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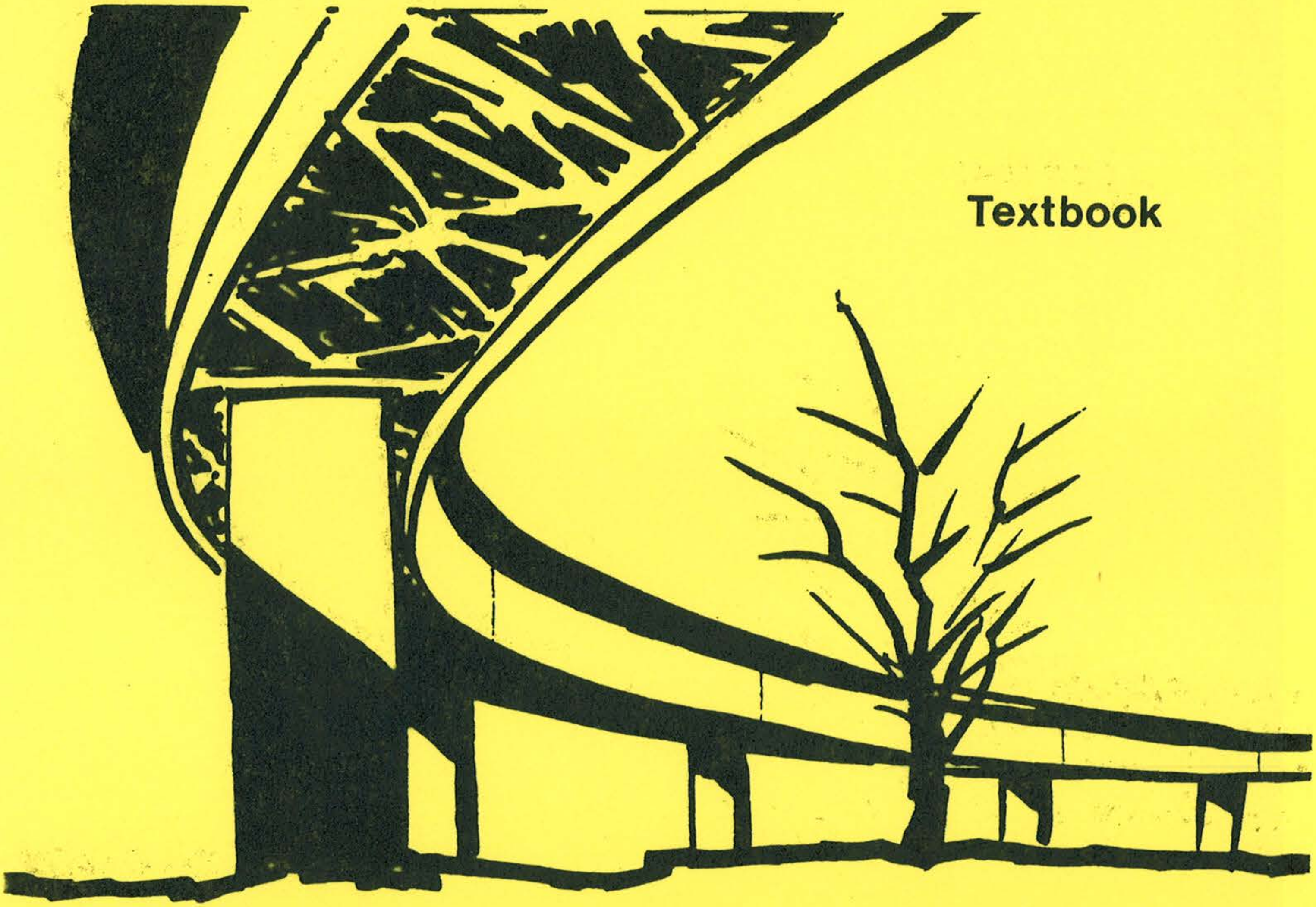
Design and Construction of Welded Bridge Members and Connections



Textbook

Offices of Research and Development
Implementation Division
Federal Highway Administration
U.S. DEPARTMENT OF TRANSPORTATION

DESIGN AND CONSTRUCTION OF WELDED BRIDGE MEMBERS AND CONNECTIONS

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DISCLAIMER

FOREWORD

There are many interrelated factors that determine the reliability and structural integrity of welded steel bridges. Foremost among these are proper materials, design, fabrication, quality control and quality assurance. Any one of these factors can contribute to fatigue and fracture of a bridge detail. Recent service failures of welded steel bridges have created a growing concern among design engineers about the possibility of catastrophic fractures in steel bridges and have led to an increasing awareness that some modifications in practices are needed.

In recognition of these problems, the Offices of Research and Development, Federal Highway Administration, issued a Request for Proposals (RFP) on July 10, 1978 calling for the preparation and presentation of a training course on "Design and Construction of Welded Bridge Members and Connections." The RFP emphasized that the course should be developed as a <u>joint venture to include the elements of both welding design and fabrication</u>. This course has been developed in accordance with these requirements.

Principal authors of the text were Roger D. Sunbury, Bridge Engineer, and Paul G. Jonas, Metallurgical Engineer. Mr. Sunbury has received national recognition for his contributions to the design and use of high strength steels in long span highway bridges. Mr. Jonas is nationally known and recognized for his outstanding contributions in the development of the arc welding process as an acceptable tool in structural engineering. Assisting Mr. Sunbury and Mr. Jonas were George S. Inenaga, Bridge Design Engineer, who is recognized for his expertise in the design of welded steel bridges; and Charles B. Kendrick, Metallurgical Engineer, who has made many contributions to inspection and to the regulation of fabrication of welded metal structures.

In total, these four engineers bring to this joint venture nearly 60 years of design experience and 50 years of construction experience, all of which occurred during a period when the art and science of bridge design and techniques of construction attained an extremely high degree of perfection. They look forward to continued progress that can be achieved partially through sharing of information by way of such vehicles as this and similar courses.

Special acknowledgment is given to Steven W. Rutter, Mechanical Engineer, for his contributions to Topic 5 of the textbook.

Dr. Russell L. Riese, who has extensive background in engineering and higher education, served as the editor, educational consultant, and evaluator for the project.

W. N. Samarzich and Associates wish to express their sincere appreciation to Mrs. Donna Stephan for her skill and dedication in typing and formating the text.

The authors wish to thank Mr. Bob Wood, Contract Manager-FHWA, Mr. Carl Hartbower, Mr. Frank Sears and Mr. John Kruegler for their constructive comments and guidance while serving as members of the Technical Review Committee during development of the course. Mr. Wood's helpful counsel has continued throughout the project.

The authors are grateful to Mr. Hartbower who provided the introductory slides that dramatically illustrate recent fatigue and fracture problems in bridges.

W. N. Samarzich and Associates expresses its gratitude to the American Association of State Highway and Transportation Officials (AASHTO) and to the American Welding Society (AWS) for permission to reproduce specific tables and figures from their publications. These tables and figures are footnoted appropriately in the text.

Finally, we wish to thank Caltrans, particularly Robert C. Cassano, Chief, Office of Structure Design, and Eric F. Nordlin, Chief, Structural Materials Branch, for their permission to reproduce some of the illustrations used in the textbook.

The major objective of this course is to address the practical design of welded members for presentation to bridge design engineers including the important design and fabrication considerations which must be taken into account in locating and detailing welded connections.

Upon completion of the course, the participants should be able to:

1. Evaluate the overall choice of design in terms of difficulty in fabrication and potential for failure from a welding standpoint.

2. State what fabrication processes are in common use today, and the limitations of such processes from a design standpoint.

3. Know the limitations of nondestructive testing methods presently being used.

4. Recognize the conditions which contribute to crack susceptibility in welded connections.

5. State the types of weld defects that commonly occur in bridge fabrication.

6. State which aspects of welded design affect fatigue and brittle-fracture.

7. Identify fracture-critical members and know the concepts of fracture control.

8. Identify problems and limitations of fabrication of selected design and weld details including workmanship problems inherent in out-of-position welding and welding in a confined space.

9. Know the alternatives to welded connections and the advantages and disadvantages in their use.

10. Know the problems inherent with field welding.

W. N. SAMARZICH & ASSOCIATES

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CLASS SCHEDULE

	Monday	Tuesday	Wednesday	Thursday	Friday
8:30		(3) 2.0 Specifications 2.1 2.2	(7) Topic 4 4.0 thru. 4.2 Design Consid.	(11) 4.4 (cont'd)	(15) Topic 6 Fracture Control
10:00		BREAK	BREAK	BREAK	BREAK
10:15		(4) 2.3 Specs-Fatigue 2.4	(8) 5.3 thru. 5.5 Welding	(12) 5.6 (cont'd)	(16) Topic 7 Summary & Evaluation
11:45		DISCUSSION	DISCUSSION	DISCUSSION	
12:00		LUNCH	LUNCH	LUNCH	
1:00	(1) Orientation Topic 1 - Problems	(5) Topic 5 5.0 thru. 5.2	(9) 4.3 Tension Mem. 4.4	(13) 4.5 Splices 4.6 & 4.7	
2:30	BREAK	BREAK	BREAK	BREAK	
2:45	(2) 1.1 Fatigue & Fracture 1.2 Course Objective	(6) Topic 3 3.0 thru. 3.2 Fatigue & Fracture	(10) 5.6 Fabrication	(14) 5.7 thru. 5.9 Quality Assurance	
4:15	DISCUSSION	DISCUSSION	DISCUSSION	DISCUSSION	
4:30					

Revised 6-9-80

TOPIC 1

THE PROBLEM

OBJECTIVES:

1. *To bring to the bridge engineer an awareness of fatigue and fracture problems that have been and are being encountered.*

2. *To inform the bridge engineer about the practical aspects of design and construction that may minimize fatigue and fracture problems.*

1.0 INTRODUCTION

A common feature on the American landscape is a bridge. Today it is estimated that there are more than one-half million steel highway and railroad bridges in the United States.

Approximately 25 years ago the steel industry introduced new alloy steels possessing qualities of high yield strength, toughness, and the ability to withstand stresses at high and low temperatures.

The bridge designer recognized the potential of this new material and soon bridge structures became more efficient and more economical through increased use of welded steel fabrication. High strength steel coupled with new fabrication techniques and new fasteners allowed the bridge engineer to design long span trusses, tied arch spans, and multispan box girders. These steel bridges have demonstrated an excellent service capability.

In spite of advances in materials, design, and fabrication, several steel bridges have experienced structural failures due to brittle fracture within the last two decades.

1.1 Bridges with Fatigue and Fracture Problems

To demonstrate the seriousness of the problems confronting bridge engineers in the design of welded steel bridges, the following examples of cracking with subsequent fracture, which in some instances lead to structural failure, are presented:

1. Interstate 79 Bridge; Neville Island, Pittsburg, Pennsylvania

   On January 28, 1977, a fracture was discovered through the bottom flange and an 11-foot deep web of a shop welded girder. The fracture was located at an electroslag shop splice in the bottom flange at the midpoint of the 350-foot center span of a three-span continuous haunched girder bridge. The fracture was of a brittle nature with little or no plastic deformation of the steel. Fracture resulted from the electroslag welds, rather than from design related details. The steel used was A588.

2. Bryte Bend Bridge; Sacramento, California

   On June 13, 1970, a brittle fracture developed in one of three tension flanges of a two-cell trapezoidal steel box girder while under construction. A Category E cross-bracing detail initiated a crack where it was welded to the tension flange. Analysis of the fracture surface indicated that a weld crack about 0.2 inches deep was present in a residual field. The weld crack initiated during fabrication or erection. When placing the concrete for the deck, the dead load stress was increased to about 28 ksi. Complete fracture of the top flange occurred. The type of steel used at this location in the structure was A517. The river spans were 281 feet and 370 feet with varying girder depths.

3. Lafayette Street Bridge; St. Paul, Minnesota

On May 7, 1975, a crack occurred in the main girder approximately 119 feet from the end of the 362-foot span. The fracture was due to the formation of a fatigue crack in a lateral gusset to transverse stiffener weld. A back-up bar was used to make a groove weld perpendicular to the bending stress in the girder. Lack of fusion in this transverse weld, which intersected two other welds, resulted in fatigue crack growth into the web and eventually caused brittle fracture of the girder. The steel used was A441.

4. Fremont Bridge; Portland, Oregon

On October 29, 1971, a brittle fracture occurred in a girder truss joint completely parting the bottom junction piece, propagated to varying heights up the vertical webs and arrested in the longitudinal welds of the plates making up the deep-girder web. The fracture initiated at a metallurgical defect produced during fabrication in a detail which was improperly oriented with respect to the direction of principal stress in the bridge girder. That is, the principal rolling direction of the steel was transverse to the length of the girder. In addition, subsequent tests showed low toughness and rejectable weld defects that had escaped quality control. The three items combined to cause the failure.

1.2 Course Description and Objective

This is a practical training course in the design and construction of welded bridge members and connections. On completion of the course, the student should have adequate knowledge to make design and construction considerations that may minimize fatigue and fracture.

1.2.1 Considerations to Minimize the Possibilities of Fracture

Fracture in structural steel bridges is a problem. However, there are differences of opinion as to the seriousness of the problem. There are also differences of opinion on how to correct the problem. In recent years, there have been numerous courses and seminars on design to correct the problem with special emphasis on fatigue and fracture mechanics. More often than not these presentations were highly theoretical presentations on research results and recommendations.

The purpose of this course is to minimize the possibilities of fracture by the application of practical considerations to design and construction. It is important that the word "minimized" be stressed because the authors do not believe nor suggest that all fracture problems will or can be eliminated. Fracture problems are nearly always a result of human errors. This is true beginning with the making of the steel, the design, the detailing, the fabrication, the erection, the inspection and in the maintenance.

Upon completion of this course bridge engineers should have an acute awareness of the practical considerations of fatigue and fracture to the extent that through their personal efforts these problems will be minimized. The ultimate objective is to have the problem of fracture brought under control so that there will be general agreement among engineers that fracture in bridges is not a problem.

1.2.1.1 Design Considerations

There are numerous design considerations that will decrease fatigue problems. Often these considerations involve good common sense based on one's experience or experience of others.

This course covers design considerations in selection and design of welded bridge members and their connections. The importance of clean-cut bridge members and their connections is emphasized. Clean-cut members are those that are simple and smooth with a minimum of weld detail.

In order to give adequate design considerations, it is necessary to have a good understanding of:

1. Design Specifications
2. Type Selection
3. Materials
4. Tension Members
5. Flexural Members
6. Splices, Attachments and Connections
7. Construction Conditions
8. Contract Administration.

1.2.1.2 Construction Considerations

Construction considerations cover the fabrication and field erection of welded bridges. Information presented in this course will give the designer a good understanding of fabrication and erection procedures including the welding operations, quality control and quality assurance. This understanding will enable the designer to make appropriate design decisions and to perform design staff functions during construction.

TOPIC 2

SPECIFICATIONS

OBJECTIVES:

1. *To acquaint the bridge designer with the history of welded construction and the development of the specifications.*

2. *To explore the future direction of specifications.*

3. *To provide information that will allow the bridge engineer to recognize those provisions which are directly and those which are indirectly related to fatigue.*

4. *To develop an awareness of each part of the specifications that may affect the fatigue characteristics of a bridge member.*

2.0 INTRODUCTION

This Topic describes the development of specifications, the development of the welded bridge, those specifications that are related in one way or another to fatigue and fracture, and design choices within specifications that may minimize fatigue and fracture problems.

2.1 SPECIFICATION DEVELOPMENT

Specifications for design and construction of welded bridges have developed slowly over the past 50 years.

Article 3.10.34, "Welds," of the 1931 <u>AASHO Construction Specifications</u> stated,

> Welding of steel shall not be done except to remedy minor defects and then only with the approval of the engineer.

In 1935, AASHO included "Standard Specifications for Arc Welding Metal Bridge Structures." Under these Specifications, the design was to be in accordance with the AASHO "Structural Steel Design" which applied to riveted design and construction except for weld stresses and weld details. These Specifications allowed tensile stresses of 10,000 psi for butt welds and 12,000 psi for compression members.

In 1936, the American Welding Society (AWS) published its first "Specifications for Welded Bridges." Progress came slowly, as a review of the Specifications indicate. Often, when welding was used, the full advantage of these fabrication techniques was not utilized. The allowable stresses did not make welded structures economically attractive in comparison with riveted construction. For several years, AASHO limited welds to specific parts of a bridge. The 1949 AASHO Specifications referred to AWS Specifications for welding and to the AASHO Specifications (riveted) for general design.

The 1949 AASHO Specifications for welding were as follows:

> 3.6.55. -- Welding -- General
>
> All welding shall conform to the current Specifications for Welded Highway and Railway Bridges, Design, Construction, and Repair, of the American Welding Society.
>
> This specification provides for welding (and gas cutting) of base metal consisting of structural carbon steel (Article 4.6.2.), or similar low carbon steel or wrought iron approved by the engineer. Wrought iron shall conform to the requirements of Division IV, Section 7.

Welding of the following items was permissible under these specifications but only if called for on the plan or in the special provisions:

1. Floor expansion devices
2. Railings
3. Built-up shoes, pedestals or expansion rockers
4. Diaphragm connection to beams or other members
5. Stiffeners except that welding transversely across the tension flanges of beams or girders, which have a flange stress of more than 75 percent of their capacity, will not be permitted
6. Filler plates
7. Stay plate and lacing connections to members
8. Connections and details of bracing
9. Caps and base plates for trestle columns except where caps supporting stringers are welded to the sides of the pile
10. Splicing of steel piling
11. Sidewalk brackets except main tension connection
12. Fastening of cover plates to rolled beams
13. Other incidental parts of the structure.

Where a definite amount of riveting was specified as a minimum for connections, the welded connection was to develop an equivalent strength.

In 1950, the <u>AASHO Interim Specifications</u> added another item entitled: "Shop Fabricated All-Welded Plate Girders." The design was still based on riveted specifications.

In 1954, AASHO removed the listing of "permissible" items and left welding selections to the discretion of the engineer. Proportions of member width to thickness ratio, etc., remained as set forth in the AASHO riveted specifications, except as otherwise covered by AWS.

In 1963, the AASHO Interim Specifications included for the first time complete specifications for the design of welded plate girders. For the welding, reference was made to the then current AWS Specifications.

By 1965, riveting was disappearing rapidly from the scene. AASHO recognized this and re-wrote the section on steel accordingly. Emphasis was on welding and high strength bolting. These specifications were written by specialists involved in bridge design and welding. By this time the use of welded plate girders was common and there were a few truss bridges constructed with all welded members. The bridges had been designed and constructed in accordance with the AASHO riveted specifications, the AWS Specifications, and supplemental specifications developed by various engineers and states. The AASHO Specifications were actually developed after the fact.

An attempt was made in the 1965 AASHO Specifications to improve on the AWS Specifications as they pertained to fatigue. The intent was to expand the specifications to cover steels other than A36. Much of this work was based on research performed largely by the University of Illinois and the Applied Research Center of U.S. Steel.

Since 1965, revisions to AASHO have been based primarily on results from research performed at the University of Illinois, the Applied Research Center of U.S. Steel, and more recently at Lehigh University.

2.1.1 Design and Construction

A review of the specifications and bridges that have been built clearly indicates the role played by progressive bridge engineers, both in design and construction, and in developing the current specifications. Various parts of the specifications for welded bridges were developed from the existing riveted specifications by engineers who had a good understanding of theory and who were cognizant of practical aspects.

Changes in bridge fabrication practices and capabilities have made possible the fabrication of members and connections that were once thought to be impossible.

New challenges to bridge engineers expedited changes in bridge fabrication and construction. Many design and construction techniques preceded their adoption as an AASHO specification for welded structures.

2.1.2 Research

Research in steel, steel details, welding, welding details and principally in fatigue led to the development of the current specifications. Research on the plate girder, (homogeneous, composite, and hybrid), box girder, curved girder, orthotropic, and other configurations has influenced specifications.

It seems that the interpretation of these research results and their application to actual bridge design and construction, has not always been properly evaluated. Consequently, it has often led to unnecessarily complicated specifications without improving the actual results; therefore, experienced practicing bridge engineers should assume a more active role in the development of specifications.

2.2 WELDED BRIDGE DEVELOPMENT

In the past two decades much has been written regarding the advantages and disadvantages of welded construction. However, with the rapid advancement of welding techniques and equipment there are certain advantages which can be claimed for welding.

2.2.1 Tension Members

The principal factor in favor of welded tension members is the savings in the weight of steel. Other fringe benefits are improved appearance and the greater freedom permitted in the choice of details and proportions, resulting in a more pleasing structure.

2.2.1.1 Rolled Sections

In the early stages of welded design, rolled sections in built-up members were used as they were in riveted construction. The only difference was that stitch welds were used in lieu of rivets. Eventually stitch welds were recognized as a source of corrosion and fatigue and subsequent designs used continuous welds. Examples of welded built-up tension members using rolled shapes are shown in Figure 2.1.

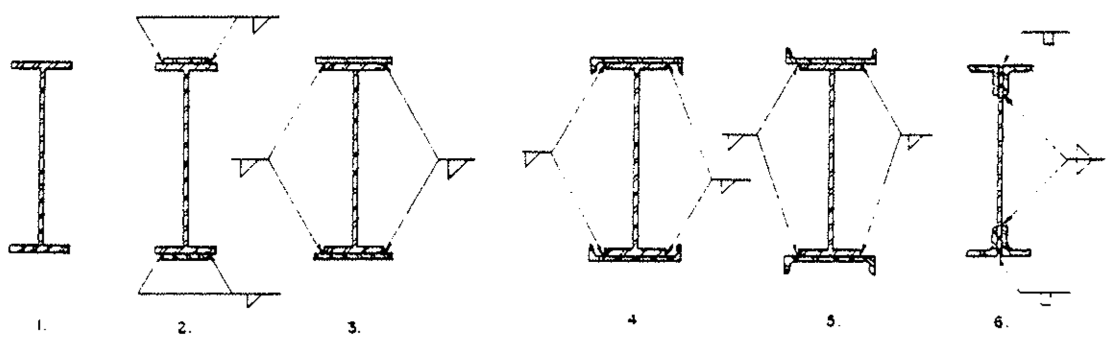


Figure 2.1 Examples of Welded Built-up Tension Members Utilizing Rolled Shapes

2.2.1.2 Welded Sections

Welded plate sections were first used in the United States, to any great extent, in the Carquinez Strait Bridge, which is shown in Figure 2.2. Members of the structure were mostly H sections, some were box sections.

End connections were made with high strength bolts, as shown in Figure 2.3, and the net area at the connections was achieved by increasing the plate thickness or by increasing the strength of the plates which were butt welded, in accordance with AWS Specifications, as shown in Figure 2.4.

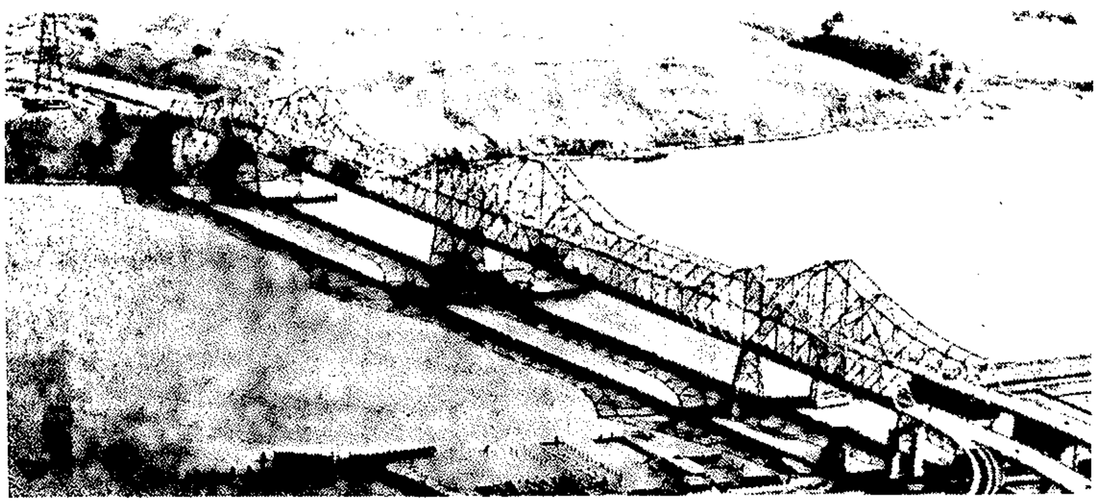


Figure 2.2 Carquinez Strait Bridge, U.S. 80, California

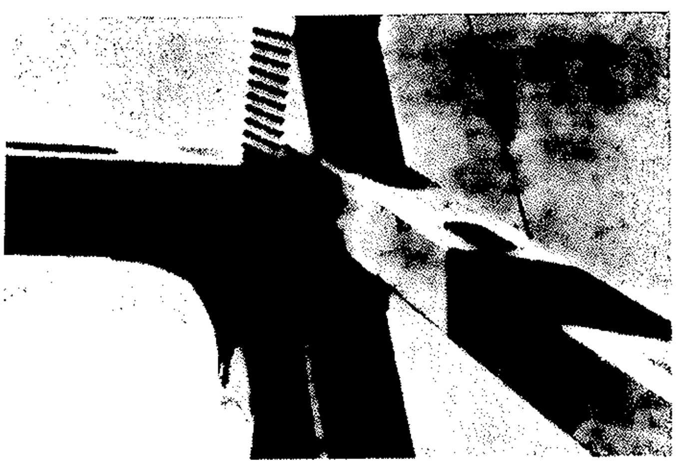


Figure 2.3 Typical End Connection

The H sections were fabricated from flame cut plates connected by fillet welds. The box sections, composed of two solid and two perforated plates, were welded at the four corners with fillet welds. Some box members were welded on the inside with automatic welders that traveled through the member. The automatic process for welding inside the box section is shown in Figure 2.5.

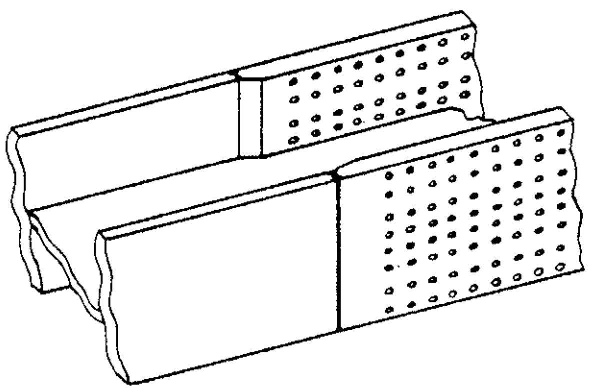


Figure 2.4 H Section

Figure 2.5 Automatic Fillet Welds Inside of Box Section

2.2.2 Beams and Girders

Beams and girders are classified as flexural members. Beams are also described as rolled beams while girders are members fabricated of welded plates whether they be plate girders or box girders.

2.2.2.1 Rolled Beams

With the advent of welding, rolled beams with partial length cover plates were available for longer spans. The terminations of the cover plates were recognized as a potential fatigue problem and continue to be of concern today.

Composite beams were made possible through the welding of shear connectors to the compression flanges. The most common shear connectors in the early days were plates or channels which have been almost completely replaced by the automatically welded Nelson Stud. Current design specifications allow their use in tension areas for continuous beams. Rolled beams were also used for continuous spans and were field spliced by welding or with high strength bolts.

2.2.2.2 Welded Plate Girder

The welded plate girder became popular in the early '50s, while the use of rolled beams for bridges diminished. The rolled beam could not compete economically with the plate girder in the western states.

Initially, the simple-span welded plate girder was developed; followed by the simple-span welded composite plate girder. The design, fabrication and erection processes evolved very rapidly in the '50s and included plate girders with or without transverse stiffeners and with or without longitudinal stiffeners.

Various strengths of steels in a girder were also utilized for the first time.

2.2.2.3 Welded Plate Girders -- Hybrid

With the different strengths of steels available, research proceeded on girders with webs of one strength level and flanges of another. This type of design is called hybrid.

Specifications were included in AASHO for the hybrid girder, but the girder has not been met with any great enthusiasm.

2.2.2.4 Welded Box Girders

During the '60s, considerable interest developed in the steel box girder; much of this was stimulated by the Europeans. Efforts were made to develop criteria that would make the box girder competitive with other types of bridges. Economic advantages were not achieved; however, box girders are used for structures with restrictions in depth and for aesthetic reasons. They are also a good choice for a curved bridge.

Specifications are included in AASHTO for the design and construction of box girders.

2.2.2.5 Orthotropic Bridges

The orthotropic bridge deck has had wide usage in Europe. For spans most popular in this country the orthotropic design has not yet become competitive. This type of bridge contains many details that require special attention with respect to fatigue and fracture.

2.2.3 Connections

Connections, especially welded connections, are a source of fatigue and fracture. Connections of greatest concern are those connected to primary members.

2.2.3.1 Connections to Flanges

Cross frames and lateral bracing connections to beam and girder flanges were recognized as potential problems. Details tended to follow closely those of riveted construction and, in some cases, simply substituted welds for rivets. Some designers believed that welds parallel to stress were not a problem. Transverse welds were not permitted.

Recognizing the inherent danger in welding gussets to flanges, some designers welded the lateral and cross frame attachments directly to the webs. This detail also has its problems. Today, there is fatigue design criteria to cover connections to flanges; however, there continues to be considerable differences of opinion about the merits of such connections.

2.2.3.2 Connections to Webs

Connections welded to webs are usually limited to vertical stiffeners, longitudinal stiffeners, and to cross frames and lateral bracing. Figure 2.6 shows the details of a typical cross frame connection.

Use of connections to webs for cross frames and lateral bracing came about as bridge engineers decided that welding directly to flanges should be avoided. Generally these connections gave no problems design-wise; however, the curved girder presented a

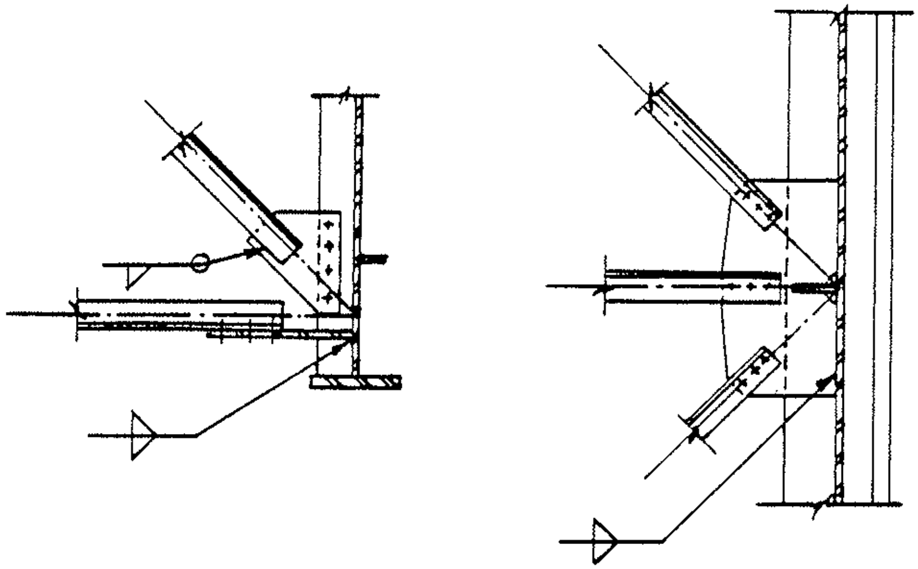


Figure 2.6 Cross Frame Connection

challenge because the cross frames are considered as primary members--a means of distributing the reactions to the flanges. Figure 2.7 presents an example of a curved girder bridge. Additional longitudinal stiffeners were added and sometimes webs were thickened. The trend today is to use connections to flanges. Criteria are included in the AASHTO Specifications.

Figure 2.7 Curved Girder Bridge

2.2.3.3 Stiffener Details

Details of transverse stiffeners for plate girders remained status quo for several years. Stiffeners were welded to the compression flange with a tight fit to the tension flange. This detail gave few problems in fabrication or service; however, some designers believed it was advantageous to cut the stiffeners back, and some researchers found no problems in welding to the tension flange. These details have been researched thoroughly for each school of thought. Specifications for both views are included in current AASHTO Specifications.

Longitudinal stiffeners were intended for compression areas to permit the use of a thin web. Details for connecting longitudinal stiffeners have not changed greatly. A problem has developed on occasions when the longitudinal stiffener is used in tension areas, primarily for aesthetics. Current specifications include the various conditions encountered in longitudinal stiffener details.

2.2.3.4 Pipe and Tubular Connections

Sway frames and laterals for most plate girder bridges have utilized rolled sections such as angles, tees, and channels. The use of pipe and rectangular tubes offer alternatives that at times are economical and very effective. The connections are relatively simple, as shown in Figure 2.8.

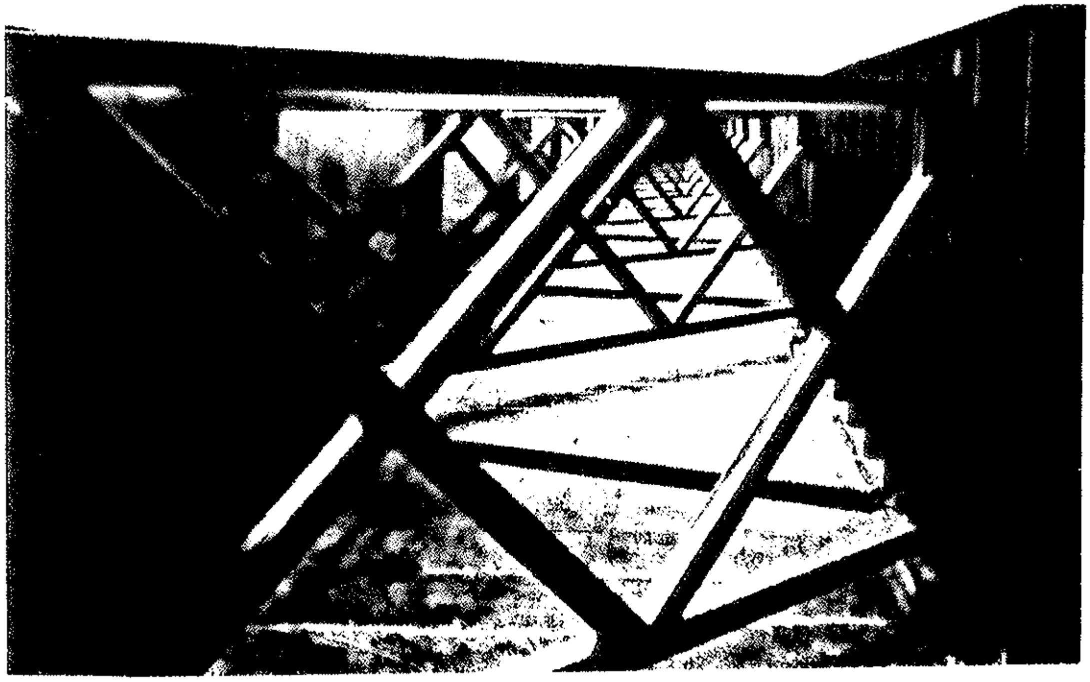


Figure 2.8 Rectangular Tube Cross Frames

2.2.3.5 Hinge Details

Hinges with thin webs and pin plates (a riveted detail), are sometimes used for welded construction. For welded detail a thickened web should be butt welded to the thinner web in order to eliminate the pin plates. Figure 2.9 shows a typical hinge detail.

2.2.3.6 Steel Caps and Attachments

The design and construction of steel caps have not changed appreciably since the 1950s. When the concept of steel cap design was new, engineers proceeded cautiously because they were concerned about some aspects of the details. Basic rules on design criteria were few; however, current specifications cover details which may be encountered. Figure 2.10 shows a typical steel cap under construction in a fabrication shop.

