NIAGARA MOHAWK

NIAGARA MOHAWK POWER CORPORATION/300 ERIE BOULEVARD WEST, SYRACUSE, N.Y. 13202/TELEPHONE (315) 474-1511

August 1, 1980

Office of Inspection and Enforcement
Region I
Attention: Mr. R. T. Carlson, Chief
Reactor Construction and Engineering
Support Branch
U. S. Nuclear Regulatory Commission
631 Park Avenue
King of Prussia, PA 19406

Dear Mr. Carlson:

Re: Nine Mile Point Unit 2
Docket No. 50-410

Enclosed is the final report concerning the Nine Mile Point Unit 2 biological shield wall in accordance with 10 CFR 50, Paragraph 50.55(e) (3). This matter was initially reported to your staff on May 30, 1979 as involving a defect in the base ring to outer shell plate weld. Investigations subsequent to the initial notification revealed that there was a potential deficiency in other welds of similar geometrical configuration. The increased scope of the biological shield wall review was reported to your staff on August 24, 1979.

After extensive engineering evaluation, it has been determined that the condition of the biological shield wall welds could not have adversely affected the safety of operations of the Nine Mile Point Unit 2 plant had it remained undiscovered. The basis for this conclusion is discussed in detail in Section VI, Analysis of Safety Implications, of the enclosed report. Also included in the report is a description of the deficiencies and the corrective action taken.

Very truly yours,

NIAGARA MOHAWK POWER CORPORATION

Donald P. Dise
Vice President Engineering

PEF:ja
Enclosure
xc: Director of Inspection and Enforcement
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

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I. INTRODUCTION

A. Statement of Problem and Summary of Conclusions

During nondestructive examination of the biological shield wall (BSW) at the jobsite, weld defects* were discovered in a number of shop welds. Investigation has shown that the weld defects existed primarily in backing bar welds; only minor defects were discovered in double bevel welds.

All shop weld joints were evaluated in accordance with AWS D1.1 and original PSAR commitments and either were shown to be acceptable or were repaired. Based on the engineering evaluation, it has been determined that the BSW weld defects could not have adversely affected the safety of operations of the Nine Mile Point 2 plant had the weld defects remained undiscovered.

*Note: "Defect" is used throughout this report to describe a weld discontinuity and should not be construed to mean that a weld discontinuity is not acceptable.

B. Overview of Report

The purpose of this report is to provide the following:

1. A description of the BSW, including its specification and Quality Assurance program requirements.

2. A detailed technical description of the weld problems.

3. A presentation of the results of an engineering evaluation and a corrective action plan, including a discussion of the overall approach, summary of the evaluation results and repairs required, and example calculations.

4. The Quality Assurance corrective action.

5. An analysis of safety implications.

C. Revisions to Interim Report

Portions of the Interim Report of April 15, 1980, require revisions to reflect the final closure plan action which was subsequently adopted and clarifications to maintain consistent terminology. In the interim report, it was inadvertently stated that the engineering evaluation was not in accordance with AWS D1.1. As stated in this report, all UT evaluations and acceptance of indications are in accordance with AWS D1.1, paragraph 3.7.6. Details of interim report revisions are listed in Appendix A to this report.

II. DESCRIPTION OF BIOLOGICAL SHIELD WALL

A. Physical Description

The BSW is supported by the reactor pedestal (Figure 1) and is attached to the pedestal by means of embedded anchor bolts (Figure 2). The BSW is an extremely stable structural system because of the lateral support at the drywell floor in the reactor building and the star truss system near the top of the drywell. The BSW consists of two concentric steel cylinders connected by horizontal and vertical stiffeners (Figures 3 and 4). The BSW is 48 ft 4 inches high and has an inner radius of 14 ft 3/4 inch and an outer radius of 15 ft 9 1/4 inches. The BSW was fabricated in three rings, each approximately 16 feet high. Each ring was shop fabricated in three 120 degree sections. The inner and outer shells and the stiffeners are 1 1/2 inches thick, A537 Class 1 steel plates connected by full penetration welds. The space between the shells will be filled with nonstructural high density concrete for neutron radiation shielding purposes. The BSW is penetrated by air duct openings, inspection openings, instrumentation line and pipe sleeve penetrations, and door openings for various piping systems. Attached to the wall are pipe restraints, a BSW extension to support the star truss and stabilizer, clip angle supports for floor beams, and insulation support brackets.

The full penetration welds used in the BSW are both single bevel (with backing bars) and double bevel types. The following table lists the various weld configurations, their abbreviations, and the figure in which they are shown.

Weld Joint	Abbreviation	Figure No.
Cover plate to base plate	CPBP	5
Vertical stiffener to inner wall	VSIW	6
Cover plate to vertical stiffener	CPVS	6
Vertical stiffener to horizontal stiffener	VSHS	7a
Horizontal stiffener to vertical stiffener	HSVS	7b
Cover plate to horizontal stiffener	CPHS	8
Horizontal stiffener to inner wall	HSIW	8

B. Functional Requirements

The functional requirements of the BSW are:

