.

In summary the programmatic elements as described in DIC procedures for each application of AWS D1.1 welding are adequate when properly implemented by the persons responsible for those activities. MSSWR's utilized in documenting structural steel weld connections were the subject of inadequate implementation of procedural requirements, resulting in the problems being addressed in this report. The research accomplished in completion of this activity revealed no inherent "root cause" generic to all programs utilizing AWS D1.1 welding, but rather indicates that the root cause of the previous acceptance of deficient structural welds was as delineated earlier in this section.

Finding #3 to CAR-19 stated, "A small number of safety related structural steel welds were not made or had missing material."

Corrective Action 3a)

"Forward the "as-built" information to the Architect/Engineer via an NCR to obtain an engineering evaluation and disposition".

All missing welds or missing material detected in the reinspections performed were documented on nonconformance reports reflecting the as-built condition found by inspectors. Of the two thousand six hundred seventy (2,670) joints reinspected (more than 11,000 welds) only two hundred seventy-three (273) welds were identified as missing where the applicable design drawing required their installation. Of the two hundred seventy-three welds not installed, one hundred twenty (120) were applicable to the polar crane girder radial stops (44%), ninety-seven (97) were due to beam seats not installed (36%), eighteen (18) were due to missing welds on six (6) pressurizer support welds (7.0%), and the remainder (38) due to missing welds on clip to beam or plate installations (13%).

Under the purview of KG&E Construction, a detailed investigation was undertaken by DIC Engineering and Management personnel to determine the root causes of missing welds and materials in each case. Significant points of that investigation included: grouping of missing welds/materials into categories to aid in research; compilation of factual data and analysis for trends/patterns; a thorough review of all applicable design change documents that may have deleted some of the items in question; visual examinations of the areas where installations should have been made; and interviews with craftsmen, craft supervision, DIC Engineering and Quality personnel for information that may have added to root causes.

Missing welds and materials were grouped into categories based on similarities that could be determined to exist in function or construction sequence. Five groups were defined, those being: beam seats and attachment welds, pressurizer support welds, Polar Crane girder radial stop welds, miscellaneous materials and associated welds, and beam to channel clip welds (for one application only). Each of these groups is discussed in detail in the following paragraphs in presenting the respective data accumulated and the conclusions drawn.

Beam seat installation welds accounted for ninety-seven (97) of the missing welds identified. Upon investigation several reasons were found as contributing factors to the root cause of failure to install beam seats as required. All beam seat connections in question were relevant to installation detail 10 on drawing C0011, which gave no required weld size, but referenced note 14. Note 14 stated, "When end reaction exceeds maximum weld size capacity provide seat angle." Discussions with personnel available who were involved with structural steel installations revealed that this note may have been incorrectly interpreted as an 'option' for beam seat installation. This resulted in a craft opinion that the beam seat was intended as a construction aid to be used only during the erection process and then removed. This contention is supported by the fact that ninety-three percent (93%) of the areas/records examined pertaining to beam seat installation revealed that the beam seats were installed prior to the beam's installation. Seventy-two percent (72%) of the embed plates investigated showed evidence of temporary welds made to attach a beam seat as a construction aid during the erection sequence, but the beam seats were not found installed upon field investigation. A majority of the beam seat associated welds missing were the beam seat to beam welds, which further indicates the questionable beam seats were tack-welded to the embed, used as a construction aid, then removed prior to welding to the beam. These above factors substantiate that the root cause of missing beam seat welds (i.e., beam seats not installed) was due to a misunderstanding of the beam seats' intended application as a permanent installation. This root cause conclusion is supported by the data accumulated and discussed in the preceding paragraphs. All missing beam seats and their respective required welds were installed as a part of KG&E and DIC Management's direction, unless prohibited by field conditions.

The missing pressurizer support welds totaled eighteen (18) welds on six (6) supports. The six (6) supports with missing welds are all of the upper supports for the pressurizer beam foundation, and all six (6) supports were found to be welded identically to each other. One inspector performed all final visual inspections of the pressurizer supports, indicating a possibility of human error being a contributor to root cause. Investigation results indicated a misinterpretation of erection details and requirements as the primary root cause of the eighteen (18) missing welds. Twenty-four (24) welds not detailed as required installations were added but not required by design drawings. The conclusion reached for root cause of the missing welds on the pressurizer supports is that DIC construction craft and Quality personnel misinterpreted the installation details and applied this misinterpretation consistently in the construction and inspection of all six supports. Nonconformance report 1SN 20509CW was generated to document these circumstances and all missing welds were installed as a part of the disposition, unless prohibited by field conditions.

The Polar Crane girder radial stops were the subject of one hundred twenty (120) missing welds. These missing welds are documented on nonconformance reports 1SN 21308CW, 1SN 21309CW, 1SN 21310CW and 1SN 21311CW. Facts gathered during the investigation of these missing welds indicate that a series of drawing revisions and misinterpretation of erection installation details resulted in DIC construction error in not making all required welds on sixty (60) radial stops. The appropriate facts are as follows:

- American Bridge Drawing E117 (C-121-8360) was revised concerning the radial stop connection. Two of the three revisions to section A were attempts to clarify the desired weld configuration at the radial stops.
- Revision B to American Bridge drawing E117 was produced to clarify where actual welds were expected.
- Revision C of Drawing E117 in part added "one side only" to the inner "C" portion of the radial stop welds.
- Bechtel Drawing C-OS2963 concerning the polar crane girder radial stop welds was altered at Revision 6 to note on Section A that the weld on the inner "C" indention was to be made on one side only.
- The MSSWR's documenting the radial stop welds made indicate erection during 2/80-3/80, before American Bridge drawing E117 clarified the installation detail on Revision E, dated 12/80.

Upon reinspection NCR #1SN 21196CW was initiated describing the deficiency in nonexistent radial stop welds. The NCR was voided in-process by the CAR-19 Inspection Supervisor due to a misinterpretation of requirements according to details on the American Bridge drawing E117, that seemed to indicate a weld installation detail requirement concurrent with the actual welds found installed during reinspection. Based upon the preceding facts, it is concluded that the root cause of missing Polar Crane girder radial stop welds is due to unclear weld detail installation requirements as projected on the American Bridge drawing E-117, and subsequent incorrect interpretation of weld installation requirements by DIC personnel.

The missing welds identified for installations involving other miscellaneous materials and welds missing are of a smaller quantity. Thorough investigation revealed the root cause of these missing welds to be due to a lack of formal follow-up and inadequate statuses of completed work and the subsequent completion of unfinished work. The missing welds on the Incore tubing supports revealed that all investigatory information supports the hypothesis that these missing welds were not installed due to oversite. The four lateral support brackets, two at each of the vertical angle supports (Incore tubing supports) located 32' - 2 3/4" north of the Reactor Center Line and 4' 10" east and west (one each direction) of Reactor Center Line on Drawing GOS2919 were added by revision to drawing GOS2924 after the supports had been presumed completed.

Nonconformance report 1SN 21273CW documents missing welds on channel clips to beam attachments. The channels that American Bridge Drawing #C121-10675 shows welded to a beam web along A2 at Elevation 2042' are bolted instead. The channel clips are bolted to the web using the same bolts as removable beams on the opposite side of the web. Research found that the installation of the channel and removable beam was late in the construction sequence of this area, also. Since the channel clips and removable beam clips are bolted through a beam web with the same bolts, the channel clip attachment welds were probably assumed to be unnecessary by the construction personnel responsible for installation.

If the removable beams had been disconnected for the purpose of construction, it would have become necessary to weld the channel clips to the beam web at that time. The beams and channel in question were installed late in the construction sequence of the area, removal of the beams never became mandatory, the welds were not a recognized priority and were never installed as required. The root cause of these missing welds is due to DIC error in assuming the bolted connections were acceptable rather than the required welds. In the miscellaneous group, investigations revealed that welds or material found missing were those welds or materials that would not impede construction progress related to that connection.

Finding #4 to CAR-19 stated, "One (1) weld was documented as having been inspected when in reality the weld was not made. (Ref. NCR 1SN 20495CW)."

Corrective Action 4a)

"Investigate the concern to determine the root cause of the error. Immediately notify KG&E Quality Assurance if any other problems of this nature are identified. Document the investigative actions. The notification of KG&E QA shall not preclude the issuance of an NCR."

The results of the CAR-19 inspection effort were tracked and each case where a missing weld or missing material was identified was researched thoroughly by DIC Engineering to determine whether documentation existed pertinent to the installation of the missing weld/material. Miscellaneous Structural Steel Weld Records (MSSWR's) were reviewed to determine if a trend or pattern existed. Nonconformance reports identifying missing welds were compared to MSSWR's to determine if there were repetitive occurrences.

Applicable drawings were reviewed for similarities in beam numbers, floor layout and beams at similar locations in an attempt to further identify possible sources of confusion. As a result of the investigations conducted only two (2) cases were identified where inspection documentation existed for welds not installed.