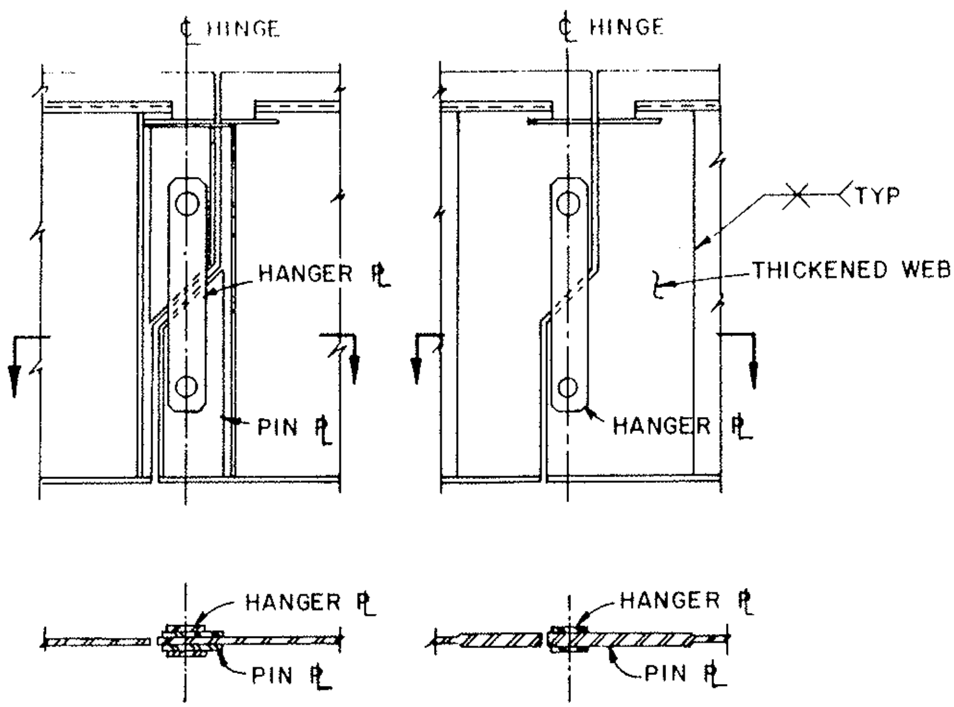


Figure 2.9 Hinge Detail

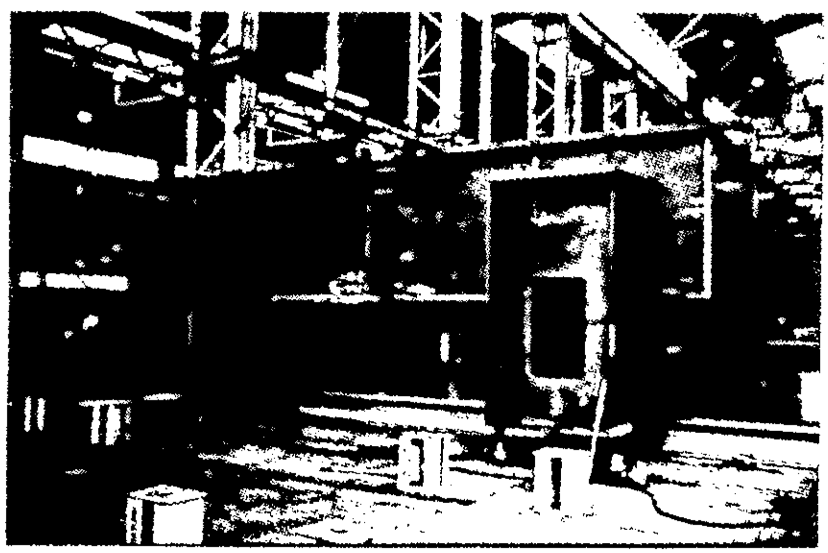


Figure 2.10 Steel Cap

2.2.4 The Development of Steels

Types of steels used in welded bridges have changed through the years to meet the structural and economic needs of designers. Bridge steels commonly used since 1950 are described in this section.

Current AASHTO Specifications include three strength levels: (1) Structural Steel, (2) High Strength Low-Alloy Steel, and (3) High Yield Strength Quenched and Tempered Alloy Steel.

2.2.4.1 Structural Steel

Welded bridges of Structural Steel have commonly used ASTM-A7, A373 or A36. These steels have minimum yield points of 33,000, 32,000 and 36,000 psi, respectively.

A7 was used in the early days; however, with emphasis on welding and weldability, A373 was developed.

A373 Structural Steel, developed in 1954, stipulated the range of carbon and manganese content to enhance its weldability and to improve its notch toughness. This material was used extensively in welded bridges, from about 1954 to 1960. For minor parts not over one inch in thickness, A7 strength steel was acceptable.

A36 was introduced in 1960 to meet the needs of designers and fabricators to increase strength and maintain weldability. Like A373, the percentage for carbon and manganese were stipulated. Today, A36 is one of the most widely used steels.

2.2.4.2 High Strength Low-Alloy Structural Steel

High strength low-alloy steels are generally thought of as the 50,000 psi yield steels. Actually, there is a spread in yield strengths from 42,000 to over 50,000 psi.

For a number of years, ASTM-A242 was recognized as the only steel in this category. In 1960, ASTM-A441 (Modified) was offered as an alternative. A441 had excellent weldability, and had a minimum yield point identical to that of A242.

In 1973, AASHTO Specifications added ASTM-A572 and ASTM-A588 giving a selection of four steels at this stress level.

The current AASHTO Specifications include only ASTM-A572 and ASTM-A588, both with minimum yield points of 50,000 psi.

2.2.4.3 High Yield Strength Quenched and Tempered Alloy Steel

High Yield Strength, Quenched and Tempered alloy steel, a 100,000 psi yield steel, was first used in the Carquinez Strait Bridge in the '50s. At that time, it had a yield of 90,000 psi and was known by its trade name as U.S. Steel T-1. Since then, specifications for this type of steel are included under ASTM-A514 and A517 and are also included in the current AASHTO Specifications. A514 is generally recognized as a structural steel and A517 as a pressure vessel steel.

2.2.5 Construction

Construction pertains to fabrication, erection, and nondestructive testing. This discussion describes the period beginning in 1950 when the use of welding, high strength bolts, and the combination of welding and high strength bolts became popular. With these types of connections, the designers could design economical, clean built-up members or sections with the type of steels desired. Welding and bolting supplanted riveting. A new era began in the construction of bridges and buildings as new fabrication processes, inspection and nondestructive testing techniques were and are being employed.

New skills had to be developed and utilized through education and training of bridge engineers, metallurgical engineers, welding engineers, nondestructive testing technicians, inspectors, welders and welding operators. New controls were then placed on the overall fabrication. Specifications had to be developed and adapted for use.

2.2.5.1 Fabrication Practices

Over the years, fabrication improvements have advanced from the first AASHO Specifications published in 1931 which permitted welding only to remedy minor defects and only when approved by the engineer.

In 1935, AASHO developed standard specifications for arc welding of metal bridge structures.

In 1936, AWS Specifications for welded bridges appeared. These AWS Specifications combined with the AASHO Specifications were used and modified by bridge engineers beginning with the early part of the 1950s.

From this phase, advancements were made in the types of steel available--greater strength levels, and improved weldability.

Fabrication processes progressed from riveted construction to welded, to high strength bolts, and to welded and bolted combinations.

Innovative jigs, fixtures and positioners for welding, assembly jigs and fixtures for shop assembly or fit up of various components of the structure were developed. New and improved welding equipment and material have expanded from manual shielded arc welding power sources and electrodes to submerged arc welding equipment power sources and electrodes, to semi-automatic welding equipment, to automatic welding equipment, to gas metal arc welding, to flux core arc welding, and to electroslag welding.

In addition to shearing, planing, and high speed rotary milling, gas cutting equipment has been developed to include free hand, mechanically guided, semi-automatic, and computerized automatic tracing with multi-cutting heads. Preheating equipment had been improved through the use of gas and electric heaters for welding.

New machinery can roll, straighten, or break back heavy plates for fabrication.

Physical layout has progressed from shop drawings and patterns to automatic drafting machines, photographic transfer systems, and lofting with the use of temperature controlled loft tapes (i.e., tapes layed out to the exact lengths that the plates are to be cut at the designated temperature). These layout methods "streamline" many traditional fabrication procedures. For instance, they facilitate drilling full size bolt holes in the plates prior to fabrication and in the plate girders that are to be spliced with high strength bolts.

2.2.5.2 Nondestructive Testing

Nondestructive testing has progressed from visual inspection of the actual weld while in progress to visual inspection of completed welds, using inspection aids such as dye penetrant and magnetic particles. Added to visual inspection was nondestructive radiographic testing. Nondestructive radiographic testing equipment ranged from heavy and bulky x-ray equipment to use of radioactive sources, such as radium, cobalt, cesium, or iridium, with the radioactive sources encapsulated and handling of the source by the fish pole technique. Later, camera and collimator techniques were developed for the handling of isotopes.

When properly applied, ultrasonic testing detects critical subsurface defects in welds and steels more effectively than any other nondestructive inspection method presently in use on bridges and in bridge fabrication. It is the only commonly used NDT method which is consistently capable of finding tight cracks, lack of fusion and other two dimensional subsurface discontinuities with sharp edges which constitute the most dangerous class of flaws because of the stress concentrations associated with them.

The methods for applying ultrasonic testing properly are described in the current AWS Welding Code. These methods provide a calibrated readout from a systematic repeatable working technique that minimizes operator variables and gives test results which can be rechecked and which will give the location, size, and orientation of subsurface flaws.

2.2.5.3 Erection and Field Welding

With the progress of welding in the early 1950s, bridge engineers and designers adapted welding to field construction of bridges where rivets were comonly used. The welding was extended to girder splices resulting in changes in erection methods.

In the late 1940s and into the 1950s, wide flange beams were commonly used for simple spans. Beams were not usually spliced and had diaphragms or cross frames welded or riveted to the beams. Field welding was limited to cross bracing, diaphragms, and bearing keeper plates.

As welding picked up momentum, steel columns and steel caps were fabricated using rolled shapes and plates. Fabricated members were trucked or shipped by rail to the construction site. Truck cranes for erection began to appear at the site to do the lighter lifts with greater mobility than that of huge crawler cranes and travelers. In the early 1950s, the bridge designer used longer simple span girders, either a WF section or a welded plate girder whose length required field splicing. Girders were either spliced by welding on the ground and then erected or were temporarily supported on false work and the field splices made in place.

Some contractors let either one end of a girder or the girder web run wild for field trim to adjust for conditions at the time of field splicing.

Continuous girders became possible as welding fabrication techniques gained acceptance. The designers used wide flange beams with partial lengths of cover plates or welded plate girders. Later high strength bolted splices were either specified or offered as an option to welding. The welded splice requires the girders to be supported by false work until the welding is completed. The false work must be capable of supporting the girders in the correct vertical and horizontal alignment without restricting movement due to weld shrinkage.

Development of the box girder has led to procedural changes during erection.

Large segments and sub-assemblies of box girders are shipped to the site for field welding. Fit-up and welding are partially completed on the ground prior to erection.

Large segments and sub-assemblies require innovative erection procedures including heavy false work and lifting equipment. Hydraulic lifting devices are sometimes used to support the erected girders and to maintain proper joint geometry for field welding. Unlike the early 1950s when the shielded metal arc was used in field welding, today's fabricators/erectors select welding processes and electrodes to accommodate different conditions.

Erection procedures and methods, quality control and quality assurance programs have changed as the welded bridge developed. The quality control and quality assurance programs must adjust to current construction practices.

2.3 SPECIFICATIONS RELATED TO FATIGUE AND FRACTURE

Article 1.7.2--"Repetitive Loading and Toughness Considerations," of AASHTO deals directly with fatigue and fracture of steel bridges and yet almost every other article has provisions, be it design, materials, or construction that affect the fatigue life of a bridge.

2.3.1 Design

The designer of a steel structure must comply with more provisions than are required for any other type of bridge structure. Most of these provisions will in one way or another alter a plain and clean member to one with many encumbrances, such as stiffeners, gussets, and connectors.

2.3.1.1 AASHTO[7]

A welded beam can be affected by the following conditions:

1. Changes in the flange areas may be accomplished by varying the thickness or width of the flange plate.

2. AASHTO allows coverplates to be added to welded plate girders.

3. Transverse stiffeners need not be in contact with the tension flange. They may be cut back between 4t and 6t from the flange to web fillet weld.

4. Field splices shall preferably be made at points of contra-flexure.

5. Live load deflection is limited to 1/800 to 1/1000 of the span length.

6. Electrode classification for the fillet welds connecting quenched and tempered steel may have strength less than the base metal.

7. Cross frames for curved beams are designed as primary members.

8. Splices and connections shall be designed for a minimum of 75% of the strength of the member.

Some other provisions are:

1. Where the metal will be exposed to corrosive atmosphere, the thickness shall be increased.

2. For back-to-back angles to be 100% effective in tension they must be connected.

3. Length, width, and thickness are specified for cover plates.

4. Heat curving is allowed for low-alloy steels.

5. Minimize overhead welding by proper location of field splices.

6. The preferred number of beams, girders, or trusses for through spans is two.

7. To provide accessibility to all parts of a structure, the member sizes and connections must be proportioned.

8. The minimum size of fillet welds is determined by the thickness of the thicker plate joined.

9. Edge distances for bolts are governed by the type of edge; sheared, flame cut, rolled, or planed.

10. Links and hangers shall be designed for 140% of the required section at the pin hole, a fracture critical member.

These are a few of the applicable specifications found in AASHTO. Although some of them may have little effect on the fatigue life, they are factors to consider in the overall design of a fatigue and fracture free member.

2.3.1.2 Supplemental Specifications of Other States

Due to the wide dispersions in test results, or lack of test results, individual states have supplemented the AASHTO Specifications with provisions which are usually more conservative.

Some of these are:

1. No attachments on tension flanges;

2. Tight fit of transverse stiffeners to the tension flange with cope holes equal to 4t to 6t;

3. Cover plates are used full length;

4. No filler plates in friction type connection;

5. Oversize or slotted holes in friction type connections shall be used on secondary members only;

6. Radius transition only for flanges of different widths. The use of tapered transition is not permitted;

7. The use of A490 bolts is not permitted; and

8. The use of Quenched and Tempered A514 and A517 is not permitted.

2.3.1.3 FHWA Policy on Fracture Control Plan

The FHWA Fracture Control Plan is a comprehensive supplement to AASHTO and AWS Codes. Its broad scope covers design, materials, inspection, and detailed welding requirements. It assigns the responsibility to the designers to implement the plan from design through bridge erection.

2.3.1.4 AWS

Welding, when authorized, shall conform to the AWS Code as modified by the AASHTO Standard Specifications for Welding of Highway

Bridges. Welding symbols are referenced to AWS Publication A2.4.

The following type joints are prohibited by AWS:

1. Partial penetration butt welds,

2. Groove welds from one side only

   a. Without backing

   b. With unqualified backing material,

3. Intermittent welds, and

4. Bevel and J-grooves other than those welded in a horizontal position.

2.3.2 Construction--Specifications Related to Fatigue and Fracture

Design specifications control the stresses that drive the fatigue and fracture mechanisms. Materials specifications control the resistance to these mechanisms, and construction specifications minimize the presence of defects that activate these mechanisms in the presence of too much stress and/or too little resistance.

Construction specifications, therefore, regulate the application of specified materials to a design by outlining various procedures that must be followed and items that must be checked during the fabrication and erection of a bridge in order to minimize the presence of defects which might initiate a fatigue crack or a brittle fracture. These outlined procedures are the various welding codes and specifications. They warrant brief reviews.

2.3.2.1 AWS - D1.1 Structural Welding Code--Steel

The American Welding Society's Structural Welding Code--Steel originated in 1928. Today it is the preeminent Code for welded bridge construction in the United States and probably in the remainder of North and South America as well. Those members of the AWS Structural Welding Committee who are responsible for its content represent a larger and more diverse forum of agencies devoted to welded steel bridge construction than is available with any other code. Thus, even though many of the requirements in this Code are derived from a successful application by one or another of the agencies represented on the Code Committee, the collective contents of the Code expresses the sum of the experiences of all these agencies. It is the base for nearly all other steel welding codes, public and private, that are used in the construction of bridges and buildings.

The 1979 AWS Code is divided into ten sections: one section defines the scope of the Code; five sections are devoted to the design and strengthening of welded connections; and four sections present construction requirements. These four sections are as follows:

1. Workmanship - deals with requirements for preparing and managing welding,

2. Technique - deals with the requirements for controlling individual welding processes,

3. Qualification - describes functions of welders, tackers, welding operators, and weld procedures (with test requirements), and

4. Inspection - describes visual and nondestructive testing techniques.

Many state transportation agencies have adopted amended versions of the AWS Code with weld metal impact strength requirements that: (1) are more stringent than 20 ft. lbs. @ 0° F; (2) require the use of low hydrogen in welding processes for all thicknesses; (3) prohibit the use of electroslag and electrogas processes in all applications; (4) require test welds which duplicate the thicknesses and joint preparations to be used; and (5) establish other more rigorous requirements.

2.3.2.2 AASHTO Specifications for Welding Steel Bridges

The AASHTO Bridge Welding Specifications are little more than a series of amendments to the AWS Structural Welding Code. Since AASHTO is primarily a user's forum, these amendments favor the use of welding procedures, fabrication methods, and inspection techniques which provide increased resistance to fatigue cracking and brittle fracture which will be discussed in Topic 5.

2.3.2.3 FHWA - Specifications for Fracture Critical Bridges

As an outcome of a number of welded bridge failures over the past 20 years, FHWA has provided a series of amendments to the AWS Welding Code which are designed to assure the survival of fracture critical bridge structures. These amendments in the main have been adopted by AASHTO and set into a special code which supercede the standard AASHTO amendments to the AWS Code.

2.3.2.4 Supplemental and State Codes

About a dozen states have prepared their own welding codes in order to satisfy what they consider to be their unique welding requirements. These codes are modeled after the AWS Code with some additional variations to supplement the AWS requirements. California's

"Standard Specifications for Welding Structural Steel" is probably typical of such codes. These specifications originated in the early 1950s as a "test method" to guide steel inspectors in the use of the AWS Code to control welded bridge construction. Over the years, it gradually became a complex contract document. It supplemented the AWS Code of that day with radiographic requirements, electrode matching requirements, and improved weld procedure testing with impact testing requirements. Ultimately, the use of both the AWS Code and the state developed "test method" became so awkward that they were combined into a single specification document.

This document incorporates more stringent weld metal matching requirements, weld metal impact requirements, and weld procedure test requirements than the AWS Code. All these changes were necessary to provide for the level of quality assurance required in the welded fabrication of the many large long-span structures erected in California over the last two decades.

2.3.2.5 ASTM - Delivery and Testing Requirements

ASTM Specifications are used primarily, to control the quality of the steels used in bridges. They do play a role in construction control, however, it is the inspector's responsibility during construction to confirm compliance of specifications with ASTM steel requirements by testing in accordance with ASTM methods. Moreover, the same ASTM test methods are used to test the weld tensile and impact specimens cut from weld procedure test plates.

2.4 CHOICES WITHIN SPECIFICATIONS

The design specifications include criteria for a variety of bridge details; however, it is the designer's responsibility to make choices best suited to a particular bridge.

2.4.1 Design

There is sometimes the tendency for bridge engineers to use specifications as a crutch or excuse. Design specifications do not treat all bridge types, materials, and details equally. The specification may allow a specific detail; however, other details may be better.

2.4.1.1 Structure Type

The specifications do not indicate any particular type of structure. Designers accept type selection as their responsibility; however, they sometimes fail to recognize that various types will not meet their needs equally. A particular type may fit the site conditions, aesthetics, and environment. An evaluation of requirements related to design, fabrication, erection, maintenance, and inspection may show that another type should be selected. It is important in making a type selection to consider, in addition to site conditions, aesthetics, and environment, the following as they relate to various bridge types:

1. Design capability
2. Fabrication Capability
3. Quality Control Capability
4. Quality Assurance Capability
5. Maintenance Capability
6. Economy.

The right selection will likely be the most economical.

2.4.1.2 Redundancy

Redundant is defined in Webster's Collegiate Dictionary as "serving as a duplicate for prevention of failure of an entire system upon failure of a single component."

A bridge can be classified as being redundant or non-redundant, the distinction being whether the bridge or portions of a bridge would collapse (non-redundant) or whether a duplicate member or load path is available to prevent collapse (redundant).

The definition is concise whereas the actual bridge is not so easily defined. Due to the bridge deck, cross-frame and lateral-brace framing into a bridge member, it is difficult to ascertain whether the failure of a bridge member or failure of a component of that bridge member would result in a collapse.

The specifications do not limit the degree of redundancy--that choice is left to the designer. If non-redundant, the designer will be restricted in his choice of details and connections.

2.4.1.3 Member Make-up

Members considered to have fatigue and fracture problems fall mostly into the category of tension members and flexural members.

2.4.1.3.1 Tension Members

Tension members are not addressed adequately by the specifications. With an understanding of the need and common-sense knowledge of materials, fabrication, and inspection, a clean-cut tension member should be designed. Because the specifications permit a wide choice, there are tension members in service and going into service that are difficult to fabricate and difficult to inspect.

Tension members are generally used as truss members, hangers and tension ties. Truss members covered in Article 1.7.44 of AASHTO,[7] while basically different, are treated alike for tension and compression.

Most tension members use bolted connections and splices. Here the designer has a variety of choices as to the design of the member ends. From the choices available, clean-cut members, including the connections, should be selected.

Thicker plates or plates with greater strength are the two most frequently used.

2.4.1.3.2 Flexural Members

Flexural members are given considerable attention by the AASHTO Specifications. Within these specifications are numerous design choices that may minimize fatigue and fracture problems. A choice permitted by AASHTO Specifications is not a guarantee that such choice is desirable or equal to others.

2.4.1.3.2.1 Rolled Beams

Rolled beams, although used extensively in bridges for more than 50 years and given a minimum of coverage by AASHTO, have nevertheless sufficient latitude within the specifications to minimize fatigue and fracture.

Rolled beams are an economical type of construction under certain circumstances and particularly in some geographical locations.

The rolled beam, except for beams with cover plates, has given excellent service. The end detail of cover plates has and is causing problems. There are a number of choices for cover plate details; some are far superior to others in minimizing fatigue and fracture. Some details will not only enhance the fatigue life but are also more economical. All too often, designers give little thought to choices available to them.

2.4.1.3.2.2 Welded Girders

The welded girder is given more coverage by AASHTO than any other type of steel bridge member. The specifications allow a wide range of design decisions. This reflects the extensive use of the welded girder which has grown in variety since 1950.

There are two distinct types of welded girders used. They are:

1. Welded Plate Girders

2. Welded Box Girders.

Plate girders can be homogeneous, hybrid, non-composite and composite. Homogeneous means that flanges and webs at any particular cross section of the girder are of the same strength level.

Box girders are used sparingly in comparison to plate girders. Within each basic type of girder there are numerous considerations that best fit the conditions, including economics.

In contrast, hybrid plate girders employ flanges with greater strength than the webs at any particular cross section.

The non-composite plate girder is designed so that the steel section alone carries the entire design load. The composite girder utilizes the concrete deck as part of the girder section being attached to the flanges with shear connectors.

The homogeneous and the hybrid plate girder may be non-composite, composite or, in some cases, a combination of each.

The box girder has an even greater variety of choices than the plate girder; however, the box girder is generally more complex in design and construction. Designers of the box girders are sometimes oblivious to its problems. Nearly all of the design choices for the plate girder can apply to the box girder.

2.4.1.4 Connections

Connections are not given much attention in the AASHTO Specifications other than in Article 1.7.2, "Repetitive Loading and Toughness Considerations."

Figure 2.11 includes 18 examples of connections and details which are correlated to allowable stress ranges. These examples have been researched intensively, providing the basis for the allowable stress ranges. Designers are in some instances accepting these illustrations as being suggested details without any thought to fabrication, erection and inspection problems. Their performance in research is no guarantee of performance in a structure.

There are numerous details and connections from which to make a choice. Consider the research but do not overlook the practical aspects as to what is good or bad about a particular detail or connection. To make the proper choice, the designer must be imaginative and aware of pitfalls in the material, fabrication, erection and the inspection. Do not select a detail that is allowed by specifications if another, in all probability, will give better service. For example, details that require welding across tension members are not recommended.

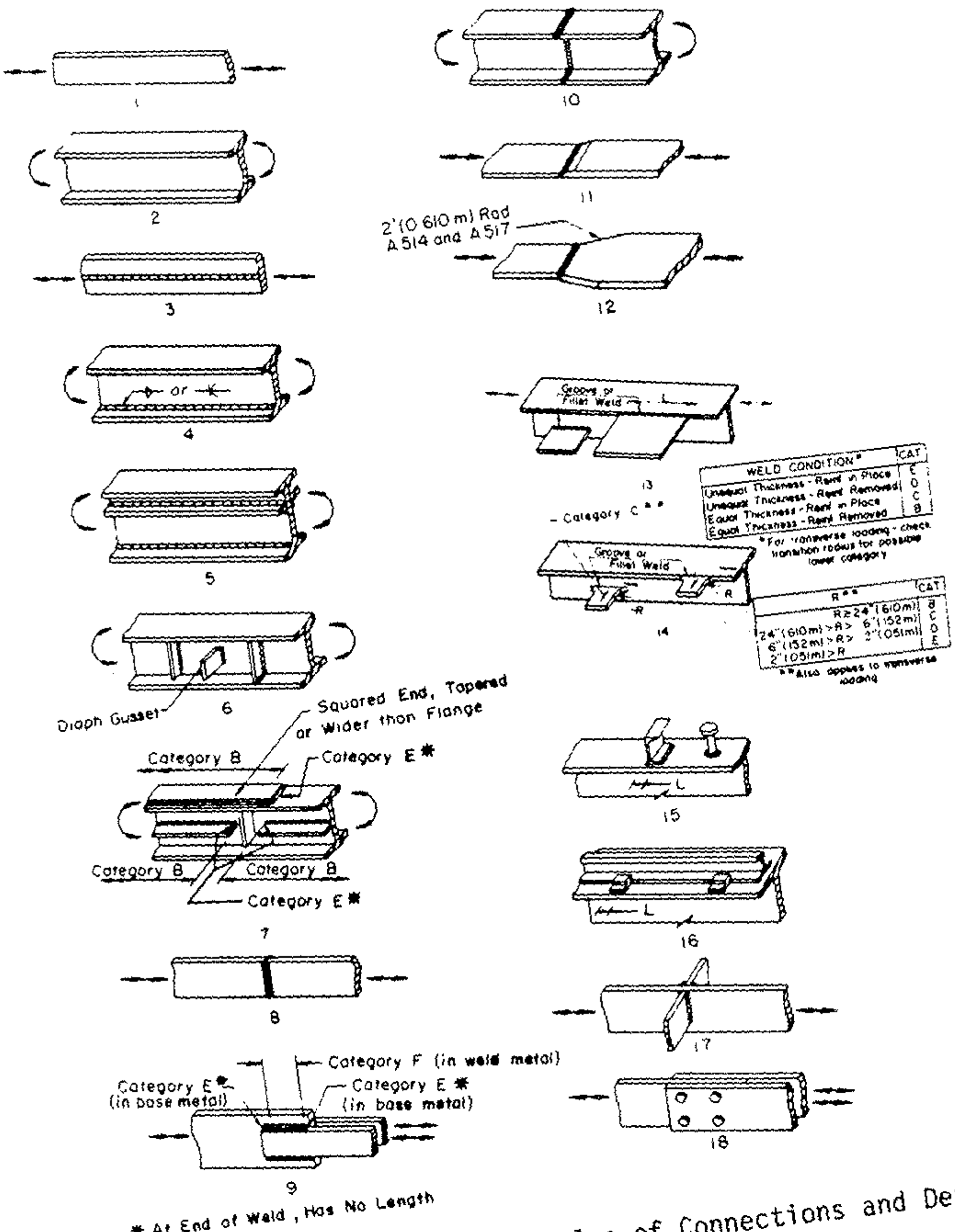


Figure 2.11 Illustrative Examples of Connections and Details

2.4.1.5 Aesthetics

Aesthetics is being achieved by adding plates and shapes to give lines and breaks to a bridge. The wide-open choice appeals to bridge designers and bridge architects; however, materials are sometimes added indiscriminately and can lead to fatigue and fracture problems. Aesthetics is a personal matter, appealing to some and not to others.

Nevertheless, aesthetics are an integral part of bridge design. The shape and proportions are very important, but indiscriminate additions are not justified. Aesthetic values must not adversely affect the structural quality as related to fatigue and fracture.

On the positive side, aesthetics can be and often is a benefit. A structure member with good proportions and shape may simplify the design, the fabrication, the erection, the maintenance, and the inspection at any phase. There are numerous aesthetic choices available and permitted by the specifications.

2.4.1.6 Fatigue

What permissible choices are there within the specifications that deal with fatigue? Naturally, the response would be to utilize materials and details included in Article 1.7.2 - "Repetitive Loading and Toughness Considerations" of the AASHTO Specifications.

Article 1.7.2 gives allowables for various connections which are a small part of the picture when related to fatigue.

Every item discussed in the preceding paragraphs allows choices that may affect fatigue. Those discussed were:

1. Structure Type

2. Redundancy

3. Member Make-up

4. Connections

5. Aesthetics.

Loading has not been discussed directly. This is an area of disagreement. The specifications allow some latitude as to live loading. AASHTO Specifications state:

> The number of cycles of maximum stress range to be considered in the design shall be selected from Table 1.7.2 B unless traffic and loadometer surveys or other considerations indicate otherwise.[7]

Distribution of loads and deflection limitations sometimes play a part in selections that affect fatigue. Most research on loadings and distribution of loads indicate current specification requirements are conservative. There are those who wish to liberalize the specifications. Data indicate that liberalized specifications can be justified; however, such action would intensify the fatigue problem and shift the emphasis to designing clean-cut members.

2.4.2 Currently Used Materials

Current AASHTO Specifications give designers a choice of three basic types of steel: (1) Structural Steel, (2) High Strength Low-Alloy Steel, and (3) High Yield Strength Quenched and Tempered Alloy Steel.

2.4.2.1 Structural Steel: ASTM - A36

A36 is the only steel classified as Structural Steel in AASHTO and is the most commonly used. It was developed for its strength (36,000 psi minimum yield point) and weldability. A36 is readily available in shapes and plates with plates up to 8" thick.

Thickness is a choice that designers must consider. The workability, weldability, and quality of A36 are not necessarily equal for all thicknesses. The designer, recognizing these differences, should select a shape that will accommodate thinner plates of A36 steel.

2.4.2.2 High Strength Low-Alloy Steel: ASTM - A572

A572 is one of two steels listed as Grade 50 in the AASHTO Specifications[7] and is classified by ASTM as High Strength Low-Alloy Columbium Vanadium Steel.

With the 50,000 psi minimum yield point steel, the thickness must be considered. Similar to A36, the workability, weldability, and quality are not necessarily equal for all thicknesses.

2.4.2.3 High Strength Low-Alloy Steel: ASTM - A588

A588 steel is a companion of A572. ASTM identifies A588 as a High Strength Low-Alloy Structural Steel with 50,000 psi minimum yield point up to 4" thick. This steel is also available in shapes and plates, and is intended primarily for welded bridges. Its atmospheric corrosion resistance is equal to approximately twice that of carbon structural steel with a copper content of 0.2% or more.

ASTM states:[75]

> Welding techniques are of fundamental importance and it is presupposed that welding procedures will be suitable for the steel and the intended service.

The designer has a choice of thicknesses which must be given consideration. Here also the workability, weldability and quality are not necessarily equal for all thicknesses.

2.4.2.4 High Yield Strength, Quenched and Tempered Alloy Steels: ASTM - A514 and A517

A514 and A517 are two steels identified in AASHTO as High Yield Strength, Quenched and Tempered Alloy Steel. A514 is described in ASTM as High Yield Strength, Quenched and Tempered Alloy Steel Plate Suitable for Welding. A517 is identified as Pressure Vessel Plate, High-Strength Quenched and Tempered Alloy Steel.

These steels possess a minimum yield strength of 100,000 psi for thicknesses up to 2½" and 90,000 psi for thicknesses from 2½" to 6", inclusive.

A514 and A517 steels are basically alike; however, A514 is tailored for bridges and A517 for pressure vessels. At one time, A517 was considered to be of better quality because it passed more stringent tests which were dictated by its intended use. Testing requirements have been changed for A514 and include a toughness requirement. There appears to be little if any reason for considering the use of A517 steel for welded bridges.

A514 and A517 steels are possible choices; however, they are limited choices in that they are only considered economical for long span bridges where the dead load is the greater part of the total load. They are steels that should be used only in clean-cut members. Thickness must be given consideration. Once again the workability, weldability, and quality are not necessarily equal for the various plate thicknesses. Unlike A36, A572 and A588 these steels may be difficult to weld if they are too thin or too thick. Some engineers maintain that the thickness should not be less than 5/8" nor more than 2"; however, these are not firm figures because much depends on environmental conditions and the fabricator's capability.

TOPIC 3

FATIGUE AND FRACTURE

OBJECTIVES:

1. *To give the bridge engineer an understanding of current fatigue specifications and how they apply to various conditions.*

2. *To provide information to bridge engineers that will lead to better decisions and selections that may minimize fatigue and fracture.*

3.0 INTRODUCTION

Topic 3 covers the development of fatigue design criteria, stress range concepts and applications of stress ratio. Historically, steel bridges have performed satisfactorily; the few failures that have occurred can be attributed directly to fatigue. There are many factors that affect the fatigue life of a member such as the materials, details, fabrication, loading, and others too numerous to list.

3.1 DESIGN CRITERIA DEVELOPMENT

As discussed in Topic 2, the design criteria were generally developed from practical considerations and later adopted by the different specification bodies such as AASHO or AASHTO, AWS and AISC. More recently, development has been through the extensive research conducted in the laboratories at Lehigh, Illinois and Drexel Universities under the auspices of the steel industry, the National Cooperative Highway Research Program, the FHWA and at various State highway laboratories.

3.1.1 History[10]

Specification considerations for designs to reduce fatigue in steel members are relatively new although the problem was first recognized 90 years ago. Since the 1930s, however, fatigue has been studied in the laboratory and the wealth of data accumulated since then has served as the basis for the present specifications.

Welding has increased the need for fatigue specifications. Although the amount of data has proliferated, the data do not cover all the areas as comprehensively as desired. The wide dispersion of test results and specimens not directly applicable makes specification writing a "state of the art" even in its present form.

As far back as 1829 the Germans evaluated the results of repeated loadings on metal. The Phoenix Bridge Company in 1885 required that the members subjected to reversals be designed for the maximum stress plus 0.6 times the minimum.

Railway engineers recorded the first fatigue failures in about 1843. Beginning in the 1930s, engineers instigated extensive laboratory studies on the factors that affect fatigue. In 1931, the Germans required the consideration of fatigue in welded structures. Only the welds were considered critical. By 1936, the AWS Specifications recognized that the base metal was also critical.

In 1944, Wilbur M. Wilson, supported by extensive research, accurately predicted the locations of cracks on riveted railway bridges. His conclusions were confirmed as numerous cracks were found during the next few years. Up until that time the only provisions for fatigue were the same as provided by the Phoenix Bridge Company except that 0.5 times the minimum stress was added to the maximum.

During the 1940s, AREA and AASHO both adopted the AWS Specifications. The British Welding Research Association (BWRA) in 1958 published extensive fatigue provisions.

During the 1960s, the different specification bodies, AREA, AASHTO, AISC, and AWS, recognized the influence of welding, type of loading, types of connections, and their importance. These bodies developed independently their own fatigue provisions.

The fatigue provisions of the 1960s were based on the stress ratio, R. The S-N curves for various stress ratios, always at R = 0, are plotted on the AWS-WRC (Modified Goodman Diagram). See Figure 3.1. The equation of the line of best fit including a factor of safety, has the form $F_r = \frac{k_1 f_{ro}}{1-\alpha R}$, where k_1 is a coefficient dependent on tensile strength, f_{ro} is the intercept on the R = 0 line, and α is the slope of the line.

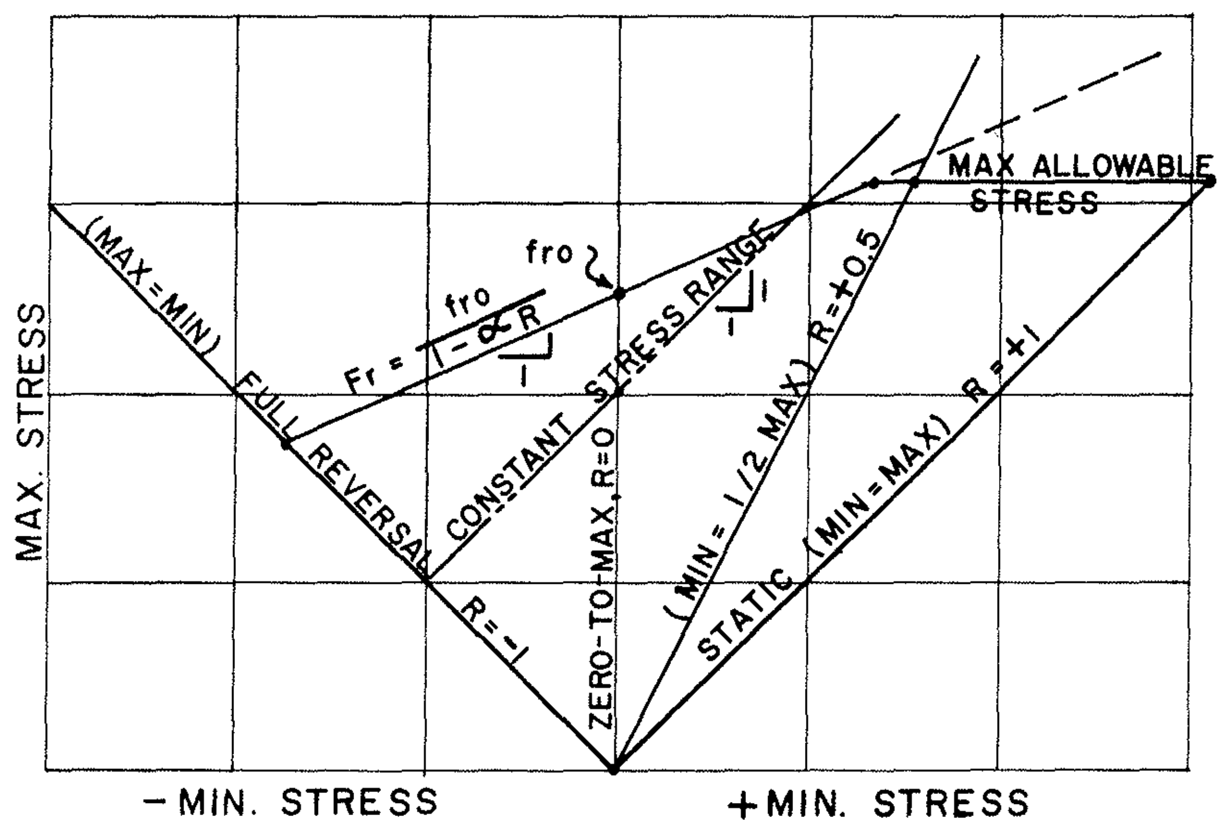


Figure 3.1 Modified Goodman Diagram

The allowable fatigue stress, F_r, in the stress ratio specification increases with increased strength of the material, F_y, and with an increase in the stress ratio.