1. Provide shielding against radiation from the reactor vessel.

2. Provide anchorage support for pipe restraints, pipe supports, flooring beams, and insulation.

3. Provide support for the star truss/stabilizer system.

4. Protect the reactor pressure vessel from pipe whip, jet impingement, and missile loads.

REACTOR BUILDING CONFIGURATION

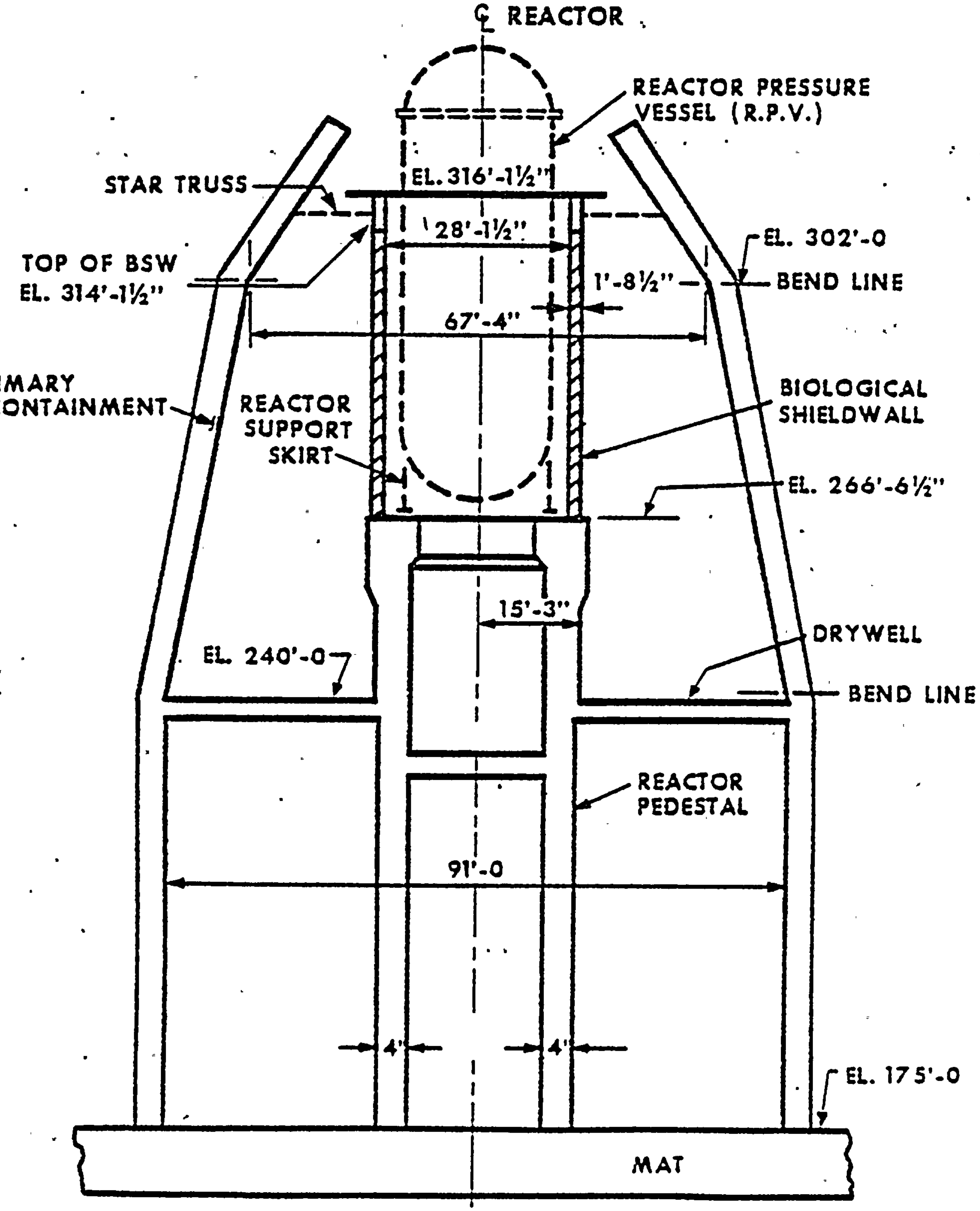


FIGURE 1

BASE DETAIL

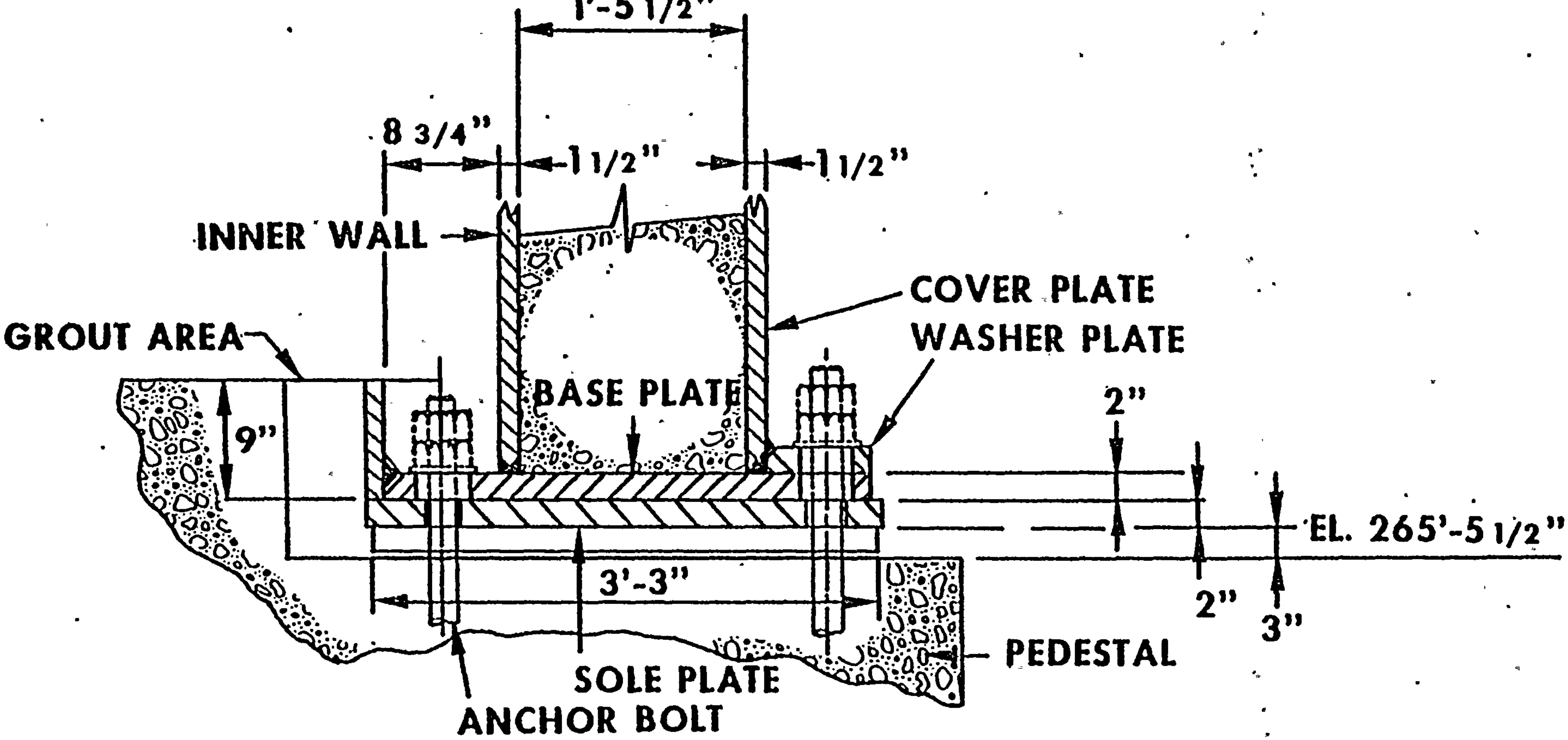


FIGURE 2

ELEVATION

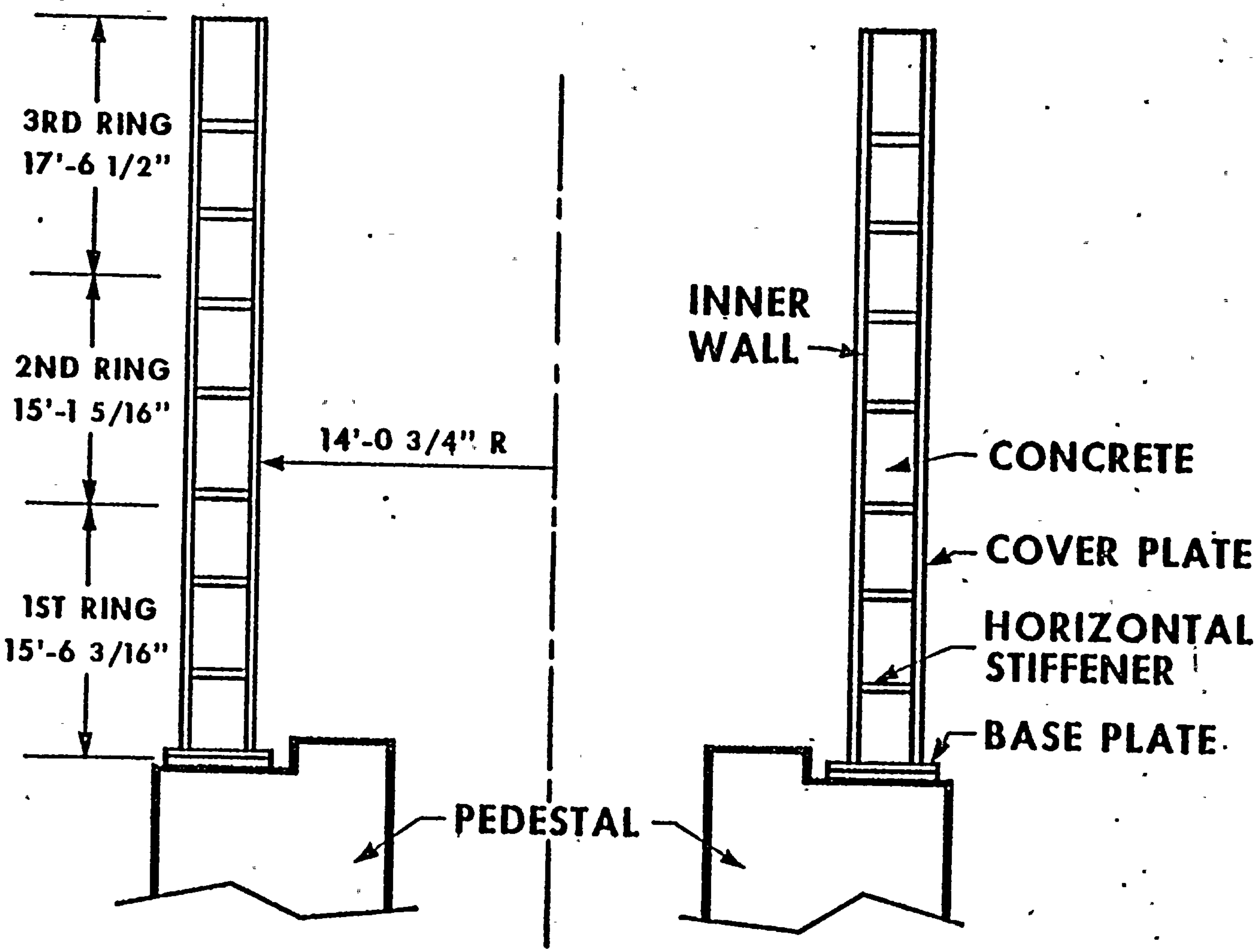


FIGURE 3

PLAN

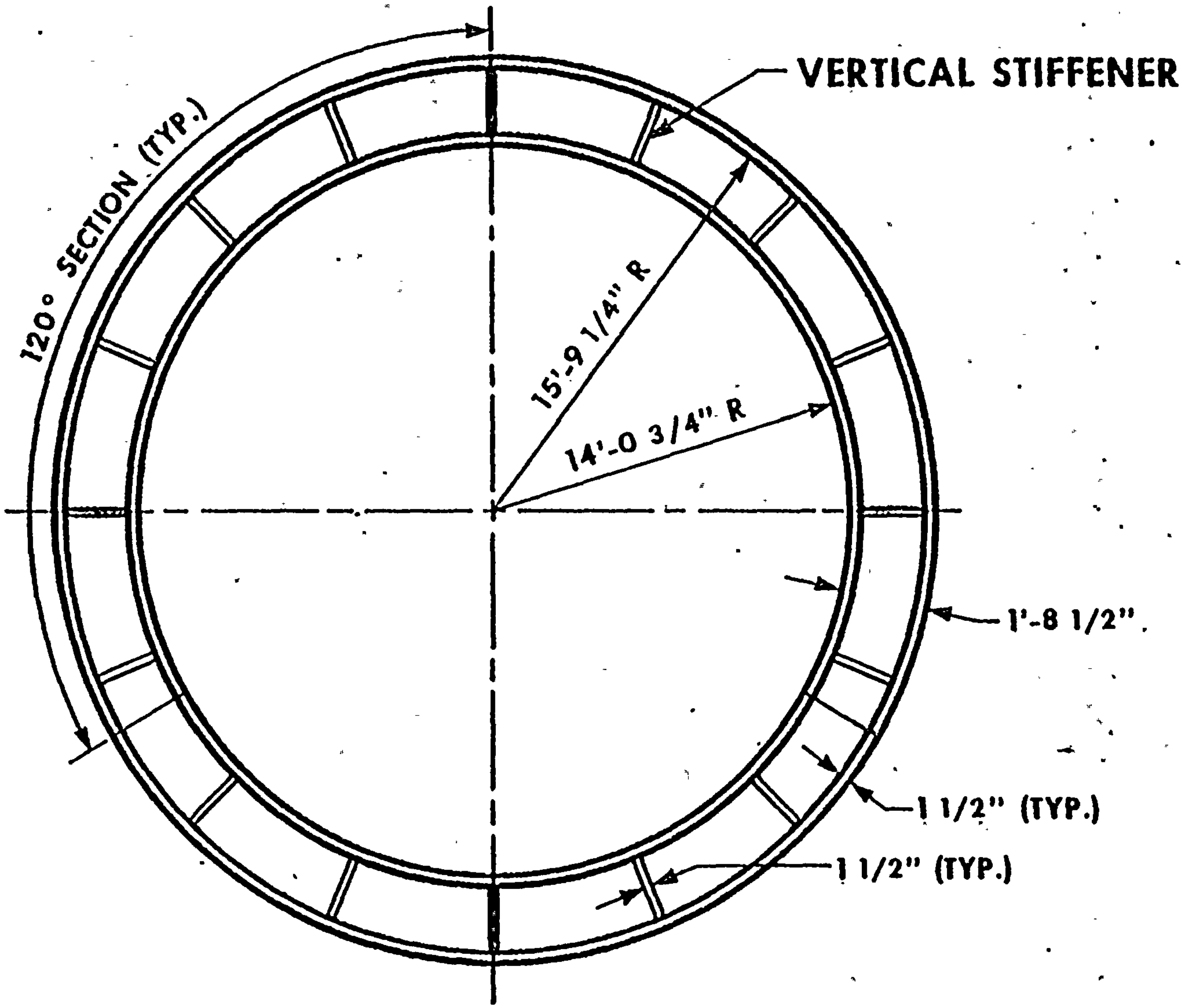


FIGURE 4

COVER PLATE TO BASE PLATE WELD

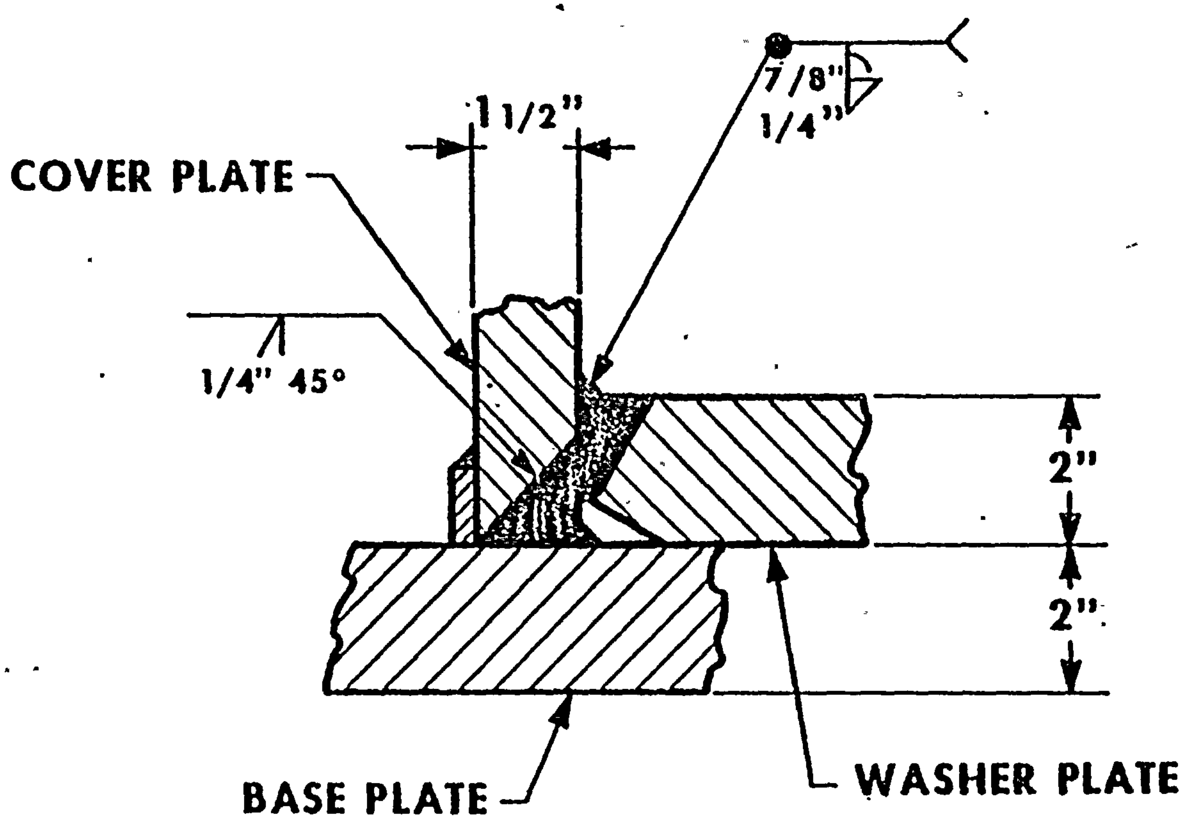


FIGURE 5

VERTICAL STIFFENER TO INNER WALL AND COVER PLATE WELDS

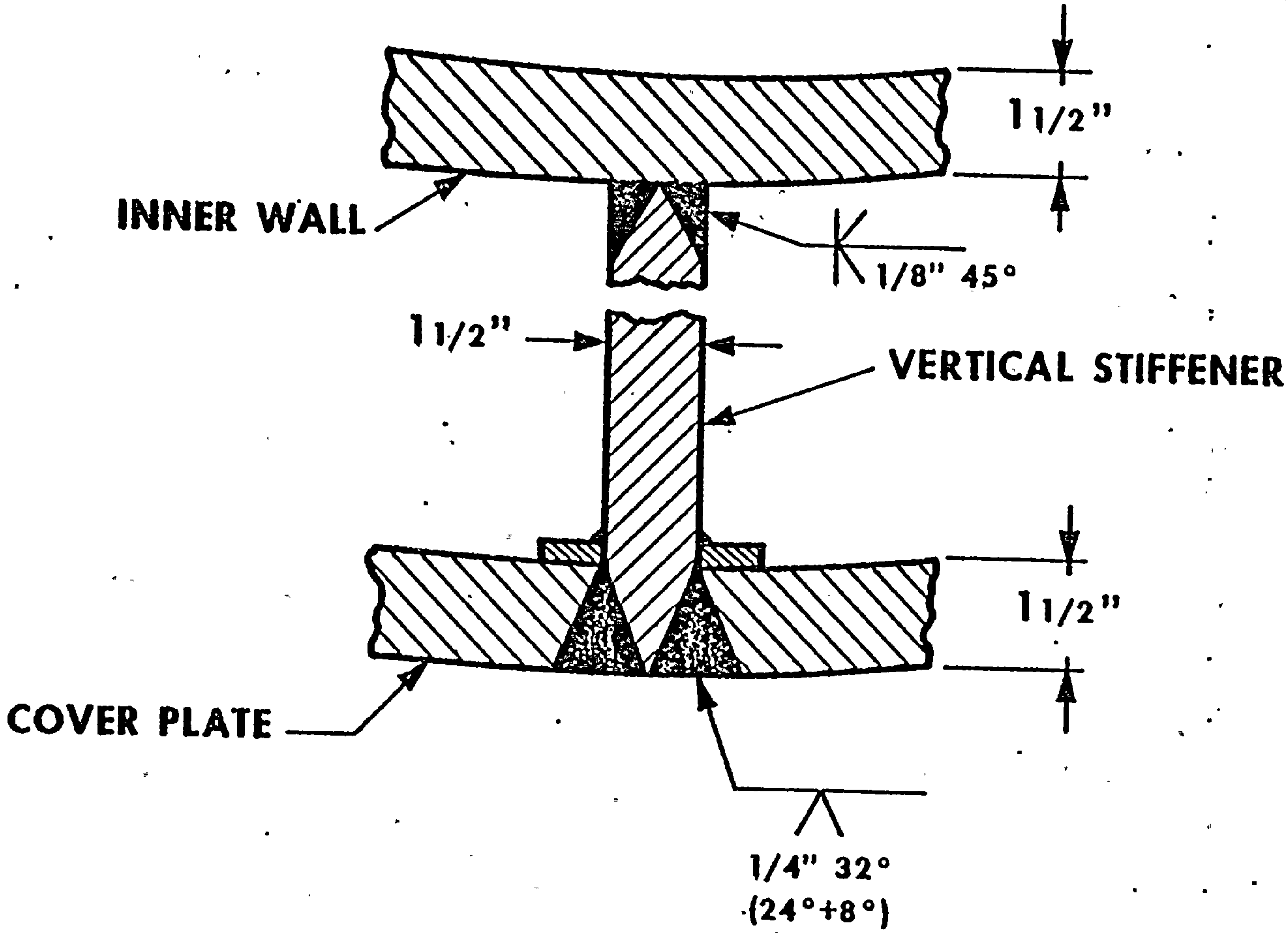


FIGURE 6

HORIZONTAL STIFFENER TO VERTICAL STIFFENER WELDS

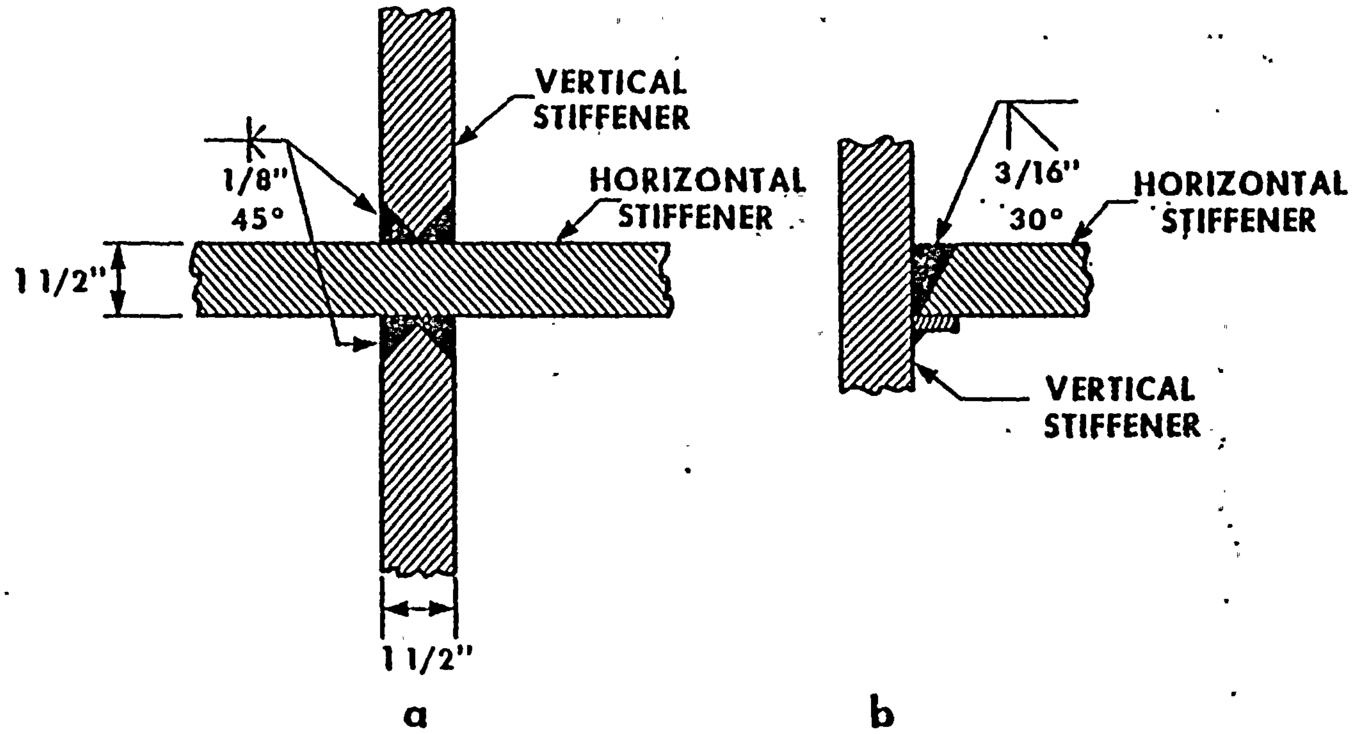


FIGURE 7

HORIZONTAL STIFFENER TO INNER WALL AND COVER PLATE WELDS

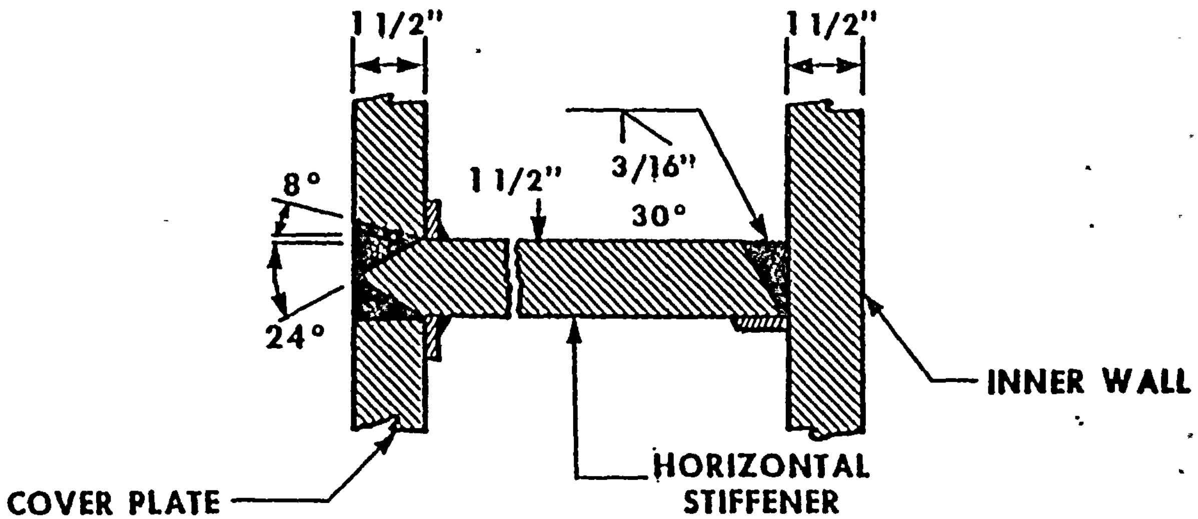


FIGURE 8

C. Summary of Design Criteria and Licensing Commitments

The BSW is designed in accordance with the AISC Manual of Steel Construction for normal operating load conditions. For abnormal/extreme environmental load combinations, the allowable stresses are increased in accordance with the factors specified in the NMP2 PSAR.

The following loads have been considered in the analysis and design of the BSW:

1. Deadload and seismic loads

2. Accident temperature cases consisting of the maximum temperature differentials between the inner and outer walls occurring as the result of a loss-of-coolant accident (LOCA)

3. Accident pressure differential between the inner and outer walls occurring as the result of a LOCA

4. Pipe restraint loads occurring as the result of restraining pipes following a postulated rupture

5. Jet impingement loads resulting when pressurized fluid from a ruptured pipe strikes the BSW.

D. Summary of Analytical Techniques

The structural analysis of the BSW was performed by the finite element method using the computer program STRUDL. The BSW was modeled using a 180 degree model with the appropriate boundary conditions for the symmetric and antisymmetric loads. Analyses for general loading conditions were conducted using principles of superposition. The inner and outer walls as well as the horizontal and vertical stiffeners were modeled using isoparametric elements.

After the computer runs of the individual load cases were made, the stresses were combined in accordance with the appropriate load combination equation from the NMP2 PSAR, Section 12.5.2.8.3.

The effect of accident temperature conditions was studied taking into account the concrete inside the BSW. It was concluded that the concrete would crack under these conditions and that the effect of including concrete on the stresses in the BSW is insignificant.

For other loading conditions, the stresses in the BSW would be reduced if concrete were included; hence, it was considered conservative not to include the effect of concrete.

E. Summary of Stresses for Design Loadings

The various loads and load combinations considered in the BSW analysis and design meet the requirements of PSAR Section 12.5.2.8.3. The stresses which were considered include those due to deadload (D), accident temperature (T), accident pressure (P), seismic (E), pipe restraint loads (Rr), and jet impingement loads (Rj). In all areas of the BSW, the stresses due to the various loads were combined in accordance with the following governing equations from the PSAR:

$$1.6S \leq 1.0D + 1.0T + 1.0P$$
$$1.8S \leq 1.0D + 1.0T + 1.0P + 1.0Rr + 1.0Rj + 1.0E$$

where S is the allowable stress based on AISC Manual of Steel Construction.

The two load combination equations above reflect abnormal/extreme environmental conditions and govern the BSW design.

The stresses in the BSW for load combination equations from PSAR Section 12.5.2.8.3 besides those listed above, such as for normal operating conditions, are very low. The conditions which control the design of the BSW and under which the stresses approach the allowables are the accident conditions. The accident temperature and pipe restraint loads produce the highest stresses governing the BSW design. The accident temperature loads produce longitudinal compressive stresses in the inner and outer walls in the 25 to 30 ksi range and longitudinal tensile stresses in vertical stiffeners in the 15 to 20 ksi range. The pipe restraint loads produce compressive or tensile in-plane stresses in horizontal and vertical stiffeners which are located directly behind the pipe restraints. Because of the large number of pipe restraints attached to the BSW, and because each restraint can have a number of loading directions and magnitudes, the stress range in the stiffeners behind the restraints varies from approximately 5 to 35 ksi in either tension or compression. Only stiffeners in the immediate vicinity of a pipe restraint are stressed near the allowable stress during a pipe restraint loading.

The stresses due to deadload, seismic, pressure, and jet impingement loadings are, in general, less than 8 ksi when combined.

F. Summary of Fabrication and Erection Strategy

On the basis of a shipping and economic study, it was determined that the BSW should be fabricated and shipped in nine 120 degree sections, each approximately 16 feet high. These sections would then be assembled to form three 360 degree rings which would be stacked and welded together to form the wall.

The fabrication sequence of each ring (consisting of three 120 degree sections) was as follows:

1. A fabrication jig was erected to align and hold the shield wall plates during fabrication.

2. The inner shell plate was erected and welded to the fabrication jig.

3. The horizontal and vertical siffeners were welded to the inner shell plate.

4. The cover plates were welded to the stiffeners.

5. The three 120-degree sections were removed from the fabrication jig for shipment.

6. The sections were transported to the site for field assembly.

The field assembly of the nine BSW sections occurred in temperature controlled enclosures, separate from the containment building. In the assembly sequence, the three sections of each ring were erected and rounded up using jacking spiders. The inner wall and horizontal stiffener welds at each vertical seam were welded, and the cover plates at the vertical seams were welded. Following fitup and welding of the first ring, it was inverted in order to level the base plate which had distorted during fabrication. The base plate was leveled by attaching shim plates, depositing weld metal, and machining the surface. Two intermediate postweld heat treatments of the first ring were performed while it was in the inverted position. Weld joint reexamination by ultrasonic and magnetic particle testing was performed on the shop welds, and a substantial amount of repair was accomplished. While work was progressing on ring 1, rings 2 and 3 were assembled, fit together, and girth welded. Ring 1 was inverted and rings 2 and 3 were stacked on ring 1. NDE and engineering evaluation of defects were accomplished and repairs were made as required. Upon completion of all fitup and repair welding, the completed BSW was heat treated and prepared for transport and placement on the reactor pedestal in the containment building.

G. Specification Requirements

The BSW fabrication specification has the following technical, workmanship, and inspection requirements:

1. All fabrication work was performed under the fabricator's QA program.

2. All welding, welding procedures, and welder qualifications shall be in accordance with AWS D1.1.

3. In addition to the AWS D1.1 requirement for 100 percent visual inspection, all full penetration welds were required to receive additional NDE in accordance with the following options:

   a. Radiographic or ultrasonic inspection

   b. Progressive magnetic particle inspection at 1/3, 2/3, and 3/3 weld joint thickness.

   (The fabricator chose to employ the UT option as well as MT of the root pass.)

4. The quality of workmanship shall conform to the requirements of the AISC Code of Standard Practice for Steel Buildings and Bridges, 1976.

5. Nondestructive examination of welds shall be in accordance with AWS D1.1, Section 6. Nondestructive test operators shall be qualified in accordance with SNT-TC-1A, Recommended Practices, Nondestructive Testing, Personnel Qualification and Certification, 1975.

6. The steel shall conform to the applicable ASTM specification as given, and shall be traceable at all times to a specific heat number. All plates shall be UT'ed to ASTM A578, Level 1. Stiffener and base plate steel shall not exceed 0.01 percent sulfur.

7. The weld filler metal shall conform to the requirements of AWS D1.1 (1975 edition).

H. QA Program Requirements

The QA Program imposed by the specification required certain actions by both S&W and the Seller, Cives Steel Company.

The fabrication specification was reviewed and approved by S&W's QA Department (Quality Systems Division) to ensure the inclusion of appropriate QA/QC requirements.

The specification and Test, Inspection, and Documentation (TID) Report required the following:

<u>Cives</u>

1. Compliance with Appendix B 10CFR50
2. Submittal of QA program
3. Transmission of QA requirements to identified subcontractors
4. Conformance of NDT to AWS D1.1 - 1975
5. Submittal of welding and NDT procedures.

<u>S&W</u>

1. Qualification by survey and audit of Cives as Seller
2. Performance of inspections (over a 26-month period) covering specific TID attributes
3. The following attributes address weld quality:

    Welding Procedure
    Electrode Control Procedure
    Qualification of Welders
    Weld Preparation
    Weld Inspection
    Random Check of Fabrication Completeness
    Inspection of Surface Defects
    NDT Test Operator Certifications.
    NDT Inspection Procedures
    NDT Inspection of Welds
    Reports of NDT Tests

In addition, S&W's Procurement Quality Assurance Department performed an annual evaluation of the Seller's quality history. Included in this evaluation was a review of:

Prior Quality Program Audits
Seller Surveillance Activities
The nature of frequency of hardware unsatisfactory inspection reports and nonconformances
Results of audits by other sources (i.e., Client, CASE, ASME, NRC) when available
Seller's responsiveness and cooperation in resolving questions or problems related to Quality Assurance.

The above data was evaluated for trends which would indicate a need for an audit, survey, or other Stone & Webster Management action.

The above summary constitutes the involvement of the S&W shop inspector, and comprised our normal QA/PQA effort on this type of structure.

Below is a summary of the man-hours expended by PQA on the Nine Mile Point Nuclear Station - Unit 2 BSW (Contract No. NMP2-S204G):

Task		Hours	Vendor	Cives % of Hours
District Supervisory		159		9.1
Out-of-Plant Related Time				
Final Document Review	95			
Inspection Prep/Report Writing-	80			
Updating and Maintaining Specification	14	189		10.8
Travel Time		420		24.1
In-Plant Time				
Records Verification	72			
Systems Verification	0*			
Status Development	0			
Hardware Inspection	613			
Total In-Plant Time		685		39.4
Clerical Support		265		15.2
Typing, filing, reproducing inspection reports, updating office copy of specification, processing vendor documentation, etc.				
Other		25		1.4
Total		1,743		100.0

The above figures do not include headquarters support activities to qualify supplier, audits, certify inspectors, monitoring inspection reports for negative trends, review quality assurance program manual revisions, etc.

*Three audits were performed but not charged to purchase order.

III. DESCRIPTION OF PROBLEM

A. Statement of Problem

Numerous NDE indications which were rejectable to Section 6.19 of AWS D1.1, and hence to the fabrication specification, were discovered in shop welds after the nine BSW sections were NDE'ed in the shop and shipped to the jobsite.

The initial indications were discovered with an MT inspection, during field work, in the toe area of the cover plate to base plate weld joint. UT results, from a sample UT examination of this weld, showed that although no UT indications were present in the weld toe, reflectors were present in the root area near the backing bar (Figure 9). As a result of this, the weld joint was 100 percent UT examined, repairs were made, and a sample was removed for metallurgical analysis. Approximately 20 percent of the length of the CPBP weld was found to be rejectable by the reexamination.

Following the discovery of the cover plate to base plate weld indications, visual indications on the horizontal stiffener to inner wall (HSIW) welds where backing bars had been removed were identified during inspection of the three third-ring sections (Figure 10). As a result, the quality of backing bar welds in the entire BSW was questioned.

An engineering investigation was performed on the HSIW welds by examining the root of the weld using progressive grinding and magnetic particle testing to determine the depth and length of the defects. (The MT results are presented in Appendix B.) The initial results showed an approximately 22 percent defect rate. Due to such a high defect rate in backing bar welds, it was concluded that all weld configurations should be investigated.