The first case is the installation of beam No. 524B2 and its connection to an embed in the Auxiliary Building. All available information indicates that DIC Quality Inspector W made a human error when documenting the inspection of this beam connection. A review of the drawings shows that the beam configuration and floor layout in the area (elevator shaft and equipment hatch) directly beneath the beam connection in question are very similar. In addition, the beam below beam 524B2 connects at the same building coordinates.

It is possible that Inspector W could have been one elevation beneath where he should have been when inspecting the connection. Out of the multiple welds inspected by Inspector W this problem occurred only once. If actions which would result in other conclusions had occurred, it would be reasonable to assume that they would have occurred repeatedly. Inspector W's signature appears on over eight hundred (800) MSSWR's. Each MSSWR could document multiple weld inspections, therefore, Inspector W very likely inspected over one thousand (1,000) structural steel welds, with the result that this type of problem occurred once. A telephone conversation between Inspector W and DIC management personnel concerning this incident revealed no information that Inspector W could offer, since he could not recall the specific connection from the more than eight hundred (800) he inspected. The root cause conclusion in this case is human error.

The second case is the installation of beam No. 95B5 to an embed in the Control Building. All available information suggests that DIC Quality Inspector X made a human error when documenting the inspection of this beam connection. The MSSWR documenting this connection shows Inspector X's confusion in that he entered the joint number incorrectly when filling out this portion of the MSSWR, then lined through, initialed and dated his error, and entered what he thought was a correct entry. Drawing K6711-XI-I-E13 details this connection, but is unclear in that it does not designate the connection number for the beam clip to embed weld, and only lists the beam seat number (91M1).

Further research revealed that Inspector X completed one hundred eighty-three (183) MSSWR's during his tenure on site, but only six (6) of these MSSWR's were related to structural steel weld inspections. This is indicative that Inspector X was possibly confused by the details on the erection drawing. It is probable that Inspector X attempted to document the welds attaching the beam clips to beam 95B5, since no retrievable MSSWR is on file for these welds. These circumstances are documented on nonconformance report 1SN 20798CW for disposition and resolution. The root cause conclusion in this case is human error.

Finding #5 of CAR-19 stated, "Objective evidence that the mechanical and structural inspection/documentation problems identified in KG&E QA Surveillance Report S-372 were rectified has not been provided."

Corrective Action 5a)

"Provide objective evidence that the mechanical and structural support welding inspection/documentation problems identified in Surveillance Report S-372 have been corrected. If such evidence is not available, research the extent of the problem and take the appropriate remedial actions." Activity 5a was broken down into two categories. 5a-1 was to review and provide objective evidence that Mechanical Deficiency Reports identified in S-372 have been correctly closed out. 5a-2 was to review and provide objective evidence that Civil Deficiency Reports identified in S-372 have been correctly closed out.

A total of forty-two deficiency reports were reviewed encompassing the departments of Civil, Civil/Welding, Mechanical, and Mechanical/Welding which are identified in S-372. Below is a brief description of the closure to each Deficiency Report (DR). (Deficiency Reports underlined.)

1. 6451 was upgraded to an NCR (1NN 4969CW) because all welds were encapsulated in concrete and deemed structurally acceptable by the A/E.

2. 6536 and 6538 were "Close in Process" because the hangers were "VOIDED"; hangers were removed mechanically, and Quality inspected the area to insure soundness of the affected structure.

3. 6559, 6557, 6560, 6568 pertained to electrical raceway hangers. DIC Mechanical/Welding inspectors performed inspections to ensure the soundness of the removal area after cut down, according to DR disposition. The reinstallation of these hangers was inspected by DIC Electrical Quality Inspectors and documented on Electrical Quality Raceway Support Checklists.

4. 6535, 6537, 6539, 6576, 6575, had dispositions calling for cut down of hangers only, therefore only the verification for the inspection of the soundness of the removal area was required.

5. 6585 disposition was "Close in Process" because no hanger could be located in the area called for by the Deficiency Report. The two closest hangers have the required documentation and their respective documentation is attached to the Deficiency Report.

6. 6249, 6250, and 6349 have MSSWR's to reflect proper closure, but the hangers are now voided. Based on this research an inspection of the applicable Building, Location, and Area (BLA) for these hangers was initiated and the hangers were verified as cut down.

7. The remaining Deficiency Reports have MSSWR's attached to reflect the proper documentation for the safety-related attachment welds. This group of Deficiency Reports numbers 26 total.

No violations of 10 CFR 50 Appendix B exist in Items 1 thru 5 as defined in the criteria of KG&E Surveillance S-372. The violations listed in S-372 pertained to welding documentation on Structural Steel. The dispositions for the deficiency reports in items 2 thru 4 require the removal of deficient welds. In some cases MSSWR's were used to document the removal so these MSSWR's show blanks (or as non-applicable) for W-100, weld technique, filler material, etc. These should not be mistaken for incomplete MSSWR's for required welding, since MSSWR's are not required for this activity.

In summary, all deficiency reports in KG&E Surveillance S-372, have been reviewed and proper closure verified. All the deficiency reports were closed properly according to the results of our investigation.

DEFICIENCY REPORT #

- 6248
- 6249
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- 6280
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V. Conclusions

The technical evaluation of WCGS structural steel significant joints, which was performed by Bechtel based upon reinspection data accumulated, established that safety related AWS D1.1 structural steel welding complies with all Quality criteria as specified in the related design documents, and is within the tolerances of acceptable deviation as determined by the Architect/Engineer. This evaluation for structural integrity was based upon this cumulative data that reflected the as-built condition of Bechtel identified structurally significant joints prior to any rework or repairs.

Two thousand six hundred seventy (2,670) structurally significant joints were identified by Bechtel and were subsequently reinspected by DIC Certified Quality Inspectors who were all also AWS certified Welding Inspectors. Eighty-two (82) of these significant joints required rework due to design allowable stresses being exceeded in the as-built condition. None of the structurally significant joints where discrepancies were identified would have failed if left uncorrected.

Research accomplished by DIC and Bechtel personnel resolved that DIC welders and welding procedures applicable to AWS D1.1-1975 welding of structural steel installations were satisfactorily qualified in accordance with AWS requirements. Additional research resulted in assurance that programs and procedures applicable to the purchase and control of weld filler materials used in AWS D1.1 applications were in compliance to AWS requirements. Investigations into site implementation of these requirements and procedures provided assurance that implementation had been effective and properly controlled by DIC during project construction activities.

The retrievability and control of Miscellaneous Structural Steel Weld Records (MSSWR's) was investigated, and a determination made that inadequate implementation of DIC construction procedures was a contributing factor to retrievability and accountability problems with MSSWR's relative to structural steel applications. Thorough analysis of each applicable program was undertaken by DIC Quality Engineerning to determine if similar programmatic or procedural requirements existed, and whether inadequate implementation had resulted in similar deficiencies. The results of these assessments determined that no programmatic problems existed in any other AWS D1.1 application relative to inspection documentation required for weld inspections. Evaluations of each application identified that more efficient documentation methods were utilized, and in each case there was more effective control of the required documentation through its initiation and processing cycles. Review of Quality Assurance historical audits and surveillances and an evaluation of procedural implementation adequacy further assured no problems existed in any other AWS D1.1 application similar to the MSSWR retrievability problem on structural steel welding.

Hardware applications of AWS D1.1-1975 requirements were also analyzed to determine if the root causes applicable to the previous acceptance of deficient structural steel welds were of potential impact in applications other than structural steel. Reinspection and Corrective Action reports existed in every case to ensure the acceptability of installed hardware where AWS D1.1 welding was utilized except in Electrical Equipment foundation welds. DIC Management determined that a subsequent investigatory effort was necessary to provide data to ascertain the possible existence of deficiencies in welding and shimming in these installations. DIC Corrective Action Report 1-EW-0046 was initiated to document and accomplish these activities.

DIC Corrective Action Reports (CAR) 1-W-0029 and 1-C-0031 were evaluated to determine why neither of these documents resulted in the appropriate identification and effective resolution of structural steel welding and documentation problems prior to KG&E Corrective Action Request 19. CAR 1-W-0029 was found to be effective for the scope of welds identified. A conclusion was reached, however, that if a larger sample size had been utilized for CAR 1-W-0029's scope of inspection activities, that corrective action concurrent with that identified for KG&E CAR-19 may have been decided appropriate as resolution for the identified problems.

With the generation of DIC CAR 1-C-0031 DIC Management recognized that documentation did not exist for all structural steel welds as procedurally required, and nonconformance reports were generated to document these inadequacies. 'Use-As-Is' dispositions were assigned to these nonconformance reports based upon the existence of defined programs and procedures that required 100% inspection and documentation of structural steel welding activities. An assumption was made that although required documentation was not 100% retrievable, the programs in place during structural steel installation/inspection activities did result in all installations being completed and inspected.

Neither CAR 1-W-0029 nor CAR 1-C-0031 required matching of MSSWR's to structural steel welds or welded connections. If this had been a required corrective action for either CAR, the problems identified in portions of KG&E CAR-19 would have been realized.