In 1973, the present specifications were adopted by AASHTO and subsequently adopted by AWS, AISC, and AREA. These concepts are based on studies conducted by John Fisher at Lehigh University. The specifications are based on the theoretical and research conclusion that the live-load stress range is the primary factor in fatigue failure.[3]

3.1.2 Research

Current AASHTO fatigue specifications, first adopted in 1973, are based to a great extent on fatigue testing and research performed at Lehigh University and Drexel Institute of Technology.[3,4]

These fatigue tests were conducted by maintaining a constant minimum stress and varying the stress range from test to test. Materials involved included A36, A441 and A514 steel. They tested over 500 beams, rolled or welded, with a majority having depths of approximately 41". A large number of the welded beams had flanges 6" wide and 3/8" thick. These beams and girders including fabrication were considered sufficiently representative of actual bridge girders to give fatigue results expected in full-size bridge girders. Many engineers question whether 3/8" fabricated material produces results similar to that for flanges of 1", 2" or 3" in thickness.

Test data indicated stress range alone; independent of type of steel, stress ratio, and other factors; controlled the fatigue life of welds and weldments as shown in Figures 3.2, 3.3 and 3.4. These research results provide the basis for the constant stress range concept used in the current specifications.

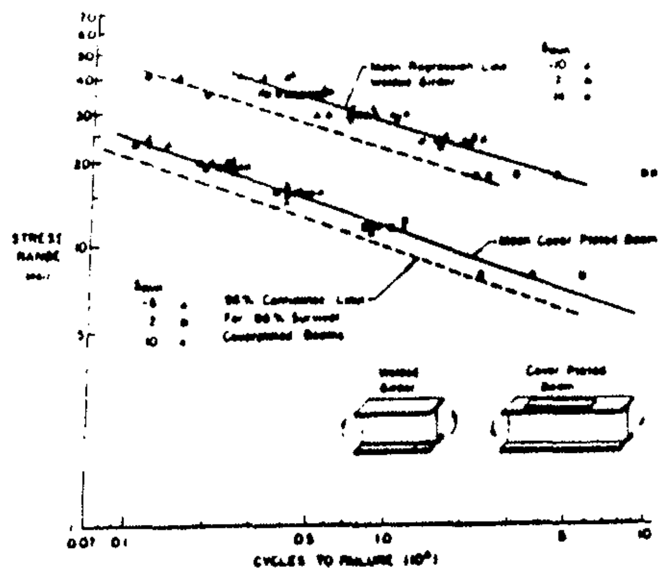


Figure 3.3[8]

Effect of Minimum Stress and Stress Range on the Cycle Life for the Welded End of Coverplated Beams and Plain Welded Beams

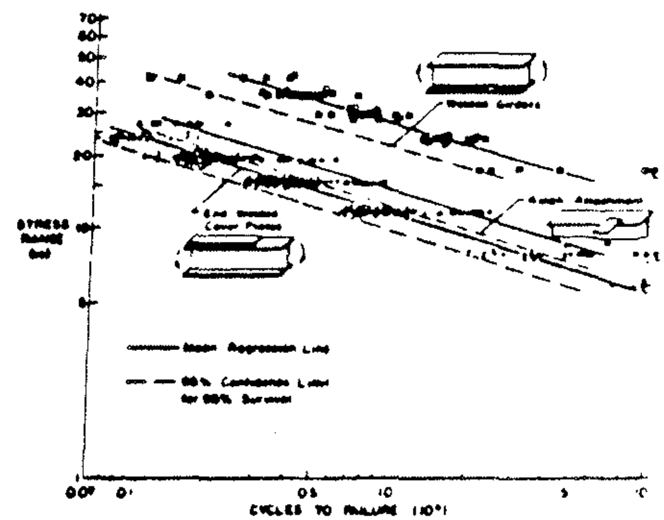


Figure 3.4[8]

Comparison of Short Welded Attachments With Cover-plated and Plain Welded Beams

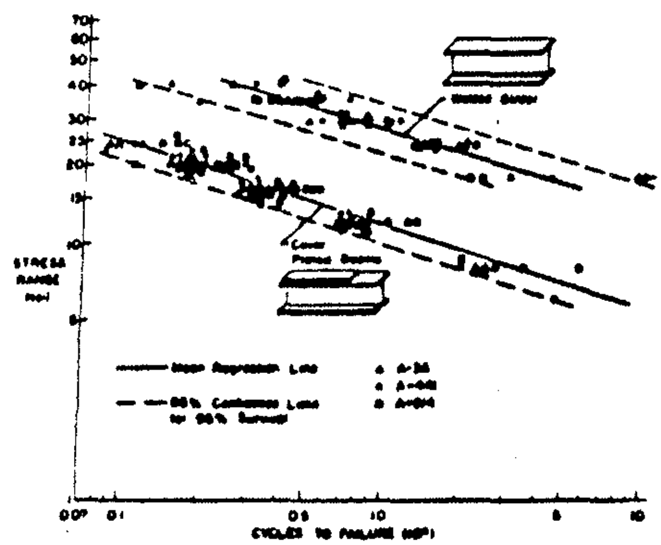


Figure 3.2[8]

Effect of Stress Range and Type of Steel on the Cycle Life of Coverplated and Plain Welded Beams

Prior to work done at Lehigh and Drexel, the majority of fatigue tests on weldments were conducted by maintaining a constant stress ratio and varying the maximum stress.[1,2,5] This method indicated that stress range is the dominant factor controlling fatigue life of welds and weldments; however, in many cases there were indications that other factors did have some effect. Professor Munse reports[2] that for numerous tests on different types of members the stress range varies for different stress ratios.

> In general, under a reversal of axial stress (R = -1), the stress range is about 20 percent greater than that for zero-to-tension axial loading (R = 0). Under a stress cycle in which stress varies from one-half tension to tension, the stress range is approximately 90 percent of the stress range for a zero-to-tension cycle.

Figure 3.5 shows a comparison of stress range as a variable and as a constant.

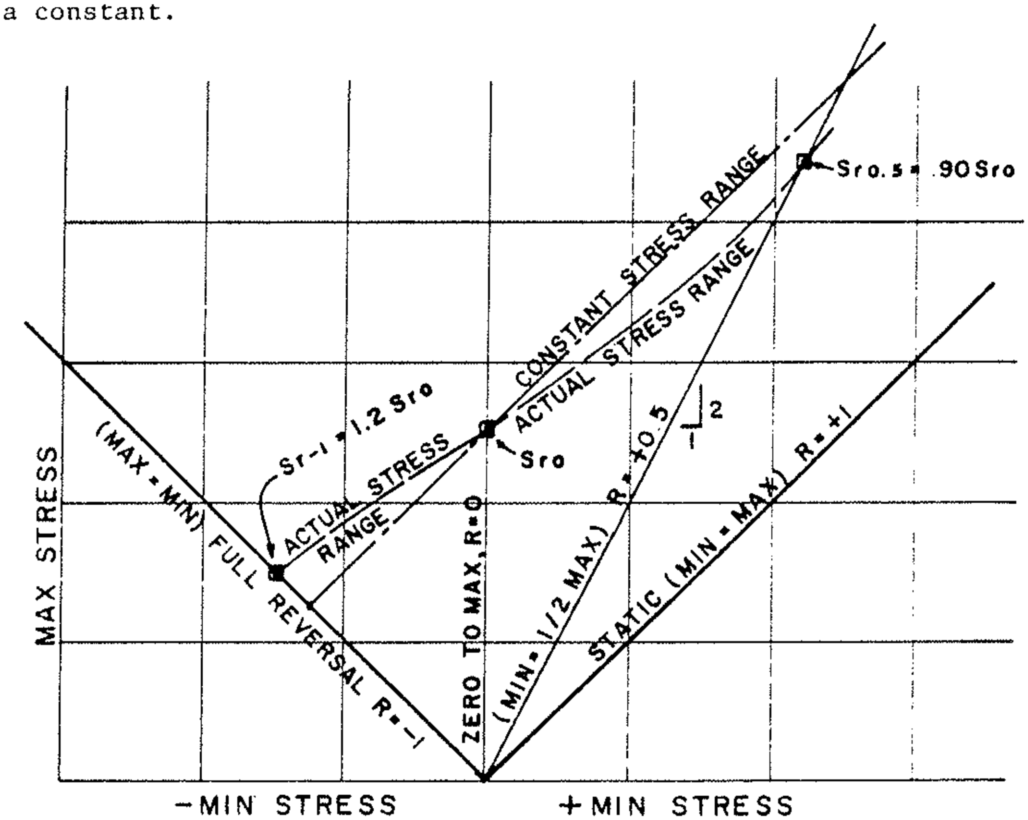


Figure 3.5 Variation From Constant Stress Range

Though there appears to be a difference of opinion, there is general agreement that use of a constant stress range is a suitable and desirable criterion in bridge design.

Any agreement on the use of constant stress range for design criteria does not imply that there is agreement on the stress ranges to use. The question remains as to how well the test specimens reflect actual conditions. Also, nearly all testing by either stress range or stress ratio has been conducted on models with a stress ratio less than 0.5. At one time, welded girder bridges were considered to be subjected to loads that produce cycles of maximum stress ranging from 0.25 to 0.5 tension-to-tension.[5] Today there are bridges with stress ratios approaching 0.8; thus there is question about the adequacy of the specified constant stress range for all levels of stress ratios. Researchers continue to seek answers to the many questions being asked.

3.2 STRESS RANGE CONCEPT

Since 1974, the major specification bodies, AASHTO AWS, AREA, and AISC, have adopted the stress range concept where only the algebraic difference of the maximum and minimum live load plus impact stresses are considered when designing for fatigue. All steels, regardless of their strength, are considered as having the same stress range. These two parameters are the major differences between previous specifications and the current criteria.

3.2.1 Stress Range

Recent studies on beams conducted by the National Cooperative Highway Research Program at Lehigh University indicated that both welded girders and cover plated beams were not affected by stress ratio or type of material.[3] These two types of beams, welded plate girders with no attachments and cover plated beams represent the extremes of fatigue details, categories B and E, respectively. While experimental values showed some scatter due to initial discontinuities and residual stresses, the values, F_{sr}, were derived from statistical analysis and based on 95 percent confidence limit for 95 percent survival.[8]

Residual tensile stresses at or near the yield point in both the weldment and the base metals and the discontinuities within these regions are the primary causes of fatigue failure. Whether the steel is A36, A572, A588, or A514-A517 makes no difference. This is the reason why the stress ratio, R, is insignificant because the maximum stress is already at or near the yield point.[8]

The allowable stress range, F_{sr}, as specified in AASHTO, depends on; (1) the number of stress cycles, (2) type of connection, (3) redundancy, and (4) toughness.

3.2.1.1 Stress Cycles

Historically, the actual live load (LL) stresses on a bridge have been less than the design stresses. It has been observed that the occasional high stress ranges are the cause of fatigue cracks and crack growth and that the damages were cumulative for the other lesser stress ranges. Due to variation in the distribution of loading, impact and occurrence of the design load, the present stress cycle criteria is thought to be below the fatigue crack growth threshold.[8]

The stress cycles used in design are 100,000, 500,000, 2,000,000, and over 2,000,000 cycles. (See Table 3.1.) Transverse members are subjected to higher stress range a greater number of times due to distribution factors, and are therefore in a higher stress cycle than the longitudinal members. The probability of the lane loading to produce the maximum design stress range is less than that of a single truck and has therefore a lesser stress cycle.

Table 3.1 Stress Cycles[7]

Main (Longitudinal) Load Carrying Members

Type of Road	Case	ADTT*	Truck Loading	Lane Loading†
Freeways, Expressways, Major Highways and Streets	I	2500 or more	2,000,000**	500,000
	II	less than 2500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000

Transverse Members and Details Subjected to Wheel Loads

Type of Road	Case	ADTT*	Truck Loading
Freeways, Expressways, Major Highways and Streets	I	2500 or more	over 2,000,000
	II	less than 2500	2,000,000
Other Highways and Streets	III	...	500,000

*Average Daily Truck Traffic (one direction).
†Longitudinal members should also be checked for truck loading.
**Members shall also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 1.3.1(B) for one traffic lane loading.

3.2.1.2 Type of Connection

The designer has no control over the stress cycle but his selection of the type of connection and the design and geometric layout will determine the fatigue life of the bridge. Ideally, the designer would try to eliminate all attachments. Finding this impossible, the designer may try bolting or to keep the attachments as short as possible or move the attachments to an area of lower tensile stress. If no other solution is available, the designer is obligated to use common sense in his choice of details to minimize the chances of fatigue failure.

The AASHTO Specifications, provide a comprehensive list of conditions, kinds of stress, and stress categories. The stress categories are listed from A through E and F. They are described verbally in Table 3.2 and pictorially in Figure 2.11. Only members and components that will be in tension need to be considered. Category A is the least fracture critical and thus the highest allowable and Category E the lowest. Category F is for welds in shear.

Table 3.2[7] Stress Categories

INTERIM
1979

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1000 or less	T or Rev.	A	1,2
Built-up Members	Base metal and weld metal in members without attachments, built-up of plates, or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,7
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges	T or Rev.	C	6
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends			
	(a) Flange thickness $\leq$ 0.8 in. (20 mm)	T or Rev.	E	7
	(b) Flange thickness $>$ 0.8 in. (20 mm)	T or Rev.	E	7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by nondestructive inspection.	T or Rev.	B	8,10,14
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2 1/2, with grinding in the direction of applied stress, and weld soundness established by nondestructive inspection	T or Rev.	B	11,12

Table 3.2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2 1/2 when reinforcement is not removed and weld soundness is established by nondestructive inspection	T or Rev.	C	8,10,11,12,14
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. (50.8 mm) and 12 times the plate thicknesses, but less than 4 in. (101.6 mm)	T or Rev.	D	13
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, is greater than 12 times the plate thickness or greater than 4 in. (101.6 mm) long	T or Rev.	E	13
	Base metal at details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length when weld soundness transverse to the direction of stress is established by nondestructive inspection			
	(a) When provided with transition radius equal to or greater than 24 in. (.610 m) and weld end ground smooth	T or R	B	14
	(b) When provided with transition radius less than 24 in. (.610 m) but not less than 6 in. (.152 m) and weld end ground smooth	T or R	C	14

INTERIM
1978

Table 3.2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	(c) When provided with transition radius less than 6 in. (.152 m) but not less than 2 in. (.051 m) and weld end ground smooth	T or R	D	14
	(d) When provided with transition radius between 0 in. and 2 in. (0 and .051 m)	T or R	E	14
Fillet Welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	
	Base metal adjacent to fillet welded attachments with length, L, in direction of stress less than 2 in. (50.8 mm) and stud-type shear connectors	T or Rev.	C	13,15,16,17
	Base metal at details attached by fillet welds with detail length, L, in direction of stress between 2 in. (50.8 mm) and 12 times the plate thickness but less than 4 in. (101.6 mm)	T or Rev.	D	13,15,16
	Base metal at attachment—details with detail length, L, in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in. (101.6 mm)	T or Rev.	E	7,9,13,16
	Base metal at details attached by fillet welds regardless of length in direction of stress (shear stress on the throat of fillet welds governed by stress category F)			
	(a) When provided with transition radius equal to or greater than 24 in. (.610 m) and weld end ground smooth	T or R	B	14
	(b) When provided with transition radius less than 24 in. (.610 m) but not less than 6 in. (.152 m) and weld end ground smooth	T or R	C	14

Table 3.2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	(c) When provided with transition radius less than 6 in. (.152 m) but not less than 2 in. (.051 m) and weld end ground smooth	T or R	D	14
	(d) When provided with transition radius between 0 in. and 2 in. (0 and .051 m)	T or R	E	14
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	18
	Base metal at net section of high-strength bolted bearing type connections	T or Rev.	B	18
	Base metal at net section of riveted connections	T or Rev.	D	18
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	9

A. CATEGORY A

Category A is the least fracture critical condition and is comprised of plain material, plates and rolled sections, with no attachments.

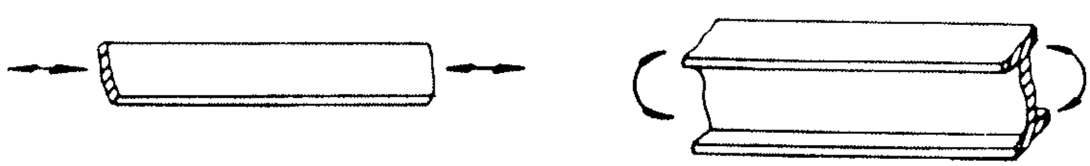


Figure 3.6[7,8] Category A

B. CATEGORY B

Category B includes built-up members with no attachments and connections that are made by longitudinal welds in the direction of stress.

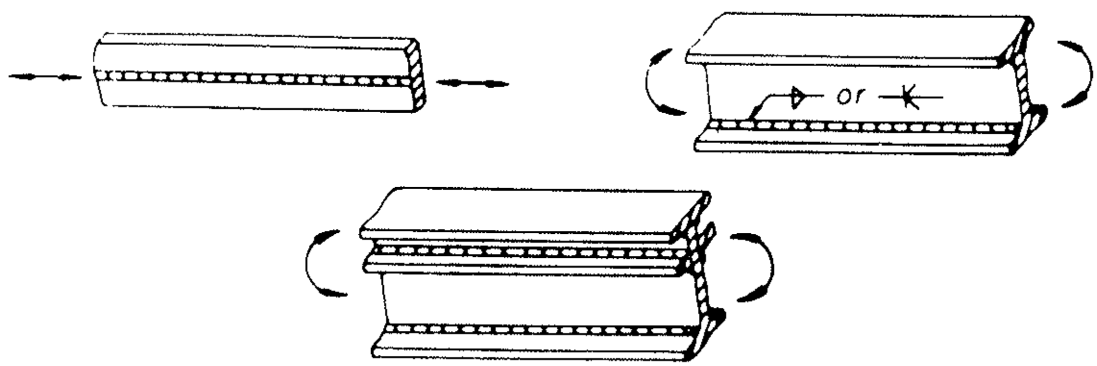


Figure 3.7[7,8] Category B

C. CATEGORY C

Category C is comprised of beams and girders with attachments such as transverse stiffeners at the toe of the stiffener to web weld.

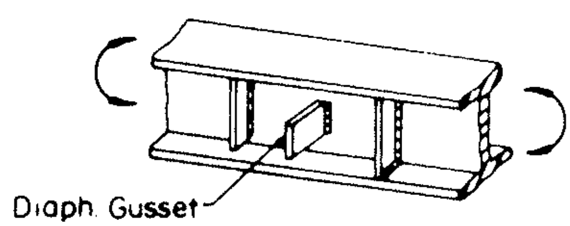


Figure 3.8[7,8] Category C

D. CATEGORY E AND E'

Category E is comprised of cover plated beams, Category E' includes cover plates greater than 0.9 inch.

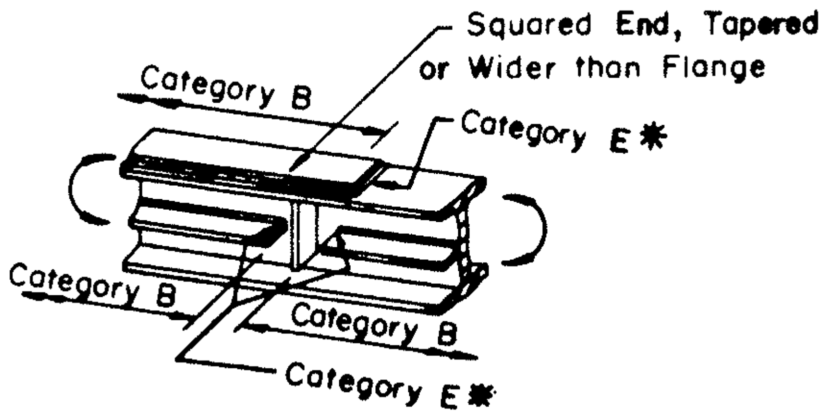


Figure 3.9[7,8] Category E

3.2.1.3 Other Details

Other important details and their categories are:

a. Butt welded flanges; ground, transitioned, and nondestructive tested are in Category B--if unground, in Category C.

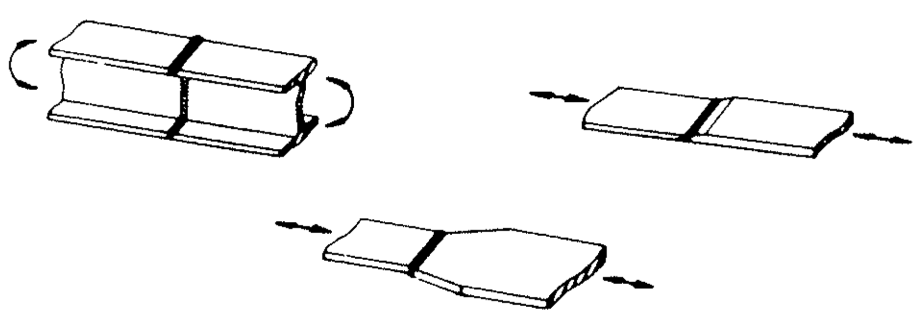


Figure 3.10[7,8] Category B or C

b. Base metal adjacent to fillet or groove welded attachments vary, depending on length, from Category C to E, the longer the attachment the more sensitive it is at the weld termination to fracture.

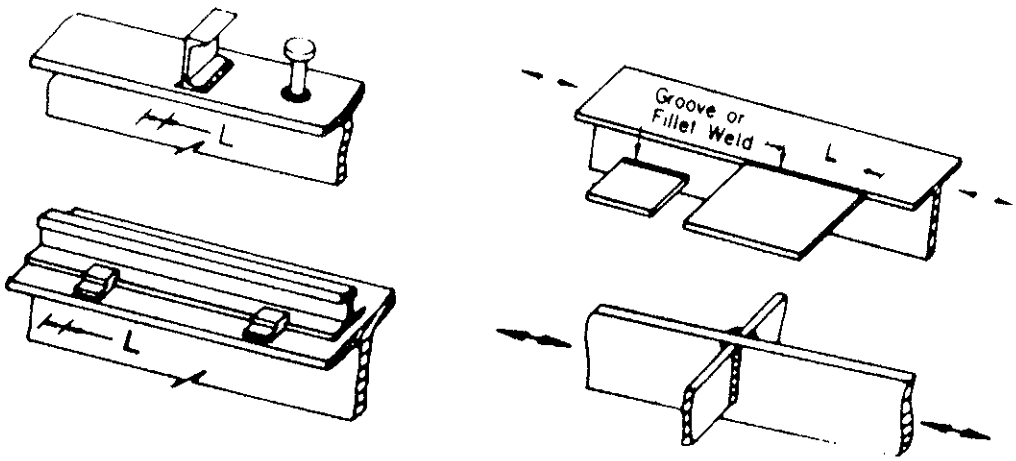
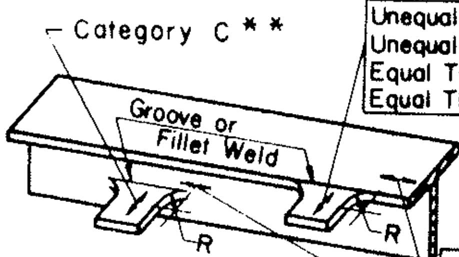


Figure 3.11[7,8] Category C to E

c. Base metal adjacent to fillet or groove welds, regardless of length, when provided with transition radii, vary from Category B to E. The smaller the transition radius, the more sensitive it is to fracture.



WELD CONDITION*	CAT
Unequal Thickness - Reinf. in Place	E
Unequal Thickness - Reinf. Removed	D
Equal Thickness - Reinf. in Place	C
Equal Thickness - Reinf. Removed	B

*For transverse loading - check transition radius for possible lower category

R**	CAT
R≥24"(.610m)	B
24"(.610m)>R> 6"(.152m)	C
6"(.152m)>R> 2"(.051m)	D
2"(.051m)>R	E

**Also applies to transverse loading

Figure 3.12[7,8] Category B to E

3.2.1.4 Redundancy

Table 1.72A1 of AASHTO is actually two tables. One is for "Redundant Load Path Structures" and the other for "Non-Redundant Load Path Structures." They are reproduced in Table 3.3 for easy reference.

The non-redundant table was formulated by decree as opposed to increasing material toughness in order to obtain further guarantees against initiation of cracks and the propagation of existing cracks or discontinuities. This was accomplished by shifting the stress cycle by one loading range. This forces the designer to select details that are less sensitive to fatigue and rules out certain choices of detail.

The designer has some degree of control through type selection of the bridge. He must decide whether he can design a non-redundant structure economically when all other parameters are considered.

Table 3.3[7] Allowable Fatigue Stress

INTERIM

REDUNDANT LOAD PATH STRUCTURES (1)

Category See Table 1.7.2A2	Allowable Range of Stress, F_{sr} (ksi) (MPa) For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	60 (413.69)	36 (248.21)	24 (165.47)	24 (165.47)
B	45 (310.26)	27.5 (189.60)	18 (124.10)	16 (110.31)
C	32 (220.63)	19 (131.00)	13 (89.63)	10, 12* (68.95), (82.74)*
D	27 (186.16)	16 (110.31)	10 (68.95)	7 (48.26)
E	21 (144.79)	12.5 (86.18)	8 (55.15)	5 (34.47)
E	16 (110.31)	9.4 (64.810)	5.8 (39.990)	2.6 (17.926)
F	15 (103.42)	12 (82.74)	9 (62.05)	8 (55.15)

*For transverse stiffener welds on girder webs or flanges.

Table 3.3[7] (Continued)

NON REDUNDANT LOAD PATH STRUCTURES (2)

Category See Table 1.7.2A2	Allowable Range of Stress F_{sr} (ksi) (MPa)			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	36 (248.21)	24 (165.47)	24 (165.47)	24 (165.47)
B	27.5 (189.60)	18 (124.10)	16 (110.31)	16 (110.31)
C	19 (131.00)	13 (89.63)	10, (68.95) 12*(82.74)	9, (62.05) 11*(75.84)
D	16 (110.31)	10 (68.95)	7 (48.26)	5 (34.47)
E **	12.5 (86.18)	8 (55.15)	5 (34.47)	2.5 (17.24)
F	12 (82.74)	9 (62.05)	8 (55.15)	7 (48.26)

*For transverse stiffener welds on girder webs or flanges.
**Partial length welded cover plates shall not be used on flanges more than 0.8 inches (20mm) thick for non-redundant load path structures.

(1) Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

(2) Structure types with a single load path where a single fracture can lead to a catastrophic collapse. For example, flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, caps at single or two column bents have non-redundant load paths.

3.2.1.5 Toughness

The susceptibility of a structure to brittle fracture depends on the notch toughness of the material, temperature, flaw size, stress level, and plate thickness.

The toughness of bridge steels ensures the elastic behavior of tension members under the design stress ranges and the average minimum service temperatures. The toughness is measured by a Charpy Vee-Notch (CVN) specimen through the absorption of impact energy (foot-pounds) at temperatures higher than the service temperature. These values are considered valid due to a temperature shift and the rate of loading. The lower the temperature the more brittle the material becomes as illustrated in Figure 3.13.

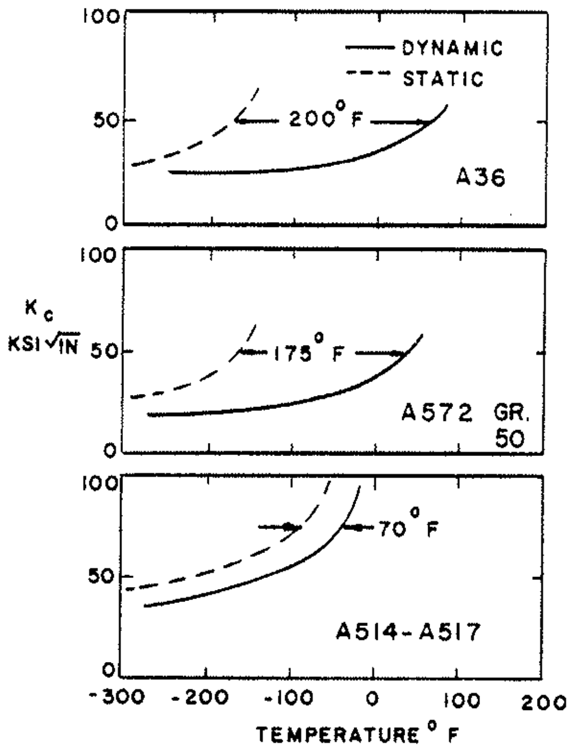


Figure 3.13[9] Loading Rate Shift for A36, A572, A514

The toughness requirement and temperature shift that have been adopted by AASHTO are considered valid by leading experts in the field of fracture mechanics and yet there are other experts who maintain that the Nil Ductility Temperature (NDT) may be well above the service temperature in the current AASHTO criteria. The designer is cautioned when designing a steel bridge to be located in extremely cold climates to use his judgment or the experiences of others when specifying impact properties and testing temperatures.

Table 1.7.2C of AASHTO[7] specifies the minimum service temperature the structure may be subjected to and the temperature zone designation--1, 2, or 3. The impact requirements will depend on the type of connection, bolted or welded, and the type of steel. For A36, as an example:

ASTM Designation	Energy Zone 1 (to 0°)	Absorbed Zone 2 (-1° to -30°)	(Ft-lb) (J) Zone 3 (-31° to -60°)
A36	15@70° (20J@21°)	15@40° (20J@4.4°)	15@10° (20J@-12.2°)

Fatigue failure generally originates in flaws or discontinuities at locations of increased stress such as at welds, arc strikes, etc. These locations, coupled with the residual stresses, are the primary cause of pop-in cracks and crack growth. The current allowable stress ranges are considered to be conservative.

In Topic 2 it was pointed out that the thickness of the material has a direct bearing on the toughness and therefore the fatigue life of a structure. Generally the thicker the material the less chance the crack tip has to deform plastically.

TOPIC 4

DESIGN CONSIDERATIONS TO MINIMIZE FATIGUE AND FRACTURE

OBJECTIVES:

To demonstrate the need for clean-cut bridge members and attachments in order to minimize fatigue and fracture.

This is to be accomplished by:

1. *Examining design considerations as per AASHTO Specifications.*

2. *Reviewing good design details with respect to fatigue.*

3. *Reviewing design details that have resulted in failures.*

4.0 INTRODUCTION

Design considerations that may minimize fatigue and fracture problems are not the same for all circumstances. Bridge engineers need a better understanding of considerations available to them so that they will more readily recognize considerations best suited for their use.

4.1 TYPE SELECTION

What does type selection have to do with fatigue and fracture problems? It may not have much effect in some cases, but in others the type selected could have a dramatic influence on the fatigue life of the bridge.

4.1.1 Bridge Type

Selection of bridge type has generally been based on site conditions, service needs, aesthetics, and economy with little or no attention to minimizing fatigue and fracture problems. Aesthetics should always be considered but it must not determine the bridge type. Any steel bridge type that meets service needs and is appropriate for the site conditions can be made aesthetically gratifying without adding unnecessary embellishments.

The type of bridge selected has seldom been based on minimizing fatigue and fracture problems. Historically, this approach to selection of bridge type has proven to be satisfactory. A review of the examples presented in Topic I shows that the type of bridge is not the source of fatigue and fracture problems.

Bridge types that are used currently for various situations are listed below and are shown in Figure 4.1.

1. Rolled Beams

2. Welded Plate Girders

3. Welded Box Girders

4. Orthotropic Deck Systems

5. Truss Bridges

6. Arch Bridges

7. Suspension Bridges

8. Stayed Girder Bridges

Current usage includes combinations of these types and a variety of versions within each type.

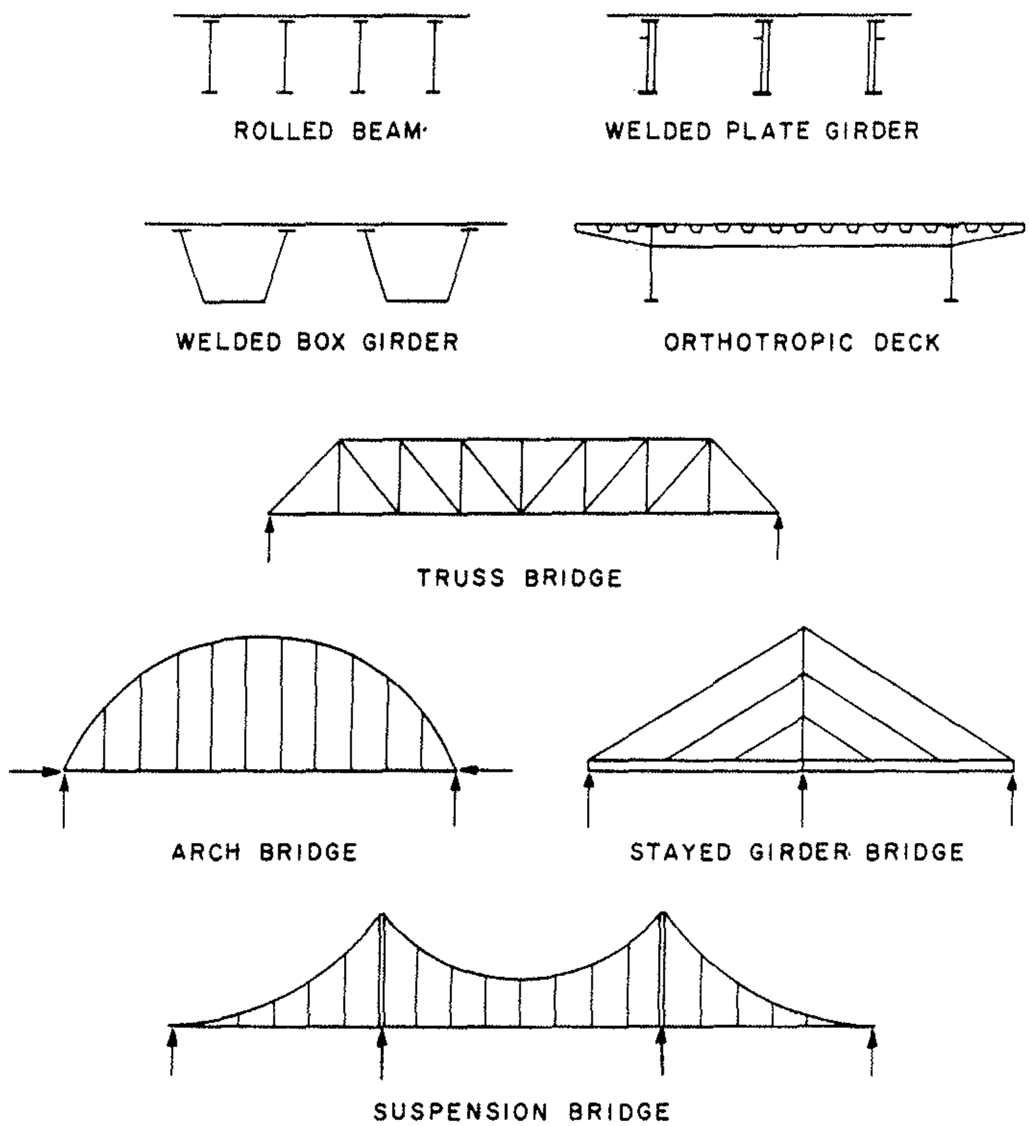


Figure 4.1 Bridge Types

In order to minimize fatigue and fracture, a bridge should have clean-cut members. The most desirable member would be the rolled section followed closely by welded built-up sections such as truss members, plate girders and box girders, free of attachments. From a practical point of view, the more welding, the greater the possibility of flaws and discontinuities.

It is unreasonable to suggest that a particular bridge type be selected on the premise that its members will be entirely clean-cut, but the designer should keep this uppermost in mind.

If the choice is between a rolled beam bridge with partial length cover plates and a welded plate girder, the plate girder would be selected under the assumption that sufficient attention is devoted to details.

Details have a strong influence in making a choice. If the designer is to choose between the plate girder and the box girder, the plate girder is usually the better choice for most cases. Box girders often have complicated details that are more difficult to design, construct, and inspect than plate girders. This is not to say that this is always the case.

Orthotropic deck systems, when compared to conventional concrete decks, are more complicated and require many more welded details. These details require extensive welding which can and do lead to fatigue and fracture problems.

Despite its short-comings, the orthotropic deck system may sometimes be appropriate. Selection of an orthotropic system for the normal highway overcrossing or undercrossing would seldom be a good choice. Experience has shown that the orthotropic deck system presents problems other than fatigue. The deck becomes dangerously slippery from frost and ice; thus, it requires extensive maintenance to keep the bridge safe. One such bridge was eventually enclosed to provide ducts for a forced air heating system.