A sampling plan approach which employed UT inspection was developed for examination of the various BSW weld configurations. In addition, four third-ring HSIW specimens were removed for metallurgical analysis. The results of the sampling plan showed that 11 of the 18 weld configurations did not meet expected confidence levels. Subsequently, the decision was made to perform UT inspection of all shop weld joints on the BSW. The results of the sampling plan UT did, however, show that HSIW weld joint indications occurred with less frequency than the frequency encountered during the initial MT investigation. It was concluded that during investigation of the weld indications, grinding caused the cracks to propagate because of the joint restraint; hence, a misinterpretation resulted which overestimated the original crack size.

A detailed time-sequenced history of events is presented in Appendix C.

COVER PLATE TO BASE PLATE WELD TOE AND ROOT INDICATIONS

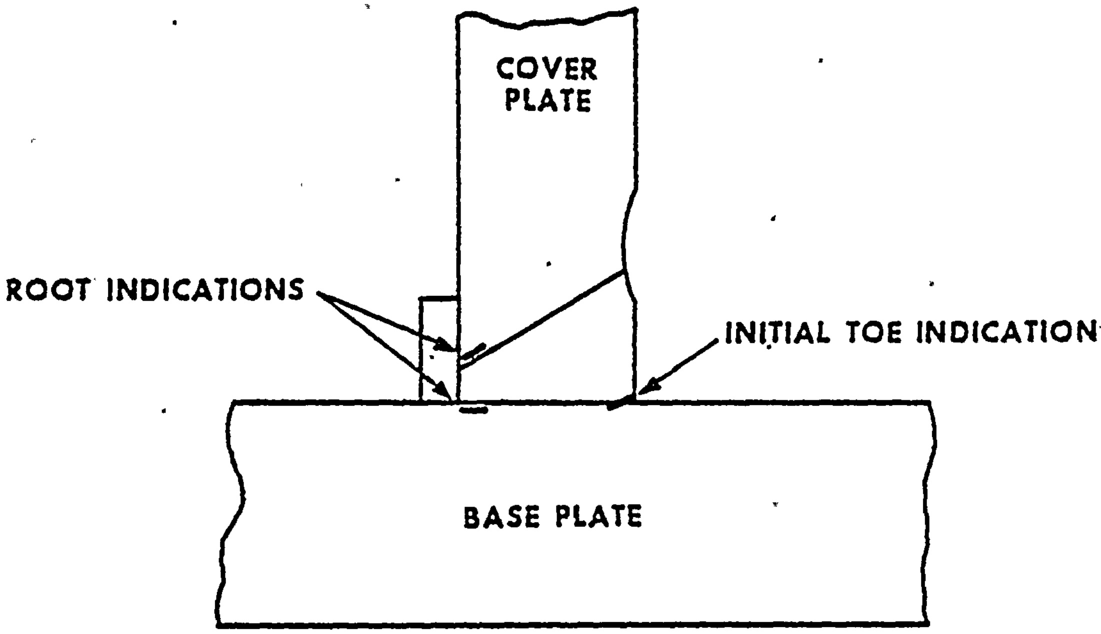


FIGURE 9

HORIZONTAL STIFFENER TO INNER WALL WELD INDICATON

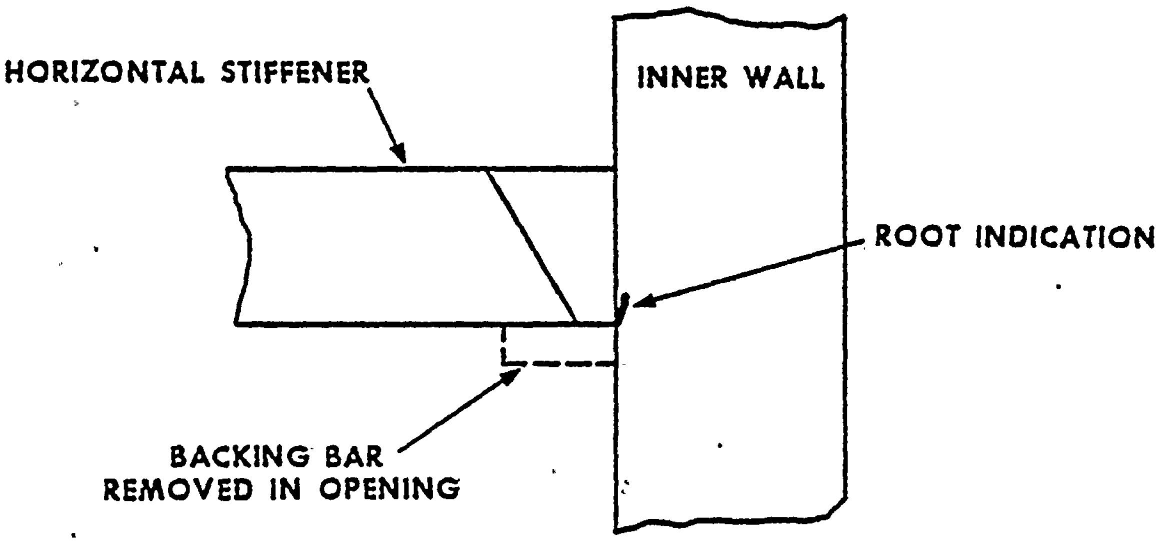


FIGURE 10

B. Metallurgical Discussion

Horizontal Stiffener to Inner Wall and Base Plate to Cover Plate Welds

A total of five metallurgical specimens which represented the worst case UT indications found during the initial engineering investigation were removed from these type welds (single vee with backing bar) to determine the nature of the indications. Four specimens were removed from the third ring horizontal stiffener to inner wall (HSIW) weld and one from the similar cover plate to base plate (CPBP) weld. Metallurgical evaluation of three of the specimens (two HSIW, one CPBP) showed that all of the indications were welding related (e.g., slag inclusions or lack of fusion) and were insignificant in size and effect. A crack was found in each of the two remaining HSIW specimens, and the cracking condition was investigated by two independent consultants. It was determined that the most probable cause of crack initiation was hydrogen and that the crack propagation was a result of the constraint of the structure. The hydrogen possibly originated from moisture which was not driven off by the preheat, but which was retained on the backing bars. (The filler material was not considered to be a possible hydrogen source.) There was no evidence of crack initiation or crack growth subsequent to the completion of welding in the shop. Since the last shop welds were completed in December 1978, it has been determined that there was insignificant risk of crack initiation or further crack propagation between the time of the investigation (late May 1980) and the time of postweld heat treatment (PWHT) (mid-July 1980). All possibility of further cracking in the unloaded structure has been eliminated by the PWHT. (The possibility of crack propagation in the loaded condition is addressed in Section IV.B.2.) Furthermore, two intermediate PWHT's previously performed on the first ring demonstrated that the PWHT process itself did not induce crack initiation or propagation and that reinspection after PWHT was not required. Therefore, the engineering evaluation and corrective action, presented in the next section, shows that this condition was adequately detected, evaluated, and repaired.

Cover Plate to Stiffener Welds

There was a high rejection rate (17 percent, 22 percent, and 12 percent on rings 1, 2, and 3, respectively) during the initial reinspection of the cover plate to stiffener welds to the standard AWS D1.1 criteria. Subsequent investigation by excavation determined that the indications were in both the weld metal and in the stiffener plate base metal. The rejection of the base metal indications was largely due to difficulty in interpreting the AWS D1.1 UT results. The laminar-type plate indications were acceptable to the ASTM A20 plate UT criteria, ASTM A578 Level I, and to the AWS D1.1 edge preparation criteria.

However, plate indications acceptable to A578 Level 1 and later subjected to AWS D1.1 standard weld UT acceptance criteria would be rejectable. Since the initial AWS D1.1 UT does not precisely categorize by type or size, or locate these plate indications, the UT inspectors conservatively assumed that the indications were in the weld metal and, therefore, rejectable.

As a result, some rework of these welds was performed in accordance with the standard AWS D1.1 UT acceptance criteria prior to establishing appropriate engineering criteria as used in the engineering evaluation addressed in Section IV. UT reexamination of some rewelded excavations resulted in unexpected stiffener plate lamellar separations which were attributed to repair weld shrinkage stresses.

In summary, it was found that the first approach, using the standard AWS D1.1 UT inspection acceptance criteria, was requiring a greater amount of repair than was necessary, and that the repairs in many cases were ineffectual in that the repair itself created a severe weld restraint condition and plate lamellar tearing concern.

Therefore, a different approach was developed to better define the indication size and location where possible as it relates to its metallurgical environment, as discussed in Section IV. An engineering evaluation of the size and location data limited the repair to that required to meet design requirements. Then, to avoid the concern of lamellar tearing due to repair weld shrinkage stresses, strict welding controls were implemented.

General Weld Quality

The welds that have been discussed in this report were made in the shop, or during field rework, repair or replacement, using either the flux-cored process with gas shielding, or the shielded metal-arc welding process. All welds examined metallurgically exhibited overall high quality. No cracking has been found in weld metal; all observed cracking has occurred in the heat affected zone (HAZ), or else outside the HAZ in the base metal.

The stabilization of the cracking condition as a result of time and the PWHT, the evaluation of this condition by stress and fracture mechanics analysis (addressed in Section IV), and strict welding controls after PWHT ensures adequate weldment quality.

IV. ENGINEERING EVALUATION AND CORRECTIVE ACTION

A. Introduction

The resolution of the BSW weld defect problem is divided into two main phases - UT of all accessible shop welds and engineering evaluation of UT indications.

1. A 100 percent UT reexamination has been performed on accessible shop welds. (Approximately 90 percent of the shop welds are accessible for UT.) Either a standard UT in accordance with AWS D1.1, Section 6.19 was performed where access permitted or a special UT in accordance with AWS D1.1 was performed.

2. Based on the UT data, an engineering evaluation was performed to determine which indications were acceptable as is and which required repairs. The evaluation consisted of both a stress analysis and a fracture mechanics analysis to establish technical justification for the acceptance of inconsequential defects. All PSAR commitments regarding loads, allowable stresses, and other technical requirements were maintained. The UT and engineering evaluation are in accordance with AWS D1.1, paragraph 3.7.6.

In addition, the results from the metallurgical analyses of the five weld specimens which were removed from the BSW have been factored into the overall plan for resolution of the problem.

B. Overall Approach

1. Stress Analysis

   The UT of the inner wall to stiffener welds provided weld defect sizes, locations, and orientations. A stress analysis consistent with the following steps was performed in the vicinity of each weld defect:

   a. The maximum tensile stress due to the combination of individual load case stresses (in accordance with the PSAR load combination equations) was determined.

   b. The maximum tensile stress was factored by the ratio of the original weld area to the reduced weld area. This reduced weld area resulted from the weld defect.

c. The resulting stress was compared to the factored allowable stresses. If the resulting stress were less than the factored allowable stress, the weld defect was acceptable from a stress analysis viewpoint; if greater, the weld defect was repaired.

If a defect were shown to be acceptable from a stress analysis viewpoint, a fracture mechanics analysis was performed using the stress and defect information to determine the defect's acceptability.

If a defect were determined to be easily accessible and repairable without undue hardship, it was repaired even though determined to be acceptable based on an engineering analysis.

2. Fracture Mechanics

In the industry today, a conservative fracture mechanics analysis is now a regular means of assuring the integrity of welded structures which realistically contain some discontinuities in the material, such as slag, porosity, or lack of fusion.

Such an analysis provides a sound basis for establishing acceptability criteria for the discontinuities and thus can eliminate unnecessary repairs.

For most structural steels under normal design conditions, linear elastic fracture mechanics (LEFM) might not be applicable. Therefore, in this analysis we use both LEFM and a technique by Dowling and Townley (Reference 1). This method, known as the Two-Criteria Approach, has been used by others (References 2 and 7), and covers the spectrum of conditions from brittle fracture (where LEFM is applicable) to completely plastic failure (where some form of limit analysis has to be used). The structure is built of ASTM A537 Class 1 steel, which has excellent fracture toughness in the longitudinal direction. In cases where it is required, the directional properties in the through-thickness direction are considered.

For the purpose of this analysis, the variety of discontinuities which might be encountered in a welded structure can be reduced to two major types: surface and subsurface defects.

It is assumed throughout the analysis that the applied stress is a tensile stress perpendicular to the defect, the applied loads are dynamic and there is no cyclic loading which could initiate fatigue cracking. In this analysis, surface and subsurface defects (Figure 11) are defined as in ASME XI, Division 1. The following relations describe LEFM approach as it applies to this structure:

In the case of a surface defect at the root of a weld, the stress intensity factor is given by

$$K_I = F_S F_E F_W F_G F(a/a_N) S \sqrt{\pi a}. \qquad (1)$$

In this equation, the defect size $a = \Delta a + a_N$ (Figure 12), where Δa is the actual defect size and a_N is the stressed portion of the backing bar. The stress $S = S_{applied} + S_{residual}$. The free-surface correction factor $F_S = 1 + 0.12\ (1-\frac{a}{\ell})^2$; the shape factor $F_E = 1/\sqrt{Q}$ (see ASME XI, Appendix A). Factor F_W is the correction for finite thickness, t, of a plate:

$$F_W = \sqrt{\frac{2t}{\pi a} \tan \frac{\pi a}{2t}} \qquad (1a)$$

In the case of a weldment with a backing bar, t = (thickness + a_N

Factor F_G accounts for the stress field gradient caused by changes in geometry of the stressed structure. In this case the factor applies to defects emanating from the root and the toe of T-welds. Analyses of such joints were performed in References 8 and 9. The most conservative of the values from these papers were used in the calculation.

Factor F (a/a_N) describes the stress distribution in the backing bar:

$$F\ (a/a_N) = \frac{2}{\pi} \left[\frac{a}{a_N} - \sqrt{(a/a_N)^2 - 1} + \pi/2 - \sin^{-1} a_N/a \right] \qquad (1b)$$

The fracture criterion can be presented as

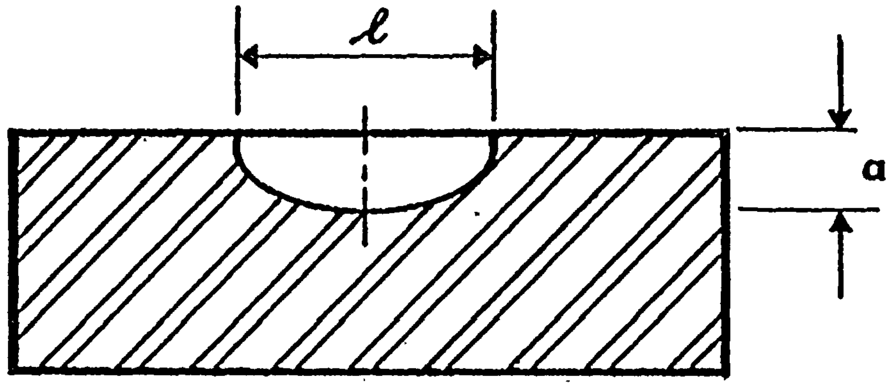
$$K_I \geq K_{Id}\ (\text{or } K_{Ic}). \qquad (2)$$

To be conservative, the dynamic fracture toughness, K_{Id}, is used in this analysis. K_{Id} is calculated from the Sailors-Corten relation (Reference 10):

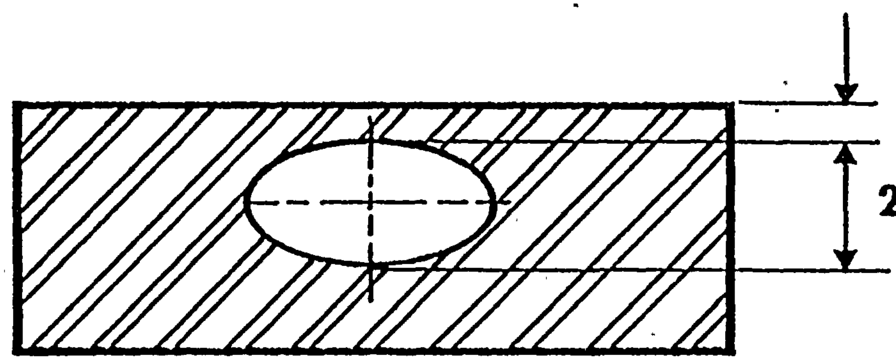
$$K_{Id} = 15.873\ (C_v)^{.375}, \qquad (3)$$

where C_v is the Charpy V-notch impact energy.

SURFACE AND SUBSURFACE DEFECTS



SURFACE DEFECT (B<a)



SUBSURFACE DEFECT (B>a)

FIGURE 11

DEFECT SIZE

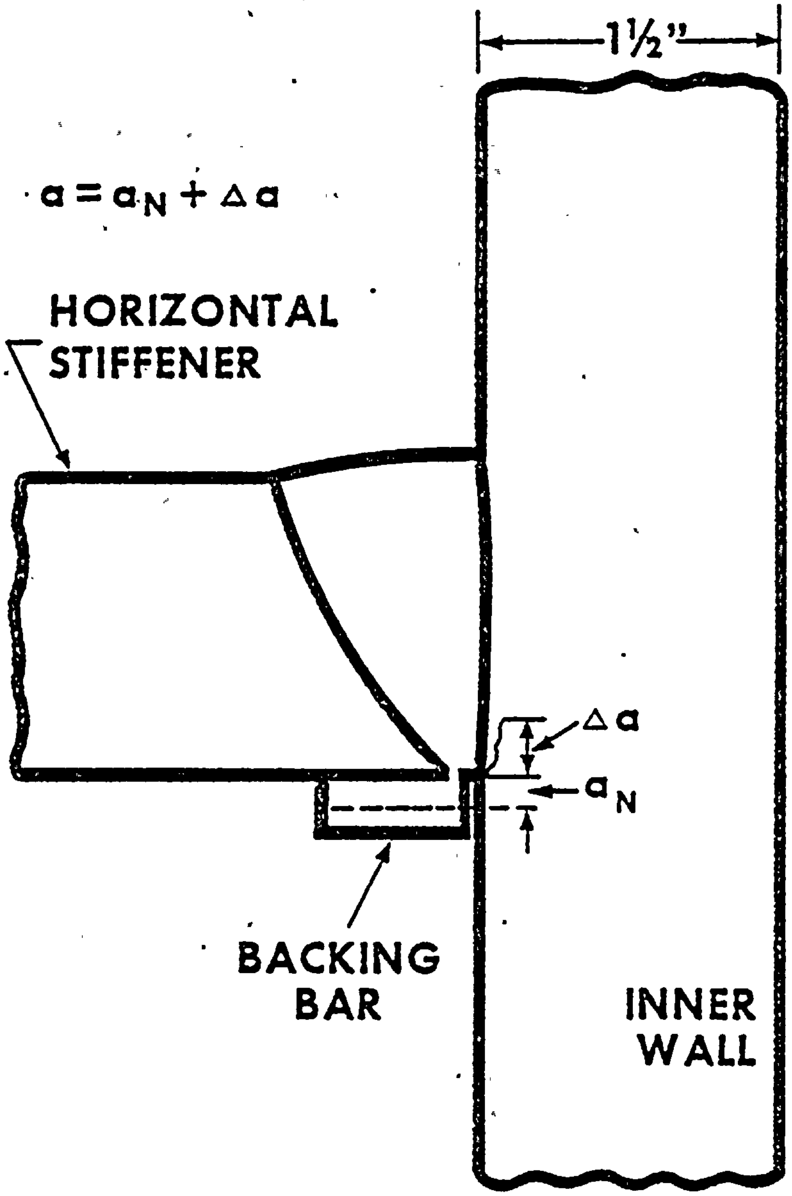


FIGURE 12

For subsurface cracks, the stress intensity factor is defined as:

$$K_I = M_m F_E S \sqrt{\pi a} \quad , \tag{4}$$

Here, M_m is the correction factor for membrane stress (see ASME XI, Appendix A). Factor F_E is the same as in Equation 1. The defect size, a, is defined in Figure 12.

The Two-Criteria relation for the critical applied stress, S_F, is:

$$S_F = S_u \times \frac{2}{\pi} \cos^{-1} \left\{ \exp\left[-\frac{\pi^2}{8} \times \frac{(S_k - S_{res})^2}{S_u^2} \right] \right\} \tag{5}$$

The ultimate stress is:

$$S_u = S_{flow} (1-a/t) = \frac{S_{ys} + S_{ts}}{2} (1-a/t), \tag{6}$$

where S_{ys} and S_{ts} are the yield strength and tensile strength, respectively. The critical stress, S_k, is calculated from the following LEFM relation:

$$S_k = \frac{K_{Id} \sqrt{1 + 4.593(a/\ell)^{1.65}}}{\sqrt{2t \left[1.12 F_G F(a/a_N)\right]^2 \tan \frac{\pi a_{cr}}{2t}}} \tag{7}$$

In the cases where bending and membrane stresses are acting on a defect, the stress intensity factor for a surface crack is given by the following LEFM equation:

$$K_I = (F_S F_W S_m + M_b S_b) F_G F(\frac{a}{a_N}) F_E \sqrt{\pi a} \tag{8}$$

where M_b is the correction factor for bending stress (see ASME XI, Appendix A), S_m is the membrane stress and S_b is the bending stress. The other parameters are defined as above.

The LEFM equation for a subsurface crack is:

$$K_I = (M_m S_m + M_b S_b) F_E \sqrt{\pi a} \tag{9}$$

(The two-criteria method is not used in cases where bending and membrane stresses are acting simultaneously because of some uncertainties in applying this method to a nonuniform stress field.)