The findings addressed in CAR-19 in addition to missing MSSWR's included deficiencies identified in previously accepted structural steel welds, missing structural welds or missing structural material, and documentation that a weld was inspected and accepted, but no weld was installed.

An evaluation of the DIC Quality inspection training program demonstrated that this program and related procedures were in compliance to ANSI N45.2.6. Further investigation concluded that Quality inspection training was appropriate and adequate during the structural steel installation time frame. An evaluation of DIC Quality inspection procedures and criteria applicable to the original structural steel installation/inspection period revealed several procedural inadequacies. A thorough analysis of the omission of each inspection criterion of AWS D1.1 structural steel applications was accomplished, with the conclusion that no adverse impact had resulted from these procedural inadequacies relative to AWS D1.1 welding inspection.

Inspection criteria to be used in the structural steel reinspection activities was procedurally defined and training of all personnel completed prior to reinspection initiation. Sufficient technical justification was established by Bechtel to validate inspection of welds through a predetermined maximum thickness of paint. An analysis of reinspection results determined the root cause of the previous acceptance of deficient structural welds to be due to DIC inspection implementation differences relative to inspection vs. reinspection techniques, and inadequate implementation of applicable DIC procedures.

Two (2) of the welds on joints reinspected were initially thought to be documented as being installed when in reality they were not installed. Research revealed no evidence to indicate that either was a case of deliberate falsification. Additional investigations resulted in a conclusion that human error was the cause of incorrectly documenting these nonexistent installations.

Reinspection found that some welds and materials were not installed as required by design documents. These errors were primarily due to craft/engineering errors relative to misunderstanding of installation drawing details and requirements. Failure to install these welds and materials, although in some cases determined to be significant in impact to design stress allowable calculations, would not have resulted in material or structural failure if left uncorrected. All missing welds will be reworked in accordance with KG&E Management's direction, unless prohibited by field conditions.

As a result of those concerns identified in KG&E CAR-19, DIC conducted an assessment of the programmatic aspects of the Piping, Hanger, Mechanical, Electrical and Civil disciplines to ascertain the adequacy of those programs instituted in the construction of Wolf Creek Generating Station. Other than the concern identified in DIC CAR 1-EW-0046 the program assessment has established a high degree of confidence in the adequacy of the overall DIC Construction program to assure compliance with 10CFR50, ANSI N45.2, FSAR, design and procedural requirements. The cause of the adverse conditions identified in KG&E CAR-19 and DIC CAR 1-EW-0046 is limited to these areas in that all other areas of work which would have been rendered inadequate or suspect due to the identified root cause have been adequately addressed through subsequent means such as retrofit or reinspection programs.

After completion of the program assessment, which addresses all aspects of the DIC Construction programs in total, and as they might have been affected by the identified root cause of deficient structural steel welds, it is the conclusion of this assessment that all significant problems have been identified and are being adequately addressed and resolved through appropriate corrective actions.

This program assessment is included in the Appendix, section VI.H of the KG&E CAR-19 Final Report, and has concluded that a satisfactory level of confidence exists to assure compliance with 10CFR50, ANSI N45.2, the FSAR, and Design and Procedural requirements.

The objective of KG&E CAR-19 was to establish by review of Construction and Quality programs, as-built conditions, nonconformance identification and correction and by design evaluation and/or rework, that all structural steel erection commitments in the Wolf Creek Final Safety Analysis Report were satisfied. Through the cumulative efforts in the resolution of CAR-19 assurance was obtained that all significant Quality criteria as specified in the related design documents were satisfied, within the tolerances of acceptable deviations as determined by the Architect/Engineer.

FORM 1177 REV 1-67

KG&E

INTEROFFICE CORRESPONDENCE

TO: G.L. Fouts

KGLKQC 84-002

FROM: R.M. Grant

DATE: October 17, 1984

SUBJECT: Corrective Action Request (CAR) No. 19

Attached is Corrective Action Request (CAR) #19 which is being issued to obtain corrective actions to problems associated with safety-related AWS D1.1 structural steel welding.

Please respond to this Corrective Action Request by completing Section 5 of the subject CAR. Your schedule for implementing corrective actions and an explanation of any actions you have already taken should be submitted to me by October 24, 1984.

RMG/dkb

cc: K.R. Brown
G.L. Koester
F.J. Duddy
W.J. Rudolph II
C.E. Parry
C.G. Patrick

KGF-60 Rev. 2/84

IE 58531

WOLF CREEK GENERATING STATION

CORRECTIVE ACTION REQUEST

CAR NO. 19

1. CONDITION DESCRIPTION:

See Attached.

2. RESPONSIBLE ORGANIZATION:

KG&E Construction

3. CAUSE OF CONDITION:

QA Program breakdown associated with safety-related AWS D1.1 structural steel welding.

4. RECOMMENDED CORRECTIVE ACTION:

See Attached.

Reviewed 10-17-84 Date

Quality Branch Representative 10-17-84 Date

5. SCHEDULE FOR IMPLEMENTATION OF ACTION:

Responsible Supervisor Date

6. NRC REPORTABLE: [✓ Yes] No
9/18/84 See Attached Telephone Call Record

7. STOP WORK ACTION TAKEN: Yes [✓ No]
If Yes, Report # ______

8. CORRECTIVE ACTION VERIFIED - Method of Verification:

Quality Branch Representative Date

Supervisor Date

9. CAR CLOSED: Yes

Quality Branch Representative Date

Supervisor Date

10. APPROVAL ______ Director - Quality DATE ______

I. CONDITION DESCRIPTION

A. Objectives

- To document a consolidated project plan for the identification, evaluation and resolution of problems associated with Safety-Related AWS D1.1 Welding.

- To provide assurance, based on objective evidence, that AWS D1.1 Welding of Safety-Related Structural Steel complies with all Quality Criteria as specificed in the related design documents and is within the tolerances of acceptable deviations as determined by the Architect - Engineer.

- To provide assurance that the documentation which supports the inspection of safety related structural steel welds is:

  - Available
  - Complete
  - Reflects appropriate information
  - Traceable to the item or activity

- To evaluate supporting elements of the DIC Quality Assurance Program to ensure that those elements were adequately and effectively implemented to demonstrate that the DIC welding of safety related structural steel, HVAC Supports, Electrical Supports, Pipe Whip Restraints and any other AWS D1.1 safety related welding activities were in compliance with the FSAR (i.e. AWS D1.1 - 1975) and the Design and Construction QA Program Manual, Section 17.1.3.

B. Definitions

Joint - A structural steel welded connection. A joint may consist of numerous welds. A joint may also be referred to as a connection.

Weld - A continuous length of weld material with only one start and one stop.

MSSWR - Miscellaneous Structural Steel Weld Record; a form used by DIC to document installation and inspection data for welds made to structural steel.

AWS D1.1 - American Welding Society's Structural Welding Code. This code covers welding requirements applicable to welded structures. It is to be used in conjunction with any complementary code or specification for the design and construction of steel structures.

Miscellaneous Structural Steel - See Attachment 3 for Complete Definition.

Structurally Significant Welds - See Attachment 3 for Complete Definition.

C. Background Information

- KG&E Surveillance Report S-372 (October, 1981) identified a Quality Program breakdown due to the following deficiencies:

  - Missing inspection documentation
  - Incomplete/improper resolution of identified electrical, mechanical and structural weld documentation deficiencies.

  The Surveillance Report resulted in the issuance of DIC CAR #9. CAR #9 pertained exclusively to the major finding of the Surveillance Report, that being electrical support weld inspection documentation. An agreement between KG&E and DIC Quality Management was reached that required KG&E to issue a CAR if the DIC resolution was unsatisfactory to KG&E.

- DIC CAR No. 1-E-009 (October, 1981) was subsequently issued to address the electrical support weld inspection documentation concerns identified in the KG&E Surveillance Report. The root causes of the problems identified in the KG&E Surveillance Report were determined by DIC to be:

  - The lack of notification by the responsible craft to Quality inspectors that welding activity was scheduled to commence.
  - Improper processing and filing of weld records.
  - The existance of a single part document as opposed to a triplicate type form to record inspections.

  The corrective measures taken by DIC involved the retraining of construction engineering personnel and the placement of limitations on the authorization level required to initiate the dispositions to Deficiency Reports. The CAR was closed in November, 1982.

- DIC CAR 1-W-0029 (March, 1983) was initiated to address some weld inspection inconsistancies in the Auxiliary, Control and Fuel Buildings. To investigate the extent of the problem 241 welds were inspected of which 147 were identified by the inspectors as deficient. To resolve the condition identified on the CAR, NCR 1SN10381FW was generated. The evaluation of the NCR involved another inspection by Welding Engineering which resulted in the determination that only 22 welds exhibited potentially significant conditions and were subsequently evaluated by the Architect - Engineer and dispositioned "use-as-is". Based on the NCR and its closure, DIC closed CAR 1-W-0029 in October, 1983.