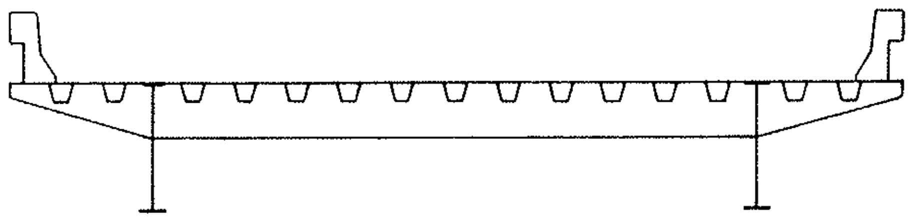


Figure 4.2 Orthotropic Deck

When spans are long enough to justify the selection of a truss, arch, or suspension type bridge, experience has shown that one of these types offers few, if any, advantages over the other with respect to fatigue and fracture. Tension members for all three types can easily be designed that are clean-cut and free of unnecessary welded connections and details.

4.1.2 Span Lengths

The length of a span is important in fatigue design, however, it is usually determined by other factors. The span length determines the type of loading (truck or lane), stress ratio, maximum allowable total stress and influences the choice of redundancy.

AASHTO Specifications explicitly call for constant stress ranges for specific details and types of loading. Not all engineers or researchers agree that a constant stress range is a safe criteria. The relationships among stress range, stress ratios, allowable total stress and redundancy with respect to span length are interesting as shown in Figure 4.3 and Figure 4.4 for simple spans only. Similar relationships exist for continuous spans, depending on span arrangements.

Figure 4.3 shows stress ratios as a function of span length.

The information in Figure 4.4 is derived from Figure 4.3. It shows the allowable fatigue stress and basic design stress for various stress ratios and span lengths.

Figure 4.4 shows that for a bridge fabricated with A36, A572 and A588 steels having a Stress Range/Maximum Stress ratio of 0.45 (R'), or less, fatigue is of no concern for Categories A, B, and C, redundant or non-redundant.

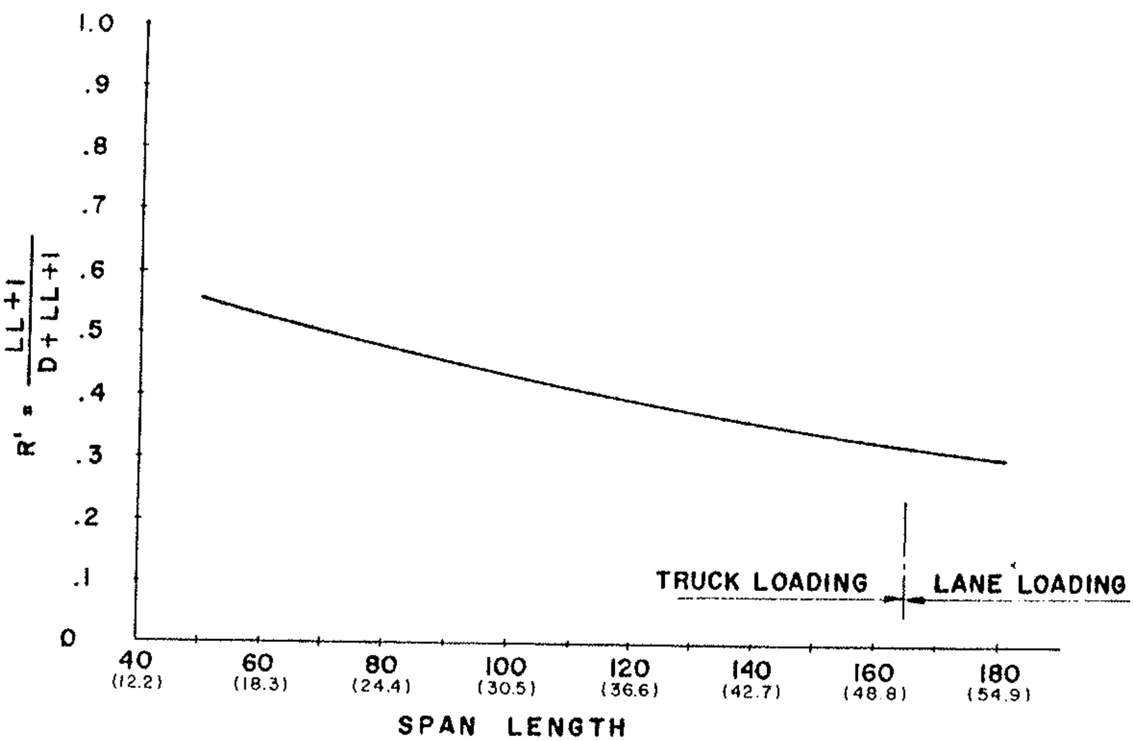


Figure 4.3 Stress Ratio vs Span Length

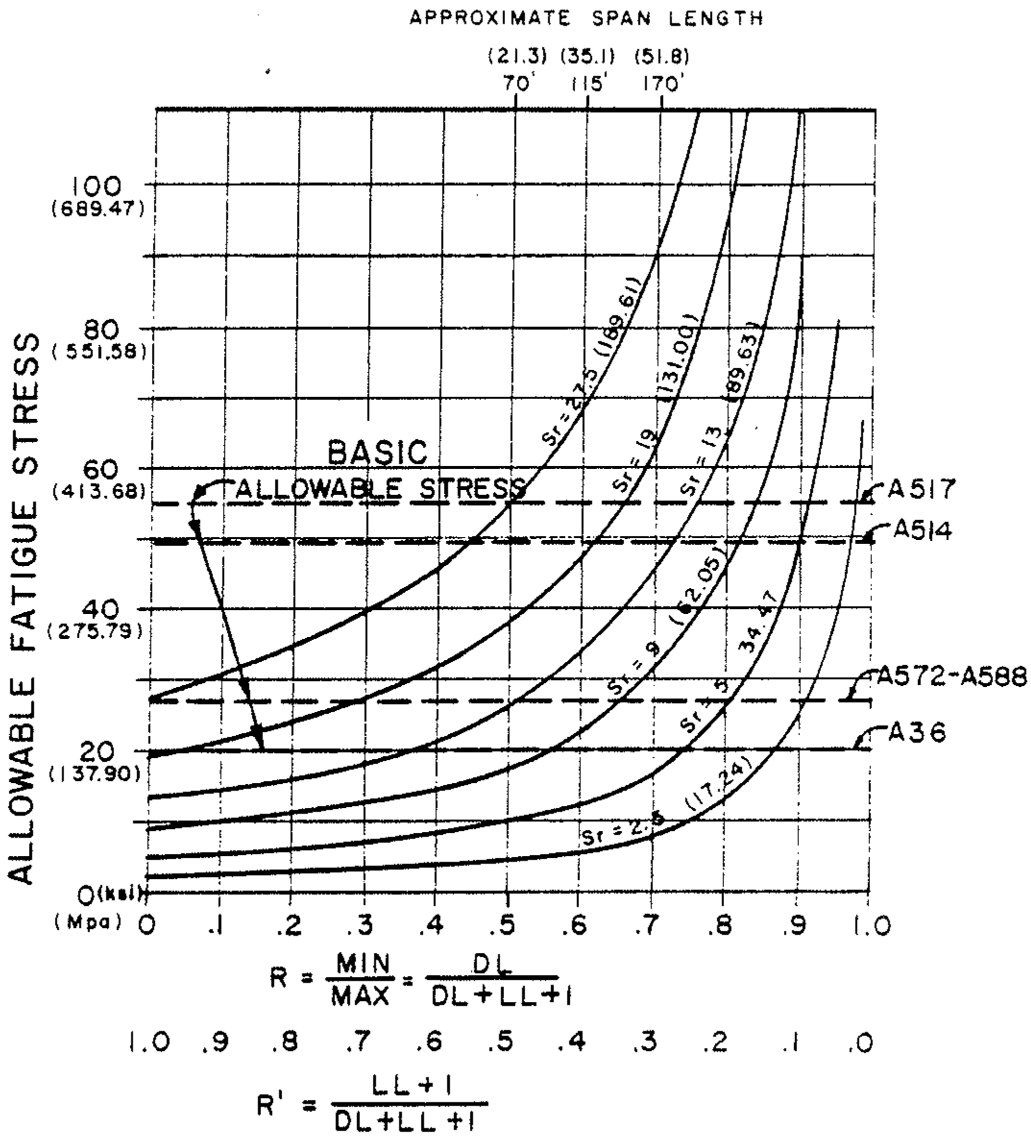


Figure 4.4 Allowable Stress as a Function of Span Length or Stress Ratio

EXAMPLE: Welded Plate Girders of A36, A514 or A588

$R' = .3$ or $R = .7$

Loading = Lane Loading Case I (AASHTO Table 1.7.2.B)

Stress Cycles = 500,000

A36

	REDUNDANT	NON-REDUNDANT
Category C	S_r = 19 ksi	13 ksi
Allowable Fatigue Stress	F_b = 63 ksi	43 ksi
Basic Allowable Stress	s = <u>20 ksi</u>	<u>20 ksi</u>

A514

Category C	S_r = 19 ksi	13 ksi
Allowable Fatigue Stress	F_b = 63 ksi	<u>43 ksi</u>
Basic Allowable Stress	s = <u>55 ksi</u>	55 ksi

A588

Category C	S_r = 19 ksi	13 ksi
Allowable Fatigue Stress	F_b = 63 ksi	43 ksi
Basic Allowable Stress	s = <u>27.5 ksi</u>	<u>27.5 ksi</u>

The basic allowable stress governs the design in all cases except for A514 non-redundant members.

4.1.3 Redundancy

Selecting a redundant structure over a non-redundant structure will decrease the likelihood of having a serious fracture problem; however, there may be a greater probability of fracture by the nature of the specifications which allow a greater stress range.

As previously stated in Topic 2, a redundant structure or member is one where in the event a member fails, the bridge or a portion of the bridge will not cause a catastrophe whereas a non-redundant structure is where the failure of a member will cause a catastropic situation.

For short to medium spans there is no problem in selecting a redundant structure with multiple beams and girders. For longer spans it may not be economical nor practical to have more than two main load carrying members. There are disagreements about the degree of redundancy of two-member systems. Actually, a two-member system may be both redundant and non-redundant. Cross framing and lateral systems often provide alternate load paths not considered in design and a failure in a region near the point of contraflexure may form a hinge.

A continuous open-top box girder with two or more cells may be considered redundant in the negative moment regions and non-redundant in the positive moment regions.

A non-redundant member may not always be better fatigue-wise than a redundant member as Figure 4.4 clearly shows. More assurance is found in clean-cut members that are easily constructed and inspected. For simple spans and portions of continuous spans, a comparison of fatigue stresses for redundant and non-redundant members shows that the basic allowable stress governs with no increase in the fatigue life due to redundancy.

The conservative fatigue design for non-redundant members has been recognized and consequently adopted by AASHTO for non-redundant load path requirements. Some researchers suggested greater toughness and others suggested a decrease in stress range to obtain greater life.

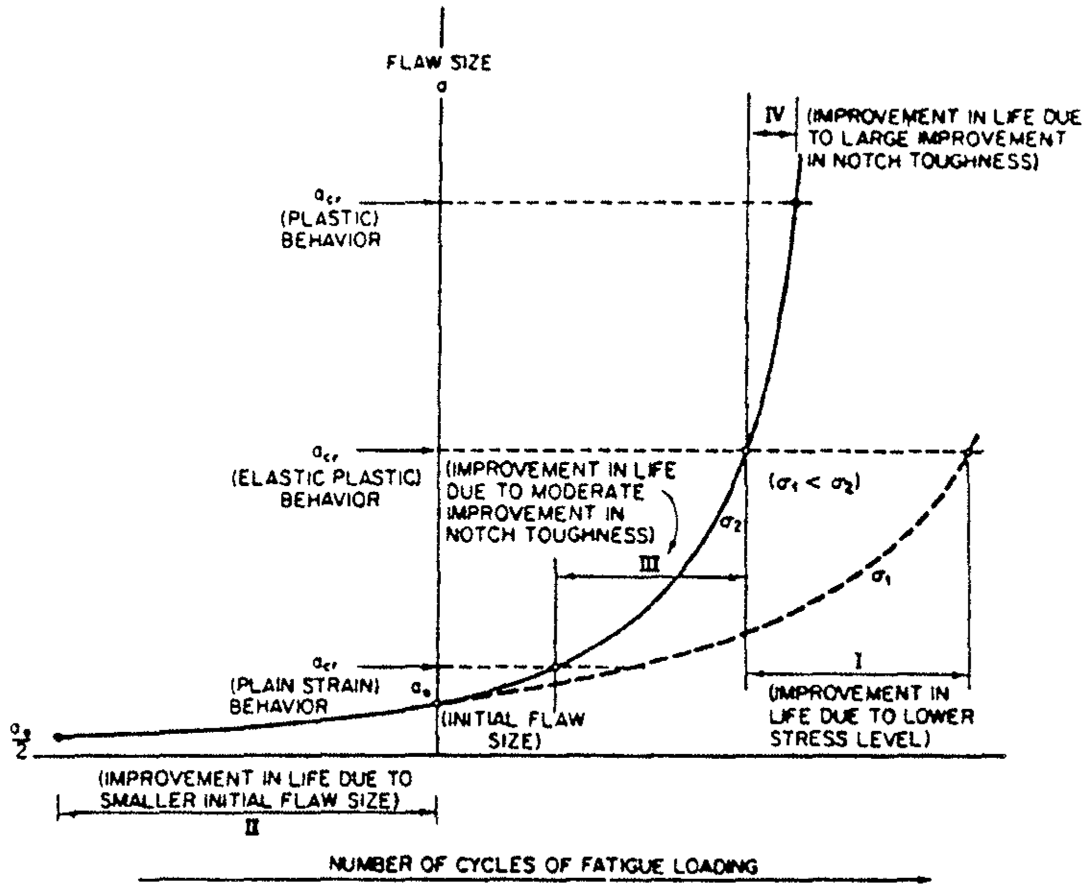


Figure 4.5[9] Fracture Mechanics for Bridge Design

A decrease in stress range was adopted under the premise that the percentage increase in life was greater than that due to an increase in toughness as shown in Figure 4.5. From the example presented, this is not always true--both proponents are correct under specific circumstances.

4.2 MATERIALS

The bridge engineer's primary interest is in the structural steels included in AASHTO; however, other steel may be used. Steels included in AASHTO are:

1. Structural Steel: ASTM A36

2. High Strength Low-Alloy Steel: ASTM A572
   ASTM A588

3. High Yield Strength Quenched and Tempered Alloy Steel:
   ASTM A514
   ASTM A517

A knowledgeable bridge engineer will consider primarily the 36 and 50 ksi yield steels; only occasionally the 100 ksi yield steel.

4.2.1 Strength Levels

A36, A572, and A588 are easily adapted to most situations.

A514 and A517 have limited use except where the dead load is the predominant portion of the total load.

All three levels may be used separately or in combinations. Mixing strength levels may be advantageous to a designer's quest for clean-cut conditions.

4.2.2 Quality

Material quality is generally thought of as the quality of the raw product. The bridge engineer considers the steel as the raw product of a bridge.

Material and construction specifications consider the quality of various steels as being equal. From the author's observations and knowledge of inspection of materials at the time of fabrication, the quality is not necessarily equal. From rolling through fabrication and erection, the probability of uniform quality favors the lower strength steels. Experienced steel and welding inspectors are more concerned about the quality of the higher strength steels.

A welding engineer has greater concern for thicker plates, and for the 50,000 psi and 100,000 psi yield steels. He recognizes these needs as they pertain to welding procedures, type of weld, electrode material, preheat, workmanship, shape of weld bead, and other factors that influence quality during construction.

Strict adherence to welding sequences and procedures is a requisite in all fabrication; this becomes more important when working with the higher strength steels.

Lack of quality workmanship may lead to transverse and longitudinal cracking in the weld and in the parent metal in the heat affected zone. Delayed cracking up to 72 hours may also be encountered.

The authors are not aware of any specifications or recommendations for different degrees of shop inspection for the different bridge steels except for radiographic and ultrasonic inspection.

Designers must recognize that the quality of materials, fabrication, and inspection differ for the various steels and evaluate these variables in accordance with design, fabrication, erection and inspection capabilities.

4.2.3 Thickness

Plate thickness was always thin with riveted construction; however, this is not the case with welded construction. Plate material is available and frequently used in greater thicknesses than is desirable. Plates are rolled up to the following thicknesses:

A36	up to 8" inclusive
A 572 Grade 50	up to 2" inclusive
A588	up to 4" inclusive
A514 and A517 100,000	up to $2\frac{1}{2}$" inclusive
A514 and A517 90,000	over $2\frac{1}{2}$" to 4" inclusive

Strict adherence to proper procedures is necessary when welding thick plates. This is not to infer that thin plates can be welded without adequate controls but rather that thick plates require more attention. With thick plates there is more weld metal, more weld bead sequences and more preheat, thus quality control and quality assurance are more difficult. Quality control and quality assurance requirements will also differ with the various strength levels of steel.

Some distinct advantages of thinner plates are:

1. Better quality

2. Greater uniformity

3. Easier workability

4. More easily fabricated

5. Easier to maintain net section with bolted splices and connections.

The majority of H-section and box-section tension members can be fabricated from plates up to $1\frac{1}{2}$ inches, and girder flanges from plates up to 2 inches.

The choice of a thin plate should be exercised. Varying the width of flanges or employment of different strengths of steel, or a combination thereof, can be used to advantage.

4.2.4 Workability

Workability affects the quality of the finished product regardless of quality control and quality assurance.

Workability as used here refers to:

1. Straightening of Plates

2. Flame Cutting

3. Weldability.

Straightening is influenced by (1) yield strength, (2) plate thickness, (3) residual stresses, (4) fabricating equipment, and (5) fabrication capabilities.

When straightening is necessary, what specifications apply? What are the heating and mechanical procedures to be followed? The techniques vary for different thicknesses and different strengths. Some steels cannot be straightened by heating without losing their characteristics. Straightening plates without appropriate controls and procedures, especially in the absence of adequate quality control and quality assurance, may end in disaster when the product is in service.

Flame cutting is influenced by (1) chemical composition, (2) uniformity in plate quality, (3) thickness, (4) cutting equipment, and (5) fabrication capabilities.

Flame cutting, whether done free hand, mechanically guided, or automatically, requires skilled craftsmen to produce a uniform squared or beveled edge. Flame cutting requires considerable experience and a good understanding of the equipment to select proper tip sizes for various plate thicknesses, to regulate the oxygen and acetylene ratio and to regulate the cutting speed.

Weldability is affected by several factors including (1) chemistry, (2) uniformity of the materials, (3) thickness, (4) Welding equipment, (5) member make-up, (6) strength level, (7) weather conditions, (8) physical surroundings, and (9) fabrication and welding capabilities.

Generally, A36 is the most workable steel as greater variations in fabrication procedures will not preclude achieving a satisfactory product.

4.3 TENSION MEMBERS

Primary tension members are usually truss members, hangers or tension ties. In addition, there are secondary members such as sway frames, cross bracing and lateral systems. The designer's major concern is with primary members and attachments thereto.

4.3.1 Rolled Sections

Rolled sections, as tension members, are sometimes used in a manner similar to that in riveted construction. A tension member may be a single rolled section or may be built up of two or more rolled sections connected with intermittent or continuous fillet welds.

Figure 4.6 shows some examples of typical rolled section tension members.

The first five of the sections shown in Figure 4.6 are considered to be clean-cut. Example 6 could be fabricated with flaws and cracks which may shorten its fatigue life.

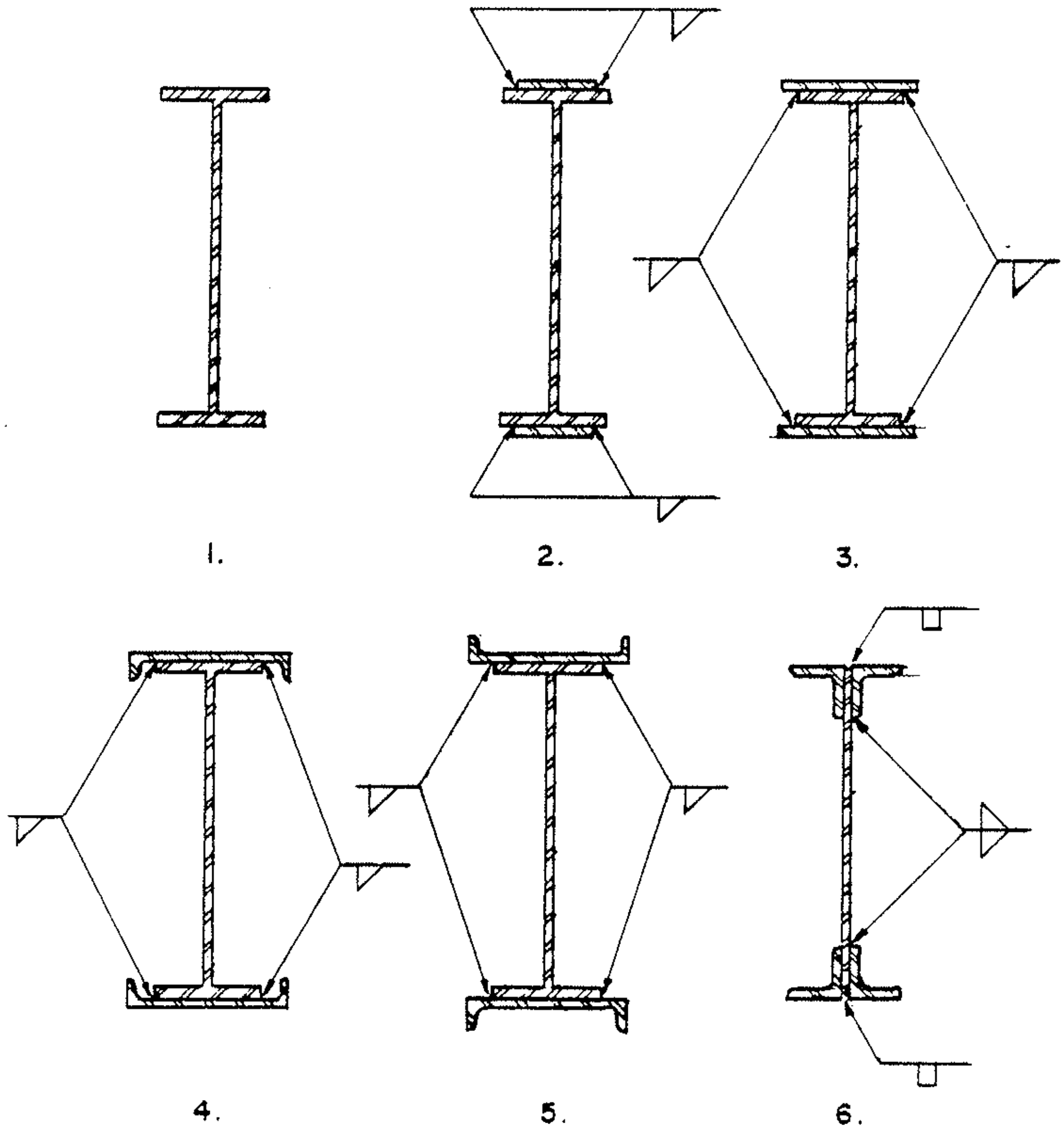


Figure 4.6 Examples of Typical Built-up Rolled Section Tension Members

Examples one through five are easily fabricated and easily inspected, provided continuous welds are specified. Intermittent welds may meet stress requirements but the probability of flaws and cracks at weld terminations is sufficient reason to prohibit intermittent welds. There is no difficulty in designing and fabricating a clean-cut tension member from rolled sections; however, once the member is selected, emphasis must be orientated toward keeping that member clean-cut, i.e., free of unnecessary connections and attachments. Connections, required area, shape and dimensions make built-up rolled sections less attractive to designers than a welded built-up plate member. Various strengths of steel can be utilized with rolled shapes but not to the extent as with welded plate members.

4.3.2 Welded Plate Sections

A common type of tension member is the welded plate section. The first major bridge in the United States to utilize this type of member was the Carquinez Strait Bridge. Tension members were mostly H shape; box sections were used also.

The availability of a wide variety of plate thicknesses and strength levels gives the bridge engineer the opportunity to design clean-cut members. The H section fastened with continuous fillet welds will meet the needs most of the time.

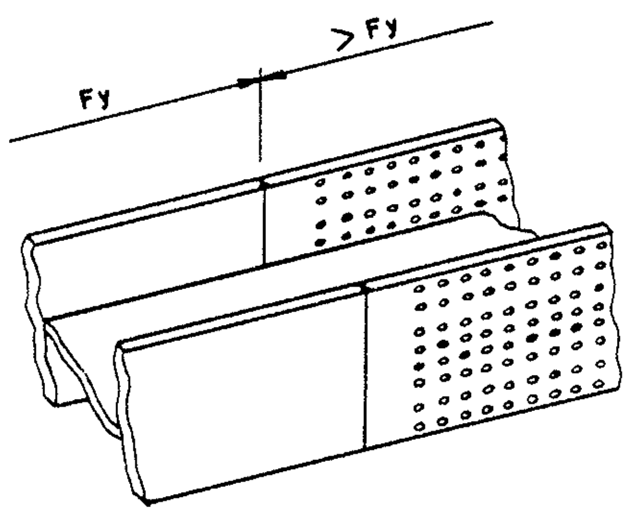


Figure 4.7 Typical H Section

Common practice is to keep the members free of attachments except at the connections. The desirability is not only for having clean-cut members but also clean-cut connections. A clean-cut welded connection is not always the answer. A well designed high strength bolted connection will often meet the needs better. Maintaining the net section with bolted connections is seldom a problem. Current AASHTO Specifications permit a reduction of 15% in gross area when using high strength bolting which minimizes the problem.

From the standpoint of fracture, a wide thin plate is more desirable than a narrow thick plate. If the size of the member introduces undesirable secondary stresses, higher strength steels may be used advantageously.

Higher strength steels and thick plates both require different fabrication and welding procedures. Indiscriminate use of thick plates, especially of A514 - A517 steel, should demand reevaluation.

A welding procedure that is prequalified for one grade of 100,000 psi yield steel may not be prequalified for a different grade due to the steel composition.

Joint geometry and plate thickness influence the mechanical properties of the weld metal and the parent metal in the heat affected zone.

Welding consumables affect the properties of a weld joint depending on the thickness of the weld joint, composition of the steel, preheat, postheat and cooling rates. This subject is discussed further in Topic 5.

Should a member go in to service with a crack, the crack will grow and, under certain circumstances, result in a failure. The addition of non-redundant criteria to the specifications does not provide an added factor of safety against fracture in all cases. Simple, clean-cut members with adequate toughness coupled with design details that are easily fabricated, erected, and inspected provide the best insurance against fatigue.

The design, fabrication, erection, quality control and quality assurance capabilities are all important aspects that must be considered in order to minimize fatigue and fracture problems.

The designer has the responsibility to produce good clean designs together with contract plans and specifications that are easily understood.

The fabricator has the responsibility to produce good clean fabrication in accordance with the contract plans and specifications. The fabricator should have the capability to monitor the fabrication with work records and to document the quality control.

Quality assurance is the owner's inspection team who should be well versed in fabrication, and in destructive and non-destructive testing techniques.

Erection procedures, including field welding, should be fully understood by both the owner and the contractor with approved quality control and quality assurance programs that include records that document completely the inspection and non-destructive testing.

4.4 BEAMS AND GIRDERS

The majority of steel bridges have either rolled beams or plate girders. There are a few bridges with box girders, and a few with orthotropic deck systems.

4.4.1 Rolled Beams

Rolled beams were common for short to medium spans in the '50s prior to the emergence of welded plate girders. Their use has continued in some regions of the United States and decreased in other regions.

The rolled beam is an excellent member with long fatigue life. It is what is done by design and during fabrication and construction that reduces its fatigue life. Partial length cover plates are recognized as a definite potential source of failure.

The fatigue and fracture problem associated with simple span rolled beams with partial length cover plates, non-composite and composite, can easily be corrected. Full length cover plates offer a simple solution. The cover plate may be of uniform size or made up of plates of different thicknesses and widths that are butt welded prior to attachment with continuous fillet welds.

Welded attachments to beam webs have caused fractures. Welded attachments can be eliminated through utilization of high strength bolted connections.

Fabrication errors and slipshod workmanship may be classified as a quality control and quality assurance problem. The fabricator must be made fully aware of the consequences of careless work.

Continuous rolled beam spans with cover plates are not modified as easily as the simple span.

Beams with cover plates in negative moment areas can be eliminated by mixing variable weight beams, beams with different strength steels, or both. There is also the opportunity to mix the rolled beam with welded plate girder construction, utilizing the plate girder in the negative moment region as shown in Figure 4.8.

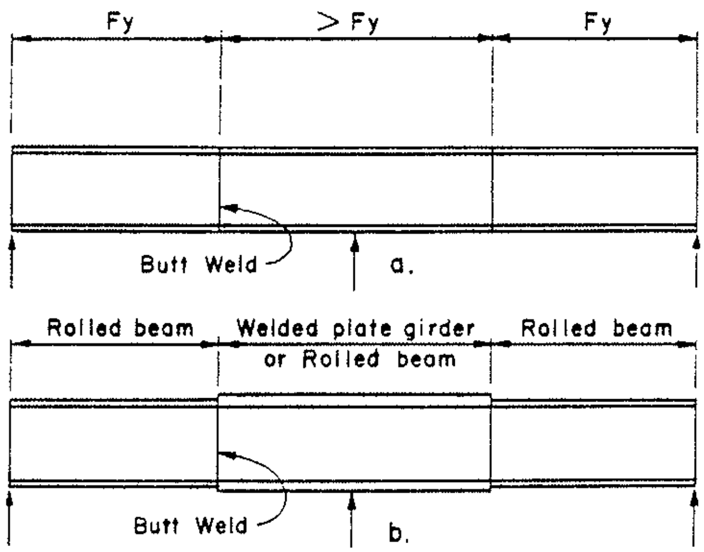


Figure 4.8 Continuous Rolled Beams

Cover plates, splices, and attachments near the points of contraflexure have not been given adequate consideration. Some bridge engineers recognize that this area deserves greater attention and have eliminated all unnecessary welds in order to minimize fatigue and fracture problems.

4.4.2 Welded Plate Girders

The welded plate girder is a very versatile bridge member. Various design criteria are permitted within AASHTO Specifications that will prolong its fatigue life. The designer must visualize the final product so that the performance will be as assumed in the design with special attention to rigidity at connections, out-of-plane bending, and secondary stresses.

AASHTO Specifications include provisions for the following:

1. Redundant or Non-redundant Girders

2. Three Strength Level of Steels

3. Simple or Continuous Spans

4. Non-Composite or Composite

5. Uniform or Variable Depth Girders

6. Flanges - Uniform or Variable Width

   Flanges - Various Thicknesses

7. Webs - Various Thicknesses

   Webs - With or Without Vertical Stiffeners

   Webs - With or Without Longitudinal Stiffeners

8. Girders, Homogeneous or Hybrid

9. Cover Plates

10. Welded or Bolted Girder Splices

11. Welded or Bolted Connections

12. Vertical Stiffeners - Detail Options

13. Longitudinal Stiffeners - Detail Options

14. Various Shear Connection Details

15. Fatigue Requirements for Redundant and Non-Redundant Girders

16. Straight Girders or Curved Girders.

Designers are expected to select girder details that best fit the conditions. They must recognize the fact that details permitted by the specifications are not always practical and in some cases may be undesirable. The plate girder described here includes webs, transverse stiffeners, longitudinal stiffeners, flanges, connections and splices.

A welded plate girder without stiffeners and attachments is an excellent choice to minimize fatigue and fracture; however, it may be impractical from an economic standpoint. Fabrication costs will decrease when stiffeners are eliminated but the decreased fabrication costs may be offset by the expense of additional steel in a thicker web. Web thicknesses can vary to satisfy shear stresses while keeping weight at a minimum.

Girder webs without stiffeners have been offered as an alternative for economic reasons. This detail could be utilized for fracture critical members. Although it has been offered as an alternative, fabricators have seldom, if ever, exercised such an option. There is an indication that had the plans shown girders without stiffeners, and the girder with stiffeners provided as an alternative, the choice may have been the opposite.

One fabricator's comment on his decision to use stiffeners was, "Our business is labor, not selling steel." Consequently, if the cost is the same, the detail with more labor is obviously selected.

Two items often overlooked are the cost of quality assurance and the possibilities of flaws or cracks caused by welding the stiffeners. The elimination of stiffeners is not a reasonable consideration for longer spans.

4.4.3 Vertical Stiffeners

Several details related to vertical stiffeners that are shown in Figure 4.9 are permitted; they all fall into Category C. They may be equal according to the design specifications, but there exists the probability that the details with welds across the tension flange will produce more defects than other details.

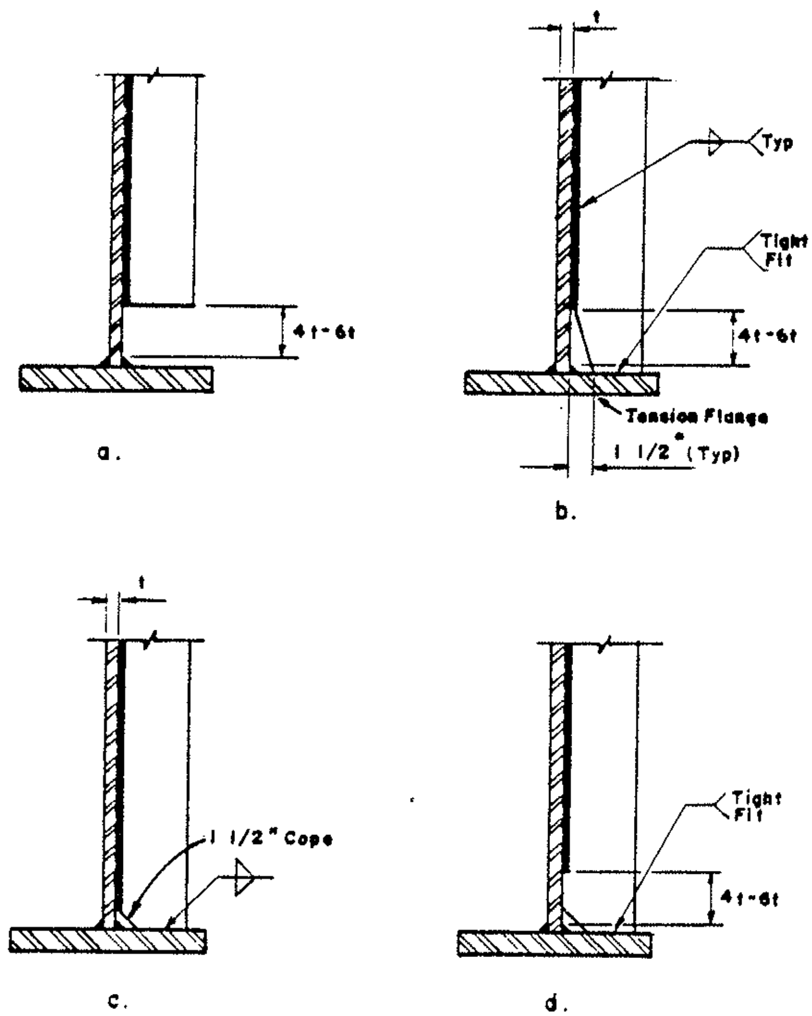


Figure 4.9 Vertical Stiffener Details

Although welding across tension flanges with fillet welds is permitted by the AASHTO Specifications, this practice is not accepted in some states nor by many designers.

Once the decision is made to use vertical stiffeners it then becomes important to select stiffener details that will present the least problem with respect to fatigue and fracture.

An evaluation of the choices indicates that full length stiffeners welded to the web and the compression flange with tight fit at the tension flange as shown in Figure 4.9, Detail (b), is the most desirable.

Detail 4.9 (a). Stiffeners cut short at the tension flange would be the author's second choice; however, this detail has sometimes given problems in transportation and handling. When transported or handled, the girder webs flex between the bottom flange and the cutback stiffeners to the extent that cracks have occurred in the webs at the termination of the stiffener to web fillet welds. Of course, there is also the possibility that the weld terminations are faulty due to weld crater cracks, not finishing the weld, leaving the weld crater exposed, insufficient weld, and unfused area at the root of the weld termination. These conditions are illustrated in Figures 4.10 and 4.11. Often the welder makes a wrap around weld at the stiffener termination, undercutting the sides and end; or the wrap around weld may be deposited on the web only and miss the stiffener entirely as shown in Figure 4.12.