In conclusion, it should be emphasized that the fracture mechanics analysis is very conservative. Conservative assumptions are used for the defect size and all defects are assumed to be sharp, which for the large majority of cases is more severe than the actual case. Also, it is assumed that the stresses due to various mechanical and thermal loads act simultaneously and that the applied tensile stress is perpendicular to the defect. The lowest value of the fracture toughness is used throughout the structure. Thus, the analysis provides additional assurance for the safety of the BSW structure.

Example

To illustrate the approach outlined in the report, a typical example is given below, which evaluates the effect of exclusion of 1/8 inch from the root area of a backing bar weld.

The material, A537 Class 1 steel, has minimum tensile strength $S_{ts} = 70$ ksi and minimum yield strength $S_{ys} = 50$ ksi. The lowest fracture toughness can be expected in the through-thickness direction.

The postulated defect is located in the HAZ (Figure 12), where toughness in the through-thickness direction, according to S&W experimental data, is better than in the base metal; the base metal toughness is used here for the sake of conservatism. Based on the experimental Charpy impact tests performed at $90^{o}F$ and available published data (Reference 11), the Charpy energy at $100^{o}F$ is:

$$C_v \geq 20 \text{ ft lb}$$

LEFM Approach

The stress intensity factor for this case is given by Equation 2 (see Section IV.B.2). The parameters in the equation are given below:

$$a = \Delta a + a_N.$$

For this particular example $a_N = 0.090$ inch. Thus, in this example:

$$a = 0.125 + 0.090 = 0.215 \text{ inch.}$$

The maximum average tensile stress is given as $S_{applied} = 25$ ksi. The residual stress after PWHT at $1,100^{o}F$ is assumed to be 10 percent of the yield strength, that is:

$$S_{res} = 0.1\ S_{ys} = 0.1 \times 50 = 5 \text{ ksi.}$$

Therefore,

$$S = S_{applied} + S_{res} = 25 + 5 = 30 \text{ ksi}$$

$F_S = 1.12$

F_E is given in Appendix A to ASME XI, but it can be also expressed by the following equation:

$$F_E = \frac{1}{\sqrt{1 + 4.593\left(\frac{a}{\ell}\right)^{1.65} - 0.212(S/S_{ys})^2}} = \frac{1}{\sqrt{Q}}$$

where ℓ is defect length.

F_W is given by Equation 1a (see Section IV.B.2)

$F_G = 1.328$ (in this case)

$F(a/a_N) = F\left(\frac{0.215}{0.090}\right) = 0.86$ (see Section IV.B.2, Equation 1b)

Finally, note that the defect is assumed to be infinitely long so that $a/_\ell = 0$.

Thus,

$$K_I = 1.12F_G F(a/a_N)S\sqrt{\pi a}\sqrt{\frac{\frac{2t}{\pi a} \tan \frac{\pi a}{2t}}{1 + 4.593\left(\frac{a}{\ell}\right)^{1.65} - 0.212(S/S_{ys})^2}}$$

$$K_I = 1.12 \times 1.328 \times 0.86 \times 30\sqrt{0.215\pi}\sqrt{\frac{\frac{2 \times 1.59}{0.215\pi} \tan \frac{0.215\pi}{2 \times 1.59}}{1 + 4.593 \times (0)^{1.65} - 0.212 \left(\frac{30}{50}\right)^2}}$$

$$= 33.2 \text{ ksi}\sqrt{\text{inch}}$$

The fracture toughness of the material is calculated from Equation 3 (see Section IV.B.2):

$$K_{Id} = 15.873\ (20)^{0.375} = 48.8 \text{ ksi}\sqrt{\text{inch.}}$$

Thus,

$$K_I = 33.2 < K_{Id} = 48.8$$

The critical defect size evaluated from Equation 1 (see Section IV.B.2) is:

$$\Delta a_{cr} = 0.47 \text{ inch.}$$

The evaluation is performed by an iteration process until a calculated stress intensity factor equals the fracture toughness of the material.

Two-Criteria Approach

The critical applied stress is given by Equation 5 (see Section IV.B.2). The parameters in this equation are given by Equations 6 and 7 (see Section IV.B.2), and S_{res} = 5 ksi. The critical defect size is then evaluated from Equation 5 (see Section IV.B.2):

$$\Delta a_{cr} = 0.42 \text{ inch.}$$

The actual defect size is shown to be much less than the critical defect size and is, therefore, acceptable. Furthermore, the ratio of the critical crack size to the actual crack size is large.

$$\frac{\Delta a_{cr}}{\Delta a} = \frac{0.42}{0.125} = 3.36.$$

Therefore, a 1/8-inch defect at the root of a weld subjected to 25 ksi average tensile stress is acceptable.

REFERENCES

1. Dowling, A. R. and Townley, C. H. A. The Effect on Defects on Structural Failure: A Two-Criteria Approach. The International Journal of Pressure Vessels and Piping, 3, 2, 1975, p 77-107.

2. Chell, G. C. A Combined Linear Elastic and Post-Yield Fracture Mechanics Theory and Its Engineering Applications. Fracture Mechanics in Engineering Practice, Editor, P. Stanley. Applied Science, London, England.

3. Harrison, R. P. A Unified Approach to Failure Assessment of Engineering Structures. Fracture Mechanics in Engineering Practice, Editor, P. Stanley. Applied Science, London, England.

4. Muscati, A. and Turner, C. E. Post-Yield Fracture Behavior of Shall-Notched Alloy Steel Bars in Three-Point Bending. Fracture Mechanics in Engineering Practice, Editor, P. Stanley. Applied Science, London, England.

5. Townley, C. H. A. The Integrity of Cracked Structures Under Thermal Loading. Fracture Mechanics in Engineering Practice, Editor, P. Stanley. Applied Science, London, England.

6. Roche, R. L. Analysis of Structures Containing Cracks - Some Comments on the J Integral Criterion. The International Journal of Pressure Vessels and Piping, 7, 1979, p 65.

7. Bloom, J. M. Prediction of Ductile Tearing of Compact Fracture Specimens Using the R-6 Failure Assessment Diagram. The International Journal of Pressure Vessels and Piping, 8, 1980, p 215-231.

8. Usami, S. et al. Transactions of the Japan Welding Society, April 1978.

9. Guerney, T. R. Finite Element Analysis of Some Joints with the Weld Transverse to the Direction of Stress. Welding Institute Research Report, E/62/75, March 1975.

10. Sailors, R. H. and Corten, H. T. ASTM STP513, p 164.

11. Lentz, J. S. Journal of Pressure Vessel Technology, Vol. 100, February 1978, p 77.

C. Inner Wall to Stiffener Evaluation

1. Approach

Based on UT data, weld defect sizes, locations, and orientations were obtained for evaluation. The special UT performed from the inner wall was the sole basis for evaluation of indications and subsequent accept or rework dispositions. Stresses in the vicinity of the defects were evaluated as a result of reduced weld area due to the defects. All PSAR commitments regarding allowable stresses were maintained. After it was shown that the stresses were at an acceptable level, the stress and defect information was used in a fracture mechanics evaluation.

2. UT Techniques Employed

All horizontal and vertical stiffeners to inner wall welds were 100 percent examined from the inner wall surface using special ultrasonic techniques in addition to those described in AWS D1.1. These techniques were especially effective in detecting planar-type discontinuities located in fusion and heat affected zones of these welds.

The examination employed 5 MHz 1/2-inch diameter, straight beam transducer and 4 MHz, 8 x 9 mm 45-degree angle beam transducers to scan the welds as shown in Figure 13. The transducer frequencies and sizes were selected to provide optimum sensitivity, resolution, and minimal interference from near-field effects. Two calibration blocks as shown in Figures 14 and 15 were prepared for establishing primary reference sensitivity levels. The calibration block in Figure 14 contains a flat bottom slot 1/8 inch wide, which was used to establish the reference level for the straight beam examination. The primary reference level for the 45-degree angle beam examination was obtained by using the slot in the calibration block oriented at a 45-degree angle. In addition, a part of the fusion zone of vertical stiffener along a 45-degree bevel that does not get adequate coverage by angle beam transducer was examined by straight beam transducer. The reference sensitivity level for this examination was established by using the notch-in-second calibration block shown in Figure 15. A distance amplitude curve (DAC) was established for each examination and all the indications above 20 percent DAC were recorded. All recordable indications were further evaluated by additional ultrasonic examination. The responses from weld discontinuities were compared to the responses from various size reference reflectors which simulated the orientation and location of the discontinuities. During this comparison, corrections were made for any differences in second attenuation

STIFFENER TO INNER WALL STRAIGHT BEAM AND 45° ANGLE BEAM SCANNING PATTERNS

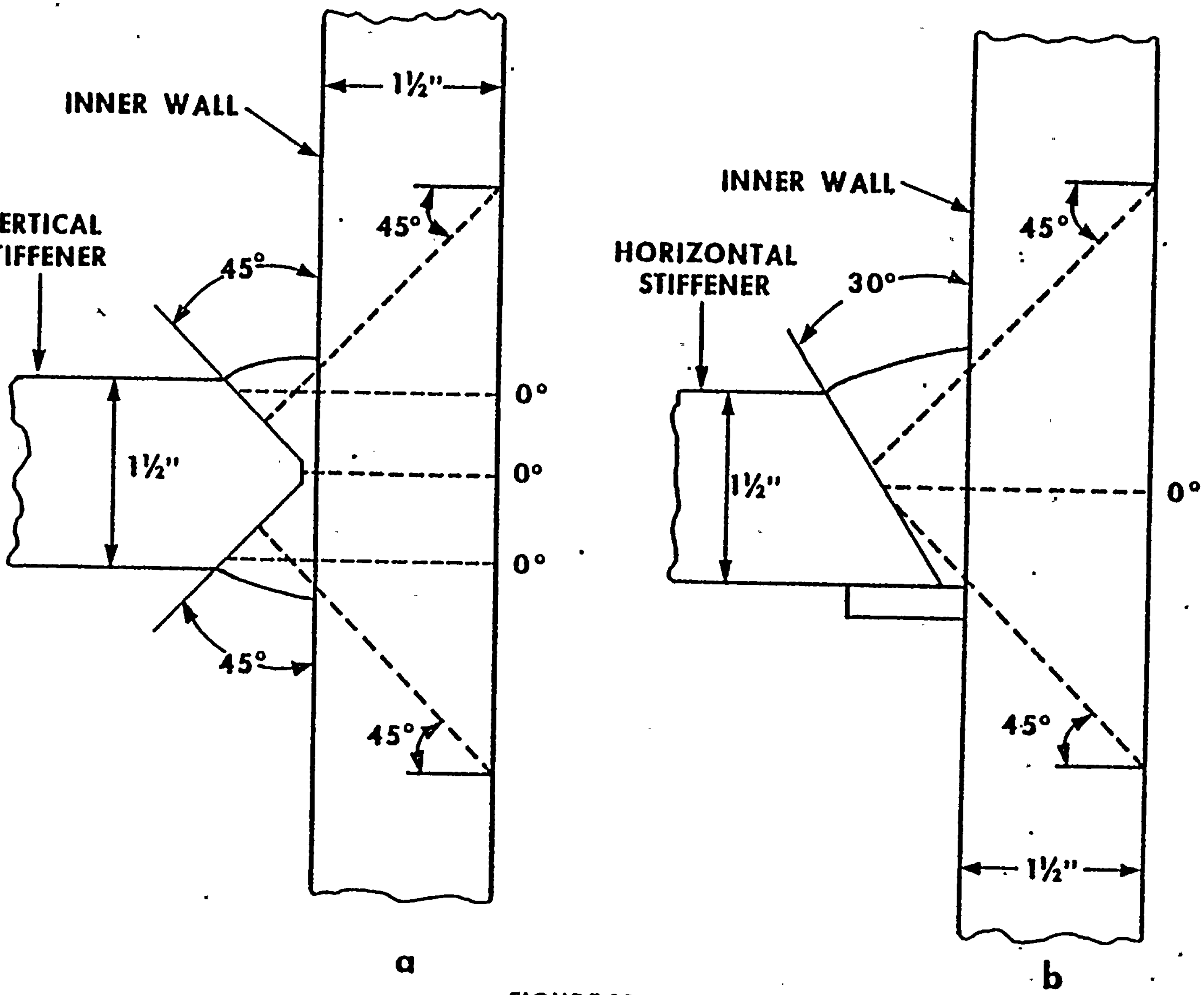


FIGURE 13

CALIBRATION BLOCK USED TO ESTABLISH PRIMARY REFERENCE LEVEL

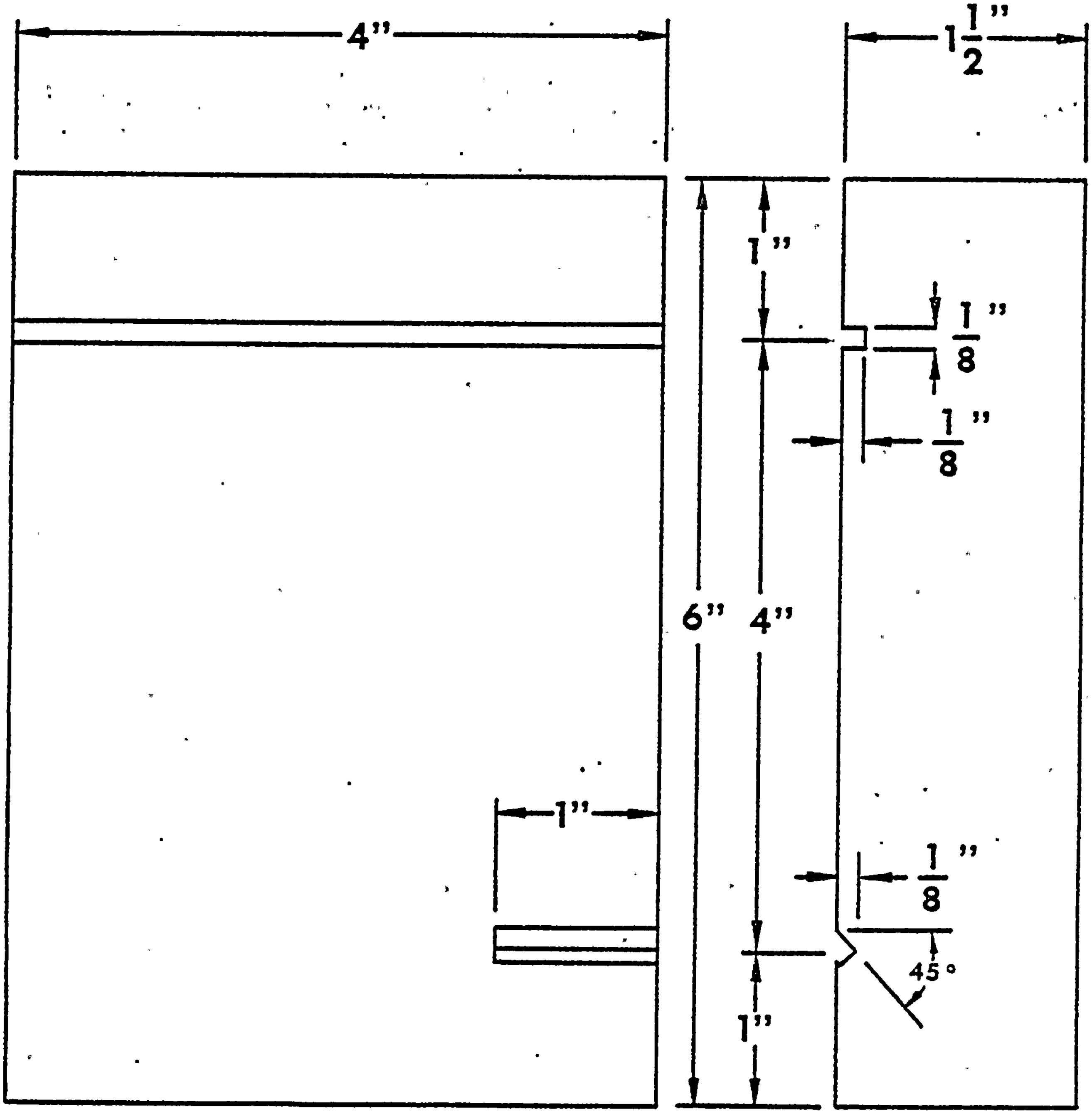


FIGURE 14

CALIBRATION BLOCK USED FOR THE EXAMINATION OF FUSION AND HEAT AFFECTED ZONE IN VERTICAL STIFFENER

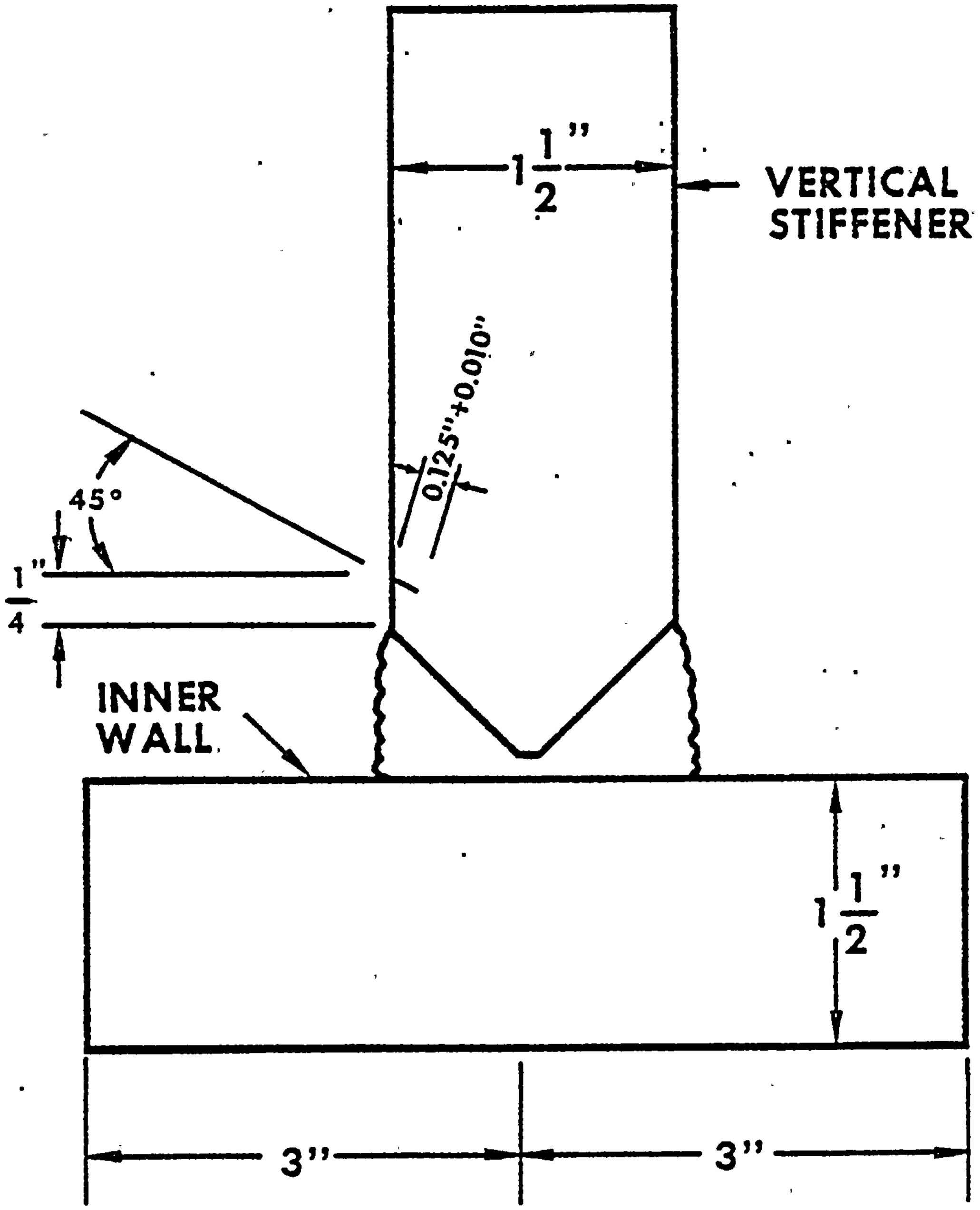


FIGURE 15

characteristics between the reference block and stiffener material.

3. Map of Defect Sizes and Locations

A map of defect sizes and their respective locations is included as Figure 16. (The defects are shown on a developed view of the BSW from the perspective of being outside the BSW, looking toward the inside.). A total of 23 indications (four in the first ring, two in the second ring, and 17 in the third ring) were evaluated and are shown on the map. Details of the inner wall to stiffener UT indication data are presented in Table 1.

4. Summary of Evaluation Results and Repairs Required

All 23 indications were evaluated by stress and fracture mechanics analyses and were determined to be acceptable. However, ten indications were reworked because they were accessible from the inside of open compartments and it was determined that rework would not be harmful to the structure. The remaining 13 indications were not reworked because of one or more of the following conditions where rework of the acceptable welds would be more detrimental than the presence of the defect:

a. Removal of a cover plate would be required to gain access to the indication

b. Cutting of an access hole in a stiffener would be required in order to chase an indication into an adjacent compartment

c. Cutting through the inner wall would be required to gain accessibility to the indication

d. Excavation of a significant amount of weld metal or base metal would be required to remove the indication, thus imposing large through-thickness shrinkage stress on the inner wall.

The repair status of each indication is summarized in Table 1.

5. Example Calculation

Subsurface Defect in the Inner Wall of the BSW

The following wxample issustrates an actual case of a reported indication in the inner wall of Ring 1. The indication was interpreted as being a subsurface defect parallel to the surface of the inner wall (Figure 17).