- DIC CAR 1-C-0031 (August, 1983) states in part:

  > "MSSWRs used to document safety related structural steel welded connections through out "Q" designated areas is inadequate. A sample survey made by (DIC) Q.C. has shown 16.4% of the required MSSWRs cannot be located for all "Q" welds in the Fuel Bldg. A survey of 6 erection/design drawings in the Reactor Bldg revealed 24% of the welds are missing documentation. In addition, M/W Quality has initiated a NCR (1SN11957CW) to document 42 missing MSSWRs for welds in the ESWS Pumphouse."

The CAR was dispositioned to write an NCR for each safety related building to address the missing MSSWR's. Although the CAR remains open, the proposed justification for closure is based in part on the closure of DIC CAR 1-W-0029.

- Current Project Actions
  - Document Reconciliation Task: On August 13, 1984, a document reconciliation effort was initiated at the direction of project management to determine which safety related structural steel welds identified on design drawings were lacking inspection documentation in the form of MSSWRs.
  - Inspection Verification Plan: On August 17, 1984, an inspection verification effort was initiated at the direction of project management to provide an accurate assessment of the "as-built" conditions of safety related strucutral steel welded connections with unretrievable MSSWR's. These activities are being performed by a combined team of DIC and Architect - Engineer AWS Certified Welding Inspectors under direct supervision of KG&E Construction QC. These activities are being performed in accordance with written instructions issued by KG&E Construction QC which reflect the criteria of AWS D1.1-1975 and the applicable Architect - Engineer design documents. The results of these verifications and the review of Surveillance Report S-372 have caused the findings in Section E of this report to be issued.

D. Requirements

The welding of safety related structural steel connections at WCGS is governed by welding code AWS D1.1-1975. The WCGS FSAR invokes this code for each safety related structure. In addition, SNUPPS project specification 10466-C-122 (Q) Rev. 0 through 14 entitled "Technical Specification for Contract for Erection of Structural Steel for the (SNUPPS) Power Plant" and specification 10466-C-132(Q), Rev. 0 through 8 titled "Technical Specification for Erecting Miscellaneous Metal for the Standardized Nuclear Unit Power Plant System (SNUPPS)" requires structural steel welds to be performed in accordance with AWS D1.1-1975, with exceptions in the criteria for undercut (para. 8.5.2) and weld convexity (para. 8.5.3).

E. Findings - Impacts - Recommended Corrective Actions

The five findings listed below were identified during the two WCGS management assessments described in the 'Background Information' section of this report and a review of Surveillance Report S-372 by KG&E QA. Collectively, these represent a breakdown of the constructor's Quality Assurance program. This condition was caused by an apparent inconsistent application of weld inspection criteria, failure to implement procedural requirements for documenting inspections, and failure to implement effective corrective actions for identified deficiencies.

Finding #1: The results of the Document Reconciliation Task indicated that 1509 of 6816 MSSWRs for safety related structural steel welds are missing. (See Attachment 3)

Impact: Without the documentation for the structural welds, the following areas are indeterminate:

- Welder identification and qualification
- Filler metal traceability
- Visual inspection results
- Qualified weld procedures specification used

Recommended Corrective Actions: Actions 1a through 1h below will adequately address all of the concerns identified in Finding #1 and the "root cause" concerns associated with Finding #2.

1a. Based on DIC program requirements, assure that all of the welders and welding procedure specifications were qualified to AWS D1.1 - 1975.

1b. Review the DIC program for the purchase and control of filler material to ensure that only acceptable filler material was used in safety related structural steel welds.

1c. Evaluate the adequacy of the DIC inspection criteria and procedures to determine if these elements could have adversely impacted either the results of the initial inspections or the results of the verification plan. Document and provide this evaluation to KG&E QA for review prior to any additional inspection implementation. Any changes in inspection criteria and procedures shall be provided to KG&E QA for review.

1d. Obtain a documented evaluation to determine the validity of inspections performed with the presence of paint on the weld.

1e. Utilize personnel certified to ANSI N45.2.6 - 1978 for the inspection of safety-related structural steel welds. Inspections shall be performed in accordance with the DIC Quality Program and training shall be performed and documented to assure that inspectors are cognizant of the DIC Quality Inspection program requirements.

1f. Perform a 100% reinspection of all structurally significant safety-related structural steel welds with missing MSSWR's. The identification of "structurally significant" welds shall be made by the Architect - Engineer (See Attachment 3). Inspect the welds per recommendations 1c, 1d, 1e, 1g, 1h and 2a.

1g. Use an NCR to obtain and document a suitability for service evaluation of inaccessable welds.

1h. Report all identified deficiencies on an NCR.

* *

<u>Finding #2</u>: An inspection verification effort of safety-related structural steel welding, undertaken by AWS certifed weld inspectors identified several areas of deficiencies. These deficiencies have been catagorized below:

- Undersized welds
- Weld defects
- Incorrect configuration
- Weld underrun
- Weld undercut

<u>Impact</u>: These deficiencies could jeopardize the structural integrity of the connection.

<u>Recommended Corrective Actions</u>: Actions 2a through 2d below will adequately address all of the concerns identified in Finding #2 and the investigative actions required by Finding #5.

2a. Determine and document the "root cause" of the previous acceptance of deficient structural welds. Analyze the HVAC Support, Electrical Support, Pipe-Whip Restraint and any other safety-related program utilizing AWS D1.1 Welding to, ensure that the same "root causes" inherent in the structural steel welding program were not generic to other programs.

2b. Perform a 100% reinspection of all structurally significant safety-related structural steel welds having NSSWR's. The identification of "structurally significant" welds shall be made by the Architect - Engineer (See Attachment 3). Inspect the welds per recommendations 1c, 1d, 1e, 1g, 1h, and 2a.

2c. Evaluate the results of the completed Inspection Verification Plan against the acceptance criteria used in Action 1c.

2d. Any identified deficiencies shall be documented on an NCR.

* * *

<u>Finding #3</u>: A small number of safety-related structural steel welds were not made or had missing material.

<u>Impact</u>: The structural integrity has possibly been jeopardized.

<u>Recommended Corrective Action</u>: The following action and the engineering disposition will adequately address Finding #3.

3a. Forward the "as-built" information to the Architect - Engineer via an NCR to obtain an engineering evaluation and disposition.

* * *

Finding #4: One (1) weld was documented as having been inspected when in reality the weld was not made. (Ref. NCR 1SN23495CW)

Impact: The inspector who made the error could have improperly documented other welds. The structural integrity has possibly been jeopardized.

Recommended Corrective Action: The following action will adequately address Finding #4.

4a. Investigate the concern to determine the root cause of the error. Immediately notify KG&E Quality Assurance if any other problems of this nature are identified. Document the investigative actions. The notification of KG&E QA shall not preclude the issuance of an NCR.

* *

Finding #5: Objective evidence that the mechanical and structural welding inspection/documentation problems identified in KG&E QA Surveillance Report S-372 were rectified has not been provided.

Impact: There is a possibility that the mechanical and structural support welding inspection/documentation problems identified in the Surveillance Report were not corrected.

Recommended Corrective Action: The following action will adequately address Finding #5.

5a. Provide objective evidence that the mechanical and structural support welding inspection/documentation problems identified in Surveillance Report S-372 have been corrected. If such evidence is not available, research the extent of the problem and take the appropriate remedial actions.

F. Recommended Corrective Action Flow Diagrams

See Attachment C.

ATTACHMENT B

1. Definition of Miscellaneous Structural Steel:

Miscellaneous Structural Steel is divided into two (2) parts for the purposes of this CAR.

A. Main Frame and Associated Members:

Main frame welds are those welds on structural steel connections which support the main building floors (concrete or grating) and roofs. For efficiency, these connections are identified on a "per drawing" basis rather than categorizing each piece of steel individually. Therefore, it is inevitable that this category will include certain "associated" connections, such as, welds other than those which support main building floors and roof, which are depicted on drawings primarily showing main building floor and roof steel.

B. Miscellaneous:

Miscellaneous welds connect steel which does not support main building floors or roofs (i.e., all structural steel welds not classified as main frame or associated welds). This does not include hand-rails, toe-plates, and similar items.