Detail 4.9 (b). A minimum cope of between 4t - 6t from the flange to web weld is provided at the tension flange in order to provide sufficient space to inspect the termination of the stiffener to web welds. It is customary to use full depth

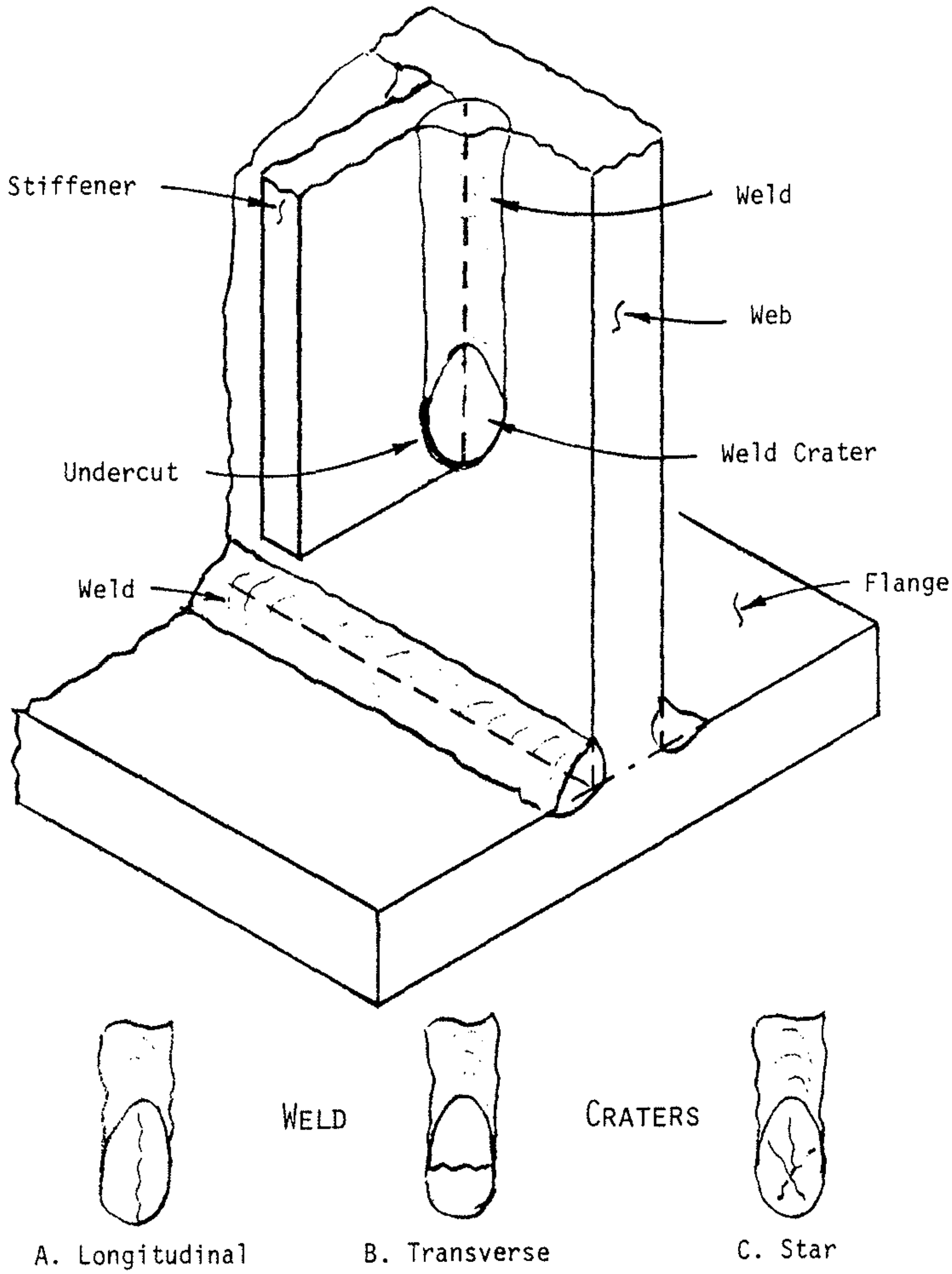


Figure 4.10 Three Types of Weld Crater Cracks

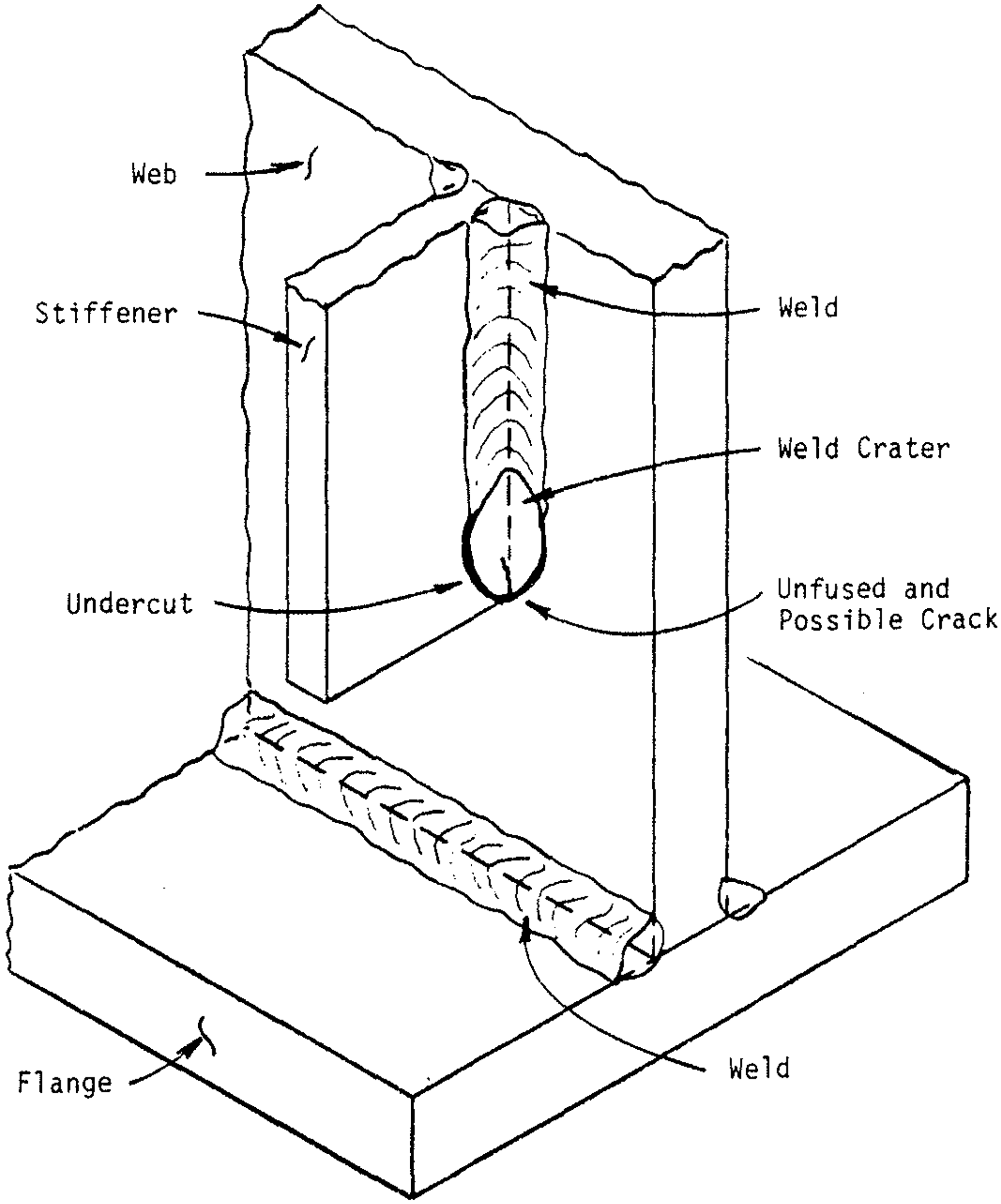


Figure 4.11 Insufficient Weld at Weld Crater

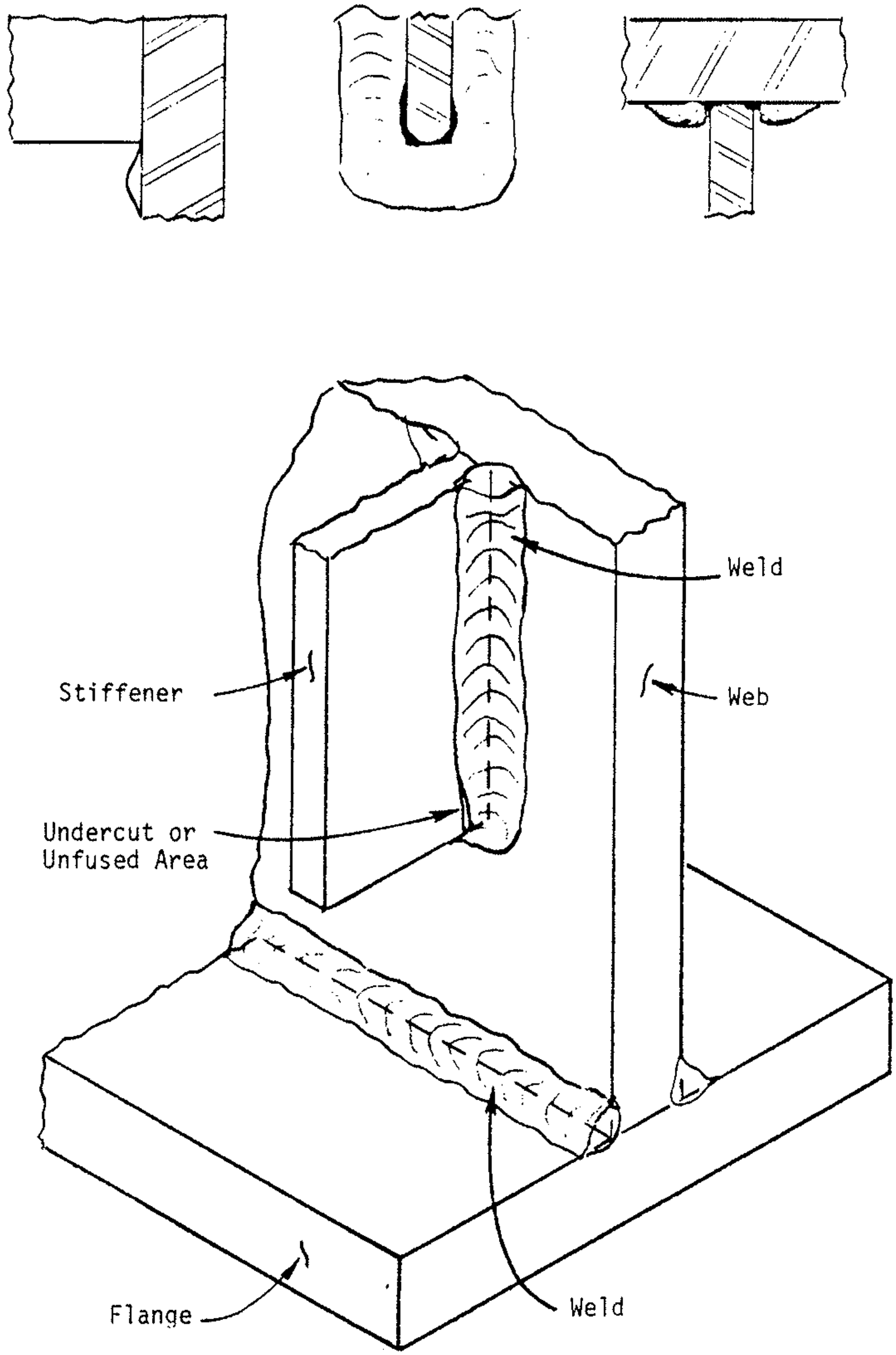


Figure 4.12 Wrap Around Weld at Stiffener Termination

stiffeners in order to keep the flange at right angle to the web. This detail prohibits any welding across tension flanges which reduces weld defects, flaws, or cracks. Crater crack, undercut, and insufficient weld are some of the conditions that may exist at the termination of the fillet welds in detail "b." Figure 4.13 shows the fillet weld termination to be below the intersection of the stiffener cope. The weld should be inspected for undercutting, crater cracking, and slag inclusions.

Terminating the fillet weld above the intersection as shown in Figure 4.14 (a) leaves a weld crater with insufficient weld metal. This condition is a possible crack starter. The crack may originate from the weld crater at the root of the weld where the stiffener edge joins the web.

Some designs have called for the stiffener to be notched at the top of the cope as shown in Figure 4.15 in order to terminate the fillet weld. The weld should be properly finished.

Detail 4.9 (c). The same conditions exist for Figure 4.9 detail (c) as in details (a) and (b). However, an additional factor has been added and that is the welding of the stiffener across the tension flange. These weld connections need special attention to assure there are no weld defects that will go in to service. Quality control and quality assurance should be very conscious about such welds by looking for weld metal cracks, fillet weld toe cracks, lack of fusion, lack of penetration, and slag inclusions. Such welds do not lend themselves to be either radiographically or ultrasonically tested with any confidence; thus, visual inspection should be supplemented by magnetic particle and dye penetrant testing methods. Prior to testing, the welds should be ground to a smooth contour.

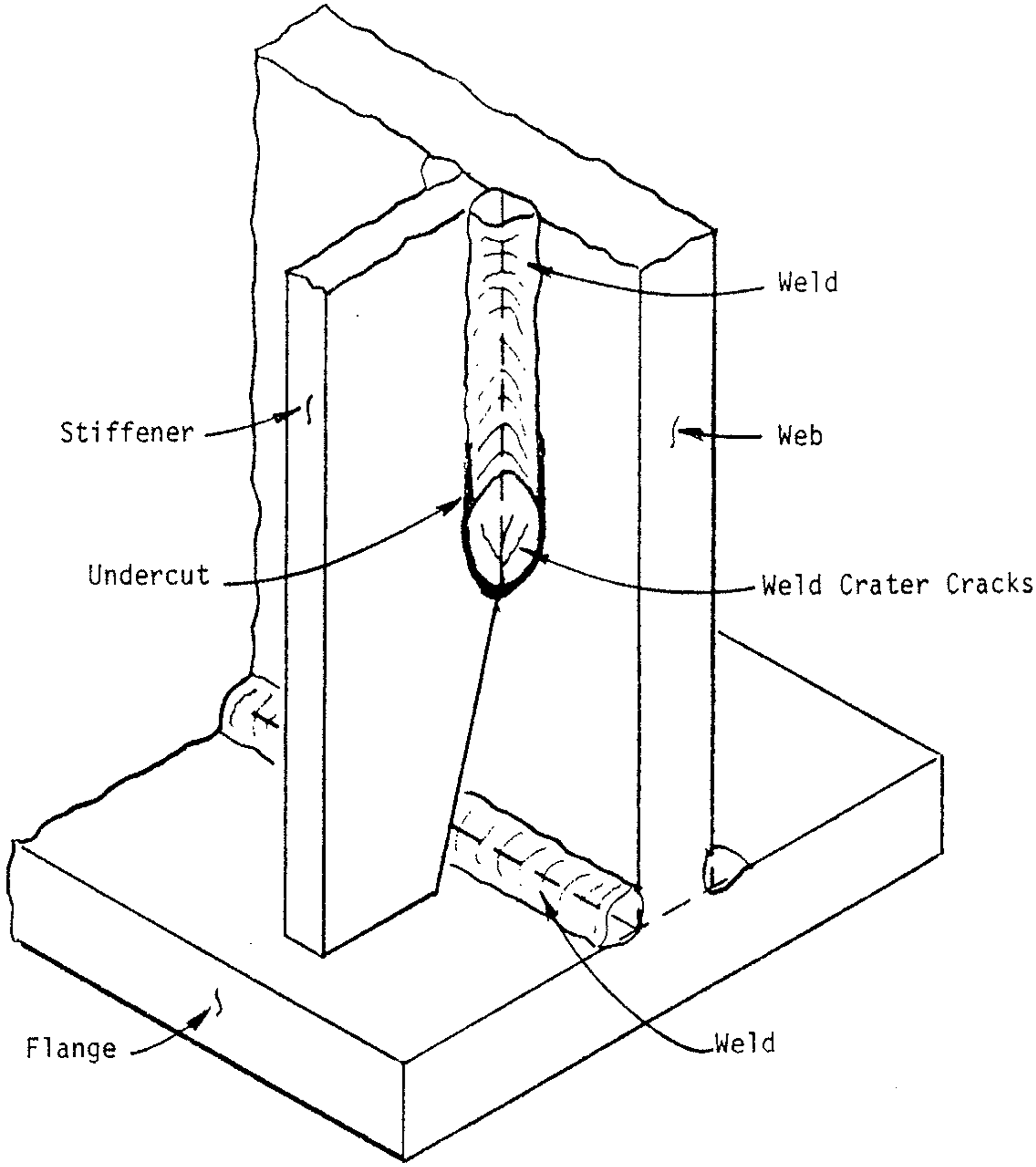


Figure 4.13 Fillet Weld Termination Below Stiffener Cope

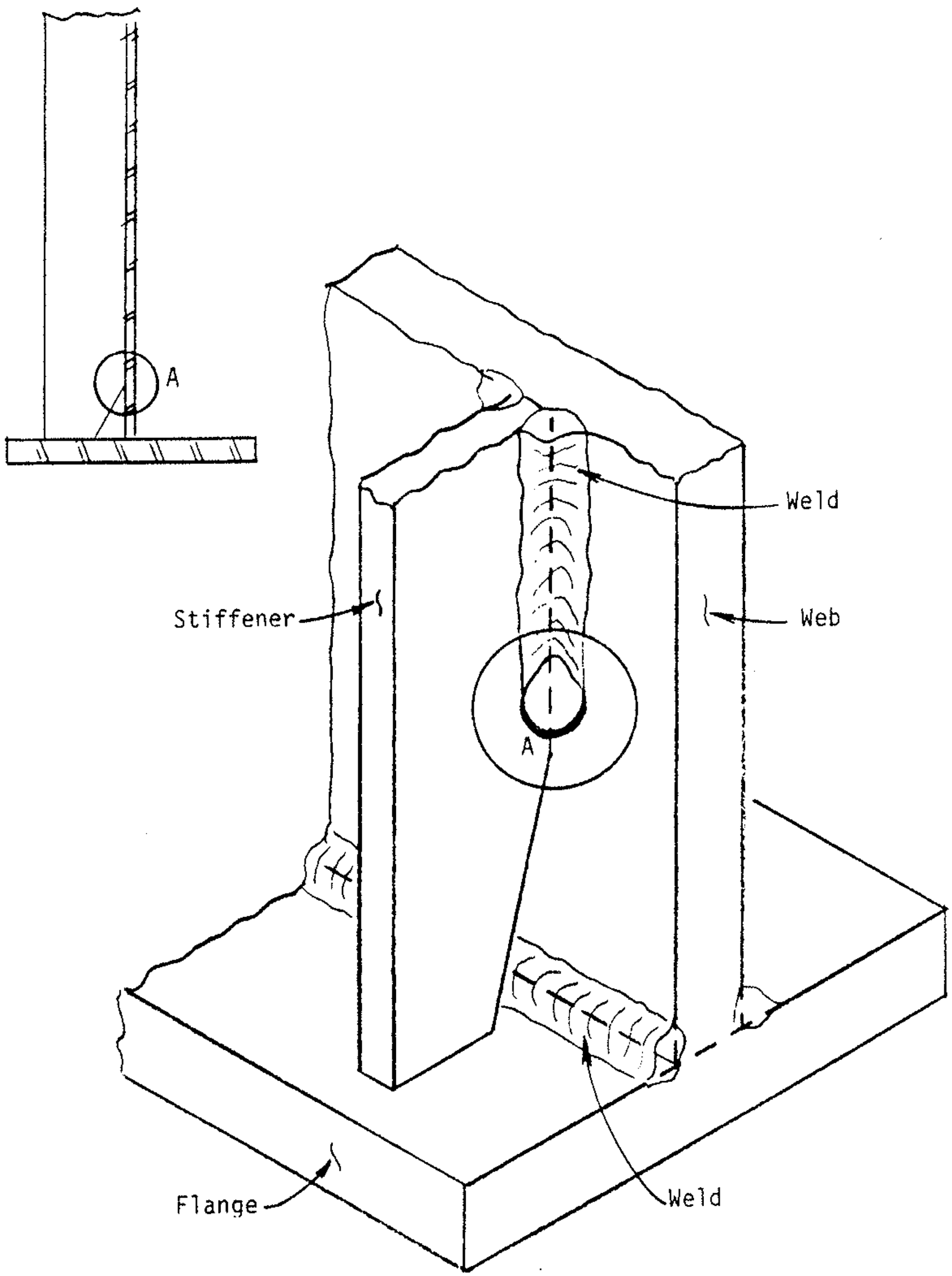


Figure 4.14 Fillet Weld Termination Above Stiffener Cope, No notch at A

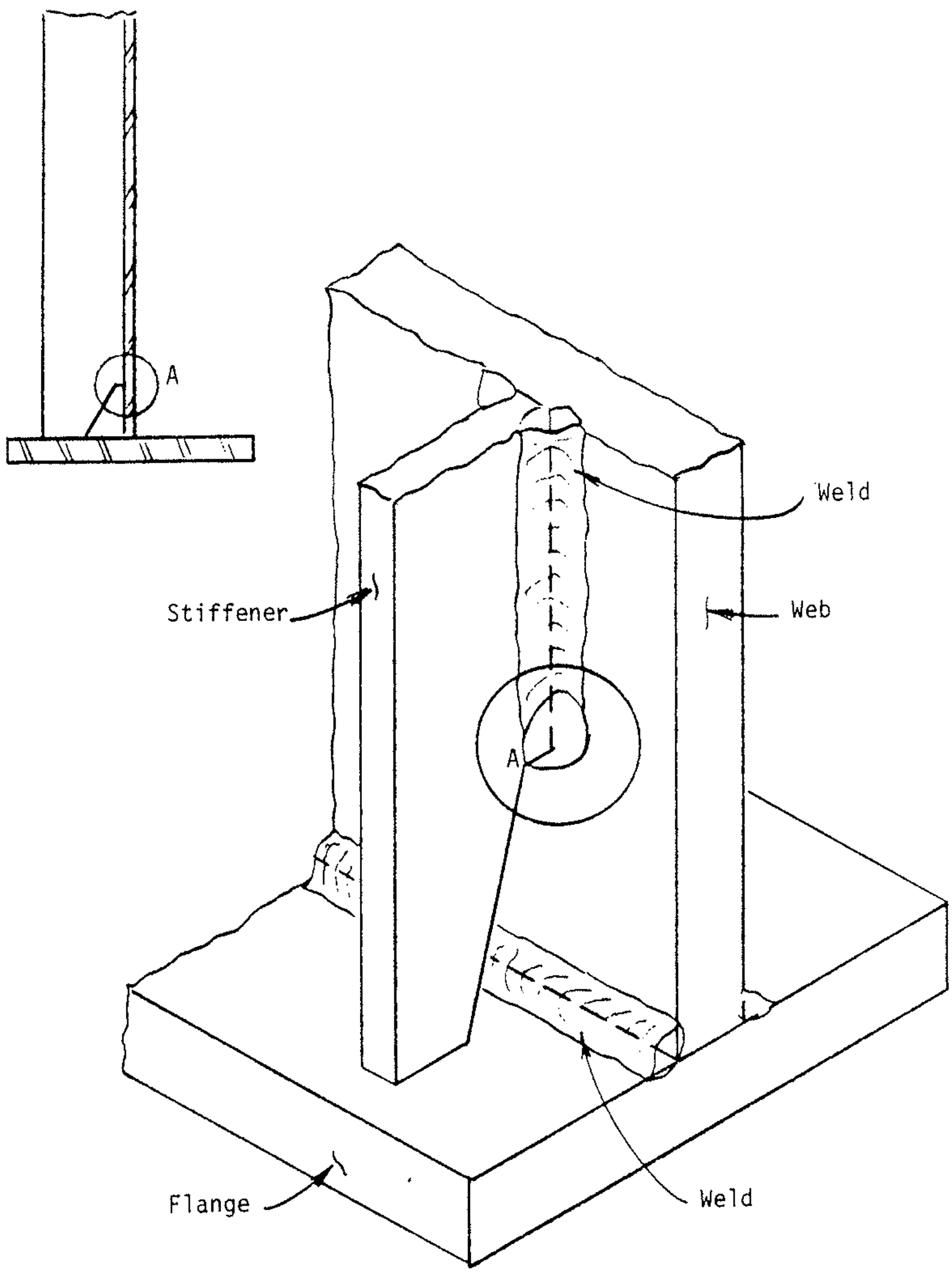


Figure 4.15 Fillet Weld Termination at Notch A

Figure 4.16 shows various types of defects one could encounter in these relatively short fillet welds across tension flanges.

Figure 4.16(A)	1.	Shrinkage crack in weld metal
	2.	Check crack in weld metal
Figure 4.16(B)	1.	Root crack
	2.	Under bead crack
Figure 4.16(C)	1.	Rollover or overlap
	2.	Toe crack
Figure 4.16(D)	1.	Slag inclusion at root of weld
	2.	Undercut--a probable cause for cracking or a crack starter
Figure 4.16(E)	1.	Slag inclusions and/or lack of fusion resulting from too low an amperage
	2.	Defective weld profile, insufficient weld leg on web
Figure 4.16(F)	1.	Shrinkage crack extending to root weld may or may not extend into base metal
Figure 4.16(G)	1.	Check cracks. Short, discontinuous, very fine, and hard to detect
	2.	Transverse weld crack may or may not extend into base metal

This detail has problems also of crater cracking, undercutting, and lack of fusion.

Detail 4.9 (d). Terminating the weld between 4t and 6t from the flange, Figure 4.9(d) could pose the problem of the unwelded

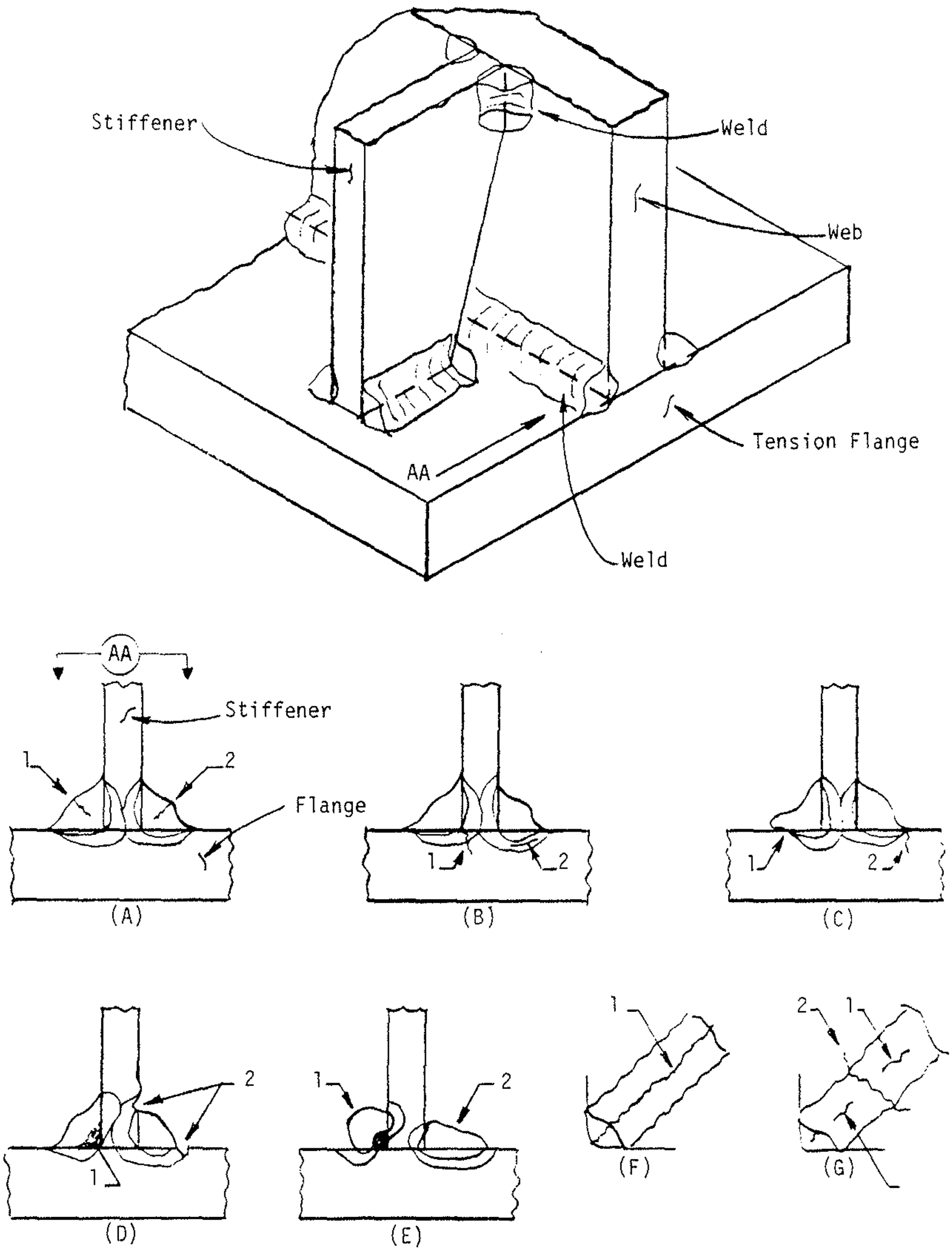


Figure 4.16 Fillet Weld Defects

portion to act as a crack starter and as an area for corrosion. Figure 4.17 shows some of these areas.

These potential fabrication problems will be discussed further in Topic 5.

4.4.4 Longitudinal Stiffeners

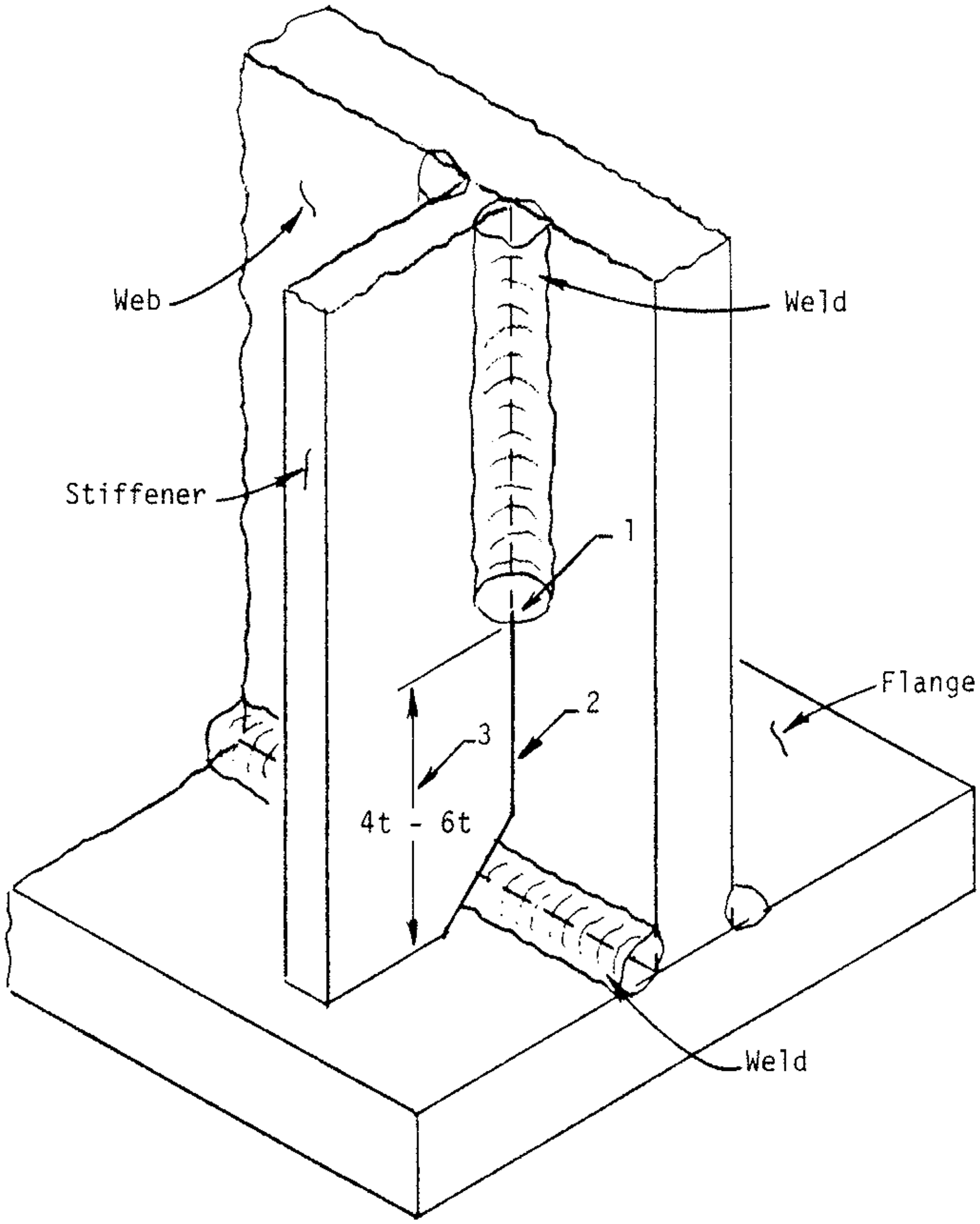
Longitudinal stiffeners act as reinforcement for the web in compression. While this indicates there should be no problem, one frequently develops from poor stiffener details and terminations in areas where the web is in tension.

For simple span plate girders where longitudinal stiffeners are located on the compression portion of the web only, there are no problems; however, on occasion, additional longitudinal stiffeners have been added in tension areas for aesthetics. Whether this is good aesthetic treatment is debatable; however, if these stiffeners are added, they should be treated as tension flanges; any welding on these stiffeners should be treated accordingly.

For continuous spans, failures have originiated from longitudinal stiffeners due to poor details and/or weld defects.

The longitudinal stiffener for positive moment area is either continued through the negative moment area for aesthetics or ended in an area near the point of contraflexure. In either case, it presents a possible fatigue or fracture problem.

Girders are generally designed with transverse stiffeners on one side and longitudinal stiffeners on the opposite side. This makes one face free for longitudinal stiffeners except for the bearing and cross frame stiffeners. If the longitudinal stiffeners are placed



1. Crack starter at unwelded portion of stiffener to web.

2. If unwelded portion does not fit properly and a gap exists, cracking could propagate. Gap is a corrosion area.

3. The welder will miss the 4t to 6t limitation unless under strict quality control.

Figure 4.17 Weld Termination at 4t to 6t on the inside face of exterior girders, the longitudinal and transverse stiffeners intersect resulting in a Category E detail which should be avoided.

The purpose of the longitudinal stiffeners and the need for clean-cut members are sometimes forgotten. With these two things in mind, the designer should strive to place longitudinal stiffeners only where required.

Figure 4.18, detail (a), is the most commonly used detail for longitudinal stiffeners.

Termination of welded longitudinal stiffeners in a tension area creates a fatigue problem. The problem can be eliminated by terminating the welded stiffeners in a compression area and using a bolted detail through the area of stress reversal as shown in Figure 4.18, detail (b). The bolted stiffener can be placed on the inside of exterior girders for aesthetics.

A second solution is to run the stiffener continuously through the area of stress reversal then curve toward the neutral axis for termination, as shown in Figure 4.18, detail (c).

Another solution, with adequate design consideration for the allowable stress range, is to continue the stiffener through the area of stress reversal, terminate with a radius as shown in Figure 4.18, detail (d), grind the weld termination, and inspect by non-destructive techniques.