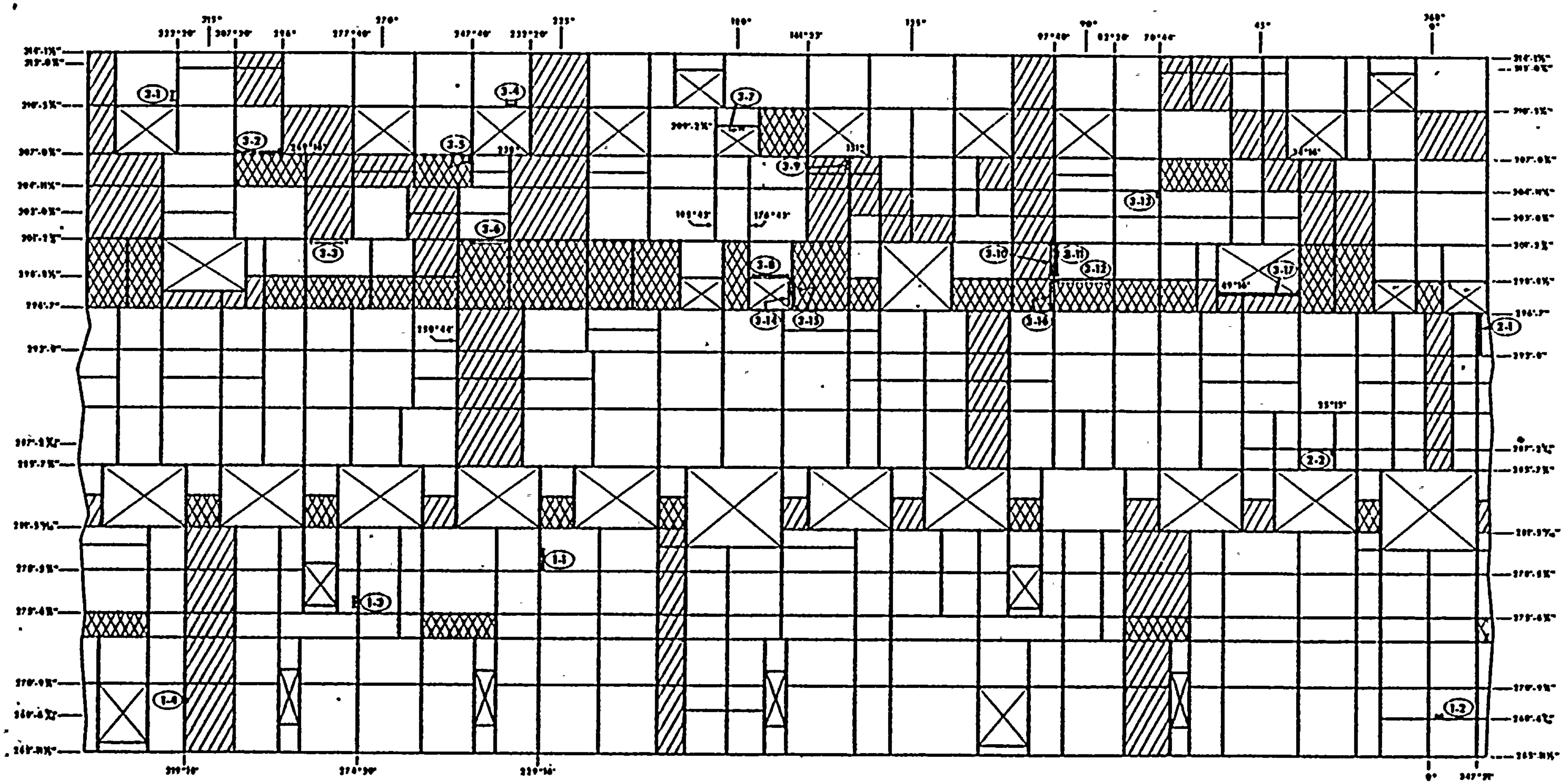


FIGURE 16 - MAP OF INNER WALL TO STIFFENER UT INDICATIONS

LEGEND

- PIPE, DOOR, OR DUCT OPENING
- FIELD INSTALLED, BEFORE-CONCRETE COVER PLATES
- FIELD INSTALLED, AFTER-CONCRETE COVER PLATES

TABLE 1 - INNER WALL TO STIFFENER UT INDICATION DATA

Indication No.[1]	Length (Inches)	Thru-Thick Depth (Inches)	Accept/Rework Disposition	Basis for Disposition
1-1	10 (int)[2]	1/8	Accept	Inaccessible, CP removal required
1-2	13/16	1/8	Accept	Inaccessible, CP removal required
1-3	1	1/8	Accept	Inaccessible, CP removal required
1-4	5/8	1/8	Accept	Inaccessible, CP removal required
2-1	28 1/4	1/8	Accept	Inaccessible, CP removal required
2-2	1 7/8	1/8	Accept	Inaccessible, CP removal required
3-1	5/8	1/8	Accept	Inaccessible, CP removal required
3-2	37 1/2 (int)	1/8	Accept	Excessive weld metal or base metal removal required
3-3	23 (int)	1/4	Accept	Excessive weld metal or base metal removal required
3-4	5 1/8	1/8	Rework	Accessible, only small amount of metal removal required
3-5	3/4	1/8	Rework	Accessible, only small amount of metal removal required
3-6	22	1/8	Accept	Excessive weld metal or base metal removal required
3-7	1 3/8	1/8	Rework	Accessible, only small amount of metal removal required
3-8	26	1/8	Accept	Excessive weld metal or base metal removal required Possible burn-through
3-9	5	1/8	Rework	Accessible, only small amount of metal removal required
3-10	29 3/8	1/8	Accept	Excessive weld metal or base metal removal required
3-11	29 3/8	1/8	Rework	Accessible, only small amount of metal removal required
3-12	43 (int)	1/8	Rework	Accessible, only small amount of metal removal required
3-13	5/8	1/8	Accept	Inaccessible, CP removal required
3-14	18	1/8	Rework	Accessible, only small amount of metal removal required
3-15	18	1/8	Rework	Accessible, only small amount of metal removal required
3-16	24	1/8	Rework	Accessible, only small amount of metal removal required
3-17	45 1/2	1/8	Rework	Accessible, only small amount of metal removal required

Notes

1. Indication numbers correspond to numbers in Figure 16.
2. (int) Indicates the defect is intermittent in the given length.
3. All indications were acceptable based on engineering evaluations.

SUBSURFACE DEFECT

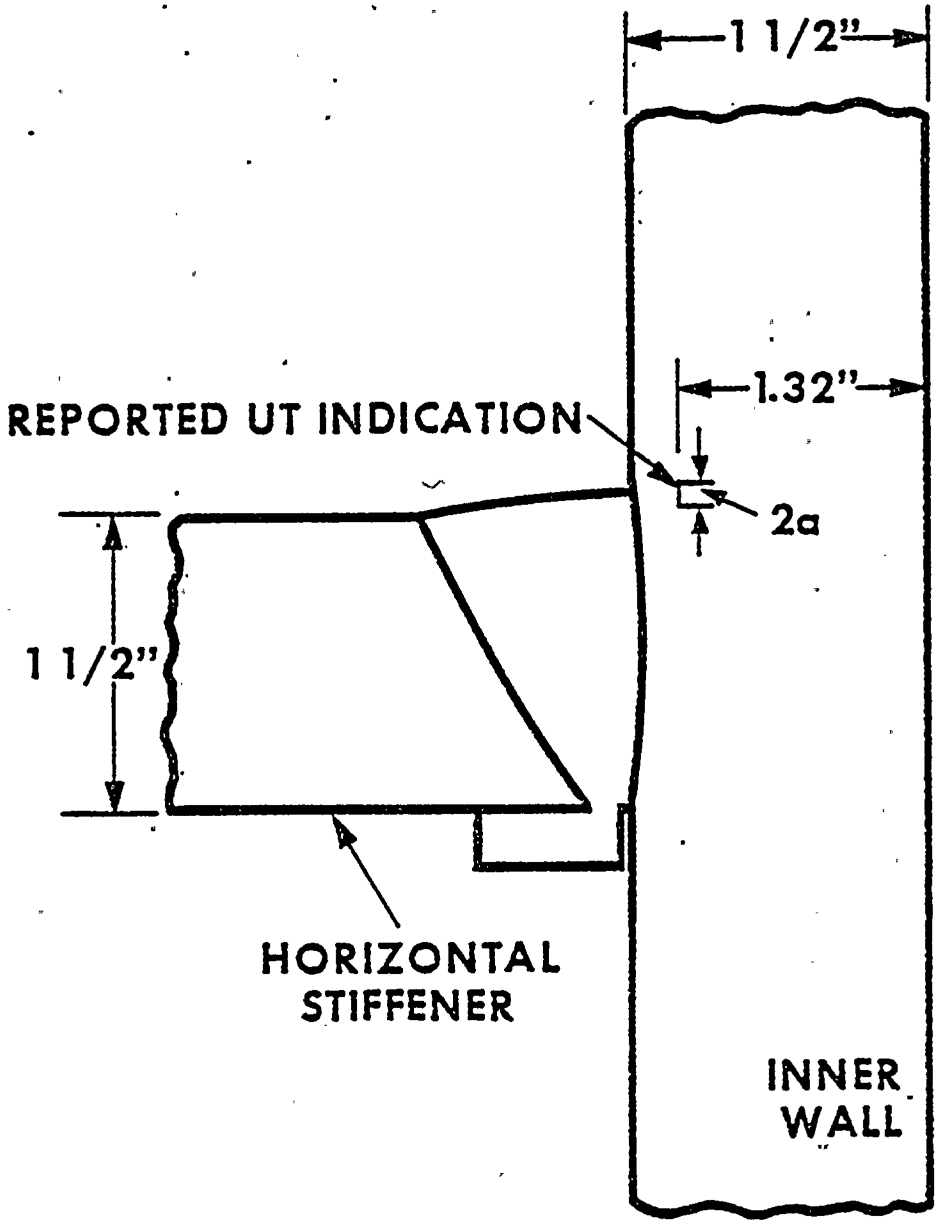


FIGURE 17

LEFM Approach

The stress intensity factor for this case is given by Equation 4 (see Section IV.B.2). For subsurface defects, the defect size, a, in Equation 4 is half of the actual reported defect size (denoted 2a). Note that the actual defect size, 2a, was reported as less than 1/8 inch, but it was conservatively assumed here that 2a= 1/8 inch and therefore a = 1/16 inch. The defect length, according to the UT report, was ℓ= 3/4 inch. Hence, a/ℓ = 0.083.

The stress, once again, includes both applied and residual stresses. The applied stress in this example is 16 ksi, and the residual stress, as in Section is 5 ksi. Thus $S=S_{applied} + S_{res} = 16 + 5 = 21$ ksi.

Factor $M_m = 1$ (defect parallel to the plate surface)

Factor F_E is given by the same equation as in the example

in Section IV.B.2 above.

Thus,

$$K_I = \frac{21\sqrt{\pi \times 1/16}}{\sqrt{1 + 4.593(0.083)^{1.65} - 0.212(\frac{21}{50})^2}} = 9.13 \text{ ksi } \sqrt{\text{in}}$$

Since the fracture toughness of the material in the through-thickness direction at the minimum temperature when the stress might develop is:

$K_{Id} = 48.8$ ksi $\sqrt{\text{in}}$ (see Section IV.B.2)

Thus, K_I is much less than K_{Id}.

The critical defect size evaluated from LEFM Equation 4, (see Section IV.B.2) assuming that the ratio a/ℓ is constant, $a_{cr} = 1\ 3/4$ inch and $2a_{cr} = 3\ 1/3$ inch.

Two-Criteria Approach

The critical applied stress is given by Equation 5 (see Section IV.B.2). The parameters in this equation are given by Equations 6 and 7 (see Section IV.B.2).

$S_{res} = 5$ ksi

The critical defect size is evaluated by an iteration process from Equation 5:

$a_{cr} = 1.7$ inch,

so that:

$2a_{cr} = 3.4$ inch.

Therefore, the 1/8 inch defect subjected to 16 ksi stress is acceptable.

D. Cover Plate to Stiffener Evaluation

1. Approach

The cover plate to stiffener welds were evaluated in a similar manner to the inner wall to stiffener welds; namely, with a stress analysis employing mapped UT data, effective weld area reduction, and stress comparisons, and with a fracture mechanics analysis employing stress and UT data.

The engineering evaluation limited the amount of cover plate to stiffener weld repair to only that which was necessary to meet the design and PSAR commitments. This limited repair was necessitated by the following events.

The relatively high percent rejection rate (17 percent, 22 percent, and 12 percent for Rings 1, 2, and 3, respectively) during the initial reinspection of the cover plate to stiffener welds was determined by excavation to be due to indications both in the weld metal and in the stiffener plate base metal. The rejection of the base metal indications was largely due to difficulty in interpreting the UT results. These laminar type indications were acceptable to the plate UT criteria, ASTM A578 Level I, and to the AWS D1.1 edge preparation criteria. However, if the same indications were in the weld metal, they were rejectable to the AWS D1.1 standard weld UT acceptance criteria. The standard AWS D1.1 UT did not precisely locate these indications; therefore, it was conservatively assumed that the indications were in the the weld metal.

Some rework of these welds was required in accordance with the standard AWS D1.1 UT acceptance criteria. UT reexamination of the reworked cavities showed unexpected stiffener plate lamellar separations which were due to the weld shrinkage stresses induced by repair.

This approach, using the standard AWS D1.1 acceptance criteria, required a greater amount of repair than was necessary; the repairs in many cases were ineffectual since the repair itself created new weld indications.

A UT program, described in Section IV.D.2, was developed to better define the indication size and location.

On repairs which were necessary, welding techniques were implemented to mitigate the weld shrinkage stress problem.

It should be noted that some cover plate welds can be UT examined only when the adjacent after-concrete cover plates are attached.

2. UT Techniques Employed

All shop cover plate welds were examined by ultrasonics in accordance with the requirements of the American Welding Society Code, AWS D1.1. In addition, all indications rejected by the standard AWS criteria were further examined by additional UT techniques to establish their relevancy and to better quantify their nature and size. Most of the AWS rejectable indications were marginal and data is not available which relates AWS defect ratings to actual flaw size.

In order to develop suitable procedures which would provide sufficient information for stress analysis and fracture mechanics evaluation of the discontinuities, the weld area was divided into three separate zones as shown in Figure 18, and the weld discontinuities tabulated for each zone. Indications in Zone 1 were not reexamined, since the ultrasonic responses from weld discontinuities may have been influenced by numerous small but acceptable plate laminations contained within the horizontal and/or vertical stiffeners. All indications in Zone 1 were removed and the welds were repaired.

Indications in Zone 2 were examined using, in all cases, both 45-degree and 70-degree beam angles as shown in Figure 19. Reference blocks containing artificial reflectors which simulate the orientation, size, and location of critical flaws as defined by fracture mechanics analysis were used during the examination. They are shown in Figure 20. Other test parameters, such as transducer frequency, probe size, and instrument calibration, were addressed to provide the resolution necessary for proper interpretation during the examination. During the examination, all responses were recorded in decibels relative to the responses from each reference block reflector.

Indications in Zone 3 were examined to establish that their physical size did not exceed a 1/2-inch through-wall dimension. This criterion was determined by fracture mechanics analysis as a meaningful threshold for gathering data. The use of a 4 MHz, 8 x 9 mm, 70-degree angle beam probe within the halfskip distance provided a sound beam within Zone 3 which was less than the 1/2-inch through-wall size; therefore, other estimated limits of reflectors larger than 1/2 inch are meaningful by the 6 dB drop method.

Also, the separation of probe positions at the 6 dB limits from point source reflectors is far less than from the 1/2-inch reflectors. Probe spacings were recorded for the 6 dB limits from all reflectors in Zone 3.

The results of all examinations were reported for further engineering evaluation as stated below.