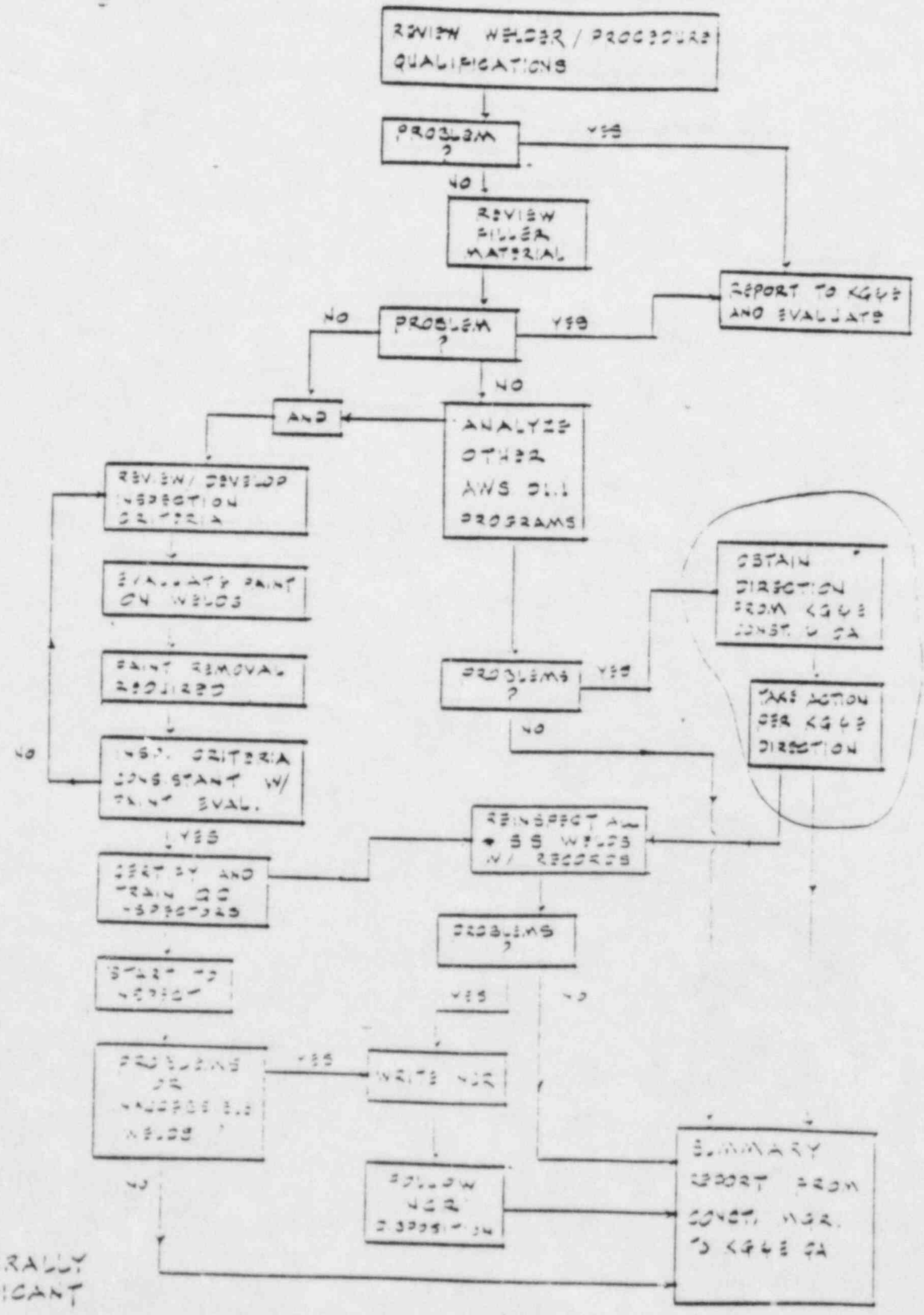
2. Definition of Structurally Significant Welds:

Those welds which are required in the completed building structure to support and protect safety related equipment and building components. Welds for temporary supports, non-safety related supports, hand-rails, toe-plates, and similar items are not considered to be structurally significant by this definition.

KG&E CAR 19 ATTACHMENT C-1

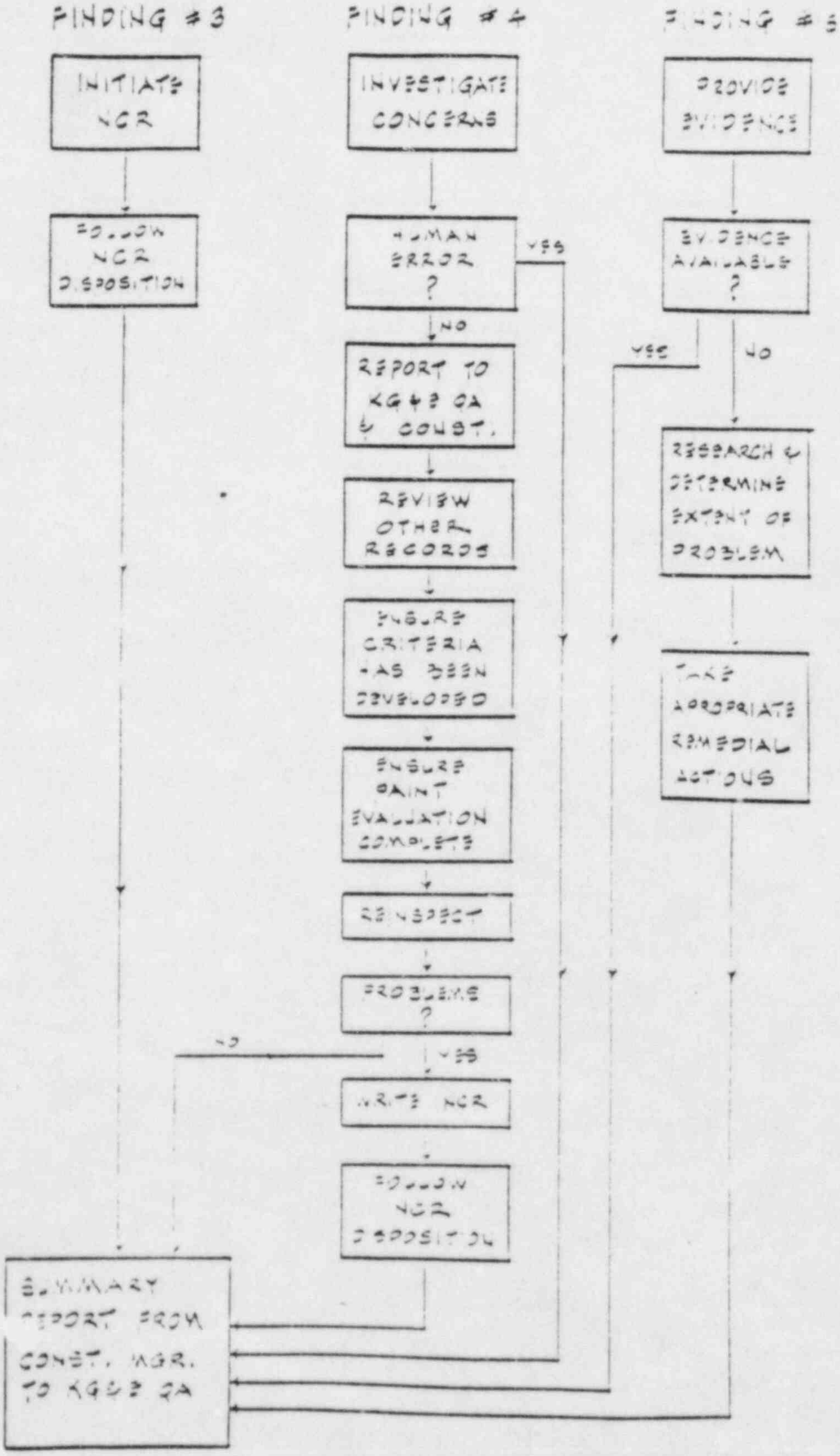
RECOMMENDED CORRECTIVE ACTION FLOW DIAGRAM

FINDING #1 & 2



KG&E CAR 19 ATTACHMENT C.2

RECOMMENDED CORRECTIVE ACTION FLOW DIAGRAM



ATTACHMENT (2)

KG&E MANAGEMENT PLAN

NOVEMBER 14, 1984 (Revision 1)

MANAGEMENT PLAN FOR THE RESOLUTION OF CAR-19

Overview

The objectives of this plan are as delineated in CAR-19. These objectives will be met by providing objective evidence that each of the corrective actions specified within CAR-19 are satisfactorily implemented. The intent is to verify that both the hardware and programmatic aspects of all safety related activities utilizing AWS D1.1 welding are in compliance with the FSAR (i.e. AWS D1.1 - 1975) and the Design and Construction Program Manual (Section 17.1B).

The attached logic chart illustrates the approach to be used in providing the above mentioned verifications. The Corrective Actions associated with each of the steps on the logic chart are identified on the chart.

All Corrective Actions shall be implemented in strict accordance with CAR-19 including review and approval of specific items by KG&E QA where requested. Flow diagrams (attachments C-1 and C-2 of the CAR) have been and will continue to be considered in developing corrective actions.

Upon completion of each of the corrective actions necessary to resolve CAR-19, reports will be prepared which summarize action taken. These summary reports will be used internally by DIC in the preparation of evaluations which will be submitted to KG&E to be used in the preparation of a final report.

Findings and Corrective Actions

The following pages include the Findings and Corrective Actions as presented in the subject CAR. The detailed activities required to implement each Corrective Action are listed beneath the Corrective

Actions. The numbering system for findings and corrective actions used in CAR-19 correspond directly with those used herein. Responsible key personnel are also provided.