Continuous longitudinal stiffeners intersect with transverse vertical stiffeners at cross-frame locations. The longitudinal stiffeners, viewed as a secondary element, is, in actuality, a supplemental flange. Discontinuities at vertical stiffeners result in high stress

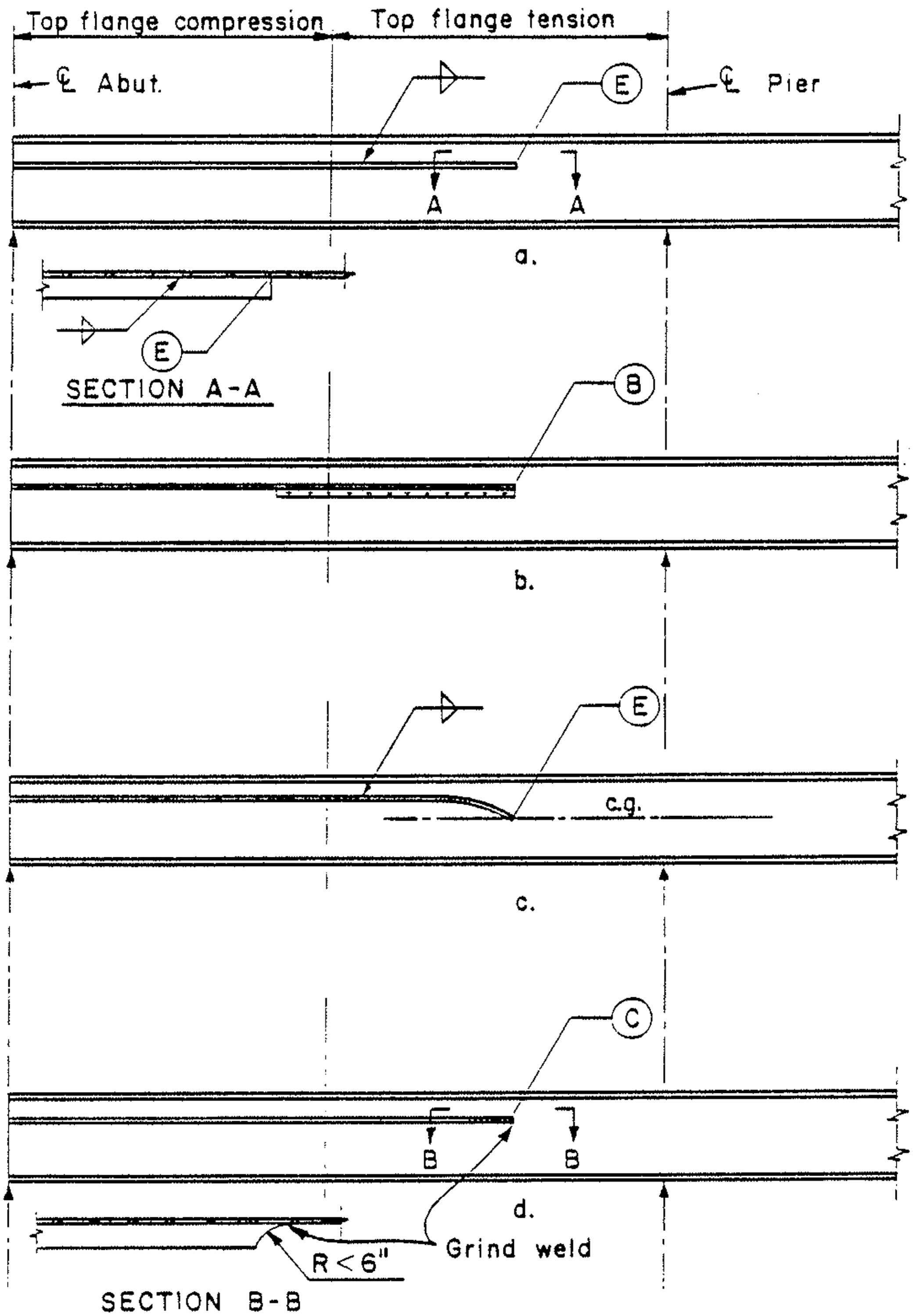


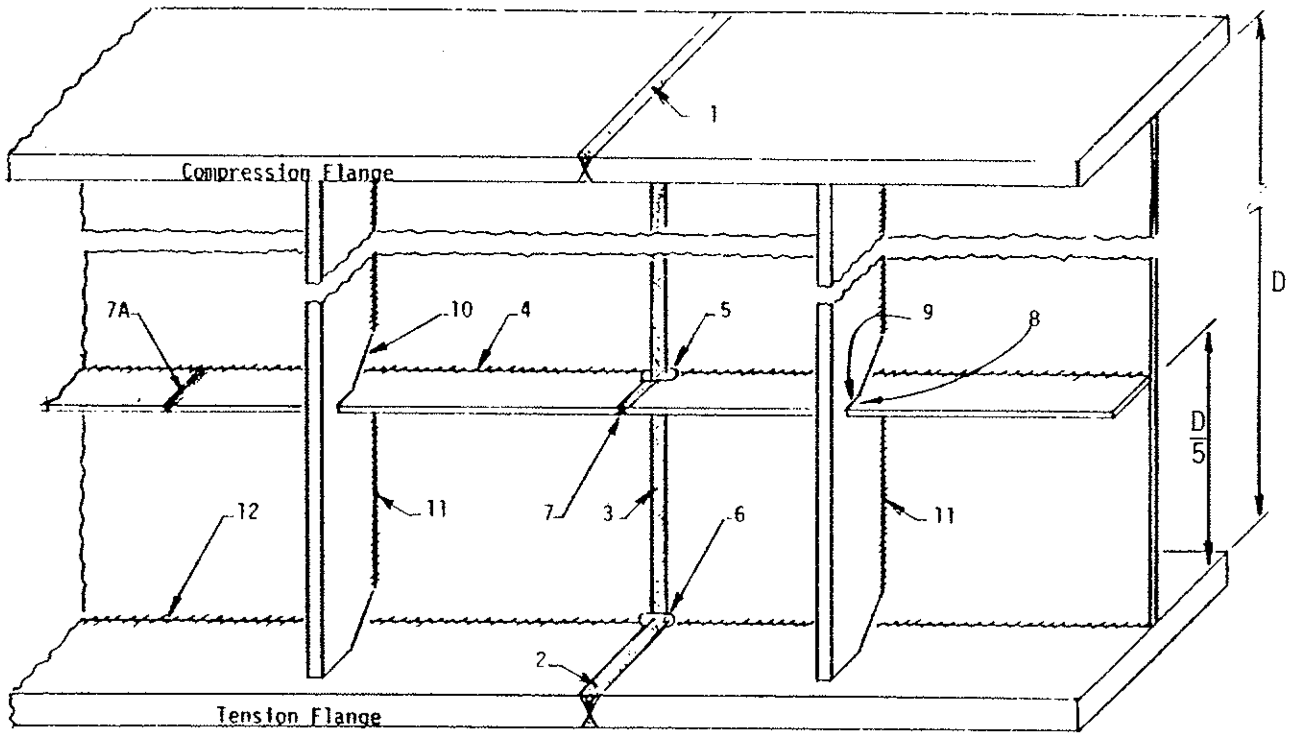
Figure 4.18 Longitudinal Stiffener Detail

concentrations at the weld terminations which are vulnerable to cracks. A practical consideration should be given as to how the welded stiffener performs. The longitudinal stiffener acting like a tension flange should be run continuously and the vertical stiffeners should be cut so that they have a tight fit on each side with cope holes for the longitudinal welds. This changes the condition from Category E to Category C. (The stress range for Category E = 8,000 psi and for Category C = 13,000 psi, assuming non-redundancy.) The welded girder without vertical or longitudinal stiffeners falls into Category B, with vertical stiffeners into Category C, and with longitudinal stiffeners terminated in a tension area into Category E; therefore, discontinuous longitudinal stiffeners should be avoided in tension areas.

As shown in Figure 4.19, the longitudinal stiffener butt weld (7) at a girder splice should be welded after (1), (2), and (3). This weld should be inspected and tested to the same conditions as for the tension flange weld (2), with all cope holes ground smooth and radiused. Fillet welds should also be magnetic particle and dye penetrant tested at regular intervals in areas of concern. A complete girder assembly drawing and welding sequence drawing should be required for all parts on bridge members no matter how insignificant the part may be.

Longitudinal stiffeners which are discontinuous and welded to vertical stiffeners as illustrated in Figure 4.20 (1) result in high stress concentrations and are vulnerable to weld cracking, especially in the tension areas. Even in a compression area, this type of welded connection exhibits weld cracking.

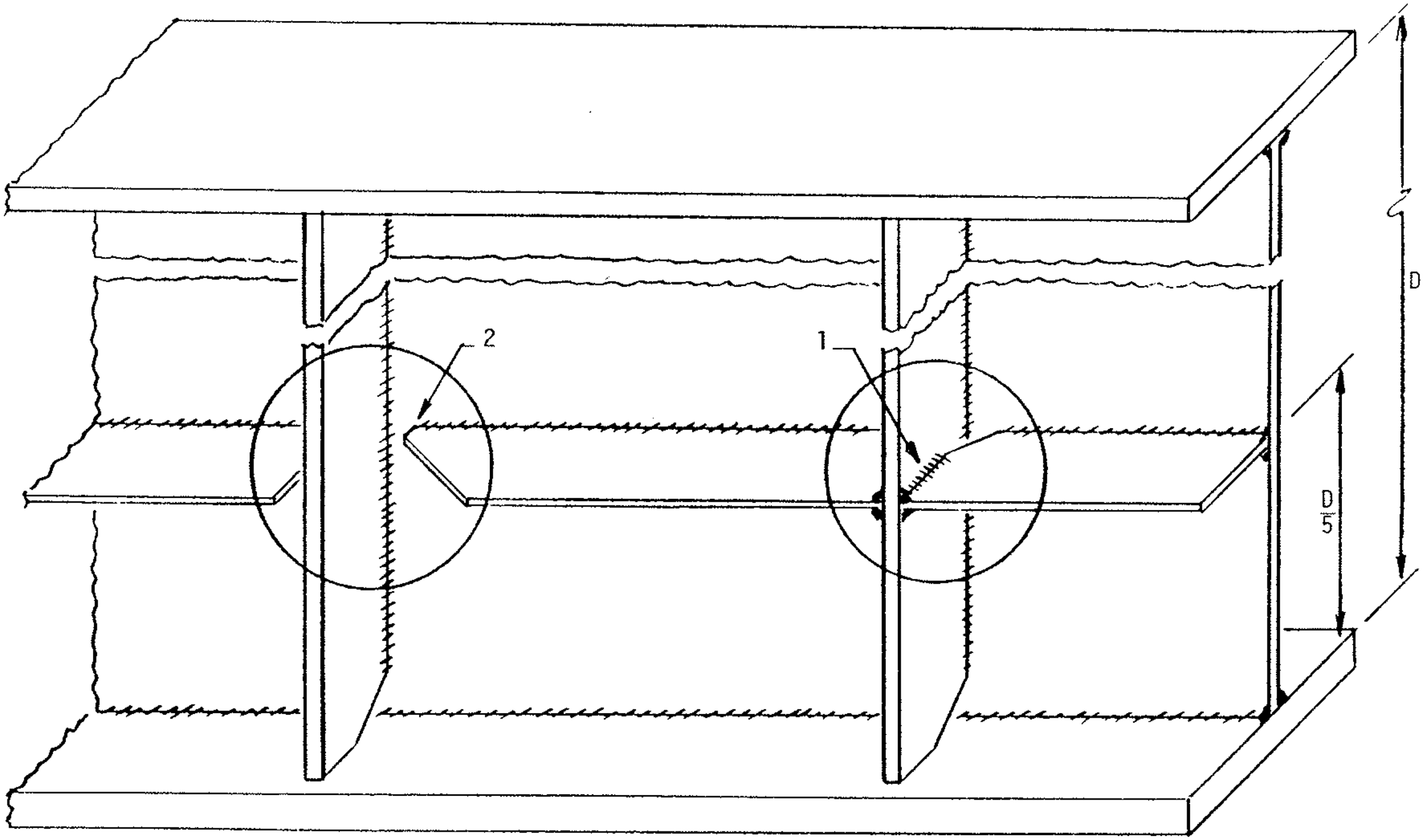
Longitudinal stiffeners which are discontinuous but cut short of vertical stiffeners as shown in Figure 4.20 (2) should be used in the compression areas only.



1. Top Flange Butt Weld (Compression)
2. Bottom Flange Butt Weld (Tension)
3. Web Butt Weld
4. Longitudinal Stiffener Fillet Weld
5. Longitudinal Stiffener Cope Hole
6. Web Cope Hole
7. Longitudinal Stiffener Butt Weld (Tension)

7A. Longitudinal Stiffener Butt Weld (Shop)
8. Longitudinal Stiffener Continuous (Tension)
9. Vertical Stiffener Cut for Tight Fit for Longitudinal Stiffener Pass Through
10. Vertical Stiffener Cope for Longitudinal Stiffener Weld
11. Vertical Stiffener Weld
12. Flange to Web Weld

Figure 4.19 Welded Plate Girder Splices and Connections



1. Longitudinal Stiffener--Discontinuous (Welded)
2. Longitudinal Stiffener--Discontinuous Cut Short of Vertical Stiffener

Figure 4.20 Longitudinal Stiffener End Detail

NOTE: When welding longitudinal stiffener splices and terminating vertical stiffener welds as well as longitudinal stiffener welds, the welding process most often used is the shielded metal arc. This process requires highly skilled welders and special techniques. Quality control and quality assurance personnel should always be conscious of these conditions.

4.4.5 Flanges

Plate girder flanges do not present problems unless there are unnecessary or undesirable attachments or connections. Problems are usually related to flanges that are thick, say 2" to 4", whereas thin tension flanges are likely to be more uniform in quality, be more workable, have better weldability, and, for the same size flaw or crack, have more life--assuming comparable stress and toughness.

Different strength levels should be considered during design in order to maintain thin flanges throughout. The steels included in AASHTO are considered readily weldable only if proper welding procedures are followed. This means welding capability, quality control and quality assurance become more important. Some welders fail to recognize the importance of the differences in welding procedures for the different steels and thicknesses.

4.4.6 Welded Box Girder

For conventional use, the welded box girder is not the most economical. The curved box girder structures may be justified for their inherent torsional strength. Long span structures, up to 1,000 feet, have been built using box girders.

There are many choices for the cross section of the box; single box, single box with multiple cells, multiple boxes, open or closed tops, orthotropic decks and concrete decks.

Field splices are complicated by the many elements that make up the box, such as longitudinal flange and web stiffeners; thin, wide bottom flanges and sloping sides. Field welding of the longitudinal flange stiffeners after the web and flange splices have been made is a potential source of cracking. Therefore, the design detail together with the assembly and welding sequences should be given special attention. Connections at the intersection of the longitudinal and transverse stiffeners in the bottom flange tensile zone should be made by bolting.

There are cases where field welding of the longitudinal flange stiffeners has taken place after the welding of the web and flange splices. These areas may be a source of cracking due to shrinkage from welding and they involve difficult welding procedures.

Welding sequences, preheat and/or postheat along with a complete quality control and quality assurance program using nondestructive testing should be employed. Quality control and quality assurance testing may be required for as much as 100 percent of the longitudnal flange stiffener welds.

The box girder requires heavy cross frames or diaphragms at the piers to prevent warping. A combination of welding and bolting may be necessary in order to prevent welding to the tension flange.

4.4.7 Orthotropic Deck

The orthotropic deck type bridge has not gained in popularity in this country as it has in Europe. The advantage of the orthotropic deck is that: (1) the deck is effective in the top tensile zone and (2) there is a reduction in dead-load weight when compared to a concrete deck.

For long span bridges the savings in weight can reduce the foundation costs. There is a saving in the structural steel but this is offset by higher fabrication costs.

The cost of an orthotropic deck may depend on the ingenuity of the fabricator and the quality of his equipment. Only the most modern and most reliable welding techniques must be employed in order to reduce the risk of locking in unnecessary residual stresses.

Both longitudinal and transverse field splices are utilized to full advantage. The transverse splice is very difficult to make because of the thin deck material and the many intersecting welds. Warping and buckling is hard to control when making long continuous welds.

The orthotropic deck will have a higher stress range or lower stress ratio due to the reduced dead load. The designer must still comply with the allowable stress ranges specified in AASHTO.

4.5 SPLICES, ATTACHMENTS AND CONNECTIONS

The majority of fatigue and fracture problems have, in one way or another, originated in splices, attachments, or connections. It is possible to design primary members that are relatively free of welded attachments. The challenge to the designer is to keep those members in that condition.

4.5.1 Splices

There are two basic types of splices--shop and field. Shop splices are generally welded; however, a high strength bolted splice is used occasionally. Field splices generally pertain to members that are too large to ship in one piece, requiring the member segments to be joined at the site. Field splices are either welded or bolted with high strength bolts or, occasionally, a combination of both.

The majority of splices are shop splices made prior to assembly into a member. Locations of plate splices are usually left to the discretion of the fabricator except for locations involving changes in thickness, width, or type of material. The designer and resident engineer will review the fabricator's splice locations at the time of the shop plan review. Generally, the fabricator's selection of splice locations is acceptable. Sometimes the length of spliced material has been considered to be too short. Any splice of material less than 12" long, such as flange material, should not be accepted. Short lengths present a problem in assuring that the primary rolling direction is parallel to the direction of primary stress.

Shop splices are treated rather casually by many designers who rely heavily on the owner's quality assurance personnel for quality material and quality workmanship. Problems arise frequently because of poor fabrication practices, lack of quality control, and inadequate quality assurance. Some problems associated with thick flanges, and especially with thick flanges of quenched and tempered steels, can be traced to failure of the fabricator to adhere to approved procedures. Steels included in AASHTO are all weldable if appropriate procedures are followed for the different steels. The fabrication and quality control capabilities of the fabricator are more important for some materials than others.

The bridge engineer should be fully aware of those materials that are more workable; those not requiring special fabrication procedures.

During development of the welded plate girder, there developed a strong preference for the welded girder splice in lieu of the riveted splice. Extensive use of high strength bolts proved satisfactory on truss bridges; however, welded plate girders continued to be constructed with field welded girder splices. Aesthetics of the welded splice were overemphasized and the economies were not always evaluated adequately.

Field welded girder splices often require extensive falsework for erection which sometimes can be eliminated completely with bolted splices. Methods of erection are generally left to the contractor; they frequently devise some very ingenious and imaginative methods.

In view of the high quality required for welded splices and adverse conditions that may be encountered, it appears advisable for the designer to include a bolted alternative in special cases.

4.5.2 Connections

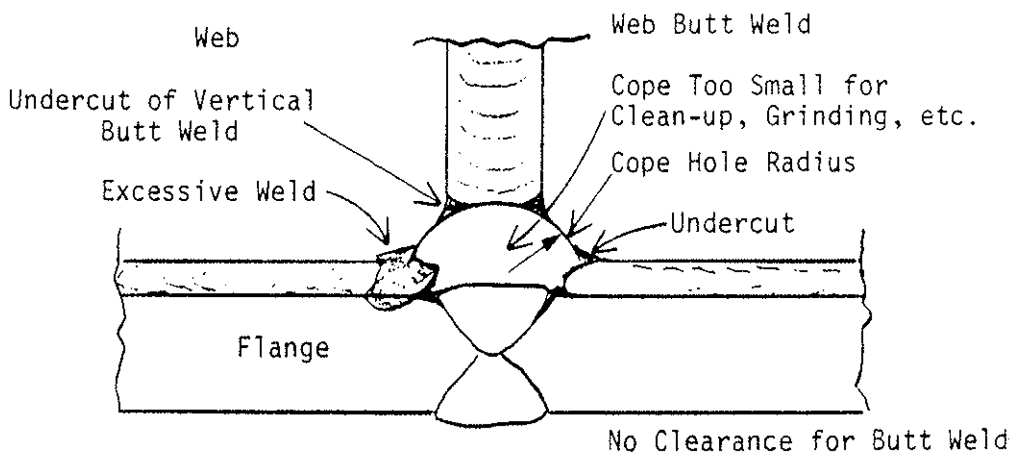
Connections are generally made with fillet welds, groove welds, high strength bolts, or a combination. Connections refer to fastening parts of members, member assemblies, and the joining of members such as truss members, floor beams, stringers, braces, and attachments.

Members are fastened with fillet welds and sometimes, to a lesser degree, with groove welds. Welds should be continuous and free of unnecessary interruptions and changes in section. The operator must avoid starts and stops during the welding operations.

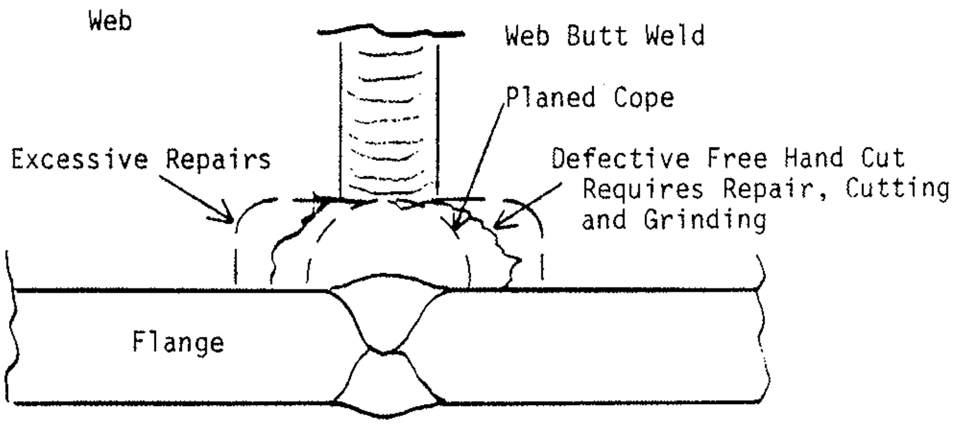
The components and geometry of a tension member determine the type, sequence, and the difficulty of making the weld. The H section is easily assembled, easily welded and easily inspected. Box section tension members are more difficult to assemble, more difficult to weld, and more difficult to inspect. Back-up bars are permitted and used in some cases to simplify welding. Back-up bars can cause problems because they sometimes create stress concentrations and cracks that are not easily detected by customary inspection methods; therefore, they should not be used on bridge tension members or in tension areas of girders. Members should be designed and proportioned so that they permit welding and inspection with reasonable ease and acceptable quality.

Diaphragms are sometimes required in order to maintain the shape of the member and to distribute applied loads. Fastening of diaphragms with high strength bolts decreases the possibilities of fatigue and fracture. Fatigue research indicates that welding across a tension member is allowable as long as it is within the allowable stress range. Research has not, from a practical viewpoint, included the probabilities of flaws, defects, and cracks that may arise from welds across tension flanges, especially those that are difficult to place and inspect.

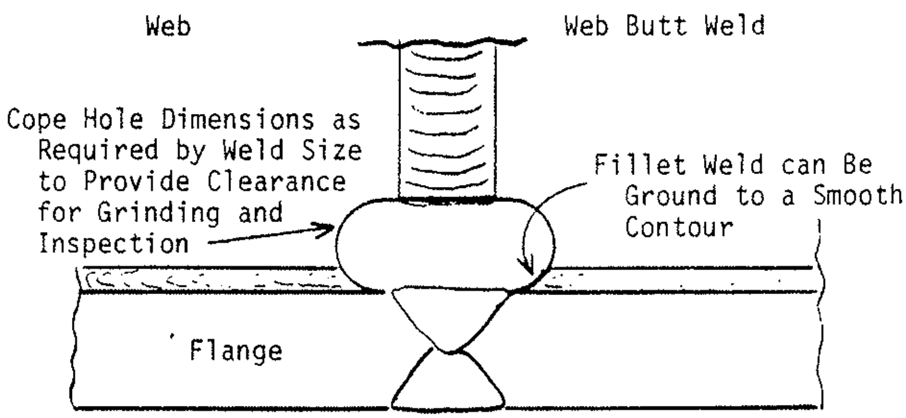
Fillet welds used to fasten plate girder flanges to webs and stiffeners or transverse and longitudinal stiffeners to webs are easily made and easily inspected for quality control and quality assurance. The bridge engineer generally considers these welds to be nearly problem free. However, welding engineers have expressed concern for these welds by requesting that details which cause discontinuities be avoided and that the details provide space for welding, grinding, and inspection. Welding engineers have indicated that terminations of fillet welds at girder splices and ends of stiffeners need special attention to assure a minimum of defects in these critical areas. Size and shape of cope holes have changed, mostly at the request of the welding engineer, so as to provide adequate clearance to place the welds; to grind the welds, if necessary; and to inspect for quality. The size and shape of cope holes have been selected on a practical basis. Cope holes, as shown in Figure 4.21,



(A) Cope Holes Too Small



(B) Defective Fabrication and Repair



(C) Ideal Cope Hole

Figure 4.21 Cope Hole Details

have been a source of problems in fabrication and field welding due to irregular cutting, lack of grinding, improper size and shape of holes and poor welding.

Connections of stringers to floor beams, floor beams to girders, and girders to caps have similarities with respect to geometry and detail as shown in Figure 4.22, as well as with respect to redundancy. For stringers to floor beams, the stringers are nearly always redundant and the floor beams are non-redundant. When floor beams are used, the girders to which they are attached are sometimes non-redundant. For beams or girders to caps, the caps are non-redundant and, depending on the designer's choice, the beams and girders may be either redundant or non-redundant. Designers often select a redundant system.

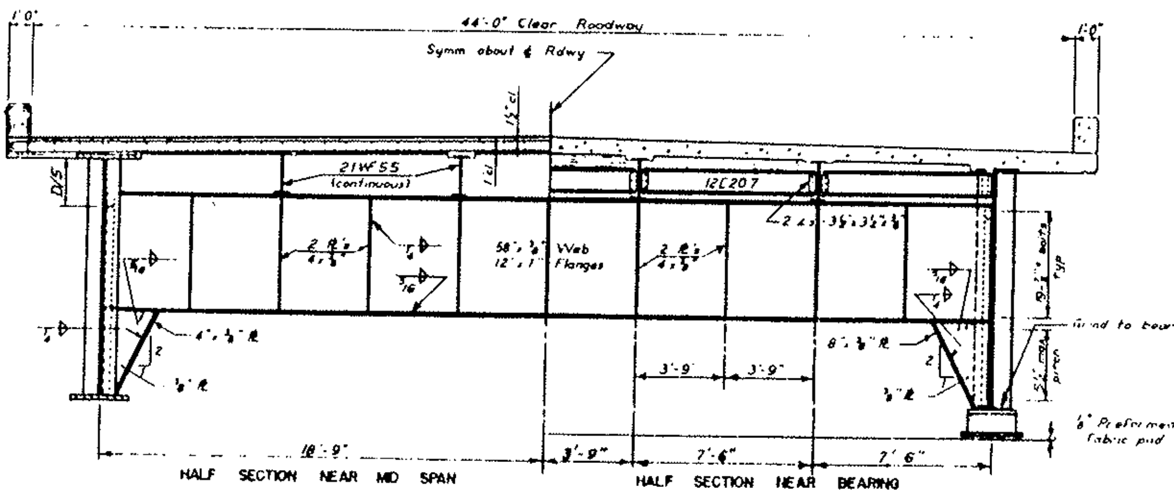


Figure 4.22 Beam to Girder Connections

Stringers to floor beam connections are often fully bolted; however, a combination of welded and bolted is not uncommon. With the combination connection, the stringers are bolted to stiffeners that are welded to the floor beams. The welds are generally in the compression area of the floor beams; otherwise, the connection stiffeners should be treated as transverse stiffeners in the tensile zone.

Continuous type floor beams, subject to positive and negative moments, present more problems than do the simple span floor beams. Any welds in the tension area should be avoided by using bolted connections.

The stiffness of the stringers and floor beams has an indirect effect on fatigue which is sometimes ignored in design. Rotation at connections has caused torsional stresses and fatigue cracks in continuous floor beam flanges in compression that were fillet welded to supports. There are generally less problems with a stiffer deck system which implies the use of lower strength steel and lower stresses.

Floor beams are either framed in or supported on the girder as shown in Figure 4.23. The framed-in connection is of more concern;

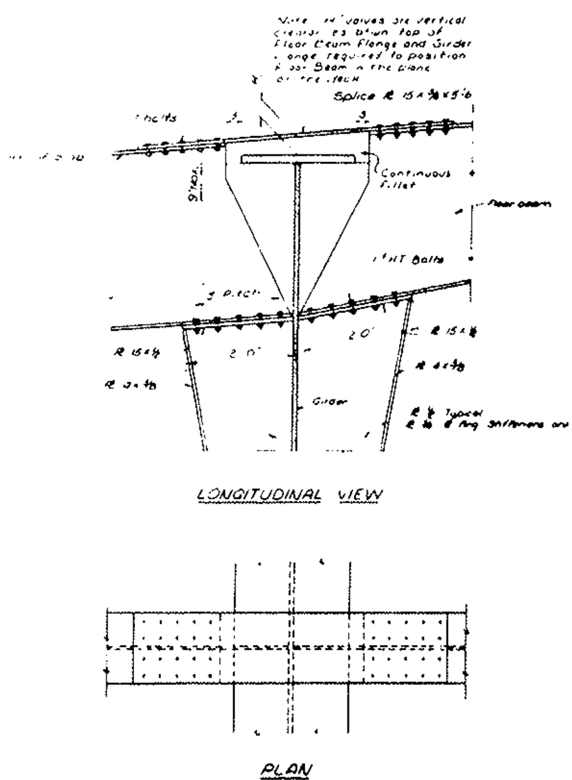


Figure 4.23 Floor Beam Connection

it may be fully bolted, fully welded, or a combination of bolted and welded. With the growing awareness of fatigue and fracture problems associated with welding, it is anticipated that the fully bolted joints will prevail unless connections are in a compression area of the girder.

Beams and girders supported by steel caps are often redundant while the caps are non-redundant. Steel caps are frequently selected for aesthetic reasons with caps in the same plane as the beams or girders. The depth of the cap may be equal or greater than the beams or girders. The bridge engineer should select a detail that recognizes the redundancy of the girders and non-redundancy of the caps. Connections of caps may be fully welded, fully bolted, or a combination of bolted and welded. The geometry of the connection has considerable influence on the quality of the welded connection. A connection with simple geometry is easier to weld and easier to inspect.

Placing girder flanges and cap flanges in the same plane complicates the connections; however, an all welded connection can be made with the flanges in the same plane if a 24" transition radius as shown in Figure 4.24 is made from the cap to the girder flange. The flange material should be cross rolled to assure good properties in both directions. For erection, it is sometimes convenient to use welded splices and sometimes to use bolted splices. Either connection can be shop fabricated ready for field welding or bolting.

Connections with flanges not in the same plane should have the flanges separated adequately to provide space for fabrication and inspection. Again, most of these can be made fully bolted, fully welded, or a combination. When any weld is made in a tension area, the designer must provide adequate cope holes to prevent weld on weld, to provide space for clean-up of weld terminations, and to provide for quality inspection.

Girder to cap connections are sometimes subjected to torsional stresses that contribute to fatigue and fracture problems. They are not always given adequate attention.

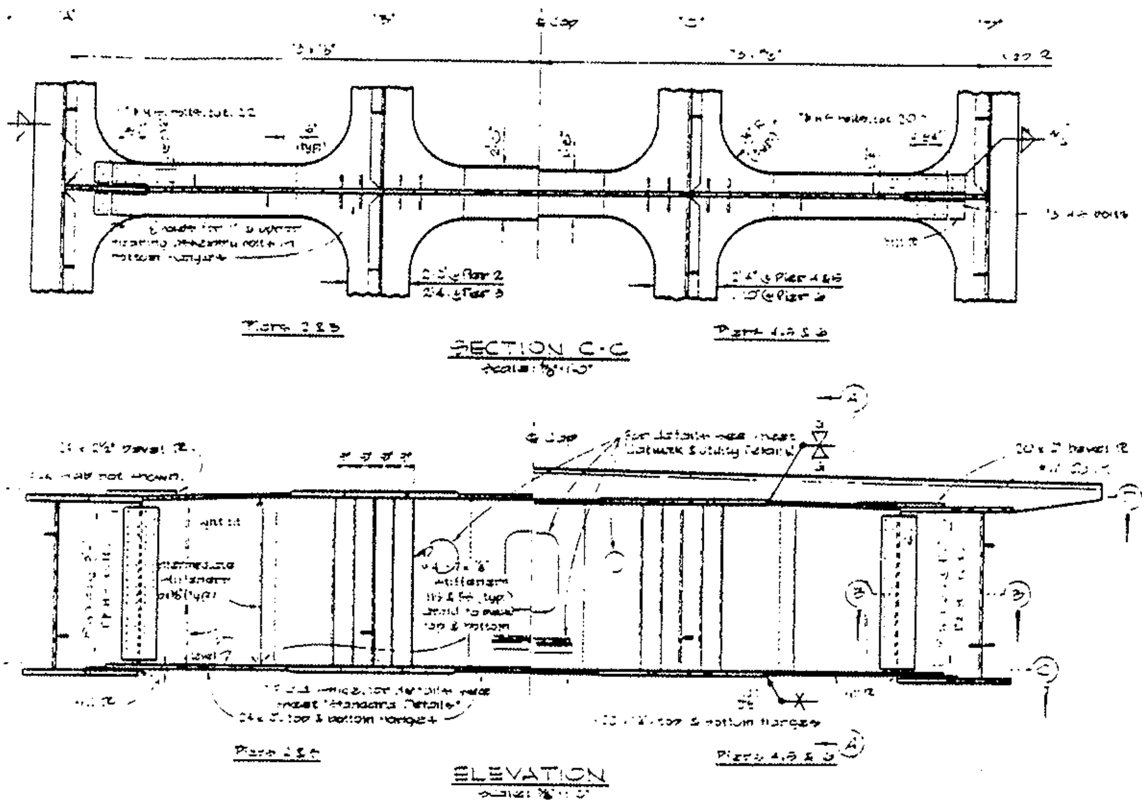


Figure 4.24 Girder to Cap Connection

4.5.3 Attachments

Attachments may be separated into two categories; those included in the design and those added during construction.

The designer has considerable control over those included in the design but little or no control over those added during construction. Additions that are not in accord with contract specifications are sometimes referred to as "illegal attachments."

Attachments associated with girders involve stiffeners and gusset plates for cross framing and lateral bracing. Attachments can cause difficulty if back-up bars are not properly used or fitted.

Figures 4.25, 4.26, and 4.27 show the results of improperly designed and/or fabricated attachments.

Figure 4.25 (A1) shows slag inclusions and lack of penetration at the root of the weld which can be sources of crack starters.

Figure 4.25 (A2) illustrates weld shrinkage warp from the gusset plate upward and lack of penetration in the root of the weld. These conditions will cause crack starters.

Figure 4.25 (A3) presents an example of improper fitup of the gusset plate in relation to the web and the back-up bar. Not only can there be lack of fusion or slag inclusions, but when the gusset is connected to the cross frame, the gusset will try to square-up which introduces strain in the toe or root of the weld. When coupled with other defects that lie in these areas, this situation can be a cause for crack starters.

Figures in 4.25 (B) show highly restrained weld connections, and as demonstrated in 4.25 (B1) and (B2) can cause distortion, weld toe cracking and lamellar tearing.

Figures in 4.25 (C) and (D) show what can be expected if large welds in thick material are not detailed properly. Weld metal shrinkage will cause weld toe cracking, root cracking and especially lamellar tearing.