COVER PLATE TO STIFFENER WELD ZONES

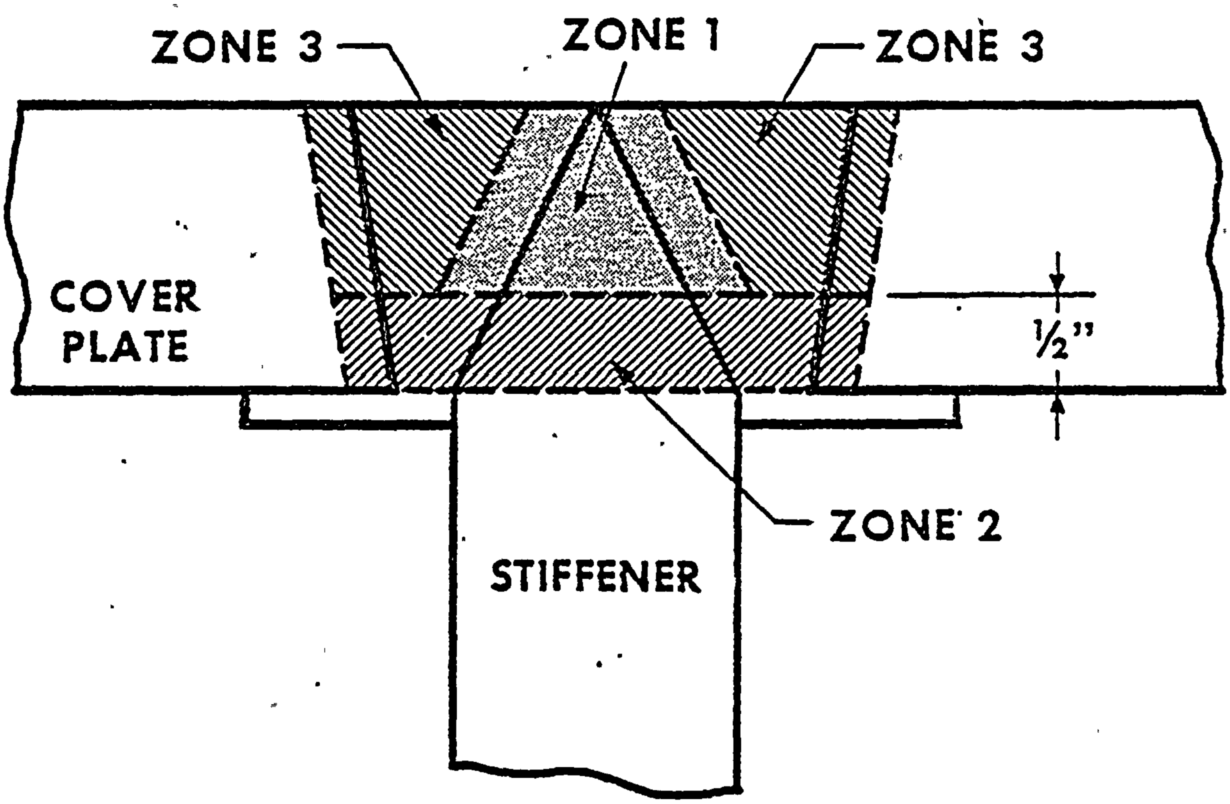


FIGURE 18

COVER PLATE TO STIFFENER WELD ZONE 2 BEAM ANGLES

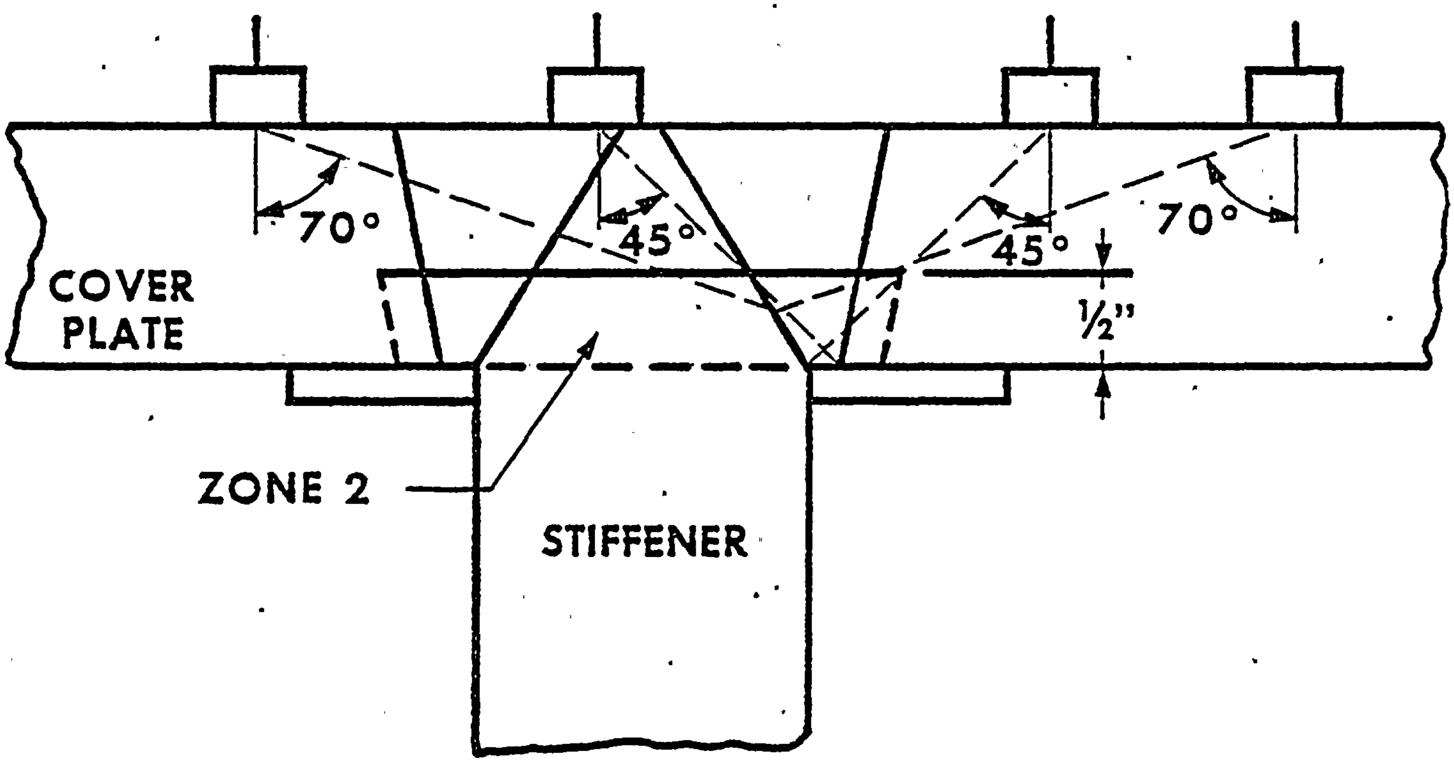
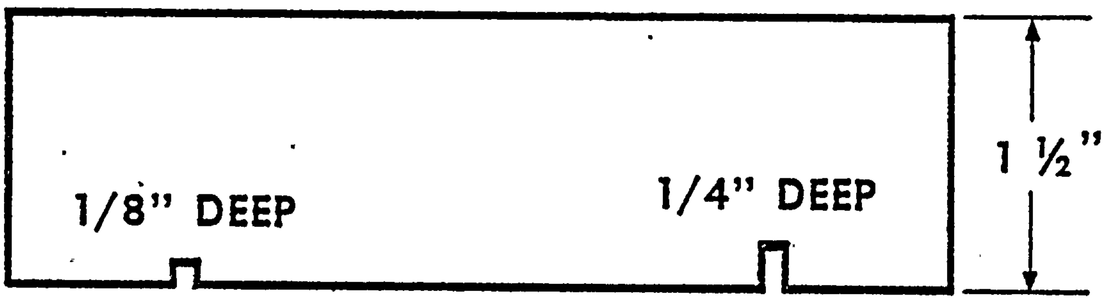
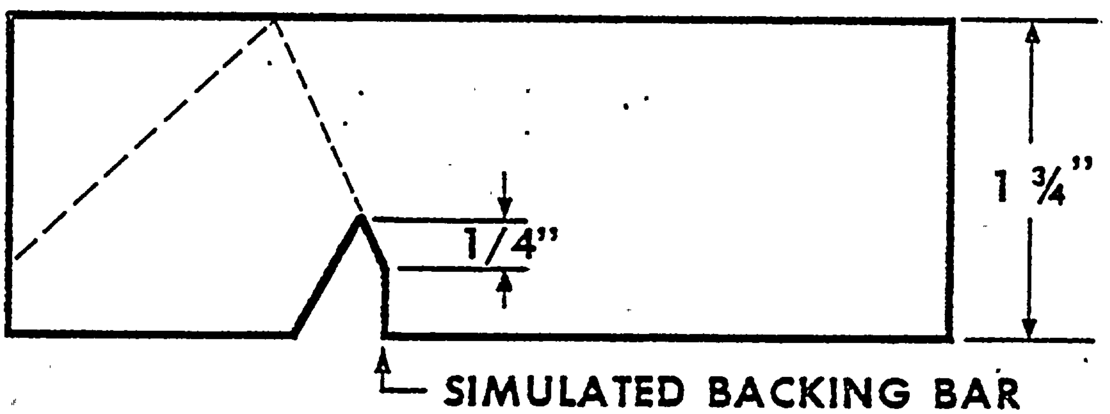


FIGURE 19

REFERENCE BLOCKS



a. 45° REFERENCE BLOCK



b. 70° REFERENCE BLOCK

FIGURE 20

Interpretations

The responses from all indications in Zone 2 were less than the 1/8-inch deep notch in the 45-degree reference block and also substantially less than the 70-degree reference block reflector.

The response from all reflectors in Zone 3 did not indicate discontinuities with through-wall dimensions exceeding 1/2 inch.

Evaluation

The through-wall size of all Zone 2 discontinuities may be considered to be less than 1/4 inch and the through-wall size of all Zone 3 discontinuities less than 1/2 inch. These values were used as part of the engineering disposition for acceptance or rejection.

3. Map of defect sizes and locations

A map of defect sizes and their respective locations is included as Figure 21. (The defects are shown on a developed view of the BSW from the perspective of being outside the BSW, looking toward the inside.) All indications in Zones 2 and 3 which require an engineering evaluation are shown. The first ring had 71 defects requiring evaluation; the second ring, 51 defects; and the third ring, 121 defects. Additional cover plate to stiffener data are summarized in Table 2.

4. Summary of Evaluation Results and Repairs Required

The total length of cover plate to stiffener welds examined was approximately 48,100 inches. Of the total length examined, 243 UT indications totalling approximately 1,470 inches were discovered in Zones 2 and 3, which was approximately 3 percent of the total length examined. Through stress and fracture mechanics analyses, 228 indications were acceptable as-is; 15 indications totalling approximately 142 inches were reworked.

A summary, including indication sizes and dispositions, of the indications is presented in Table 2.

In Zone 1 of the cover plate to stiffener welds, 48 indications totalling approximately 450 inches were discovered and reworked.

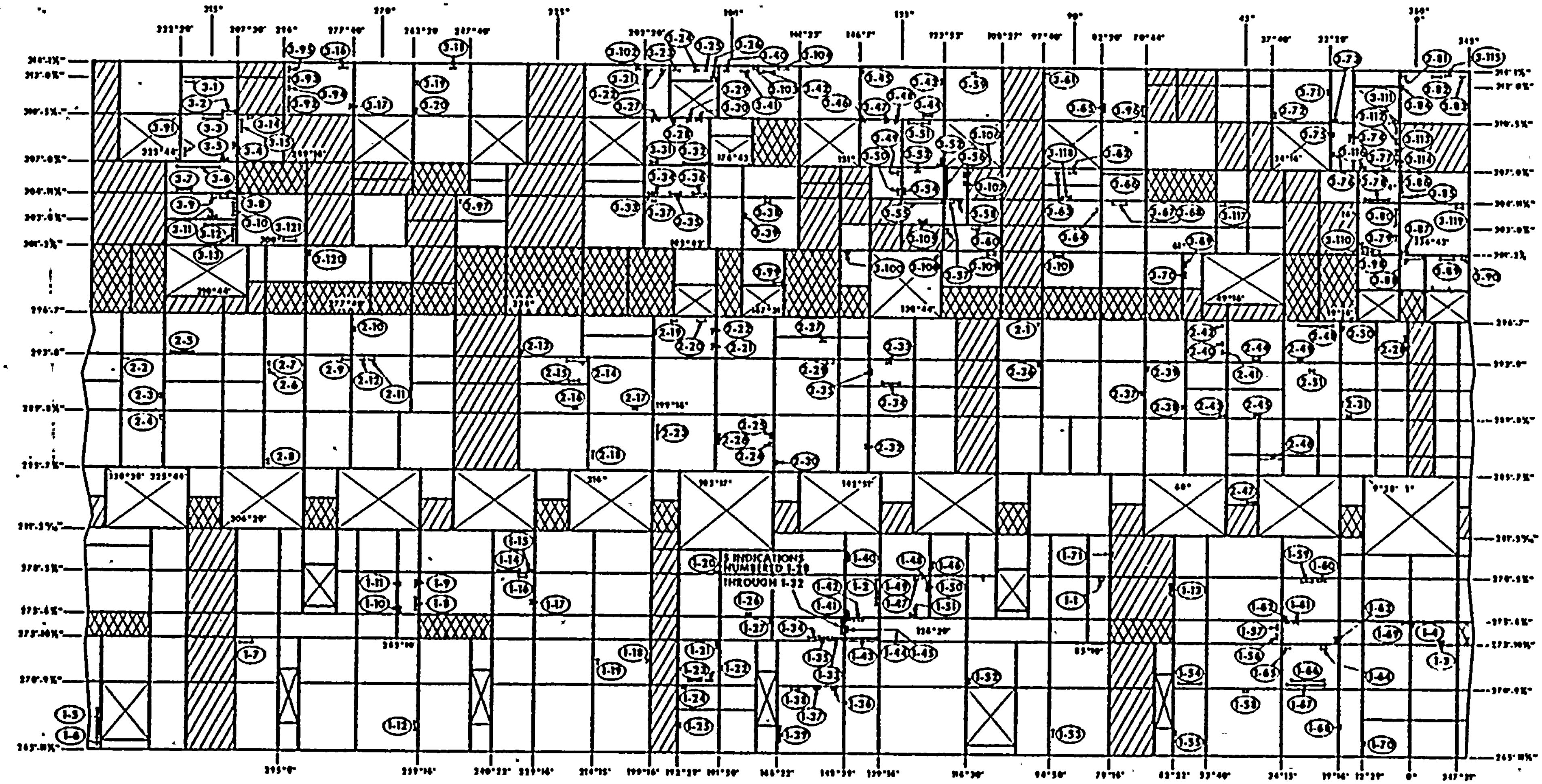


FIGURE 21-MAP OF COVER PLATE TO STIFFENER UT INDICATIONS

LEGEND

- PIPE, DOOR, OR DUCT OPENING
- FIELD INSTALLED, BEFORE CONCRETE COVER PLATES
- FIELD INSTALLED, AFTER CONCRETE COVER PLATES

TABLE 2 - COVER PLATE TO STIFFENER UT INDICATION DATA

(All indications were acceptable from an engineering evaluation unless otherwise noted.)

Indication No.(1)	Zone(2)	Length (Inches)
1-1	2	1
1-2	2	26
1-3	2	2
1-4	3	1 1/4
1-5	2	30 1/4
1-6	3	1
1-7	2	12 3/4
1-8	2	11 1/4
1-9	3	4 3/8
1-10	2	2 3/4
1-11	3	1 1/2
1-12	2	7
1-13	3	4 5/8
1-14	2	18
1-15	3	1
1-16	2	5 1/2
1-17	3	1 1/4
1-18	2	1 3/8
1-19	3	1 3/8
1-20	3	1 1/4
1-21	2	2 1/2
1-22	2	1 1/8
1-23	3	10
1-24	2	1 1/8
1-25	3	4
1-26	2	1 1/4
1-27	3	2 3/4
1-28	2	1 3/4
1-29	2	1 1/2
1-30	3	2
1-31	3	3
1-32	3	2
1-33	2	8 1/4
1-34	2	4 1/2
1-35	2	8 1/4
1-36	3	1 3/4
1-37 (3)	2	3 1/4
1-38	2	4 1/4
1-39	3	15
1-40	3	7 1/4
1-41	3	2 5/8
1-42	2	7 1/8
1-43	3	27 1/2
1-44	3	2
1-45	2	1 1/4
1-46	3	4
1-47	3	1 1/4
1-48	3	1
1-49	2	1 1/2
1-50	2	8 3/8
1-51	2	3 1/2
1-52	3	2 1/2
1-53	3	5 1/2
1-54	2	17 3/8
1-55	3	2 1/4
1-56	2	1 3/8
1-57	3	4 1/4
1-58	3	2 1/4
1-59	2	7
1-60	2	8 1/8
1-61	2	2 1/8
1-62	3	1 1/4
1-63	2	1
1-64	2	1 3/8
1-65	2	1 3/4
1-66	2	22 3/8
1-67	2	27
1-68	2	3 1/2
1-69	2	2 1/2
1-70	2	3 1/4
1-71	2	1 3/4
2-1	2	3/4
2-2	2	1 1/4
2-3	2	1
2-4	2	2 1/4
2-5	2	25 3/8
2-6	2	1 1/2
2-7	3	1 1/4
2-8	3	3 1/2
2-9	2	4 1/2
2-10	2	1 1/2
2-11	3	1 3/4
2-12	3	2 3/8
2-13	2	1 3/4
2-14	3	16
2-15	2	5 3/4
2-16	2	2
2-17	2	2 1/4
2-18	2	4
2-19	2	3 1/2
2-20	2	8
2-21	2	4 3/4
2-22	2	1 3/4
2-23 (3)	3	14 3/8
2-24	2	2 1/4
2-25	3	1 3/4
2-26	2	8 7/8
2-27	3	4
2-28	2	2 1/2
2-29	2	3
2-30	3	1 3/8
2-31	3	1
2-32	2	2 3/4
2-33	3	1 1/2
2-34	2	15 1/4
2-35	2	6 1/8
2-36	2	2
2-37	3	1 5/8
2-38	3	2 1/2
2-39	3	1 1/8
2-40	2	1 3/4
2-41	3	3
2-42	2	2
2-43	3	1 3/8
2-44	3	1 1/2
2-45	2	4 3/4
2-46	2	15
2-47	2	6 3/4
2-48	3	24 3/4

Notes

(1) Indication numbers correspond to numbers in Figure 21.
(2) Zone 2 indications have a through-thickness depth of 1/4 inch; zone 3 indications have 1/2 inch.
(3) Indication was unacceptable from an engineering evaluation and reworked.
(4) Indication was acceptable from an engineering evaluation but reworked because it was adjacent to a zone 1 indication.

Indication No. (1)	Zone (2)	Length (Inches)
2-49	2	1 3/8
2-50	2	1 1/2
2-51	2	40
3-1	2	5 1/4
3-2	3	13 3/4
3-3	2	10 3/8
3-4	2	2 7/8
3-5	2	3 1/2
3-6	3	33 5/8
3-7 (4)	3	10
3-8 (4)	3	12 1/2
3-9 (4)	2	1 1/4
3-10	2	18 1/2
3-11	3	4 1/4
3-12	3	28
3-13	2	18 3/4
3-14	3	2 1/8
3-15	3	14 1/2
3-16	2	4 7/8
3-17	2	3 1/8
3-18	2	2 3/4
3-19	3	1 1/2
3-20	2	2
3-21	2	2 3/4
3-22	2	2 1/2
3-23	2	2 5/8
3-24	2	3 1/4
3-25	2	1
3-26	2	1 3/8
3-27	2	3 3/4
3-28	2	1
3-29	2	2
3-30	2	3 3/8
3-31	2	1
3-32	2	21 5/8
3-33	3	3 1/8
3-34	2	3 5/8
3-35	2	5 1/4
3-36	2	10 3/4
3-37	3	1 1/8
3-38	2	3 1/2
3-39	2	3 3/8
3-40	2	1
3-41 (3)	2	5 1/2
3-42 (3)	2	10 1/8
3-43 (4)	2	1 5/8
3-44	2	8 1/8
3-45	2	1
3-46	2	1
3-47	3	1 1/2
3-48	2	1
3-49	2	3 3/4
3-50	2	4 7/8
3-51	2	15 1/2
3-52	2	1 3/4
3-53	2	1 1/2
3-54	3	5 1/4
3-55	2	32 1/2
3-56 (4)	2	20 7/8
3-57	3	8 3/8
3-58	2	1
3-59 (3)	2	1 3/4
3-60	2	5 1/4
3-61	2	1 1/4
3-62	3	1 1/2
3-63	2	1 1/2
3-64	2	1 1/2
3-65	2	4 1/4
3-66	2	37
3-67	2	10 1/2
3-68	2	10 1/2
3-69	3	3 3/4
3-70	3	4 1/2
3-71	3	2
3-72	2	6 1/2
3-73	3	4 1/4
3-74	3	6 3/4
3-75	2	2 1/4
3-76	2	3
3-77	3	3 3/4
3-78 (4)	3	23 1/4
3-79	2	1 3/4
3-80	3	15 1/2
3-81	2	3/4
3-82 (3)	2	7 1/4
3-83	2	3/4
3-84	2	2 3/4
3-85 (4)	3	7 1/2
3-86	3	18 1/2
3-87 (4)	2	3 1/4
3-88 (4)	3	19
3-90	3	21 3/4
3-91	2	4 1/4
3-92	2	1 1/2
3-93	2	7/8
3-94	2	3/4
3-95	2	7/8
3-96	2	4 1/2
3-97	2	1 3/4
3-98	2	1
3-99	2	1 1/2
3-100	2	3
3-101	2	15/16
3-102	2	1 3/4
3-103	2	2 3/4
3-104	2	11
3-105	2	7/8
3-106	2	3/4
3-107	3	1
3-108	2	1
3-109	2	3
3-110	3	2 3/4
3-111	2	1
3-112	3	5/8
3-113	3	1
3-114	2	7/8
3-115	2	1 1/4
3-116	2	1
3-117	2	5/8
3-118	3	9
3-119	2	57 3/4
3-120	2	2 1/2
3-121	3	22

Notes

(1) Indication numbers correspond to numbers in Figure 21.
(2) Zone 2 indications have a through-thickness depth of 1/4 inch; zone 3 indications have 1/2 inch.
(3) Indication was unacceptable from an engineering evaluation and reworked.
(4) Indication was acceptable from an engineering evaluation but reworked because it was adjacent to a zone 1 indication.

5. Example Calculation

Surface Defect in the Cover Plate

This example illustrates an actual UT indication in the cover plate located in Zone 2 of the weldment (Figure 22). The assumed defect depth in Zone 2 is 1/4 inch (see Section IV.D.2). The tip of the defect does not reach the surface, but since the distance between the tip and the surface is less than half of the assumed defect depth, the defect is categorized as a surface defect (see ASME XI). Therefore, the total depth is 0.325 inch.

This weldment is exposed to combined membrane and bending stresses due to a postulated accident when the outside wall is at a higher temperature than the inner wall. In such a case Equation 8 (Section IV.B.2) gives the stress intensity factor. The parameters in this equation are determined as follows:

$$a = \Delta a + a_N$$

$$\Delta a = 0.325 \text{ inch}; \; a_N = 0.181 \text{ inch (in this case)}$$

$$\therefore a = 0.325 + 0.181 = 0.506 \text{ inch}$$

Applied membrane stress, S_m appl., is − 2.7 ksi.

The residual stress, S_{res}, is assumed to be 5 ksi; therefore, the total membrane stress is:

$$S_m = S_m \text{ appl} + S_{res} = -2.7 + 5 = 2.3 \text{ ksi}$$

Bending stress, S_b, is 33.3 ksi, so the total maximum stress is:

$$S = S_m + S_b = 2.3 + 33.3 = 35.6 \text{ ksi.}$$

The length of the indication according to the UT report is 4.25 inches, so $a/\ell = 0.119$ and

$$F_S = 1 + 0.12\,(1-a/\ell)^2 = 1.093$$

$F_W = 1.04$ (Section IV.B.2, Equation 1a)

$$M_b = 0.84$$

$F_G = 1.04$ (in this case)

$F(\frac{a}{a_N}) = 0.885$ (Section IV.B.2, Equation 1b)

$$F_E = \frac{1}{\sqrt{1 + 4.593(a/\ell)^{1.65} - 0.212(S/Sys)^2}}$$

EXAMPLE OF COVER PLATE TO STIFFENER UT INDICATION

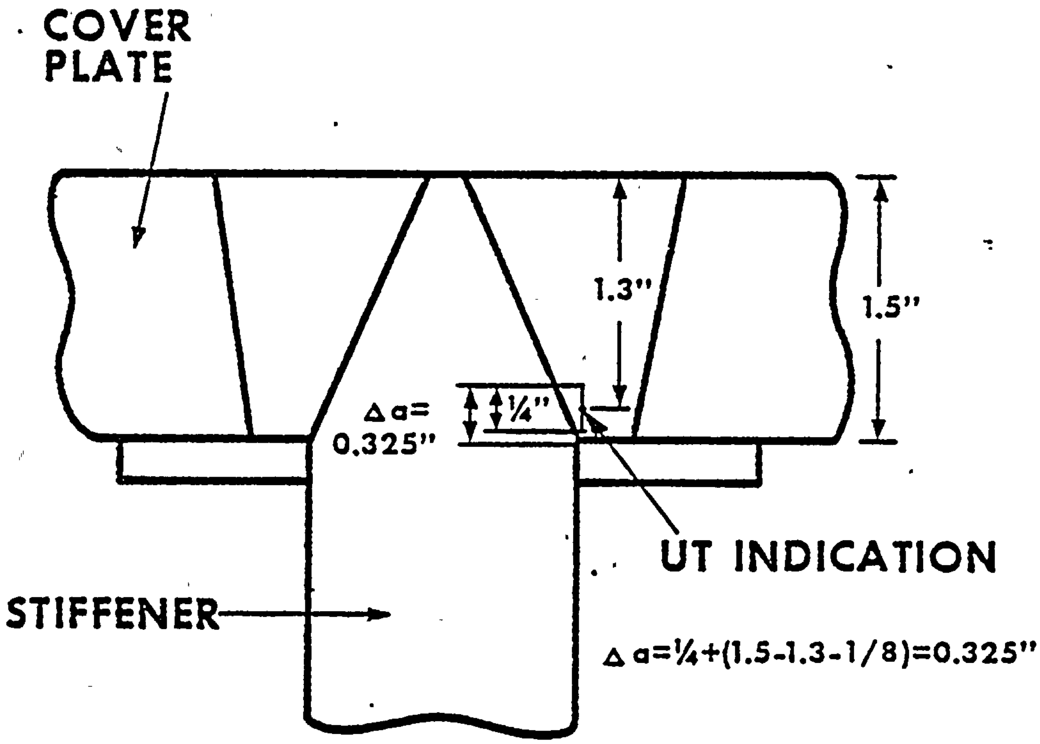


FIGURE 22

$$F_E = \frac{1}{\sqrt{1 + 4.593(0.119)^{1.65} - 0.212(\frac{35.6}{50})^2}} = 0.985$$

$$K_I = (F_S F_W S_m + M_b S_b) F_G F(\frac{a}{a_N})\ F_E\ \sqrt{\pi a}$$

$$= (1.093 \times 1.04 \times 2.3 + 0.84 \times 33.3) \times 1.04 \times 0.885 \times 0.985 \times \sqrt{0.506\pi}$$

$$= 34.98 \text{ ksi } \sqrt{\text{in}}$$

It is conservatively assumed that the critical fracture toughness is the same as in the through-thickness direction of the inner wall plate, that is, K_{Id} = 48.8 ksi $\sqrt{\text{in}}$.