Finding #1: "The results of the Document Reconciliation Task Force indicated that 1509 of 6816 MSSWR's for Safety Related Structural Steel Welds are missing."

RESPONSIBILITY

1a) "Based on DIC program requirements assure that all of the welders and welding procedures were qualified to AWS D1.1."

K. Hollingsworth
B. Newton

1a-1 DIC develop AWS D1.1 attribute checklist and review welding procedure and welder qualification procedure against this checklist; include documentation of procedure review cycle.

K. Hollingsworth
B. Newton

1a-2 DIC perform statistical sampling plan in accordance with MIL-STD-105D to verify qualifications of welders appearing on randomly selected MSSWR's.

G. Stanley
M. Pitre

1a-3 Bechtel review and comment on DIC Welding Procedure Specification and Welder Qualification Procedure as to compliance to AWS D1.1.

D. Mauldin

1a-4 Provide report summarizing the results of the above.

1b) "Review the DIC program for the purchase and control of filler material to ensure that only acceptable filler material was used in safety related welds. Assure that both safety related and non-safety related filler materials were properly controlled to preclude improper application."

K. Hollingsworth B. Newton	1b-1	DIC review procedures for the purchase and control of filler and base materials and prepare description/justification.
G. Stanley	1b-2	Bechtel review procedures for the purchase and control of filler materials and comment.
D. Mauldin	1b-3	Prepare summary report.
	1c)	"Evaluate the adequacy of the DIC inspection criteria and procedures to determine if these elements could have adversely impacted the inspection results. Document and provide this evaluation to KG&E QA for review prior to inspection implementation. Any changes in inspection criteria and procedures shall be provided to KG&E QA for review prior to implementation.
D. Mauldin J. Ayres	1c-1	Develop AWS and site specification attribute checklist related to inspection requirements. Review DIC inspection criteria and procedures in accordance with checklists.
J. Ayres	1c-2.0	Document this evaluation.
J. Ayres	1c-2.1	Summarize results of 1c-2.0 and provide results to KG&E QA.
J. Ayres	1c-2.2	Continue further actions as a result of 1c-2.0 evaluations.
J. Ayres	1c-3.0	Discuss evaluation with KG&E QA.
[illegible] [illegible] Malecki	1c-3.1	KG&E QA provide input comment on evaluation of 1c-3.0 to [illegible].
J. Ayres	1c-4	Prepare changes/revisions as necessary and submit to KG&E QA for review.

D. Mauldin
J. Ayres

1c-5 Prepare summary report items 1c-1 through 1c-4.

L. Pardi

1d) "Obtain a documented evaluation to determine the validity of inspections performed with the presence of paint on the weld."

K. Hollingsworth
B. Newton

1d-1 Obtain information from other utility/AE's that have developed a validation plan.

B. Newton
G. Brown

1d-2 DIC Welding Engineering and Bechtel Review; add site specific requirements/justification as necessary and develop site position letter.

G. Stanley

1d-3 Submit letter to KG&E for review and approval.

D. Mauldin

1d-4 Prepare summary report items 1d-1 through 1d-3.

1e) "Utilize personnel certified to ANSI N45.2.6 - 1978 for the inspection of safety related structural steel welds. Inspections shall be performed in accordance with the DIC Quality Program and training shall be performed and documented to assure that inspectors are cognizant of the DIC Quality program requirements."

D. Mauldin

1e-1 Incorporate DAR-19 Inspection Verification Plan into DIC Procedure QCP-VII-100, "Inspection of Welding Process."

M. G. Westhoff
J. [illegible]

1e-2 Inspection personnel to be certified to ANSI N45.2.6 - 1978 in accordance with DIC certification program based on education and experience levels.

L. Easterwood
J. Fletcher

1e-3 Site specific qualifications will be limited to the re-inspection of structural steel welds in accordance with the requirements of QCP-VII-200.

D. Mauldin

1e-4 Prepare summary report items 1e-1 through 1e-3.

1f) "Perform a 100% reinspection of all structurally significant safety related structural steel welds. The identification of "structurally significant" welds shall be made by the Architect - Engineer."

G. Brown
J. Fletcher

1f-1 Identification of "structurally significant" welds by the Architect - Engineer.

"Structurally significant" joints are defined as all field welded joints which support or potentially support safety related equipment and building components. This basically includes all field welds on structural and miscellaneous steel with the exception of handrail, toeplates, grating, checkered plate, stairs, ladders and monorail supports. These are non-Q items which typically see significant service loads during the construction process. Some are designated as II/I; however, II/I seismic loads are considered to be less severe than service loads. Monorails have been load tested as part of startup procedures.

The joints are selected by Bechtel based on a review of erection drawings prepared by the structural and miscellaneous steel fabricators.

L. Easterwood
J. Fletcher

1f-2 Perform re-inspections in accordance with the CAR-19 Inspection Verification Plan.

- Use the project nonconformance program to obtain and document a suitability for service evaluation of inaccessible welds.
- Report all identified deficiencies on an NCR.

Bechtel will perform a case by case evaluation of each joint inspected to determine if:

- as-built condition meets design allowables.
- if the as-built condition is a significant deficiency in accordance with 10CFR50.55(e).
- any rework is required.

D. Mauldin
J. Fletcher

1f-2.1 Summarize data from 1f-1, 1f-2.

T. McBride
D. Armstrong

1f-3.0 Collect relative data from TCR's, CVR's, NCR's for additional structural welds and furnish to Bechtel.

T. McBride
D. Armstrong

1f-3.1 Collect information and furnish to Bechtel for evaluation to determine if any additional structurally significant welds were made. Reinspect any additional welds as directed from Bechtel evaluation.

D. Mauldin

1f-4 Prepare summary report on data from items 1f-1, 1f-2, 1f-3.

Finding #2: "An Inspection verification effort of safety-related structural steel welding, undertaken by AWS certified welding inspectors identified several areas of deficiencies. These deficiencies have been categorized below:"

- Undersized welds
- Weld defects
- Incorrect configuration
- Weld underrun
- Weld undercut

RESPONSIBILITY		CORRECTIVE ACTIONS
D. Mauldin J. Ayres	2a)	"Determine and document the "root cause" of the previous acceptance of deficient structural welds. Analyze the HVAC Support, Electrical Support, Pipe-Whip Restraint and any other safety-related program utilizing AWS D1.1 Welding to ensure that the same "root causes" inherent in the structural steel welding program were not generic to other programs."
D. Mauldin J. Ayres	2a-1	Review evaluations of DIC inspection program as performed in 1c. Determine if procedures could contribute to "root cause".
D. Mauldin D. Garrett	2a-2	Review inspection training and certification procedures to verify compliance to ANSI N45.2.6 - 1978.
D. Mauldin J. Ayres	2a-3	Analyze the deficiencies found in structurally significant safety related structural steel welds as documented in the CAR-19 Inspection Verification Plan utilizing the original MSSWR, the Re-Inspection Data Sheets, and the Architect Engineer evaluation.

J. Ayres	2a-4	Identify all safety related activities utilizing AWS D1.1 welding.
J. Ayres	2a-5	Review previously compiled information relative to inspection and acceptance of HVAC and Electrical Supports, and Pipe Whip Restraints and any other safety related program utilizing AWS D1.1. Examples of compiled information include Construction Self Assessment, task force reports, QA audits and surveillances.
D. Mauldin J. Ayres	2a-6	Summarize results of any previous investigations/reports related to welding/inspection of above items.
D. Mauldin J. Ayres	2a-7	Analyze programmatic elements utilized in the erection welding of structural steel and HVAC and Electrical Supports, Pipe Whip Restraints and other items. Develop list of programmatic differences and determine extent to which these differences would influence 'root causes'.
D. Mauldin	2a-8	Provide summary report items 2a-1 through 2a-7.

Finding #3: "A small number of safety related structural steel welds were not made or had missing material."

RESPONSIBILITY	CORRECTIVE ACTIONS
	3a) "Forward the "as-built" information to the Architect/Engineer via an NCR to obtain an engineering evaluation and disposition."

<u>L. Easterwood</u> J. Fletcher	3a-1	Missing welds or material detected in the inspections performed in 1f shall be documented on NCR(s) showing the "as-built" information. These NCR(s) shall be given to the AE for evaluation and disposition.
<u>D. Blizzard</u> F. Raycner	3a-2	Verification of incorporation of design changes.
<u>D. Armstrong</u>	3a-3	Evaluate and determine probable cause of 3a-1.
<u>D. Mauldin</u>	3a-4	Prepare summary report.

<u>Finding #4</u> One (1) weld was documented as having been inspected when in reality the weld was not made. (Ref. NCR ISN 10495CN)

<u>RESPONSIBILITY</u>		<u>CORRECTIVE ACTIONS</u>
	4a	"Investigate the concern to determine the root cause of the error. Immediately notify KG&E Quality Assurance if any other problems of this nature are identified. Document the investigative actions. The notification of KG&E QA shall not preclude the issuance of an NCR."
<u>D. Mauldin</u> D. Armstrong	4a-1	Evaluate the results of the CAR-19 Inspection Verification Plan (i.e., those inspections performed in 1f) and determine whether a pattern of deficiencies is found.
<u>D. Armstrong</u> F. Raycner	4a-2	Identify further actions required if a pattern of deficiencies is found.
<u>D. Mauldin</u>	4a-3	Prepare summary report.