Figure 4.26 shows highly restrained weld joints in: (1) longitudinal "T" stiffeners, and (2) corner box welds. These joints will

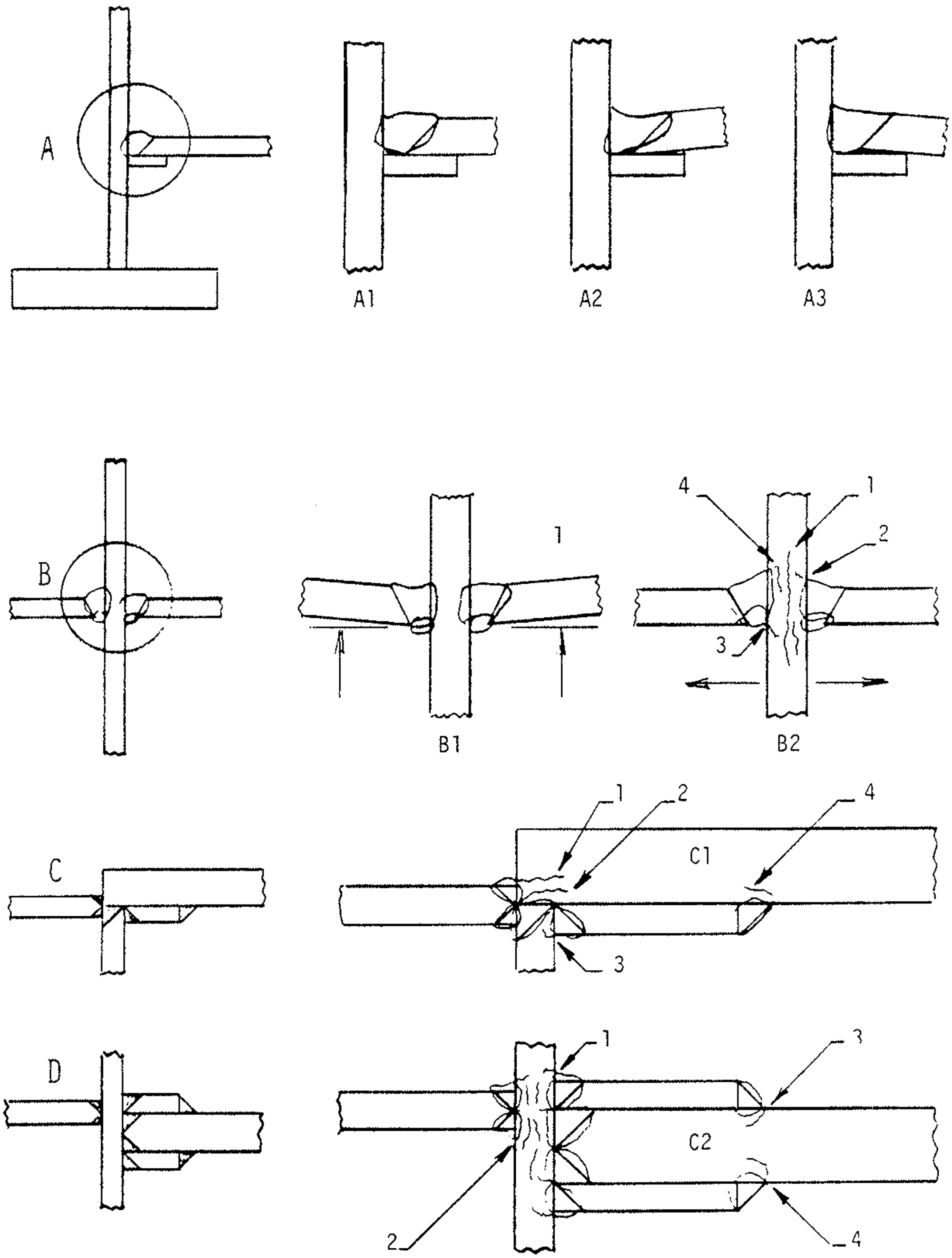


Figure 4.25 Highly Restrained Welded Connections

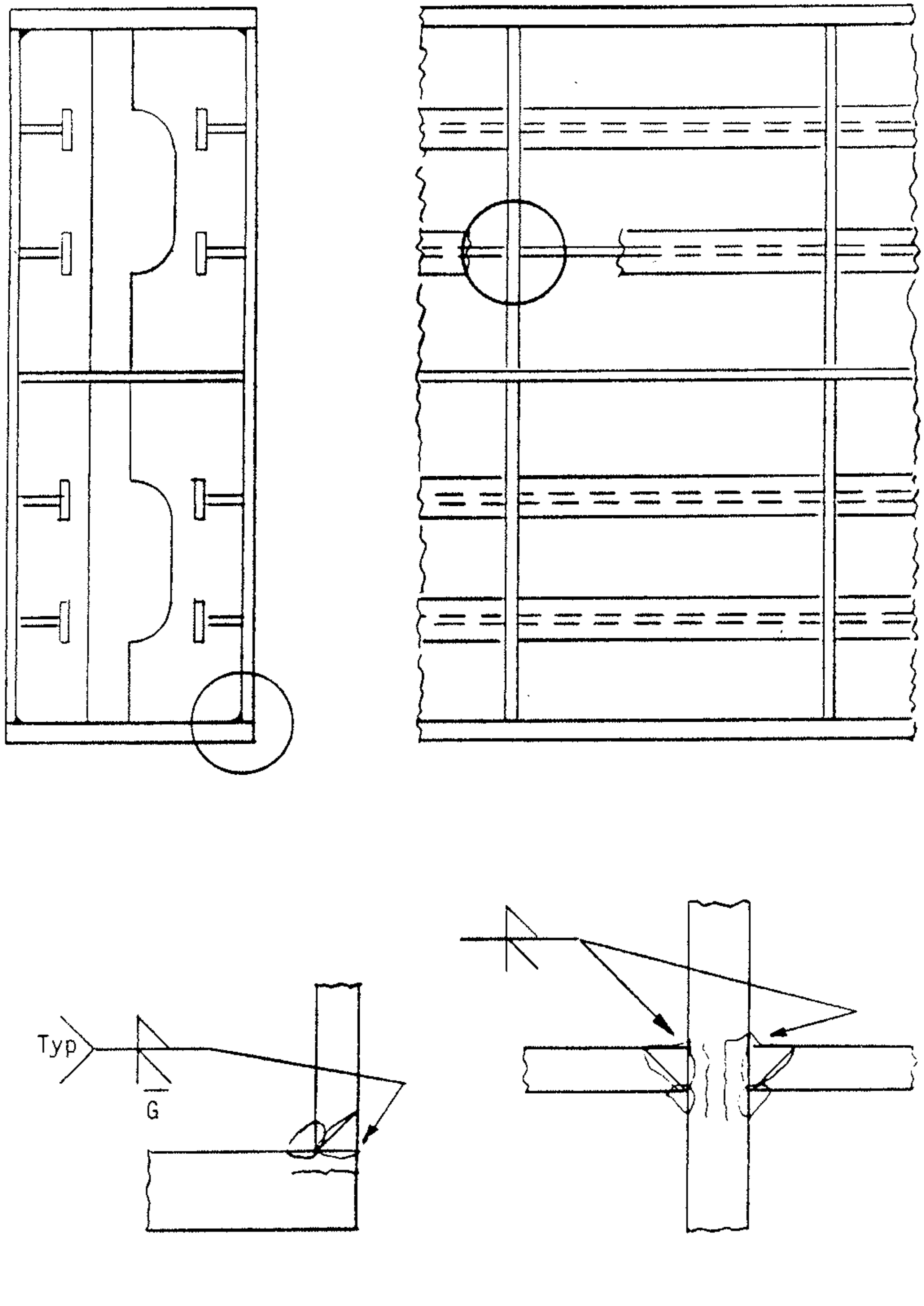
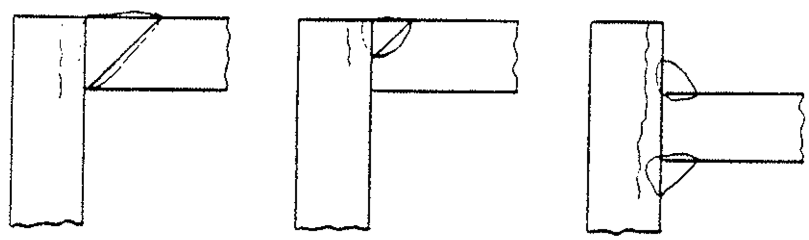


Figure 4.26 Highly Restrained Connections - Box Girders

lead to either weld toe cracking or lamellar tearing in a box girder.

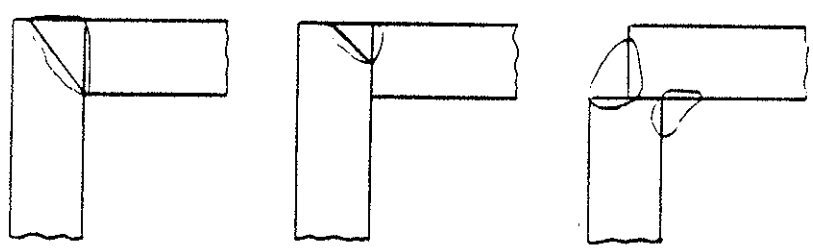
Welded connections in thick material, if not detailed and welded properly, will result in lamellar tearing which is a separation in the parent or base material caused by weld metal shrinkage.

Figures 4.27 (A1), (A2) and (A3) show weld joints susceptible to lamellar tearing while Figures 4.27 (B1), (B2) and (B3) show simple solutions that prevent lamellar tearing.



(1) (2) (3)

A. Joints Susceptible to Lamellar Tearing



(1) (2) (3)

B. Joints Improved to Prevent Lamellar Tearing

Figure 4.27 Joints Susceptible to Lamellar Tearing and Solutions

With the development of the welded girder came research data indicating welded attachments to flanges were not desirable. In the beginning, gussets for bracing were fillet welded directly to the flanges. Some bridge engineers were concerned about welding across the tension flanges and limited welding parallel to the direction of stress only. If a cross frame needed to be attached to a flange, a small plate was welded to the end of the transverse stiffener and then welded to flanges with longitudinal fillet welds as shown in Figure 4.28.

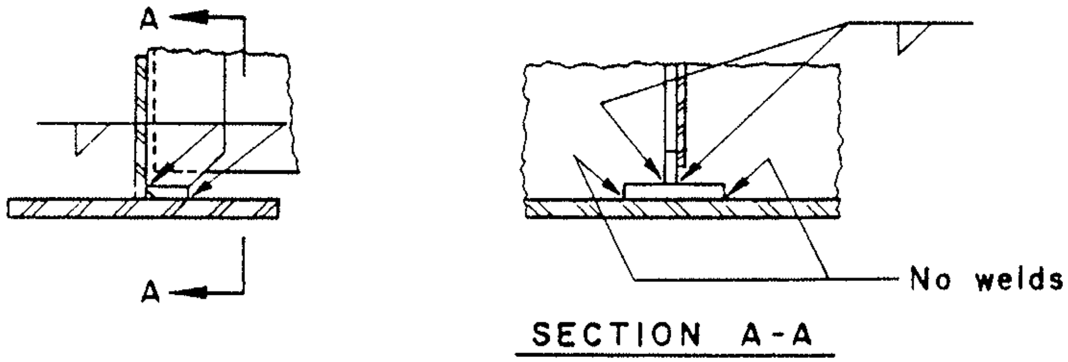


Figure 4.28 Stiffener to Flange Connection

Research indicated these longitudinal welds were also a potential fatigue problem; consequently, cross framing and lateral bracing connections were raised to clear the bottom flange by approximately 6 inches. Six inches provides adequate working space for most flange widths; however, on longer spans where wider flange widths are used, 6 " of clearance is insufficient. Welds are very difficult to make, are sometimes incomplete, and back-up bars are used. Inspection of these welds is difficult.

Additional research showed that attachments to the tension web created a fatigue problem. In addition, for curved structures, welded attachments to webs were not adequate for the forces involved. Various details were used to accommodate the forces; however, they led to other problems. One solution was to insert a thicker plate in the web capable of transferring load to the flanges, as shown in Figure 4.29; however, this caused fabrication problems in that it locked up stresses in the web and prohibited continuous automatic fillet welding of flanges to the web. The heavy web insert had to be welded by use of either shielded metal-arc welding (SMAW), manual, or submerged arc welding (SAW), semi-automatic. If the welding sequence was not maintained the thin web plate would buckle around the heavy insert.

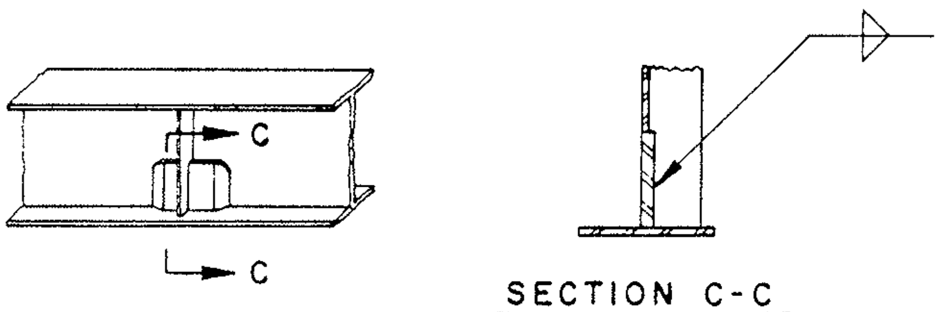


Figure 4.29 Web Modification

The time has come to take a good look at what is required to accommodate forces and how to minimize fatigue and fracture problems.

It appears that details similar to those used with riveted girders may be as good if not better than others. Attaching gusset plates directly to a flange, a riveted girder detail, with high strength bolts is a simple direct method that may be the best solution.

For those who consider bolting inappropriate, a butt welded connection to the flange with a transition radius can be used. This detail can be easily fabricated and easily inspected to assure good quality.

The authors consider stud shear connectors, attached to the tension flange, as a potential problem. Their reasons are based on the fact inspection of welds is minimal and it is not uncommon for studs to break off during the construction period leaving craters with indications of small cracks. Some states and some bridge engineers prohibit studs on tension flanges except in low stress areas and allow them to be used only with lower strength steels. Attaching the studs in a low tensile area, that is, those areas well within the allowable stress range, may give the designer a false sense of security since the product is of unknown quality.

Supplementary attachments, not specified by the contract plans, and sometimes prohibited by contract specifications, are added occasionally during fabrication and erection. Attachments are generally in the form of dogs, brackets, lifting cleats and other miscellaneous fabrication and erection aids. Added attachments may also be associated with stay-in-place forms and rebar supports. Welds connecting these attachments are sometimes referred to as "illegal welds." Illegal welds are sometimes used to repair fabrication errors, e.g., plugging holes. Other illegal welds are tack welds used to hold materials in position during fabrication.

Supplementary attachments are a serious problem. The contract plans and specifications should acknowledge these problems by providing adequate controls. All added attachments should be shown on the shop plans and be reviewed by the bridge engineer and the welding engineer.

While added attachments and welds may be prohibited by contract requirements, their importance to the fatigue life of the member is not recognized by some fabricators and inspectors. Some welders have an attitude that steel is steel and any attachment will not be harmful. Very likely the welder does not know that the added attachments and welds are prohibited. In addition to the requirement that all attachments and welds be shown on the shop plans, it may be advisable to require that appropriate cautions be noted on the shop plans to inform the welders and inspectors.

4.6 CONSTRUCTION CONDITIONS

A bridge engineer must be aware of construction conditions in order to give adequate consideration to design selections and decisions.

4.6.1 Fabrication Conditions

Knowledge of the capabilities and reputation of potential fabricators may influence design decisions. Fabrication plants differ greatly in physical resources. Some are large, have the latest and best equipment, and are capable of fabricating the smallest to the largest girder. Other plants are small, have a limited amount of equipment, and are only capable of fabricating small jobs or portions of jobs. For some, the fabrication is performed under controlled conditions and for others with make-shift protection from adverse weather.

The size of member, thickness of plates, and type of steel are basic factors that limit a fabricator's capability. Some fabricators become specialists with specific types of steel and may not be equipped to handle A514 - A517 quenched and tempered steels. Flame cut plates of A514 - A517 steel often require straightening which may require special equipment.

There is no reason to assume that the larger, better equipped plants consistently turn out superior products; smaller fabricators may do as well. The capability of the fabricator may prompt design changes or alternatives that are compatible with these capabilities.

4.6.2 Quality Control

The most elaborate fabrication plants with all of the latest automatic equipment will not necessarily produce a quality product unless they have a good quality control program. This applies to all plants regardless of size and equipment and to erectors. Some fabricators and erectors have excellent quality control programs while others have little to offer and rely on the owner's quality assurance programs. These are circumstances that confront designers and have a direct bearing on design decisions. At the design stage, unusual or difficult fabrication should be discussed with fabricators as well as with the materials and welding engineer in order to ascertain whether or not adequate quality control is available, or if the design should be based on the anticipated quality control.

4.6.3 Quality Assurance

Quality assurance is the owner's responsibility whether performed by employees or by a consultant. The capability available for quality assurance should be known by the designer. The lack of quality assurance could influence the designer to use material and details less susceptible to fatigue and fracture at the expense of economy. The cost of future problems could easily overshadow any anticipated economy. Low strength steel should be given serious consideration in the absence of a reliable quality assurance program. In particular, quenched and tempered plates should be avoided if the quality control and quality assurance programs are questionable.

4.6.4 Site Location and Conditions

The site location and conditions often play a major role in bridge type selection and related details. It is important to be aware of any conditions that could affect girder length, width, and height during transportation from fabricator to bridge site. The designer must have reliable information on how conditions affect erection with welded splices and with bolted splices. Elaborate falsework, required for welded girder splices, can be eliminated by using bolted splices. Location of girder splices influences the erection scheme. For this reason, the locations of field splices are often left to the discretion of the erector, subject to approval by the engineer.

Site location may be such that the availability of qualified welders is questionable. Such circumstances are reason enough to avoid field welding, particularly of primary members.

4.6.5 Erection Methods

At the time of design most bridge engineers envision a particular method of erection; however, they do not usually design for any specific method. The designer knows there are various erection methods and realizes that the erectors are a resourceful group. Occasionally, a designer does not have any erection method in mind and relies on the contractor's ingenuity. Apparently this process works but sometimes there are no bidders for lack of a practical method of erection.

The mobility of truck cranes has improved to where it is not uncommon to use more than one in order to eliminate falsework. A welded girder splice does not fit this method of erection; therefore, a bolted option should be provided for erection convenience.

Welded girder splices slow erection progress and place undue pressure on the welder. For some bridge projects the welding has been the controlling operation. This should be avoided.

4.6.6 Weather Conditions

Weather conditions such as temperature and humidity play an important role in erection and, in particular, in the quality of welding. Weather conditions have a direct effect on the quality of welds and, at times, a dramatic effect on the workmanship of the welder. Proposed field welding under adverse weather conditions should be discussed thoroughly with the welding engineer.

4.7 CONTRACT ADMINISTRATION

The owner, directly or indirectly, should establish contract administration procedures that will ensure adequate participation by the designer in the preparation and review of shop plans. The designer must also be available to the resident engineer as the need arises.

As a guide, the following checks on shop plans are usually made:

1. Review the contractor's erection procedure. Be sure that it satisfies the assumption for continuity made in design. If the design criteria are not met, the contractor must submit calculations for any revised cambers and stresses.

2. Verify that all materials shown in the working drawings conform to the size, thickness and type of steel shown on the contract plans or with requirements of an approved erection procedure.

3. Investigate the amount and method of camber to confirm compliance with the contract plans or with values computed to accommodate an approved erection procedure.

4. Examine the size of all welds. If a welding sequence other than that shown on the contract plans is proposed, it should be reviewed by the welding engineer.

5. Determine the end rotation of pinned ends, hinges and bearing stiffeners due to dead load deflections for structural as well as aesthetic reasons.

4.7.1 Shop Plan Preparation

Before the fabricator submits any shop plans, it is important that the resident engineer convene a prefabrication conference consisting of the designer, the owner's welding engineer, the contractor's fabricator and the erector.

The lines of communication and authority and the manner in which the work is to be conducted by both the contractor and the owner are established at this conference. From this, the basic administrative procedures are established and maintained during contract work.

The erection scheme, contract changes and anticipated changes, inspection procedures, and vague or ambiguous specifications must be discussed and any necessary corrective action should be specified.

4.7.2 Shop Plan Review

The review of shop plans is the last favorable opportunity for the designer to make minor changes in, or revisions to, his design. Changes made after the approval of the shop plans can be costly and may require corrective work detrimental to the fatigue and fracture quality of the bridge members.

Upon submission of shop plans by the fabricator, the designer should check sizes, dimensions, details, and materials as shown on the contract plans. The welding engineer must check the connections, especially the welds and weld sizes, qualification of joints, welding processes, positions and sequences. The resident field engineer will review the plans with emphasis on the erection procedures and temporary connections.

TOPIC 5

CONSTRUCTION CONSIDERATIONS TO MINIMIZE THE POSSIBILITY OF FATIGUE

OBJECTIVES:

1. *To provide sufficient information to enable bridge designers to evaluate the fabricator's capability.*

2. *To outline the procedures that the engineer can follow to insure the appropriate level of communication with the fabricator which is required for contract control.*

3. *To acquaint designers with various welding processes and inspection methods used in bridge fabrication.*

5.0 INTRODUCTION

This topic covers the fabrication and field erection of welded bridges. The designer or engineer must have a good understanding of shop fabrication, the welding operations, quality control and quality assurance. Too often the engineer has relied on inept NDT technicians, inspectors, welders, and others to perform important work and make decisions.

5.1 SHOP QUALIFICATION

It is the fabricator's responsibility to produce quality fabricated steel construction. The fabricator should have the personnel, organization, experience, procedures, knowledge, and equipment capable of producing quality workmanship. Prior to beginning a project, the fabricator should demonstrate to the owner his ability to produce quality fabrication in accordance with the contract documents (plans and specifications). This can be accomplished by requiring the fabricator to be certified by the American Institute of Steel Construction (AISC) or another suitable certification program.

5.1.1 Review of Shop Plans

The approved shop plans that are returned to the shop inspector (owner) must bear the stamp "approved" and be dated on every sheet of the plans. These approved shop plans are now as important as the design plans and these plans are the only ones used in shop fabrication. The working drawings (shop plans) shall show any changes proposed in the work, details for connections not dimensioned on the design plans, the sequence of shop and field assembly and erection, welding sequences and procedures, and the location of all butt welded splices on a layout drawing of the entire structure.

Both the fabricator's and the owner's shop inspectors should have copies of the latest approved shop plans. All inspectors (both quality control and quality assurance) should maintain a diligent check on shop plans to ensure that the latest plans are being used in the construction. The shop inspector should also be checking the shop plans against the design plans for any details that may have been overlooked during the shop plan review.

5.1.2 Fabrication Conference

At the prefabrication conference between the fabricator's and/or contractor's personnel and the owner's personnel it is of the utmost importance to discuss the overall shop fabrication, plans, specifications, quality control, quality assurance, etc., and to come to a general understanding of any problems on behalf of the fabricator and/or contractor and owner.

The prefabrication conference is usually requested by the fabricator and/or contractor. Prefabrication meetings are attended by fabricator or contractor personnel representing management, engineering, production, inspection, quality control and field operations. Owner personnel taking part in this meeting should include the senior resident engineer, bridge design engineer, welding engineer, the quality assurance chief engineer or his representative for quality assurance (shop inspector) and a nondestructive testing technician.

5.1.3 AISC Quality Certification (Shop)

The AISC shop certification program provides a comprehensive method of evaluating capabilities of a given plant and organization. The judgment of the AISC inspection-evaluation team evaluating a given plant is the sole factor determining a plant's rating. AISC relies heavily on outside plant personnel for making these judgments. While plant personnel may be most cooperative when the AISC team is inspecting the plant, their cooperation may not continue when they are under pressure to maintain a schedule and show a profit. Plant personnel may be very capable of quality fabrication, but sometimes they must be coerced into using these capabilities.

5.1.4 Shop Fabrication Quality Control Plans

At the outset of each contract, the fabricator should be required, within the specifications, to submit an outline of the quality control measures planned for the entire job. This accomplishes many things and generally verifies the fabricator's understanding of the inspection requirements that are included in the specifications. If any item is overlooked within the program, resubmittals are required until all items are addressed. It is very difficult to obtain details on the quality control tasks to be carried out by the fabricator in writing since most fabricators do not want to include anything beyond what they consider to be minimum inspection in order to complete the job in minimum time (and on schedule).

The quality control plan should include information on materials shortage, documentation, personnel qualifications, details on nondestructive testing procedures, details on the staff organization of inspection and engineering personnel, lists of equipment to be used in fabrication and inspection, and as much information as possible on how and when all quality control tasks are to be performed. Again, it is difficult to acquire all these details in writing from a fabricator, but it is important to obtain every commitment possible in the way of quality control.

5.1.4.1 Personnel Qualifications

Personnel qualifications should be emphasized in accordance with the nature of the work. For large nonredundant or critical structures, a registered engineer should be directly responsible for the project. In any case, inspection functions should be as independent as possible from production or engineering functions. Inspection is usually last on the fabricator's priority list, but qualifications for inspection personnel are as important as the qualifications of other personnel assigned to the contract. It is through the inspections performed that defective welds are found and repaired in the best possible way.

Welding inspectors for quality control (fabricator's personnel) or for quality assurance (owner's personnel) should be certified by AWS to ensure some level of competence in reading, interpreting, and following specifications. The current AWS certification program is much improved over the original program. For a critical structure, all welding inspectors should be AWS certified. For less critical structures, the chief inspector should be AWS certified and responsible for the inspections performed by other welding inspectors.

The qualifications of nondestructive testing technicians must be spelled out in the contract specifications and included in the quality control program. There is a weak link within the current system of certification of nondestructive testing personnel, but there are ways of reinforcing the system. The American Society for Nondestructive Testing (ASNT) has set guidelines for certification of nondestructive testing (NDT) technicians. However, the only technicians which ASNT will certify are Level III technicians. Level II and Level I certifications are the employer's responsibility. Generally, a Level III technician is much more competent than Level II or Level I technicians, but this has only recently become the case.

Since 1978, a company may designate an uncertified Level III examiner for certification of Level I and Level II technicians, but without ASNT Level III certification, this examiner may not be responsible for inspections. Prior to 1978, Level III technicians were not required to pass any examination and needed only minimal practical experience in NDT. A company could designate the Level III technician for the company (usually an engineer, since an engineering degree fulfilled ASNT's experience requirement) and this person acted in a supervisory capacity and administered examinations for certification of all Level I and Level II technicians in the company.

It cannot be overemphasized that no technicians below Level II should be allowed to perform any NDT on bridges. This should be a specification requirement. A practical test of competence of any technician should be given by the customer (owner) to verify the capabilities of each technician. This option should be provided through the contract specifications. The weakness in the NDT technician certification system (that of certifications being the employer's responsibility) can be reinforced by this practical exam, either performed on a flawed sample weld or on a flawed weld which occurred during fabrication. On occasions, obviously incompetent NDT technicians are assigned to bridge welding inspection and have to be "weeded out." The competency test provision provides for appropriate actions. As in any field, integrity and conscientiousness cannot be measured for any NDT technicians. These traits become apparent only after observing and working with a particular technician. Without qualified and conscientious NDT technicians as part of the quality assurance staff, no real confidence in the fabricator's NDT technicians can be established. Most NDT results are difficult to verify by simply observing another technician's work. There must be some close scrutiny of testing performance and spot checking by quality assurance to verify test results.

5.1.4.2 Nondestructive Testing Procedures

Nondestructive testing (inspection), NDT (or NDI), procedures to be included in the contract work must be detailed to some extent within the contract specifications. Also, the amounts of NDT to be performed on welds in a particular member must be spelled out.

Details of all NDT procedures to be used throughout the project should be included within the quality control submittal. The submittal should include methods of reporting and recording NDT results and any details not addressed in the specifications as well as basic requirements of the specifications. (Again, to verify the fabricator's understanding of the specifications.)

The four most common and most successful methods of NDT are radiographic (RT), ultrasonic (UT), magnetic particle (MT), and dye penetrant (PT) testing. Although exact flaw sizes cannot be determined for use in fracture mechanics formulas and calculations of stress intensity factors, good estimates of flaw sizes can be made from good NDT techniques.

No one NDI method is a complete flaw detection method. Each method is useful for detection of particular types of flaws, reliance on one method over another is a mistake.

The method used most extensively for NDI on bridge weldments is radiography (RT). Its capabilities are limited because detectable flaws must either have volume, such as slag or porosity as shown in Figure 5.1, or be oriented so that the rays from the radiation source are parallel to the flaw itself, as with incomplete penetration as shown in Figure 5.2. Cracks or incomplete fusion type flaws may not be discovered radiographically if they are tight (lack volume) or are oriented in a position other than parallel to the direction of radiation. An advantage of radiography is that a permanent record is obtained which shows actual size, orientation, and location of weld defects (if suitable match marks are used for locating the radiographic film). The methods of locating and referencing radiographs should be included as part of the quality control submittal.

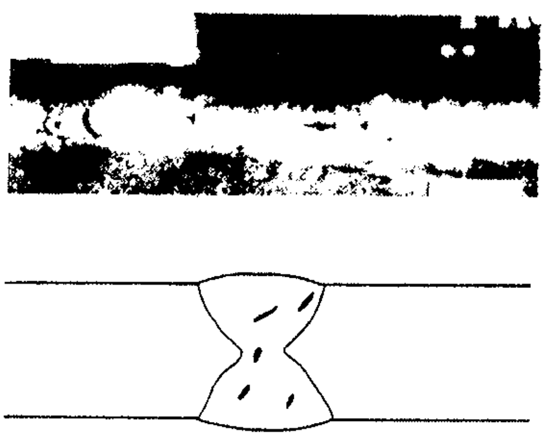


Figure 5.1 Slag Inclusions

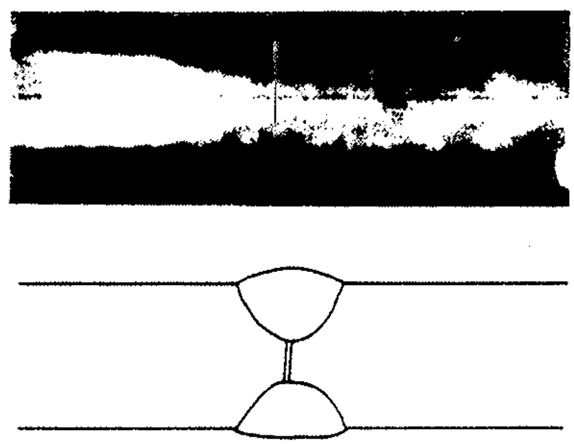


Figure 5.2 Incomplete Penetration

One very important requirement that should be included in the specifications is that of grinding welds flush for NDT. Interpretation of radiographs is eased greatly when welds are ground flush prior to testing. Radiographic contrast and sensitivity are assured by proper use of penetrameters next to a ground weld joint as illustrated in Figure 5.3. Grinding is even more important for welds that are ultrasonically inspected. This allows complete scanning of a weld without false indications appearing on the ultrasonic instrument because of a weld crown or undercut as shown in Figure 5.4. Grinding may also be necessary on welds subject to magnetic particle or dye penetrant inspection in order to eliminate short (but shallow) undercut and surface roughness which will appear as indications during testing.

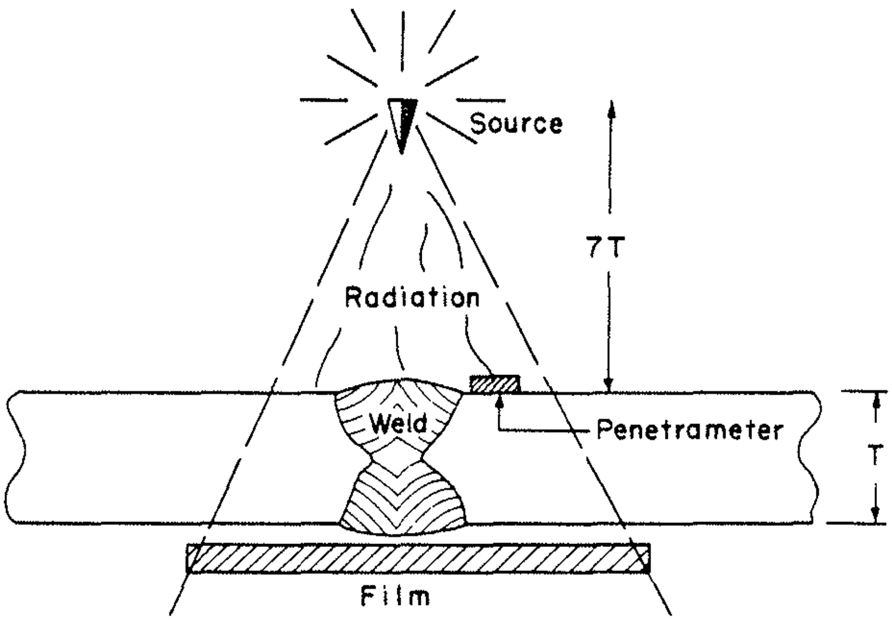


Figure 5.3 Radiographic Test--Distance of Source from Object

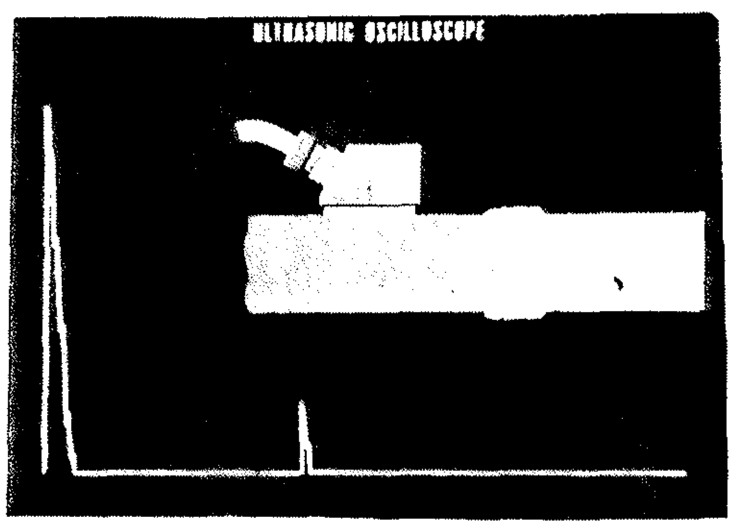


Figure 5.4 Weld Scan

5.1.4.3 Staff Organization for Nondestructive Inspection

The organization of inspection personnel should be similar to that shown in Figure 5.5. Note that the quality control staff and functions are as independent as possible from production staff and functions.

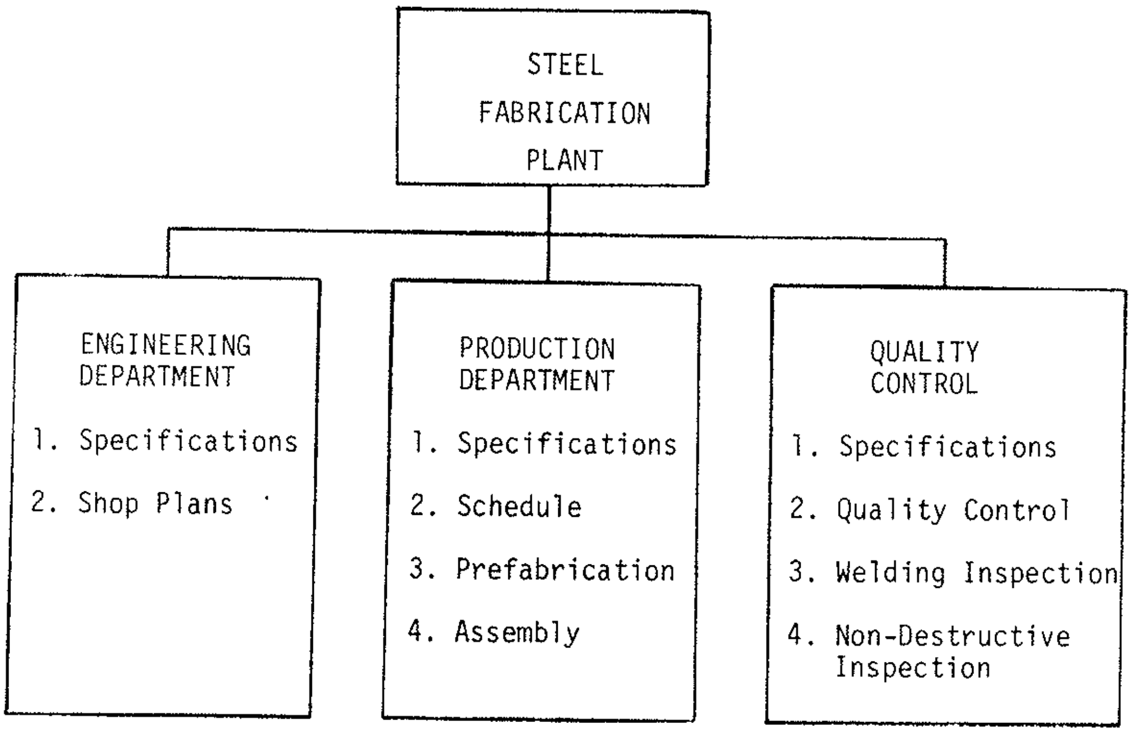


Figure 5.5 Shop Organization Plan

5.1.4.4 Equipment

The equipment to be used for quality control testing and inspection should be itemized in the quality control plan and should include manufacturer, model number, serial number, and other pertinent data. Good inspection equipment is available today and should be utilized. Strict adherence to equipment specifications increases the confidence level of the test results. For example, the focal spot size of an x-ray machine or the radioisotope source size must be limited by the specifications to reduce geometric unsharpness in a radiograph.

Just as a point source of light casts a very sharp shadow, a point source of radiation causes a sharp image on radiographic film. A one-eighth inch focal spot size or isotope source size is reasonable. The other limiting factors on geometric sharpness are weld thickness and source-to-film distance, which also must be addressed by the specifications.

As far as UT is concerned, good equipment is available. Occasionally, transducers with poor resolution cause problems and occasional differences between quality assurance test results and quality control test results. These discrepancies usually arise because of slight differences in transducer shoe angles. (Figure 5.6 shows a typical shoe angle with transducer.) The discrepancy should be resolved by relying on the test that produces the more severe (lower

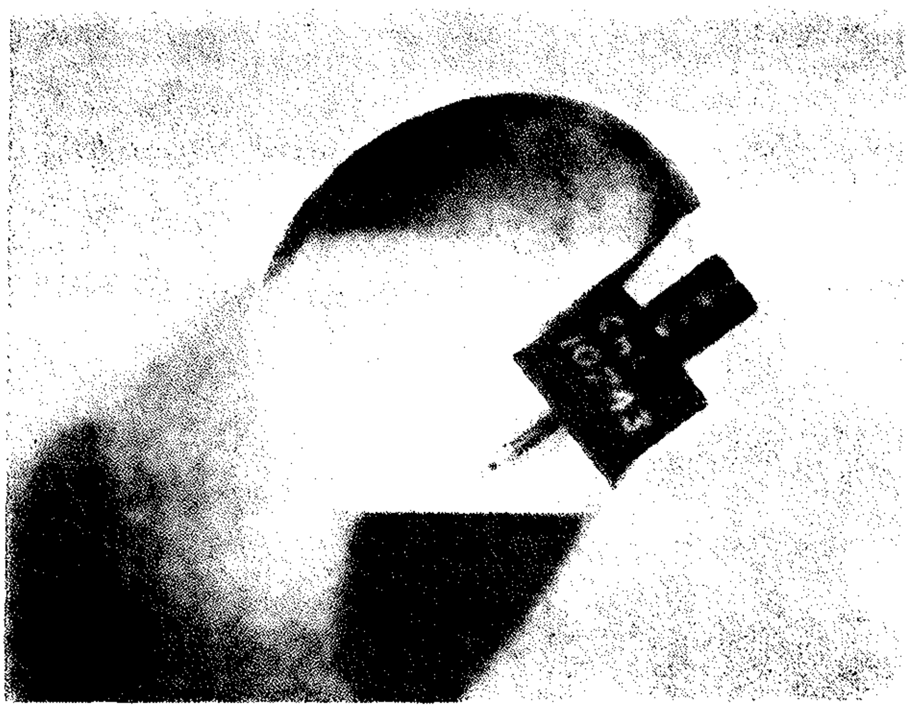


Figure 5.6 70° Shoe Angle

db number) result. No acceptance of a flaw in a critical bridge member should be based on a 1 db difference in defect level--if a 1 db defect rating would cause rejection of a weld, the defect should be repaired. Calibration of the UT instrument should be checked on an approved ultrasonic reference block prior to and at the conclusion of inspections performed on each weld or at 1/2 hour intervals, whichever occurs first. Transducers should be checked for angle, index point, and resolution periodically (at least after every 40-50 hours of use). The ultrasonic instrument should be calibrated by an authorized service center at least once a year.

Equipment for magnetic particle testing or dye penetrant testing usually needs only to be checked for proper working condition. Quality assurance measures to check for proper equipment operation will be discussed later.