Therefore,

K_I = 34.98 is less than K_{Id} = 48.8

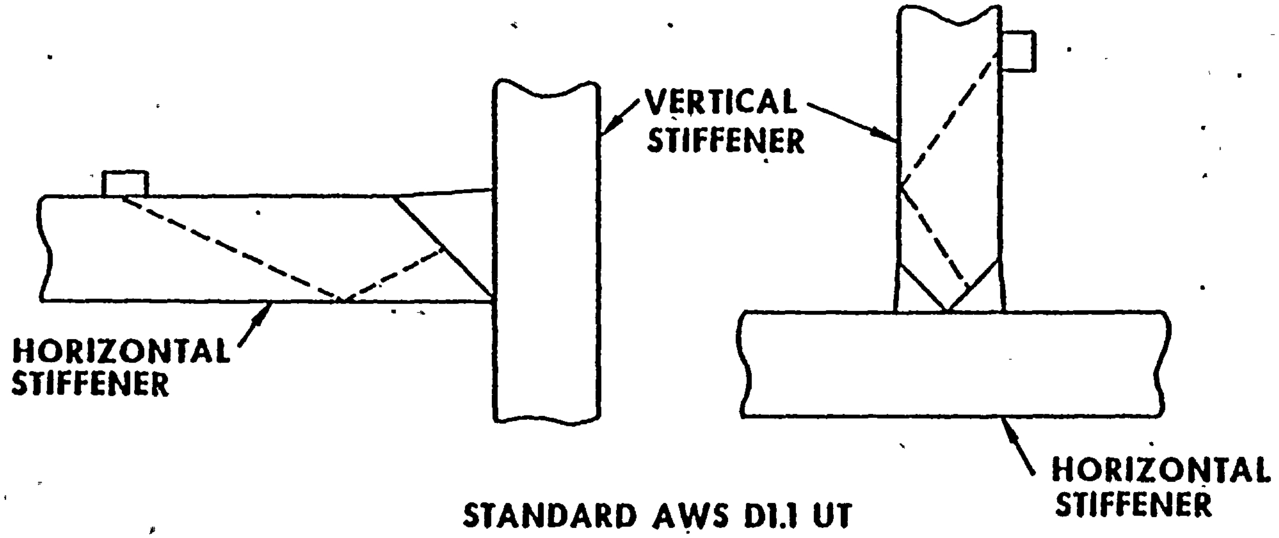
E. Stiffener to Stiffener Evaluation

1. Approach

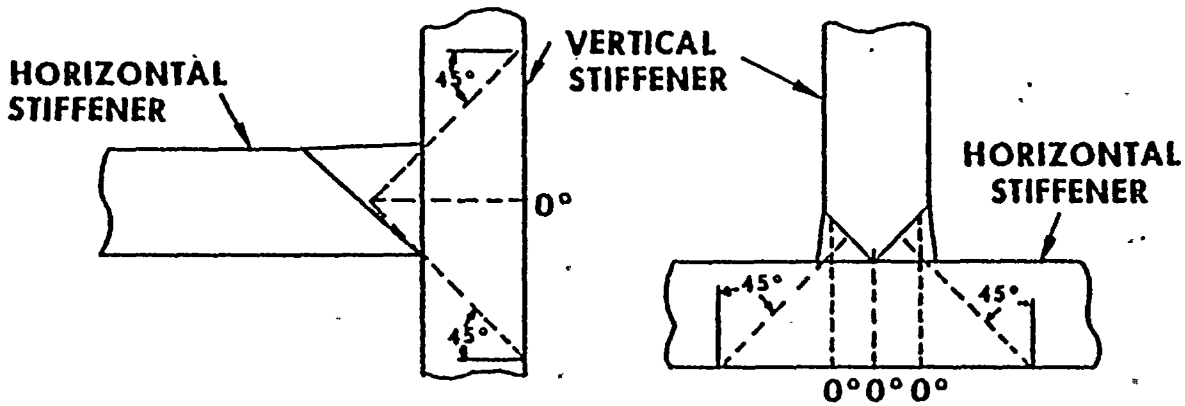
Investigation has demonstrated that the overall weld quality of the stiffener to stiffener welds is very good. While all of the stiffener to stiffener welds previously received a 100 percent UT examination in the shop, all of the accessible welds were reexamined by UT in the field. (Over 40 percent of the stiffener to stiffener welds were accessible for a UT examination.) Only minor indications were discovered during the reexamination, and the incidence of occurrence of the weld indications was extremely low. Less than 3 percent of the length of welds reinspected were rejectable to the AWS D1.1 standard UT (Figure 23), which is within the range of repeatability expected from a UT reexamination. No defects were discovered as a result of the special UT examination (Figure 23).

Structurally, the stiffener to stiffener welds are less critical than the cover plate and inner wall to stiffener welds. The load transfer mechanism is primarily one of shear along the stiffener to stiffener weld. It has been determined that a major fracture mechanics problem does not exist in this particular configuration. Even with extremely conservative assumptions, such as neglecting the inherent structural stability provided by the restrained, cellular-type configuration of the stiffener to stiffener welds, a crack of approximately 3/4 inch through-thickness depth can be tolerated for the entire length of the weld. UT data has shown that no weld defect even approaching 3/4 inch can be expected.

TYPES OF UT FOR STIFFENER TO STIFFENER WELDS



STANDARD AWS D1.1 UT



SPECIAL UT
(FOR WELDS INACCESSIBLE TO STANDARD AWS D1.1 UT)

FIGURE 23

In order to absolutely demonstrate the acceptability of the BSW, the structural integrity of the inaccessible stiffener to stiffener welds was addressed in the following manner:

> Shear stresses due to temperature and pipe rupture loads were used to determine the required weld area. (The stresses due to deadload, seismic, and pressure loads were negligible.) Considering the unlikely occurrence of a large weld defect, the weld area available was determined based on the largest defect found by UT for this configuration, a 1/4 inch through-thickness indication discovered in the inner wall to stiffener examination. By conservatively assuming that the defect extends for the entire length of the stiffener, the weld area available is approximately 84 percent of the original weld area; 80 percent was used for the analysis. For each inaccessible stiffener, it was shown that there was adequate weld area available to react to the postulated accident loads.

2. Map of Defect Locations

The locations of the indications discovered by UT are shown in Figure 24. (The defects are shown on a developed view of the BSW from the perspective of being outside the BSW, looking toward the inside.) Additional stiffener to stiffener indication data are summarized in Table 3.

3. Summary of Evaluation Results and Repairs Required

All of the indications originally discovered by the standard AWS D1.1 UT were reworked. The MT data which was collected during the rework was invalid because of the crack aggravation due to grinding as described in Section III.B. Therefore, the maximum UT indication size of 1/4 inch from the stiffener to stiffener inner wall data (Table 1) was conservatively used in the analysis. All of the inaccessible stiffener to stiffener welds were determined to be acceptable based on engineering evaluations similar to that presented in Section IV.E.4.

4. Example Calculation

For weld joint 278/12B (read as "weld joint at elevation 278'-5 3/4", azimuth 12° 29', below the horizontal stiffener"), the minimum required weld areas are determined for the following two loadings:

a. <u>Shear Stress due to Pipe Rupture</u> (Figure 25)

Consider the restraint with applied moment M of 24,000 in-k and pullout load P of 744 kips. The equivalent force couple F at the extreme flanges is

$$F = \frac{M}{s} = 298 \text{ k}$$

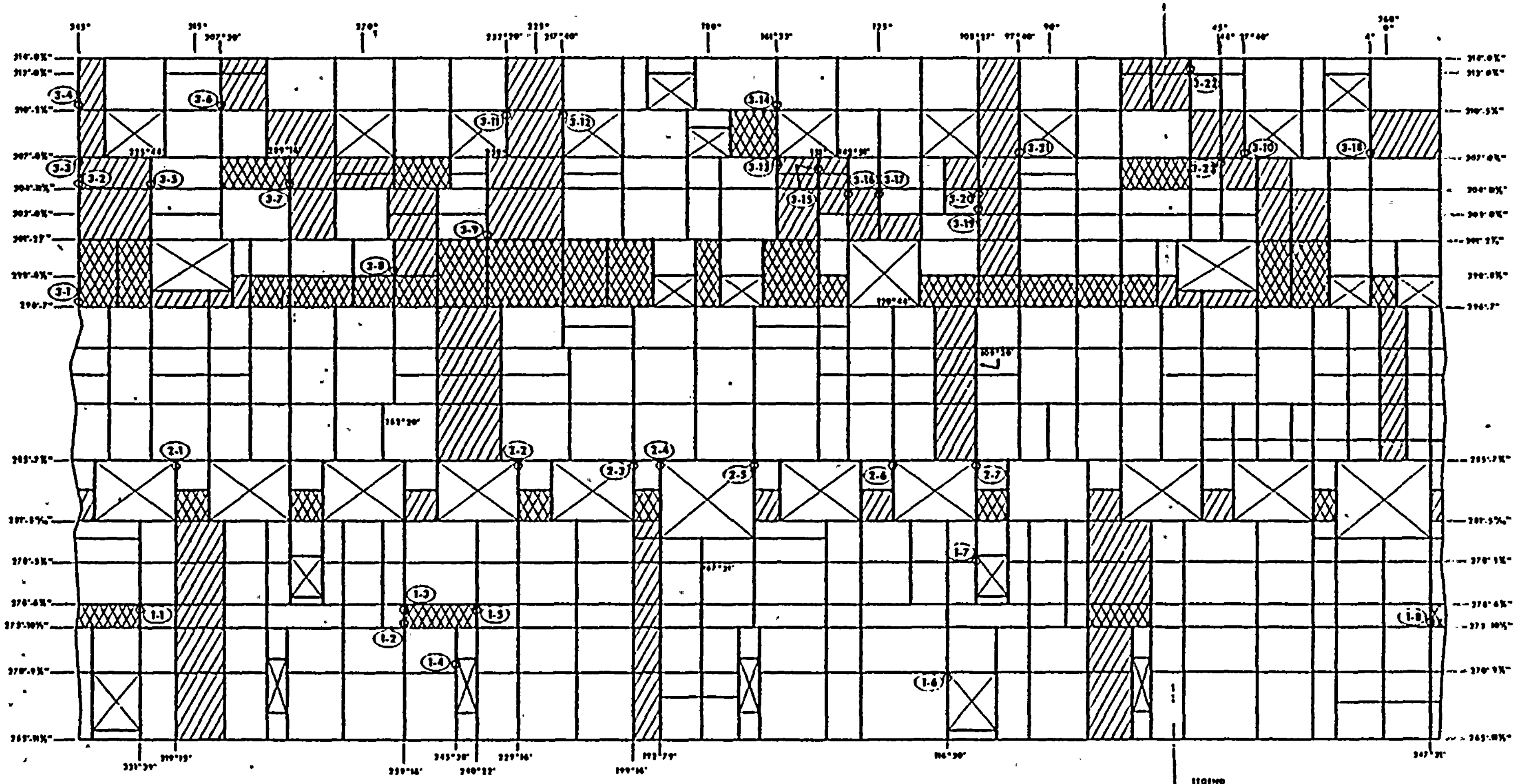


FIGURE 24 - MAP OF STIFFENER TO STIFFENER UT INDICATIONS

TABLE 3 - STIFFENER TO STIFFENER UT INDICATION DATA

Indication No.(1)	Length(2) (Inches)	Through-Thick.(2) Depth (Inches)
1-1	3 3/8	0-1/8
1-2 (3)	-	-
1-3 (3)	-	-
1-4	10 (int)	1/8-1/4
1-5 (3)	-	-
1-6 (3)	-	-
1-7 (3)	-	-
1-8	9 7/8 (int)	0-1/8
2-1	1 1/2	1/8-1/4
2-2	4 1/4	1/4-1/2
2-3	4 1/4	0-1/8
2-4	16 1/4	1/4-1/2
2-5	11 1/4	1/4-1/2
2-6	7	1/2-3/4
2-7	5 1/4	1/4-1/2
3-1	9 3/8	1/2-3/4
3-2	3	1/4-1/2
3-3	3 1/4, 3 3/4	1/2-3/4
3-4	4	0-1/8
3-5	(3)	0
3-6	10 3/4	1/4-1/2
3-7	7 3/4	0-1/8
3-8	4	0-1/8
3-9	3, 2 1/4	0
3-10	3	0-1/8
3-11	3	1/8-1/4
3-12	3	0-1/8
3-13	3	1/4-1/2
3-14	4 1/2	0
3-15	4 7/8	1/2-3/4
3-16	6	0
3-17	5 1/2	0
3-18	2 1/2	1/4-1/2
3-19	6 1/2	0
3-20	7 3/4	1/8-1/4
3-21	8	1/8-1/4
3-22 (3)	-	-
3-23	4 1/2, 3 3/4	0-1/8

Notes

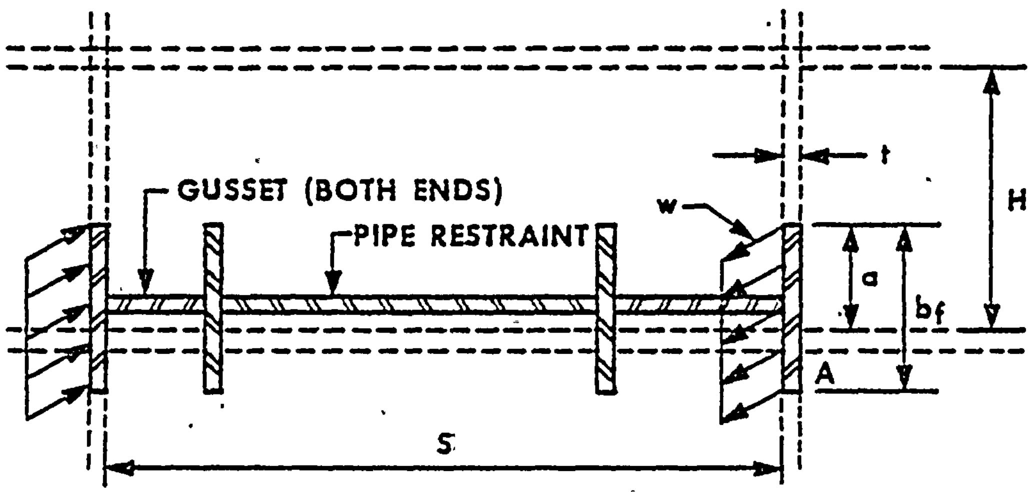
1. Indication numbers correspond to numbers in Figure 24.

2. The length and through-thickness depth were determined by the progressive magnetic particle investigation of the UT indications.

3. Indication was reworked without data having been recorded.

4. (int) - Indicates that the UT indication is intermittent along the weld length.

PIPE RESTRAINT WITH GUSSET



s = SPACING BETWEEN RESTRAINT FLANGES
H = CLEAR DISTANCE BETWEEN HORIZONTAL STIFFENERS
b_f = FLANGE LENGTH
a = LENGTH OF UNIFORMLY DISTRIBUTED LOAD
w = UNIFORMLY DISTRIBUTED LOAD
t = THICKNESS OF STIFFENER

FIGURE 25

The stress σ in the flanges is

$$\sigma = \frac{F + P}{t\, b_f} = 18.6 \text{ ksi}$$

From simple beam theory, the reaction at point A, R_A, is

$$R_A = \frac{wa}{2H} (2H - a) = 205 \text{ k}$$

The required net weld area, A_{net}, is

$$A_{net} = \frac{R_A}{\sigma_s} = 5.7 \text{ in}^2$$

where σ_S = factored allowable shear stress

The percent weld area required for R_A is

$$\frac{A_{net}}{A_{weld}} \times 100\% = 22\%$$

b. <u>Shear Stress due to Accident Temperature</u> (Figure 26)

The free increase in length of cover plate AB (Δ_{AB}) is equated to the sum of reduction in length of cover plate AB (Δ_{AB}^{w}) due to developed force plus the vertical displacement (Δ_{AB}^{st}) of the stiffener.

$$\Delta_{AB} = \Delta_{AB}^{w} + \Delta_{AB}^{st}$$

where $\Delta_{AB} = \alpha\,(\Delta T)\, H$

$$\Delta_{AB}^{w} = \frac{\tau H t}{A_w E} \,(H/_2)$$

$$\Delta_{AB}^{st} = \tau H t \left[\frac{d^3}{3EI_{st}} + \frac{d\,(1.2)}{A_{st}\, G} \right]$$

This equation reduces to

$$\tau = \left\{ \frac{\alpha(\Delta T)\, E}{t} \right\} \times \left\{ \frac{1}{\frac{H}{2\, A_w} + \frac{d^3}{3\, I_{st}} + \frac{3.12d}{A_{st}}} \right\}$$

where τ = shear stress
α = coefficient of thermal expansion
E = modulus of elasticity
d = distance from inner wall to cover plate
A_w = area of wall section
A_{st} = area of stiffener
I_{st} = moment of inertia of stiffener
ΔT = increase in temperature of outer wall

SHEAR STRESS DUE TO TEMPERATURE DIFFERENTIAL

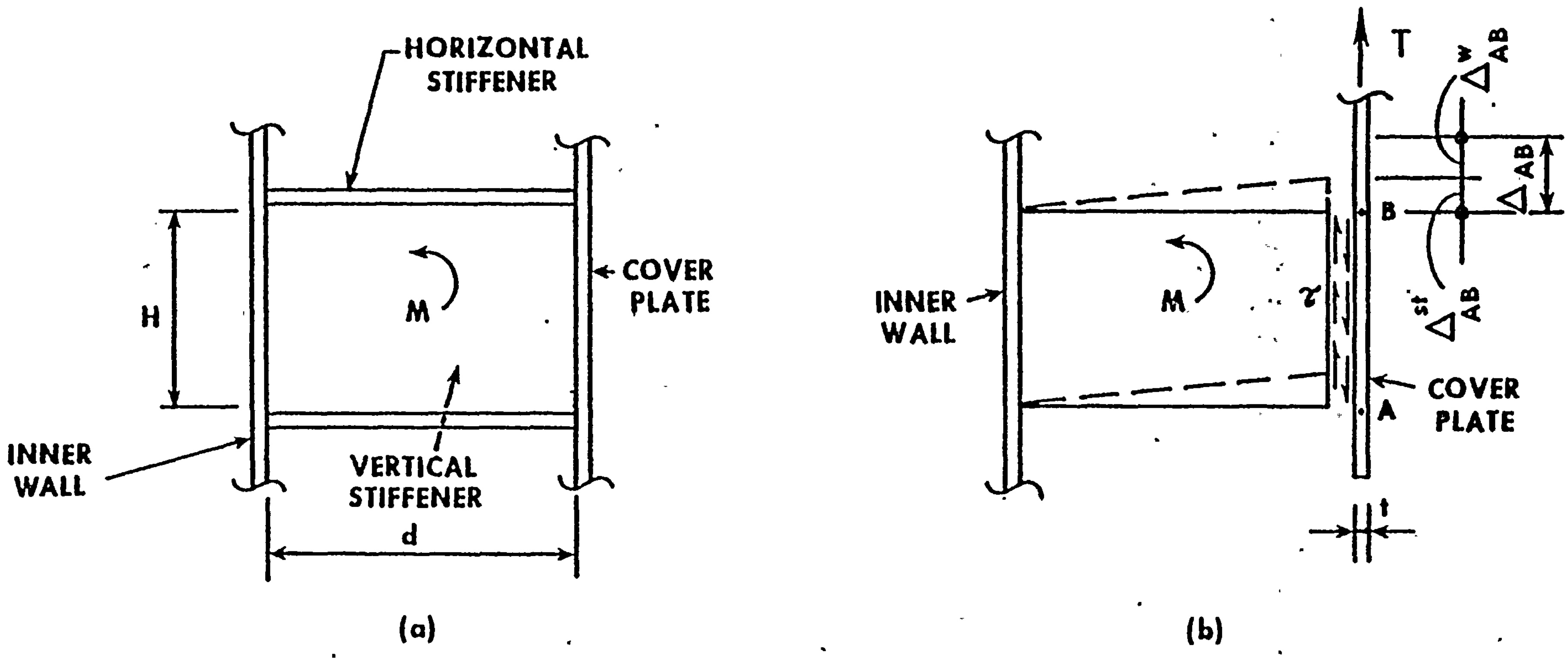


FIGURE 26

In this case, $\tau = 9.2$ ksi

The total force, F, developed in the weld is

$$F = \frac{M}{H} = \frac{\tau A_{st}\, d}{H} = 241.5\ k$$

The total force, F, is reduced by the allowable pressure force, F_{allow}, to give the net force,

$$F_{net} = F - F_{allow} = 61.5\ k$$

where F_{allow} = allowable force due to stiffener bearing on the inner or outer wall

The required net weld area is

$$A_{net} = \frac{F_{net}}{\sigma_s} = 1.9\ in^2$$

The percent weld area required is

$$\frac{A_{net}}{A_{weld}} \times 100\% = 7.5\%$$

Therefore, the total percent weld required is 22 + 7.5 = 29.5 percent. Based on a net weld area of 80 percent (total weld area less 20 percent for indication area), the factor of safety FS against shear for the weld is

$$FS = \frac{\text{net area}}{\text{required weld area}} = 2.7$$

F. Conservatisms in Engineering Evaluation

Numerous conservative assumptions which were made in the stress and fracture mechanics analyses are listed as follows:

<u>Stress Analysis</u>

1. In the thermal stress analysis, it was assumed that while the inner and outer walls are being heated during an accident condition to produce the maximum temperature differential, the stiffeners remain at the same (operating) temperature; hence, higher thermal stresses result.