<u>Finding #5:</u> "Objective evidence that the mechanical and structural welding inspection/documentation problems identified in KG&E QA Surveillance Report S-372 were rectified has not been provided."

<u>RESPONSIBILITY</u> <u>CORRECTIVE ACTIONS</u>

5a) "Provide objective evidence that the mechanical and structural support welding inspection/documentation problems identified in Surveillance Report S-372 have been corrected. If such evidence is not available, research the extent of the problem and take the appropriate remedial actions."

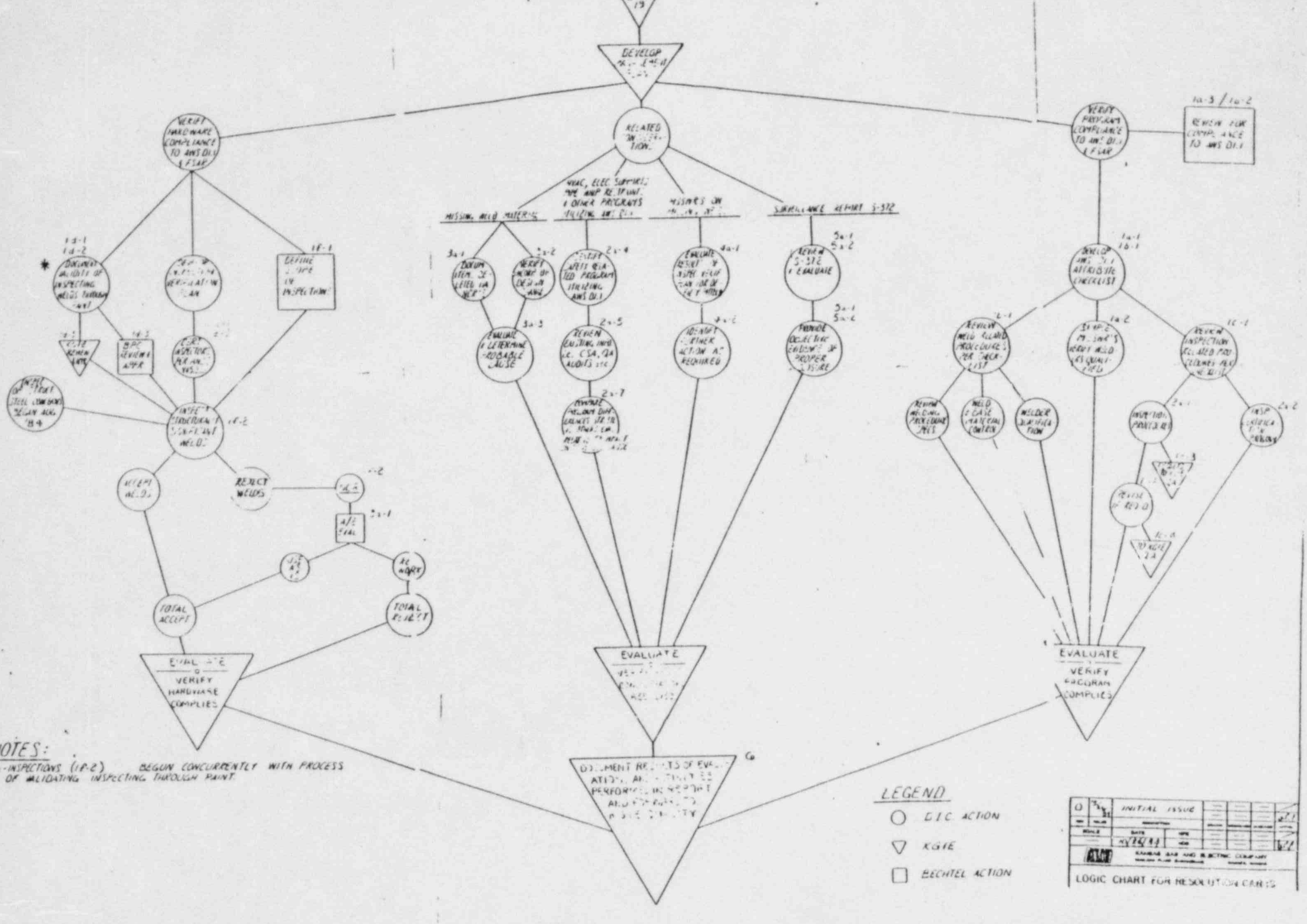
<u>D. Mauldin</u> 5a-1 Review and provide objective evidence that Mechanical Deficiency Reports identified in S-372 have been correctly closed out.

<u>D. Mauldin</u> 5a-2 Review and provide objective evidence that Civil Deficiency Reports identified in S-372 have been correctly closed out.

<u>D. Mauldin</u> 5a-3 Prepare summary report.

<u>RESPONSIBILITY</u> <u>#6</u> <u>REPORT</u>

<u>D. Mauldin</u> A final comprehensive report including all evaluations performed and the results of activities conducted to provide objective evidence to satisfy the corrective actions required [illegible] will be prepared and submitted to [illegible]. The report will also include an evaluation of Construction [illegible] programs in areas other than AWS D1.1 welding to determine the potential of programmatic deficiencies.



NOTES:
* - INSPECTIONS (1f-2) BEGUN CONCURRENTLY WITH PROCESS OF VALIDATING INSPECTING THROUGH PAINT.

0	[illegible]	INITIAL ISSUE
SCALE	DATE [illegible]	

KANSAS GAS AND ELECTRIC COMPANY

LOGIC CHART FOR RESOLUTION CAR 19

WC-AP-002.5
Rev. 8-16-84

PROCEDURE CHANGE NOTICE

NOTICE NUMBER PCN 0014/R1

PROCEDURE NUMBER QCP-VII-200

REVISION NUMBER 20

JUSTIFICATION FOR CHANGE:

To include inspection criteria to be in compliance with Bechtel Specification 10466-C-122.

REQUESTED CHANGE(S):

Add Paragraph 3.2.5

DANIEL CONSTRUCTION CO.
DOCUMENT CONTROL
NOV 2 1984

PROCEDURES AFFECTED BY THIS CHANGE NOTICE:

NONE

(Additional procedures to be listed on back, if necessary.)

APPROVALS:

ORIGINATOR 11-2-84 DATE

PROJECT MANAGER 11/2/84 DATE

PROJECT QUALITY MGR. 11/2/84 DATE

KG&E CONSTRUCTION MGR. 11/2/84 DATE

NOTE: Upon issuance by Document Control, responsible holders of this document are required to place this document immediately following Page 1 of the affected procedure(s), identified above.

1.0 SCOPE

This instruction provides direction for the inspection and documentation of AWS D1.1 structural steel welds identified for inspection by Bechtel Engineering in accordance with KG&E CAR-19.

2.0 RESPONSIBILITIES

2.1 The KG&E Construction Quality Control, Lead Welding Quality Cont Engineering shall be responsible for the implementation of this instruction.

2.2 Personnel certified in accordance with ANSI N45.2.6-78 - "Qualificat of Inspection, Examination, and Testing Personnel for Construction Phase of Nuclear Power Plants" and American Welding Society QC1 - "Standard for Qualification and Certification of Welding Inspectors" shall be responsible for performing the inspections and documentation activities defined in this instruction.

3.0 GENERAL

3.1 Inspections specified in this instruction shall be performed visually utilizing fillet weld gauges, steel rulers or steel tape measures capable of measurements within 1/16 inch increments. Undercut gages will be capable of measuring 1/32".

3.2 The welds shall be inspected in accordance with the design drawing and paragraph 8.15.1 of the AWS - Structural Welding Code D1.1-75 (See Page 3) with the following exceptions:

3.2.1 Paint on welds - Paint will exist on most of the welds to be inspected, in these cases visual examination shall be made and the results documented in accordance with Section 4.0. Engineering evaluation of the inspection results is performed knowing that paint does exist on most welds.

Foreign material (fireproofing, etc) may remain after cleaning. This condition is acceptable if the foreign material does not preclude, in the judgment of the inspector, a determination of the weld status per the inspection criteria outlined in this appendix.

If the foreign material precludes this determination, the weld shall be recleaned prior to inspection.

3.2.2 Convexity - Fillet welds need not satisfy the convexity limitations of AWS D1.1, Section 3.6.1 Reference Bechtel Specification 10466-C-122 and 10466-C-132 paragraph 8.5.3.

3.2.3 Undercut - Undercut shall not exceed 1/32 inch. Reference Bechtel Specification 10466-C-122 and 10466-C-132 paragraph 8.5.2.