2. In the heat transfer analysis to produce the accident temperature differential across the shield wall, the vertical stiffeners were ignored; hence, a greater temperature differential occurred, and thus the stresses were overestimated.

3. In the LOCA annulus pressurization analysis, no credit was taken for effects of heat sinks; only flow diverter door venting out of the shield wall penetrations was assumed; the blowdown calculation included the effects of inventory and subcooling; and the break opening time was assumed instantaneous. Hence, a higher annulus pressure resulted.

4. In the pipe rupture analyses, if a dynamic analysis was not performed, a dynamic load factor of 2 and a static impact factor of 1.3 were used; minimum material yield property factors of 1.3 for bolts and straps and 1.1 for aluminum honeycombs were used. Hence, higher stresses due to pipe rupture loads resulted.

5. In the stress analysis, the peak values of various loads, such as accident temperature, accident pressure, and pipe rupture, were combined in accordance with PSAR load combination equations resulting in conservatively high stresses. In the event of an accident, however, the peak load values occur at different times.

Fracture Mechanics Analysis

1. The ultrasonic responses from all weld discontinuities were far less than the responses from known reflectors in calibration blocks.

2. If there was any doubt about the location and orientation of a weld discontinuity, the worst location and orientation were assumed.

3. It was assumed that all the defects were sharp, which was usually not the case.

4. In cases where through-thickness properties were applicable, the lowest known value of the fracture toughness in through-thickness direction was used.

5. Dynamic rather than static fracture toughness was used in the analysis although actual strain rates are expected to be at least an order of magnitude lower than those corresponding to dynamic fracture toughness.

6. It was assumed that the postulated failure conditions develop at 100°F although the temperature for a postulated accident is expected to be greater than 135°F. The fracture toughness corresponding to the 100°F temperature was used in the calculations.

7. It was assumed that the applied tensile stress was perpendicular to the plane of the assumed defect.

Note that assumptions 1, 2, 3, and 7 result in high values of stress intensity factors and assumptions 4, 5, and 6 give a very conservative estimate of fracture toughness. In addition to the above conservatisms another margin of safety is introduced by limiting the size of allowable defect to a fraction of the critical defect size for every weld which has been evaluated.

V. QUALITY ASSURANCE/CORRECTIVE ACTION

A. Introduction

Through the inspection/rework program described within this report the biological shield wall (BSW) will comply with all PSAR commitments and criteria. However, in order to reduce the possibility of recurrence of the type of problems encountered with the BSW, a plan of action has been developed.

The purpose of this section is to summarize the nature and possible causes of the weld problems, to discuss the condition of the BSW while it was in the seller's shop, to explain why the problems were not discovered in the shop, to review the corrective action which was implemented, and to present a plan of action and certain steps already implemented to reduce the possibility of recurrence.

B. Summary of Weld Problems

The two types of welds with evidence of problems were the single bevel backing bar welds, which included horizontal stiffener to inner wall (HSIW) and cover plate to base plate (CPBP) welds, and the cover plate to stiffener welds.

<u>Horizontal Stiffener to Inner Wall (HSIW) and Cover Plate to Base Plate (CPBP) Welds</u>

These welds exhibited cracking which propagated from the weld root. As discussed in Section IIIC, it was concluded that the most probable cause of cracking initiation was hydrogen. The hydrogen possibly originated from moisture which was not driven off by preheat. It was also concluded that the source of the hydrogen was not from the weld filler material or the handling of the weld filler material.

The cracks appeared to have propagated as the result of joint restraint. In addition, it is now believed that during investigation of the weld indications, grinding caused the cracks to propagate, resulting in misinterpretation which overestimated the original crack size.

Other weld indications encountered in these welds included general workmanship-related indications, such as slag inclusions and lack of fusion. The presence of this type of indication was shown through engineering evaluation to be acceptable without compromising the weld joint integrity.

Cover Plate to Stiffener Welds (Backing Bar)

The problems encountered in the cover plate to stiffener welds were in both the weld metal and base metal. The cracking and workmanship-related indications which were present, were caused by the same reasons as those found in the BSIW and CPBP welds. However, as discussed in Section IIIC, the stiffeners had plate laminations which were acceptable to a plate UT acceptance criteria (ASTM A578, Level I) but rejectable to a weld UT acceptance criteria. Such laminations are inherent in plate material and are the result of plate manufacturing processes. Some stiffeners also exhibited lamellar tearing which was the result of a combination of the weld joint configuration, weld shrinkage stresses in the plate through-thickness direction, and the plate laminations.

C. Why the Problems Were Not Discovered in the Shop

Although defects such as slag or lack of fusion occurred in the shop, it is possible that some indications did not exist in the shop but occurred at some later time due to hydrogen inducement, erection/handling stresses, or field welding. Cives inspection system did identify indications which were repaired in the course of manufacture; however, it is possible that some indications were missed. It should be noted that some indications may have been noticed by Cives but passed in the belief that they were acceptable under the Code. Investigation of this matter is continuing, and any substantive new information will be submitted to the Commission.

D. Corrective Action for the BSW

In order to resolve the shield wall problems, an extensive program of ultrasonic examination and engineering evaluation, described in Section IV, was conducted. The BSW was analyzed or reworked to assure that all safety requirements are met.

E. Actions to Reduce the Possibility of Recurrence

Program Enhancements Previously Implemented

There have been certain long-range S&W Standard QA program modifications that occurred within the program to meet continuing new industry demands, regulations, codes and standards, and modifications suggested by S&W program audits, as well as ASME, licensees, and regulatory audits.

The following describes certain significant long-range modifications put into effect by S&W since August, 1978.

1. S&W PQA Inspector training, qualification, and certification in NDE disciplines were intensified in 1978. The resultant program modifications introduce more extensive training to cover all NDT disciplines within the division (and PQA district offices) and most specifically certification in accordance with ASNT-TC-1A guidelines. While PQA Inspectors are not normally performing hands-on inspection, this enchanced training should assist in detecting supplier NDT problems.

   Implemented - August 1978

2. S&W PQA has now implemented a formal PQA inspection planning activity. This function provides for PQA engineering review of project specifications, codes and standards, and past vendor performance with an output consisting of an inspection plan. This plan allows the inspector to concentrate his efforts on surveillance activities and documentation.

   Implemented - April 1979 (QAD-7.14, Revision A)

3. S&W inspection reporting system, which allows centralized data analysis and input into the Quality Information Center, has been modified to provide a variety of inspection report forms.

   These newly developed forms allow for simplified computer entry and general ease of completion since they are geared to specific inspection actions such as hardware inspection, process surveillance, and system surveillance. S&W's PQA effort continues to emphasize seller surveillance and conformance evaluation toward effective implementation by the vendor of his contractual quality control responsibilities. These new report formats are designed to facilitate this approach.

   Implemented - February 1980 (QS-14.2, Revision 0)

4. The S&W QA procedural system has been restructured to provide flexibility in accommodating unique project requirements without sacrificing the inherent advantages of a standard program.

   Implemented - June 1980 (Project Procedure No. 64)

Additional Program Enchancements Not Yet Fully Implemented

Overall, the Nine Mile Point - Unit 2 (NMP2) Quality Assurance (QA) program has been effective in identifying deficiencies. Nevertheless, the following enhancements are being added to this program so as to reduce the possibility of recurrence of material and fabrication-related deficiencies:

Improved communication and systematic evaluation of current QA problem events

Additional engineering involvement

Additional direction to PQA inspection and project engineering personnel

More extensive process surveillance in Seller's facility

Procedure and Program improvements

These additions are detailed below:

1. Improved Communcations and Systematic Evaluation of Current QA Problem Events

   The S&W Project QA Manager will establish a program to review, on a month-to-month basis, all Category I and other selected Nonconformance and Dispositions (N&D), Engineering and Design Coordination Reports (E&DCR), Problem Reports, Inspection Reports (IR), and audit findings (NRC, Client and S&W) for identification of potential generic problem areas as well as short and long term trends that affect quality. This information will be processed into a monthly summary report that contains a trend analysis and a listing of N&Ds and E&DCRs to be reviewed at a meeting chaired by the S&W Project QA Manager.

   This meeting will take place monthly unless there are no substantial problem areas to review. Mandatory attendance for the Potential Problem Review Meeting (PPRM) will include the S&W Project Engineer, PQA Manager, Superintendent FQC, or their designees. NMPC QA and Project personnel will also be in attendance. This committee wil review the listed N&Ds and E&DCRs for generic type problems and provide immediate evaluation for corrective action in the form of, but not limited to, specification change and/or additional inspection requirements for the field site or the specific Seller's shop. The results of these meetings will be documented.

   Initial Implementation - August 1980
   Full Implementation - October 1980

2. Additional Engineering Involvement

The S&W PQA Division has reviewed its operations to determine where system improvements might avoid seller problems. As a result of this review, the following will be applied to all new major Category I orders and selected orders already awarded.

Seller Selection Process

Expand the present S&W QA survey program to include verification of the seller's manufacturing capability. The survey team will consist of a qualified quality assurance auditor, the responsible Engineer on the project, and the engineering specialist as required. The team will verify the presence of adequate engineering, design, manufacturing, and quality assurance capabilities.

Post-Award Activities

a. Include S&W Engineers on project and/or S&W equipment specialists as members of the PQA Post-Award Conference.

b. Include S&W Engineers on project and S&W engineering assurance personnel as required in audits for recertification.

Initial Implementation - August 1980
Full Implementation - December 1980

3. Additional Direction to PQA Inspection and Project Engineering Personnel

S&W PQA and project engineering personnel will review existing and future Category I specifications for problems inherent in heavy section weldments, including the QA/QC requirements related to:

Welding joint design
Welding control and inspection
Fabricator NDE procedure review and approval
Fabricator conference to manufacturing plans
Vendor personnel qualifications
Exercise of stop work procedures under appropriate circumstances

Initial Implementation - September 1980
Full Implementation - November 1980

4. More Extensive Process Surveillance in Seller's Facility

A review of existing and future Category I specifications will be performed and a determination made of those orders that will require increased S&W PQA surveillance. In order to implement this increased surveillance, S&W will prepare a plan tailored to each pertinent order and submit it to NMPC for concurrence.

Initial Implementation - August 1980
Initial Specification Review and Revised Inspection Plans will be implemented by November 1980

5. Other Procedure and Program Improvements

a. For future and selected existing Category I orders where ultrasonic testing (UT) is utilized, the specification will establish hold points at which, prior to performance of any UT, the S&W PQA inspector, aided by Nondestructive Testing (NDT) Division engineers, will evaluate the effectiveness of the seller's UT techniques, operators, and general implementation of the test parameters. All UT procedure qualifications (techniques) will be approved by the S&W Nondestructive Testing (NDT) Division engineers at the preestablished hold point as required by specification for each different configuration.

b. For future and selected existing Category I specifications S&W QA will establish notification points for application of all NDE methods other than UT. The NDT Division Engineer will verify NDE applications on a random basis after coordinating the activity with the S&W PQA inspector. Procedure qualifications (techniques) will be reviewed on a selected basis dependent on observations at the preestablished notification point.

c. Require S&W PQA attendance at any pre-award meeting between S&W Engineers on project and seller.

d. Require sellers to notify S&W Project Management of any changes in their quality assurance management.

Initial Implementation - August 1980
Initial Review of Existing Specifications will be Implemented by November 1980

6. Increased Surveillance and Audit

Additionally, the NMPC Quality Assurance Department will increase its surveillance and audit program to verify implementation of the foregoing program enhancements.

Initial Implementation - September 1980

VI. ANALYSIS OF SAFETY IMPLICATIONS

Based on an extensive engineering evaluation, it has been determined that the BSW weld defects could not have adversely affected the safety of operations of the Nine Mile Point 2 plant had the weld defects remained undiscovered. In the context of the safety evaluation and 10CFR50.55(e), the term "undiscovered" rather than "uncorrected" is used since the evaluation has shown that some weld defects need not be corrected. The basis for this conclusion is discussed in detail below for each weld configuration.

As discussed throughout this report, the primary weld problems involved the following backing bar welds: (1) horizontal stiffener to inner wall and cover plate to base plate and (2) cover plate to stiffener. Although minor weld indications were discovered in other weld joint configurations, the incidence of occurrence of the indications is within the range expected for repeatability of a reexamination by UT. These insignificant weld defects have a negligible effect on the structural integrity of the BSW and would not have been a safety hazard if the defects had gone undiscovered.

Horizontal Stiffener to Inner Wall and Cover Plate to Base Plate Welds

As discussed in Section IV.C of this report, the horizontal stiffener to inner wall welds were evaluated based on UT examination results. From Table 2, all weld indications are acceptable based on stress and fracture mechanics analyses. Therefore, if the weld indications had gone undiscovered, a safety hazard would not have existed.

The cover plate to base plate weld, which was the initial problem discovered, was repaired based on 100 percent UT. Evaluation has shown that, even if the defects had been undiscovered, a gross structural failure could not have resulted and, therefore, a safety hazard would not have existed.

Cover Plate to Stiffener Weld

As discussed in Section IV.D of this report, AWS D1.1 weld defects were grouped according to location of the defect in the weld joint. (Refer to the discussion of Section IV.D.2 and Figure 18). Those weld indications located in Zone 1 of the weld joint were repaired, and those indications located in Zones 2 and 3 were evaluated.

From Table 3 it may be seen that all but 17 indications in Zones 2 and 3 of the cover plate are acceptable based on engineering evaluation. By a realistic consideration of the margins which exist in the analysis, it has been determined that these 17 indications could not have propagated and would not have been a safety hazard.

Addressing indications found and repaired in Zone 1 of the weld, it has been determined that these indications would not have been a safety hazard had the indications remained undiscovered. Evaluation of the worst possible defect which could have occurred in Zone 1, taking account for realistic margins which exist in the analytical procedures, shows that the gross structural integrity of the BSW would be maintained and a safety hazard would, therefore, not exist.

VII. CONCLUSIONS

The exhaustive investigation and corrective action outlined in this report demonstrates that the BSW will provide radiation shielding and maintain structural integrity for all conditions described in the PSAR. All shop weld joints were evaluated in accordance with both AWS D1.1 and the original PSAR commitments and either were shown to be acceptable or were repaired. It has been determined that the BSW weld defects could not have adversely affected the safety of operations of the Nine Mile Point 2 plant had the weld defects remained uncorrected.

The QA program has been modified to reduce the possibility of recurrence of future weld-related problems such as those discovered in the BSW.

APPENDIX A

REVISIONS TO INTERIM REPORT

ITEM NO.	LOCATION IN INTERIM REPORT	INTERIM REPORT VERSION	CORRECTED VERSION
1	p. 1, sect. I.C., 2nd paragraph	Stiffener to stiffener evaluation using inner wall UT data	Stiffener to stiffener evaluation using accessible stiffener to stiffener weld UT as well as inner wall UT data
2	Figure 2	"inner wall plate" and "outer wall plate"	"inner wall" and "cover plate"
3	Figure 2	2 inch grout thickness between sole plate and top of pedestal	3 inch grout thickness
4	Figure 3	"inner wall plate" and "outer wall cover plate"	"inner wall" and "cover plate"
5	Figure 5	washer plate to cover plate and washer plate to base plate weld details are shop welds	washer plates were removed and reattached using weld details shown in final report Figure 5
6	Figure 6 title	"Vertical Stiffener to Inside and Outside Wall Plates"	"Vertical Stiffener to Inner Wall and Cover Plates"
7	Figure 6	"inner wall plate" and "loose outside wall cover plate"	"inner wall" and "cover plate"
8	Figure 8 title	"Horizontal Stiffener to Inside and Outside Wall Plates"	"Horizontal Stiffener to Inner Wall and Cover Plates"
9	Figure 8	"inside wall plate" and "loose outside wall cover plate"	"inner wall" and "cover plate"
10	p. 5, sect. IV.A, paragraph 2	Same as Item No. 1	Same as Item No. 1

ITEM NO.	LOCATION IN INTERIM REPORT	INTERIM REPORT VERSION	CORRECTED VERSION
11	p. 6, sect. IV.A, 1st paragraph	1. defect sizes larger than 1/8 inch will be mapped	1. all defects, regardless of size, which underwent engineering evaluation were mapped
		2. the 1/8 inch criteria deviates from AWS D1.1	2. defects are allowed in welds if the provisions of AWS D1.1, paragraph 3.7.6 are met
12	p. 6, sect. IV.A, 2nd paragraph	Same as Item No. 1	Same as Item No. 1
13	p. 6, sect. IV.A, 3rd paragraph	Same as Item No. 11, No. 1	Same as Item No. 11, No. 1

APPENDIX B
BSW HWIS WELD JOINT MT DATA[4]

Ring	Total Length Examined	Total Defect Length	1/8-1/4[1]	1/4-3/8	3/8-1/2	1/2-3/4	3/4-1
1	204	5	5	0	0	0	0
2	283	2	2	0	0	0	0
3[2]	2172	475.1	273.1	60	28	104	10
3[3]	950.3	460.4	391.6	31.5	35.3	2	0

1. 1/8 - 1/4 is read "1/8 inch $<$ defect $\leqq$ 1/4 inch"
2. This data does not include the data shown on the next line.
3. MT data in the 30 compartments made accessible by cover plate removal.
4. All units are in inches.

APPENDIX C

History of Events

The discovery of weld defects in the biological shield wall and subsequent action taken can be described in three separate phases: discovery of a potential problem, engineering investigation, and sample plan approach.

A. Phase I - Discovery of a Potential Problem (May 1979)

Based on UT indications in the cover plate to base plate welds and visual indications discovered in the third ring horizontal stiffener to inner wall welds, the quality of backing bar welds for the entire biological shield wall was investigated.

B. Phase II - Engineering Investigation (June 1979 to August 1979)

The purpose of the investigation was to determine if a weld quality problem existed for the biological shield wall backing bar welds. The sequence of events for the second phase is as follows:

June	1.	A specimen was removed from the base plate to perform a metallurgical examination of cover plate to base plate indications.
June - July	2.	UT and MT inspections of the horizontal stiffener to inner wall welds for all three rings were performed. The inspections were performed on random accessible welds.
July		Based on unacceptable defects discovered by MT in the third ring horizontal stiffener to inner wall welds, additional MT inspction was performed on all accessible third ring horizontal stiffener to inner wall weld joints (approximately 2,000 inches out of a total of 6,000 inches).
July	3.	The cover plate to base plate weld joints were dispositioned to require 100 percent UT inspection and repaired as required. The UT was performed in accordance with AWS D1.1 with a 1/8 inch exclusion allowed at the root of the weld (based on engineering evaluation). This UT was completed in September 1979.
		By evaluation of data obtained to that time, it was concluded that the horizontal stiffener to inner wall welds on the first and second rings were acceptable but the third ring horizontal stiffener to inner wall welds were rejectable and would require complete reinspection and rework, as required.

C. <u>Phase III - Sample Plan Approach</u> (August 1979 to December 1979)

Since the initial engineering investigation was primarily concerned with the root of the welds, inspections were performed using MT methods. Subsequent investigations to determine the quality of the entire volume of the welds were made using UT methods in accordance with the sample plan. The purpose of using a sample plan approach was to verify the quality of backing bar welds by using a more rigorous, systematic approach. A nationally recognized sampling approach using confidence levels consistent with levels previously employed was chosen.

The sequence of events for the third phase was as follows:

August - September	1.	An effort was made to establish confidence levels based on the data available from the engineering investigation. An additional 91 inches of MT inspection on the first ring horizontal stiffener to inner wall welds was performed to fulfill the root sample size requirements. Based on the MT sample plan, the horizontal stiffener to inner wall welds for the first and second rings were acceptable and for the third ring, rejectable.
September - October	2.	Thirty cover plates were removed from the third ring to provide accessibility for reinspection and rework of the horizontal stiffener to inner wall welds.
October - November	3.	The sample plan approach was extended to all weld configurations, including weld joints without backing bars. The weld joints were compiled into 18 weld groups based on the ring (i.e., first, second, or third), the joint configuration (i.e., single bevel weld with backing bar or double bevel weld without backing bar), and the thickness of the plates being connected (i.e. 1 1/2-inch to 1 1/2-inch plates or 1 1/2-inch to 2-inch plates).
November	4.	Two specimens were removed from the third ring horizontal stiffener to inner wall welds to perform a metallurgical examination. (Two additional specimens of the horizontal stiffener to inner wall welds were removed for metallurgical examination, one in March 1980 and one in April 1980.)

November – December	5.	UT inspection of the 18 weld groups was performed by a certified Level II inspector. UT data previously taken during the engineering investigation was also incorporated into a sample plan approach.

Based on the sample plan UT data of record, 11 of the 18 weld groups were rejected, 4 were accepted, and 3 were not applicable to the sample plan. Due to the high number of rejected weld groups, it was determined that all welds, which were accessible for inspection, would be inspected by UT.

[illegible]