3.2.4 "Nail Holes" (Construction Aids) - "Nail Holes" in embeds in some instances will be located where a weld is required. The "nail holes" may remain open provided the weld on both sides of the "nail hole" is increased from the size shown on the drawing by 1/16 of an inch for a length of 2 inches per side.

Reference Bechtel drawing C-1003 Miscellaneous Steel General Note 11.

3.2.5 Gouges - Gouges in base materials shall not be longer than 3 inches nor deeper than 3/16 of an inch. Gouges in weld metal shall not reduce the section thickness of the weld below specified size.

Gouges exceeding the above shall be noted on the Weld Data Sheet (Exhibit A or B) including details such as depth, length, and location on base or weld material.

Document discrepancies on a nonconformance report.

3.3 When inspection reveals a rejectable weld or joint configuration (excluding clip to embed top or bottom weld overrun), the entire joint is to be "as-built" inspected and all weld sizes and lengths documented. The "as-built" weld sizes should be accurate as possible and should reflect any significant oversized welds within the joint.

3.4 Any missing material identified as a result of inspection per the design drawing shall be identified on an NCR.

3.5 Document any welder identification (D number) marked on the joint.

4.0 DOCUMENTATION

4.1 The inspection results will be documented on an inspection report similar to Exhibit A or B and submitted to KG&E Construction Quality Control for final review for completeness and accuracy. These reports must be completed in a consistent manner and as a minimum shall contain the following information.

1. Description and size of weld deficiency.
2. The dimension (distance) between beam and embed (to be reported as "set-back gap").
3. Drawing and detail number.
4. Date of inspection.
5. Name and certification number of the AWS CWI who performed the inspection.
6. Accept/reject.

4.2 When any condition is found that does not meet the acceptance criteria outlined in paragraph 3.2 and 3.3 of this instruction, a nonconformance report will be generated in accordance with the applicable project procedure and forwarded to Bechtel for evaluation. In order to reduce the amount of paperwork, it is acceptable to generate one nonconformance report per building.

4.3 The documentation generated as a result of this Inspection Verification Plan shall become an attachment to CAR-19.

5.0 EXHIBITS

5.1 Exhibit A - Weld Data Sheet

5.2 Exhibit B - Weld Data Sheet

8.15 QUALITY OF WELDS

8.15.1 **Visual Inspection.** All welds shall be visually inspected. A weld shall be accepted by visual inspection if it shows that:

8.15.1.1 The weld has no cracks.

8.15.1.2 Thorough fusion exists between weld metal and base metal.

8.15.1.3 All craters are filled to the full cross section of the welds.

8.15.1.4 Weld profiles are in accordance with 3.6.

8.15.1.5 The sum of diameters of piping porosity does not exceed 3/8 in. (9.5 mm) in any linear inch of weld and shall not exceed 3/4 in. (19.0 mm) in any 12 in. (305 mm) length of weld.

8.15.1.6 Fillet welds in any single continuous weld shall be permitted to underrun the nominal fillet size required by 1/16 in. (1.6 mm) without correction provided that the undersize weld does not exceed 10 percent of the length of the weld. On web-to-flange welds on girders, no underrun is permitted at the ends for a length equal to twice the width of the flange.

3.6 WELD PROFILES

3.6.1 The faces of fillet welds may be slightly convex, flat, or slightly concave as shown on page 4, with none of the unacceptable profiles also shown on Page 4. Except at outside corner joints, the convexity shall not exceed the value of 0.1S plus 0.03 in. where S is the actual size of the fillet weld in inches. (See Page 4).

3.6.2 Groove welds shall preferably be made with slight or minimum reinforcement except as may be otherwise provided. In the case of butt and corner joints, the reinforcement shall not exceed 1/8 in. (3.2 mm) in height and shall have gradual transition to the plane of the base metal surface (see Page 4). They shall be free of the discontinuities shown for butt joints on Page 4.

3.6.3 Surfaces of butt joints required to be flush shall be finished so as not to reduce the thickness of the thinner base metal or weld metal by more than 1/32 in. (0.8) or five percent of the thickness, whichever is smaller, or leave reinforcement that exceeds 1/32 in. However, all reinforcement must be removed where the weld forms part of a faying or contact surface. Any reinforcement must blend smoothly into the plate surfaces with transition areas free from edge weld undercut. Chipping may be used provided it is followed by grinding. Where surface finishing is required, its roughness value[11] shall not exceed 250 μin. (6.3). Surface finished to values of over 125 μin. (3.2 μm) through 250 μin. shall be finished parallel to the direction of primary stress. Surfaces finished to values of 125 μin. or less may be finished in any direction.

3.6.6 Welds shall be free from overlap.

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[11] ANSI B46.1 Surface Texture, in microinches (μin.).

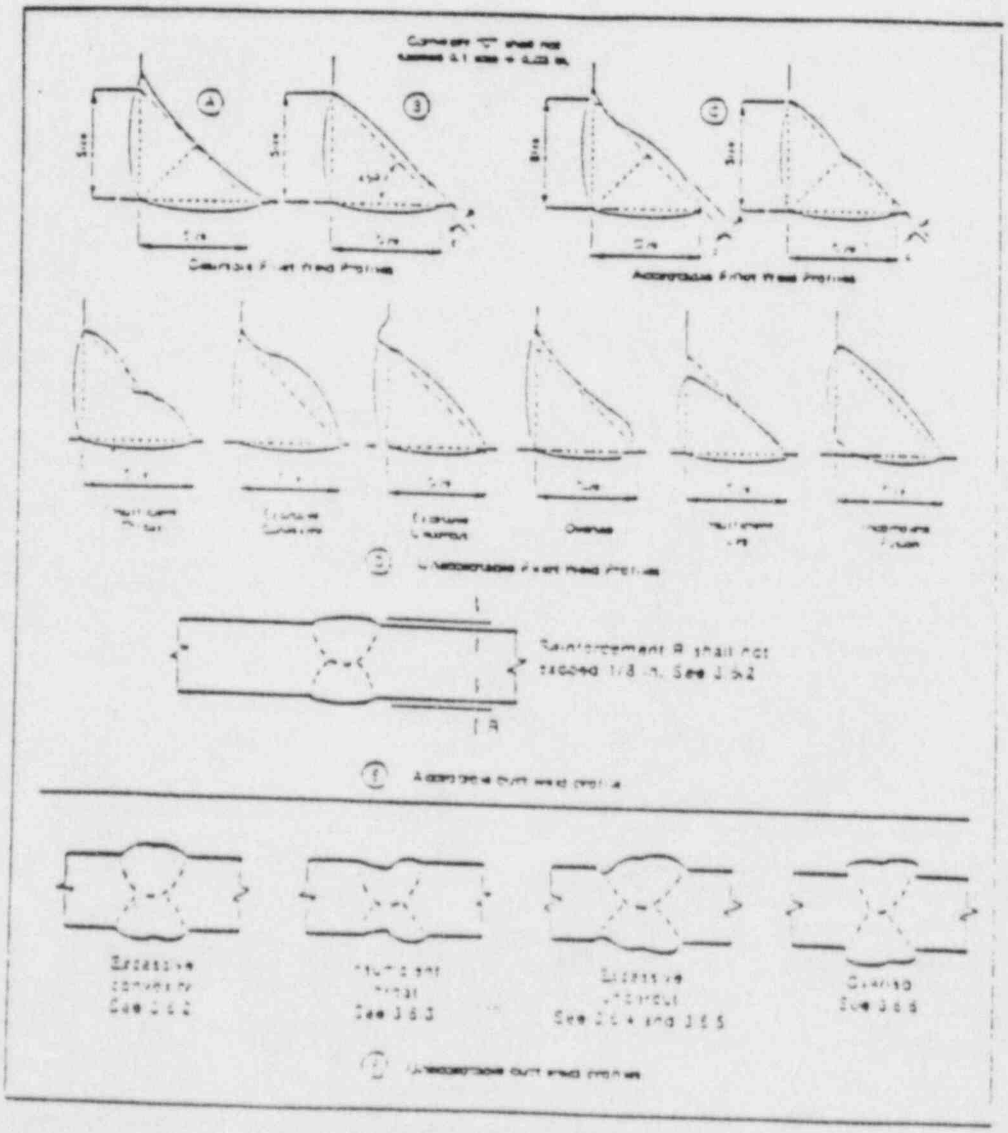

Fig. [illegible]—Acceptable and unacceptable weld profiles

Connection # ____________ WELD DATA SHEET AREA ____________

APPENDIX VIII
EXHIBIT A

	[illegible]	[illegible]	[illegible]
1-			
2-			
3-			
4-			
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17			
18-			

INSPECTOR ____________ DATE ____________ DRAWING No ____________
CWE No ____________ ELEV. ____________ DETAIL No ____________

Additional Comments on Reverse Side.

Joint or Weld Number	Access	Cover	Surface State	Lack of Throat	Undercut	Bead Profile	[illegible]	of Fusion	Incomplete Penetration	Crack	Inclusions	Concavity	Remarks

APPENDIX VIII
EXHIBIT B