5.1.4.5 Scheduling of Quality Control

Inspection tasks to be performed by the fabricator (quality control) must be scheduled so that sufficient time is allotted for each task. All too often, inspection personnel are blamed for production delays which could have been avoided by proper scheduling. Proper scheduling also allows quality assurance personnel (customer representatives) to observe quality control tests and inspections, to gain confidence in the test results (when acceptable results are obtained) and to be aware of inspection problems that arise (when tests are performed improperly or when test results indicate defective welds).

An important specification requirement to consider is that of performing radiographic inspection prior to ultrasonic inspection when both are done on a particular weld. Usually this will minimize the time necessary for ultrasonic inspections since radiography will locate severe defects and those defects (slag and porosity) less easily evaluated by UT.

5.2 MATERIALS VERIFICATION

5.2.1 Steel

Construction specifications usually require the contractor to furnish the engineer with a list of his sources of materials in sufficient time to permit identification and verification of compliance with specifications before that material (steel in this case) is incorporated into the contract work. For steels this means that the contractor or fabricator must notify the engineer when any steel is purchased or received for the contract. If the steel is bought at the mill, the engineer should arrange for inspection at the mill. If the steel has been shipped, he should arrange for inspection of the fabricator's supply. In either case a mill test report identifying the type and heat of steel purchased, including its chemical composition and mechanical properties, should be available to the engineer for every piece of structural steel used on the contract.

5.2.1.1 Mill Test Report

Before the quality assurance inspector accepts a welded steel component in partial fulfillment of a contract, he must be able to verify the identity of the steel in that component. The inspector should establish records on the origin of all steel when it came into the fabricator's supply so that he can match it with a mill test report or a certificate of compliance from the engineer's files. Steels are generally tagged, color coded, or stamped with a heat and slab number that can be matched with a purchase order or a material's source notice which shows the steel manufacturer, type, and a heat number. With this information, the inspector must obtain the matching mill test report or a certificate of compliance and a shipping release record that verifies compliance with specified requirements. If one or more of these documents is missing, he may have to arrange for a check sample to be taken from the material in order to verify compliance. In any case, the steel must be matched eventually with a mill test report in order to verify its identity and confirm that it is being used as intended.

5.2.1.2 Material Check Sampling

ASTM A673 specifies a standard location in the corner of a rolled plate for the origin of the specimens that are tested to establish the mechanical properties of the steel in that plate (and in all plates of similar thickness throughout that heat under heat lot testing). Steel manufacturers, however, have been reluctant to warrant these same properties throughout the plate or in any other plates of that thickness that remain from the heat. Their reluctance appears well founded because plates rolled from the last ingots poured from a heat may have significantly poorer properties than the plates from earlier ingots due to progressive oxidation of the slag in the ladle during the pouring operation. Steel from later ingots may have impact strengths which are as much as 20 ft. lbs. below the impact strength of the steel from the first ingots. The authors have examined several cases that confirm these variations where steels which complied with AASHTO requirements in a mill test report fell below these requirements in check tests taken from different places in different plates from the same heat. Thus, some state departments of transportation have established a policy of check testing a sample of one or more out of every ten plates used in the construction of a bridge. The plates to be tested are designated on the project plans sent to bidders so that they can arrange for the purchase of extra plate lengths to permit the necessary sampling. These samples are cut and tested in the same manner as the standard drop samples. If the results cause concern there is generally sufficient material to permit the preparation of additional specimens for a more analytical examination of the material.

5.2.1.3 Certificate of Compliance - Fabrication

The fabricator should be required by the specifications for the contract to furnish mill orders, and mill test reports, so that all steel plates can be properly identified before fabrication commences. The fabricator should also be required to certify that all steel incorporated into the structure has been manufactured in conformance with, and tested for compliance with, the contract specifications.

The certificate of compliance should be signed by the manufacturer (fabricator) of assembled materials and shall state that the materials used comply in all respects with the requirements of the specifications.

The certificate of compliance and its disposition should be developed by the engineer. A sample form is shown in Figure 5.7.

Job No.________________

Date________________

Shipment No. ________________

Bill of Lading No.____________

STRUCTURAL STEEL CERTIFICATION

I certify that all fabricated structural steel in Lot No.____________ has been manufactured and tested in accordance with the specifications for Contract No.____________, including all specifications which are a part of that contract, and is in conformance to the requirements of said specifications and test methods.

____
(Signed)

____
(Position)

____
(Company)

Figure 5.7 Certificate of Compliance, Structural Steel

5.2.2 Welding Consumables (Materials Verification)

5.2.2.1 Certificates of Compliance

A certificate of compliance for welding consumables consists of a certified test report similar to the example shown in Figure 5.8. it shows the results of tests performed on a welding consumable to verify its compliance with the requirements for AWS classification. The fabricator should furnish the engineer with one such certificate for each manufacturer's batch or control number represented in that supply of welding consumables he intends to use on the contract. These certificates should include the following:

1. The manufacturers of the consumables being certified

2. The types of consumables being certified

3. The AWS specifications and classes of the consumables being certified

4. The batch or control numbers of the consumables being certified

5. The dates and places of manufacture of the consumables represented by these batch or control numbers

6. The number of the fabricator's purchase order filled with consumables bearing these batch or control numbers

7. The results of each test performed to verify compliance with AWS requirements for the class of consumables being certified (not a generalized statement of compliance listing the minimum requirements)

(Manufacturer's Name

and Address)

CERTIFICATE OF COMPLIANCE TO REQUIREMENTS FOR WELDING ELECTRODES
Supplied to: ____________________

Date ____________ Quantity ____________ Order No. ____________ Project No. ____________

This is to certify that (Trade Name or No.) AWS classification (EXXXX) as supplied under the above order number, is of the same classification, manufacturing process, and material requirements, as the electrodes tested on ______________, 19___.

All tests required by Specification AWS A5.1 or AWS A5.5, were performed in conformance with this specification, and the above electrode met all the requirements. The electrodes are marked in conformance with AWS A5.1 or AWS A5.5.

The chemistry and mechanical properties of the deposited weld metal were as follows:

	5/32"		3/16"		1/4"	
	DC+	AC	DC+	AC	DC+	AC
Tensile Strength P.S.I.						
Yield Strength P.S.I.						
Elongation % in 2"						
Charpy V Notch Ft. Lbs. at _____°F.						
Manganese %						
Silicon %						
Nickel %						
Chromium %						
Molybdenum %						
Vanadium %						
Fillet Tests - Position as required						
Radiographic Test						

Fillet Test, Radiograph, chemistry and mechanical properties are not required for the following sizes:

____________________ Operations Supervised by

____________________ Chief Engineer

____________________ Director

Figure 5.8 Certificate of Compliance, Welding Consumables

8. The date and location where these tests were performed and the testing agency

9. The signature and registration number of the engineer in charge of the testing and the signature of the manufacturer's representative if not tested by the manufacturer.

Such testing should be performed on consumables of the same brand, class, and origin and within a year of the date of manufacture of those consumables represented by the batch numbers shown on the certificate.

Whenever the engineer or his representative receives such a certificate, they should confirm the presence of the packages, reels, or sacks bearing the designated batch or control numbers in the fabricator's supplies and monitor the use of these supplies in order to discourage substitution of uncertified consumables.

5.2.2.2 Types of Materials

Ten different AWS Specifications are used currently to designate the one-hundred-thirty classes of welding consumables that are allowed with the six kinds of welding processes that may be used in the fabrication of welded bridges. These ten AWS Specifications govern the makeup of nine permissible classes of manually shielded metal-arc electrodes for welding carbon and alloy structural steels; eight permissible classes of wire and thirty-seven permissible classes of flux for submerged arc welding of carbon and alloy structural steels (i.e., seventy possible combinations); eleven permissible classes of gas metal-arc wires for welding of carbon and alloy structural steels; thirty-two permissible classes of flux cored arc welding electrodes for welding of carbon and alloy structural steel; three permissible classes of wire and two permissible classes of flux for electroslag welding of carbon structural steel; and twenty-eight permissible classes of electrodes and wires for electrogas welding of carbon structural steels. The <u>AWS Structural Welding Code</u> allows the use of even more classes of welding consumables. The numbers here do not take into account the different brands, grades, and/or sizes within each class of consumables that must be considered in verifying these materials. Fortunately, however, the problem of locating and inspecting the fabricator's consumables in the process of verification is not as difficult as the foregoing numbers imply. The needs of design and the capabilities of the average shop generally limit the fabricator to the use of not more than three welding processes with manual and submerged arc welding being the most popular although the use of flux cored arc welding is expanding rapidly as improved electrodes are developed to overcome the variable toughness and ductility problems of the early flux cored welds. Nevertheless, the inspector should have copies of the certificates of compliance before he performs his verification inspection so that he can identify and know in advance what class of consumables he is to verify and to familiarize himself with the requirements and weaknesses of those consumables.

5.2.2.3 <u>Verification Inspection</u>

Verification inspection can take two forms. The first form is not a true verification because it generally involves having the fabricator identify his supply of consumables so that the inspector can list the consumables and batch or control numbers for which the fabricator must supply compliance certificates. This is really a quality control rather than a quality assurance function and it represents an abuse of quality assurance time. Nevertheless, it is frequently necessary to perform such an inspection in order to forestall later difficulties that may occur if the fabricator elects to proceed before he has certificates for his consumables.

The second and proper form for a verification inspection is that which the inspector performs to verify the presence and condition of consumables for which he has certificates.

Having identified the consumables, the inspector should record their general condition and the conditions under which they are stored. He should note:

1. Is prevailing weather dry or humid, hot or cold?

2. Is storage area enclosed or exposed to the elements?

3. Are manual and flux-cored electrodes sealed in cans or boxes?

4. Are electrode cans sealed or of the reclosable type?

5. Are sealed cans intact?

6. Is submerged arc flux fused, or bonded; neutral; alloyed; and/or active?

7. How is it stored - on the ground, on racks, in paper sacks, or in cans?

8. Does the storage facility have devices to reduce the humidity to levels that are low enough to prevent deterioration of the stored consumables?

9. Does this drying device record temperature and humidity?

10. Is any rust visible on stored wires or electrodes in open containers?

11. Are storage conditions such that a moisture analysis should be performed on stored fluxes or electrodes?

The inspector should be aware of the tendency of welding fluxes to take up any available moisture in the form of hydrates. Drying after a long exposure to humidity is not always effective because many of the hydrates formed do not break down at drying temperatures. In the case of coated low-hydrogen electrodes the hydrates frequently take the form of rust on the core wire under the flux as shown in Figure 5.9.

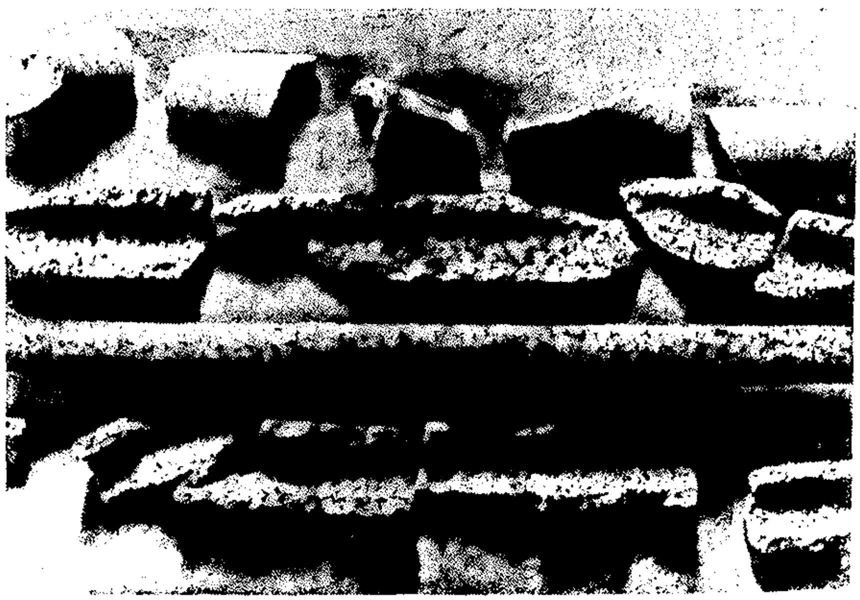


Figure 5.9 Electrodes Showing Rusty Core Wire

The inspector should note during his verification inspection what facilities the fabricator has available to dry or condition his electrodes and fluxes. Does he have a standard drying oven for low-hydrogen electrodes? Does he have a dryer to condition the flux? Will the type of flux he is using withstand any mechanical and thermal stresses imposed on it by the dryer? (Some fluxes will powder.) Are any of his drying ovens sealed from the moisture in the surrounding atmosphere? (Heating hygroscopic material in a moist atmosphere can actually accelerate hydration at low temperature ranges. One can illustrate this by pouring water over a freshly polished surface on a piece of warm steel, then try to remove the resulting rust, hydrated iron oxide, with heat.)

Hydrogen from the combined water in hydrated oxides in the flux or in rust spots on the electrode or on the steel at the weld, or from the breakdown of drawing lubricants on the welding wire, will be absorbed by the molten steel in the weld. In theory, hydrogen embrittlement should only be a problem with high strength welds and steels having yields in excess of 120 ksi. In practice it is also detrimental to the toughness of steels and welds at lower strength levels.

5.3 WELDING PROCESSES

5.3.1 General

Six different welding processes are used in welded bridge construction. Listed in order of their frequency of use, they are:

1. Shielded Metal-Arc Welding SMAW
2. Submerged Arc Welding SAW
3. Flux Cored Arc Welding FLAW

4. Gas Metal Arc Welding GMAW

5. Electroslag Welding ESW

6. Electrogas Welding EGW

An example of the current specifications that defines as well as limits the use of these various processes is provided in Figure 5.10. It could be argued that electroslag welding has supplanted gas-metal arc welding in bridge fabrication even though FHWA has restricted its use on federally funded bridges.

5.3.2 Shielded Metal-Arc Welding

This manual all-position welding method remains the most flexible and most widely used bridge welding process. It requires less capital investment then any other process. It is portable. In the hands of moderately skilled welders, it is one of the most reliable welding processes which accounts for its popularity in field welding during erection, especially at remote locations.

Its principle disadvantages are that it is a relatively slow welding process, that it wastes electrode material in the unburned stubs, and that it requires the services of reasonably good welders which are sometimes hard to find.

The Japanese have attempted to overcome these shortcomings by developing devices to feed electrodes, repeater fashion, into an automatically controlled head so that under special circumstances, one welding operator can operate several such heads simultaneously.

In the United States and Europe, however, the disadvantages described above have caused this process to be supplanted by flux-cored arc

Base Metal	Manual Shielded Metal-Arc	Submerged Arc	Gas Metal-Arc	Flux Cored-Arc	Electroslag AWS A5.25-78	Electrogas AWS A5.26-78
ASTM A36, A500, A501 A53, A139	AWS A5.1-78 E7015, 16, 18 or 28	AWS A5.7-76 EMXXK-F60 to 66 or F70 to 76	AWS A5.18-79 ER70S-2, 3, 6 or 7	AWS A5.20-79 E6XT-1, 5, 6 or 8 E7XT-1, 5, 6 or 8	FESX2-EMXXK-EW	EGX0S-2, 3 or 6 EGX2S-2, 3 or 6 EGX0T-2 thru 5 EGX2T-2 thru 5
ASTM A441	AWS A5.1-78 E7015, 16, 18	AWS A5.17-76 EMXXK-F70 to 76	AWS A5.18-79 ER70S-2, 3, 6 or 7	AWS A5.20-79 E7XT-1, 5, 6 or 8	FES72-EMXXK-EW	EG70S-2, 3 or 6 EG72S-2, 3 or 6 EG70T-2 thru 5 EG72T-2 thru 5
ASTM A572 Grade 50 A588 painted, all welds A588 unpainted, single pass fillet welds only	AWS A5.1-78 E7015, 16, or 18	AWS A5.17-76 EMXXK-F72 to 76	AWS A5.18-79 ER70S-2, 6 or 7	AWS A5.20-79 E7XT-5, 6 or 8	Not Permitted	EG72S-2, 3 or 6 EG72T-2 thru 5 (Not Permitted on A588)
ASTM A514 fillet welds A588 painted or unpainted all welds	AWS A5.5-77 E8016-C3 or E8018-C3	AWS A5.23-77 F82 to F815 EW-W or EN11-N11	AWS A5.28-79 ER80S-N11 E80C-N11	AWS A5.29-80 (T) E8XT-W or N11	Not Permitted	Not Permitted
A514 Longitudinal groove on plate over 2-1/2" thick	AWS A5.5-77 E9018-M	AWS A5.23-77 F94 to F915 EM1-M1	Not Permitted	AWS A5.29-80 (T) E9XT-M1	Not Permitted	Not Permitted
A514 plate over 2-1/2" thick A514 longitudinal groove on plate 2-1/2" or less thickness	AWS A5.5-77 E10018-M	AWS A5.23-77 F104 to F1015 EM2-M2	AWS A5.28-79 ER100S-1 or 2	AWS A5.29-80 (T) E10XT-M2	Not Permitted	Not Permitted
A514 plate 2-1/2" or less in thickness	AWS A5.5-77 E11018-M	AWS A5.23-77 F114 to F1115 EM3-M3	AWS A5.28-79 ER110S-1	AWS A5.29-80 (T) E11XT-M3	Not Permitted	Not Permitted

(T) Tentative based on pending AWS Specification

Figure 5.10 Filler Metal Requirement for All Welds welding in most shops and even in the field. Manual shield metal-arc electrodes for steel bridge welding are made with eleven kinds of flux cover and with strength levels from 60 ksi to 120 ksi. The 11 kinds of flux coatings on manual electrodes can be divided into the following types:

	Principle Components	Electrode Class	Maximum % Moisture
(1)	Cellulose	X010, X011	2 to 5%
(2)	Titania	X012, 6013	1.0%
(3)	Titania & Iron Powder	X014, X024	0.5%
(4)	Iron Powder	X027	0.5%
(5)	Iron Oxide	X020	1.0%
(6)	Low Hydrogen Iron Powder	X018, X028	0.6%
(7)	Low Hydrogen Titania	X016	0.4%

All of these electrodes are listed under the AWS A5.1 Specification. Only the low hydrogen electrodes are supplied to meet impact requirements. Thus, it is insufficient to merely specify AWS A5.1 which was common in older specifications. In the hands of an average welder, manual electrodes can be used to produce more consistent welds than any other process.

5.3.3 Submerged Arc Welding

This is probably the most prevalent shop welding process. It produces better welds more consistently than any other automatic or semiautomatic welding process. Its principal disadvantages are the large capital investment required, the fact that it must be operated in the flat position necessitating extra jigging and crane capacity to position pieces for welding, and the substantial quantities of grandular fluxes that must be stored under controlled humidity in order to prevent deterioration.

The chemical composition of the welding wires can be divided into seven categories and into six strength levels ranging from 70 to 120 ksi.

The five composition categories of submerged arc welding wires used for bridge steels having yield strengths of 50 ksi or less are:

1.	Low Manganese	EL
2.	Low Manganese with deoxidized wire	ELxxK
3.	Medium Manganese	EM
4.	Medium Manganese with deoxidized wire	EMxxK
5.	High Manganese	EH

Submerged arc welds with the low manganese wires (0.3 to 0.6%) have either marginal or insufficient toughness to meet AASHTO requirements when applied with neutral fluxes. Such welds display inconsistent toughness values when made with active and/or alloy fluxes.

The toughness of submerged arc welds made with high manganese wires (1.75 - 2.25% Mn) are sensitive to variations in section thickness, preheat, welding speed, and weld heat input. These shortcomings cause welds to display wide variations in toughness. They frequently do not meet specified requirements for weld toughness.

Submerged arc welds made with wires of medium manganese content (0.85 to 1.40%) with less active fluxes display the most consistent levels of toughness under a wide variety of shop conditions.

Submerged arc fluxes are prepared by:

1. Prefusing the components and grinding or shotting to size

2. Bonding the components with water glass or potassium silicate, pelletizing, and ground to size

3. Agglomerating the components by sintering with a ceramic binder.

Prefused and agglomerated fluxes generally contain little or no alloy and must therefore be used with alloy wires if welds are to meet stringent mechanical requirements. Such fluxes are exceedingly uniform, non-hygroscopic, and mechanically stable. Hence, they keep well in storage requiring a minimum of precautions against absorption of moisture.

Bonded fluxes must be used when fluxes are to be alloyed. Bonded fluxes are mechanically weak, hygroscopic, tend to powder easily, and are difficult to store without absorbing moisture. Unalloyed or EL wires are frequently used with these types of fluxes in order to take advantage of the alloy contributed by the flux. Unfortunately, the alloy composition and the mechanical properties of the weld may vary widely with inconsequential changes in procedure.

5.3.4 Gas Metal-Arc Welding

Gas metal-arc welding is a semi-automatic or automatic welding process that utilizes a consumable metal electrode to make a weld under a protective blanket of inert or non-oxidizing gas. It may be used in any position. The shielding gases used are generally helium or argon with a small percentage of oxygen. Helium is generally used to shield welds made on certain metals such as aluminum because it improves the transfer of heat from the arc, a desirable feature when welding metals that have high conductivity. Argon is generally used to shield welds made on steels. Carbon dioxide may also be used in some cases and steam has been used as a shielding gas in some third world countries.

5.3.4.1 Short-Arc vs Spray-Arc

Gas metal-arc or MIG welding (as it is more commonly called) may be used in either of two modes. These are the "spray arc" mode in which the current densities and voltages are high enough to separate melted metal from the tip of the electrode in the form of an ionized spray. In the "short-arc" mode the tip of the electrode melts in the arc and is mechanically transferred to the puddle by the wire feed as the electrode shorts out in the weld puddle.

The short-arc mode provides a high deposition rate while the spray-arc mode provides welds with better mechanical properties.

Theoretically, MIG welds made with the spray-arc technique should provide better welds than any other welding process except TIG (Tungsten Inert Gas). In laboratory tests and under precise controls utilized in critical applications like those encountered in the space industry and in naval ship building, this has been true.

In the average bridge application, however, gas metal-arc welding has often been disastrous because of the inability to hold the shielding. Field welding, for instance, often involves welding in substantial winds with inadequate protection. Consequently, the gas shielding may simply blow away leaving the arc unprotected. Gas metal-arc welding in shops has been more successful. But, open construction areas and windy locations that prevail in many bridge shops makes loss of the gas shield a problem even in shop welding. Secondly, most bridge shops operate under production quotas that cause them to strive for high deposition rates. Under these conditions, the weld passes are often large and subjected to excessively rapid cooling rates because the speed with which they are made and the relatively large heat loss rate promoted by the absence of an insulating flux blanket. This makes these welds excessively hard and sensitive to any defects when applied to bridge steels.

High strength submerged arc-welds (for use on A514 steels, for instance) are made by using electrode wires and/or fluxes containing extra nickel, molybdenum, manganese and/or chromium.

5.3.5 Flux-Cored Arc-Welding

This is an all position semi-automatic or automatic welding process that utilizes a hollow tubular electrode to hold the shielding flux. It is used with or without shielding gas. The process has been used commercially for about 20 years. Welds made with the early version of these electrodes were extremely brittle with Charpy V-notch impact strengths of 2 to 10 ft lbs at 32°F. The flux requirements for good welding with this process were not clearly understood. Within the last two years, these electrodes have been improved radically so that reasonably good welds can be made by using this welding process. The resulting weld metal toughnesses, while not spectacular, are satisfactory for most applications with impact strengths from 10 to 40 ft lbs at 32°F.

American and Japanese fabricators are beginning to utilize this welding process in fabricating off-shore drilling rigs and ships.

Unfortunately, when applied to bridge fabrication, the process has not proven consistent enough to assure the level of toughness required for use in fracture critical bridge components. The promise remains, however, and many shops big and small are beginning to replace manual shielded metal-arc welding equipment with equipment utilizing this process without gas shielding. This equipment can be used both semi-automatically and automatically. It eliminates storage and procurement problems involved in using shielding gas granular flux or flux covered electrodes.

5.3.6 Electroslag

This welding process involves making a weld by using the electric resistance of a molten conductive slag to melt filler metals and fuses the weld puddle into the faces of the joint. The welding is performed in the vertical position. It is completed in one pass. Welding speed varies from 1/2 inch to 2 inches per minute depending on the thickness of the joint. The weld puddle is contained on the sides by stationary or movable copper shoes as shown in Figure 5.11. The movable shoes are water cooled and move up the sides of the joint with the weld puddle. The equipment is shown in Figure 5.12. The weld metal heat-affected zones of welds made by this process are subject to extreme grain growth and segregation. Thus, the weld metal tends to be extremely brittle with impact strengths that seldom exceed 15 ft lbs even at 70°F and frequently show only 5 to 10 ft lbs at this temperature.

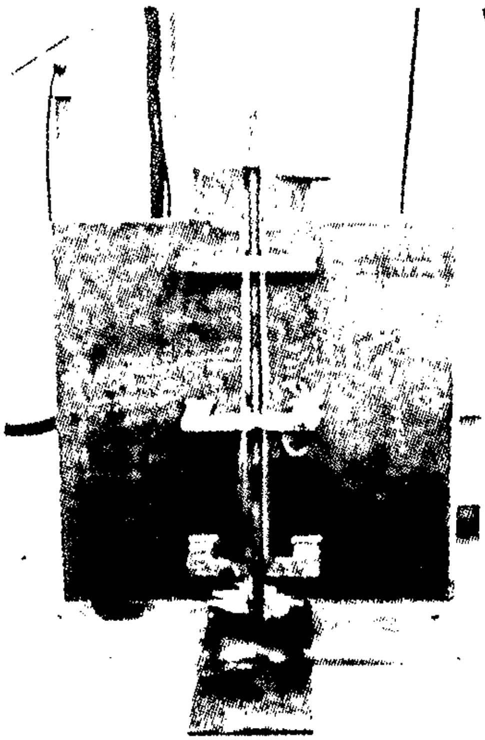
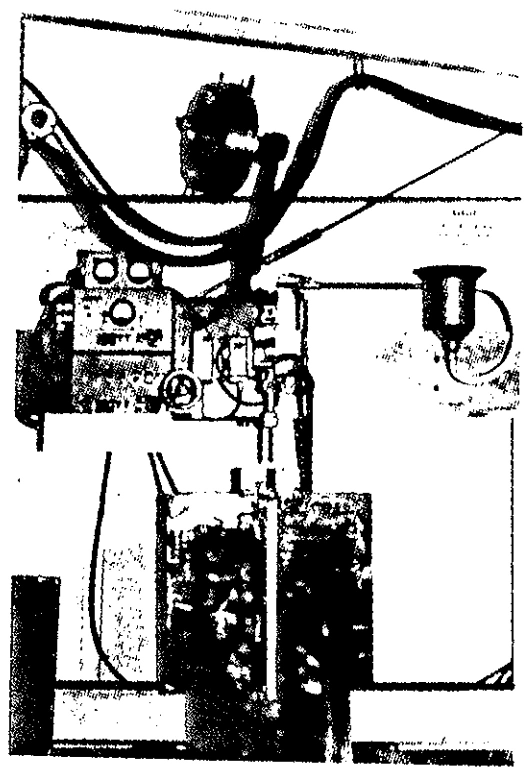


Figure 5.11 Electroslag Weld Plate With Copper Backing Shoe Removed to Show Filler Metal Guide Tube

Figure 5.12 Electroslag Weld Joint Showing Welding Equipment in Place



5.4 QUALIFICATION OF WELDERS AND WELDING OPERATORS

5.4.1 General

5.4.1.1 Definitions

5.4.1.1.1 Welder

A welder is one who makes a weld with a hand held device. This device may be a clamp held electrode for shielded metal-arc welding, a wire "gun" for gas shielded arc or flux cored arc-welding, a tungsten electrode "gun" for tungsten inert gas welding, or a "squirt gun" for submerged arc welding.

5.4.1.1.2 Welding Operator

A welding "operator" is one who makes a weld by operating a machine that welds without being guided or manipulated by hand during the welding operation. The operator has merely to set the electrode position and align the track to guide the machine over the joint to be welded, load the necessary welding consumables into the machine, preset it to operate at the appropriate speed, amperage, voltage, wire feed rate, and start it. His subsequent duties require him to continuously inspect the weld so that he can correct welding deficiencies as they appear by altering the operation of the welding machine.

5.4.1.2 Qualification and Work Records

5.4.1.2.1 Roster Submission

Welders and welding operators should be "qualified" before they are allowed to make welds on bridge components. Customarily, the fabricator will prepare a roster of his welders and operators listing each person's qualifications and submit this information to the engineer for review before work begins. Lacking this information, the engineer should notify the fabricator in writing that work cannot proceed until this information is submitted and approved by the engineer.

5.4.1.2.2 Examination of Records

The qualification test record of each welder or welding operator should be examined to see if he is qualified for the particular welding processes he will use in fabrication. This qualification covers the steels, thicknesses, consumables and positions that will be used with each of the assigned welding processes. It is customarily required that a welder or operator have no break longer than six months in his work record (one year for pile butt welders). When this time interval has been exceeded, the engineer may require requalification.

5.4.1.2.3 Requalification

5.4.1.2.3.1 Determination

The decision whether or not to require requalification and whether or not such requalification should be applied loosely or rigorously depends on the depth and breadth of experience shown on the individual's work record plus evidence of an ability to produce welds that have been free of defects as shown on NDT records for prior work or the statement of an inspector who is familiar with the person's ability and his performance on previous contracts.

5.4.1.2.3.2 Documentation

If the welder's or operator's qualification is renewed without or with limited requalification testing, the engineer should prepare a document indicating that the qualification has been renewed without a full requalification test. The engineer may limit activities of the person who qualifies in this manner.

5.4.1.3 Testing

5.4.1.3.1 Qualification Welding

When it has been established that tests must be performed to qualify a welder or welding operator, the fabricator and the engineer's inspector must designate a time for the preparation of the qualification weld so that the inspector can witness the welding. The inspector should record the type of qualification weld being made; i.e., name of candidate, process, joint, steel, thickness, consumables used, positions, etc. He should be present most of the time in order to insure that the entire weld is made by the candidate and in the position being qualified. He should also be alert to father for son, brother for brother, and friend for friend substitutions in the preparation of qualification welds. The inspector should mark the qualification weld in some manner so that its identity can be confirmed on the bend test specimens that must be machined from it.

5.4.1.3.2 Qualification Testing

The test specimens prepared from the qualification welds generally go directly to a responsible testing agency. The engineer's inspector is notified when this occurs so that he can be present to confirm the identity of the specimens and the results of the tests. Since this is not always convenient, one State has established a practice of requiring that tested specimens be submitted to the inspector for review if he was unable to be present for the testing. This State also requires that its testing agencies comply with the requirements of ASTM E 329. This specification seeks to improve the accountability of testing agencies by establishing a minimum complement of testing equipment with periodic calibration requirements for this equipment and establishes minimum requirements for the education and experience of those persons in responsible positions at such agencies. These requirements tend to eliminate incompetent agencies.

5.4.1.3.3 Radiographic Testing

Most welding specifications permit the use of radiographic testing of the entire qualification weld in lieu of mechanical testing of the specimens. Radiographic methods will detect a lack of soundness. Conversely, a purely mechanical test may miss the internal porosity and defects caused by poor welding. Both methods can be applied advantageously, radiography to eliminate needless testing of unsound welds and mechanical testing to eliminate those welders and operators who misapply welding variables.

5.4.1.4 Strength and Steel Limits

All welder and welding operator qualifications for work on fracture critical structures should be limited to the strength levels of the electrode materials used to make the qualification weld even though this exceeds AWS requirements. This is desirable not because the higher strength electrode materials are more difficult to use but because the higher the required weld strength the greater the need to assure weld quality in order to avoid brittle fracture. Defects that are inconsequential in low strength welds are liable to initiate fractures in high strength welds. The errors in welding technique that create these defects are more likely to be revealed by requiring qualification tests on high strength welds. Extra welder and operator qualifications should be required for welds on fracture critical structures which are to be made of steels with minimum specified yield strengths in excess of 50 ksi or of steels like A588 or A514 with alloy contents that are high enough to cause excessive hardening and embrittlement in the weld heat affected zone.

5.4.2 Welder Qualification

5.4.2.1 Thickness and Process Limits

Welder qualification testing includes testing for competency in manual shielded metal arc-welding (i.e., stick welding) and in various semiautomatic welding methods. Qualification welds must be made on 1" thick test joints if the welder is to weld on joints over 3/4" thick. Otherwise, the qualification welds may be made on 3/8" thick test joints. Two side bend test specimens are cut from the 1" qualification welds. One root and one face bend are cut from the 3/8" qualification weld.

5.4.2.2 Position Limits

5.4.2.2.1 SMAW, GSAW, FCAW

SMAW, GSAW, and FCAW (shielded metal arc-welding, gas shielded arc-welding, and flux cored arc-welding, respectively) processes are suitable for welding in all positions. It follows that qualification welds must be made in the most arduous position(s) the candidate is assigned to weld in, and the assigned ascending rank order of difficulty among the four groove welding positions goes from flat, to horizontal, to overhead, to vertical. Hence, a successful flat groove qualification weld qualifies the welder only for flat groove welding but a successful vertical groove qualification weld qualifies the welder for groove welding in any except the overhead position. Overhead and horizontal groove qualification welds are also not interchangeable but either qualifies the welder for flat groove welding. The same order of difficulty for fillet welding positions make horizontal and flat fillet weld qualifications interchangeable thus differing slightly from groove weld requirements.

5.4.2.2.2 SAW

Welder qualification for semi-automatic submerged arc-welding (i.e., "squirt" welding) does not extend beyond the flat position for groove welding or the flat and horizontal positions for fillet welding simply because gravity does not permit the use of a granular flux cover for anything other than flat groove welds or flat and horizontal fillet welds.

5.4.2.3 Electrode Class Limits

A graded series exists among the various classes of electrodes used in manual shielded metal arc-welding generally due to the difference in the fluidity and conductivity of the fluxes used. This series appears in every welding specification and it circumscribes welder qualifications in the same manner as position limits. In general the all-position low-hydrogen electrodes are the most difficult to handle and hence a successful qualification weld made with these electrodes will qualify the welder to use any other class of manual shielded metal-arc electrode having the same strength level.

Qualification for welding with flux-cored electrodes presents a similar problem, but at least one State sidesteps this difficulty by limiting all welder qualifications for semiautomatic welding procedures to the consumable combination actually used to make the qualification weld.

5.4.2.4 Control of Variables

The candidate in every welder qualification test should be able to make all of the welding machine adjustments required to secure a good weld. He should be required to adjust his own machine settings when performing the qualification weld.

5.4.3 Operator Qualification

5.4.3.1 Process and Position Limits

Operator qualification tests an operator's ability to set, align, and operate a fully automatic welding machine for one or more of the following processes:

1. SAW - submerged arc-welding

2. GSAW - gas shielded arc welding

3. FCAW - flux cored arc-welding

4. ESW - electroslag welding

5. EGW - electrogas welding

Process 1 can only be used in the flat position. Processes 2 and 3 are generally used in the flat position with occasional use in the vertical and horizontal positions. Processes 4 and 5 are used only in the vertical position.

5.4.3.2 Simultaneous Qualification of Operators and Procedures

Setting up an operator qualification test is time consuming and wasteful of materials. Consequently, specifications permit the simultaneous qualification of welding operators and welding procedures. Many fabricators elect this option.

5.4.3.3 Welding Operator Qualification

Operator qualification welds for steels with minimum specified yields of 50 ksi or less may be made on any bridge steel having a minimum yield strength between 42 and 51 ksi. Qualification welds for steels with yield strengths over 50 ksi should be made on the same type and class of steel the operator is to weld on the contract.

5.4.3.4 Procedure Requirements

The welding procedures used to make operator qualification welds should duplicate the welding procedures established by the specifications.

5.4.3.5 Operator Qualification Except Electroslag/Gas

5.4.3.5.1 Thickness Requirements

The standard operator qualification weld for each automatic butt welding process, except for electrogas and electroslag welding, consists of one butt welded joint not less than 15 inches long made in position using the maximum thickness to be welded in that position, for each position that the operator is to qualify, except that the thickness of the qualification weld should always be greater than 3/8" but need not exceed 1" even when thicker joints are to be welded by the operator. A successful qualification weld qualifies the operator for welding on equal or lesser thicknesses or on all thicknesses if the qualification weld was 1" or more in thickness.

5.4.3.5.2 Position Requirements

If an operator prepares a successful qualification butt weld in other than the flat position, he also qualifies for butt welding in the flat position and fillet welding in the flat, horizontal and test positions. If the successful qualification butt weld was made in the vertical position, the extra qualification extends to horizontal as well as flat butt welding.

5.4.3.6 Operator Qualification for Multiple Head Welding

It should be noted that special tests are advisable for automatic welding processes that use machines which make two or more separate welds simultaneously. In such cases it is advisable to require the operator to qualify both welds in order to test his skills at setting and aligning the machine to make two good welds simultaneously and at monitoring and adjusting the machine to maintain proper operation. These kinds of welds are generally fillet welds and welder qualification test methods for fillet welds are usually used to evaluate their quality. Most shops have only one or two of these machines. It follows therefore, that only a few operators are required for these machines. Most contractors find it advisable to exercise their option to qualify operators and welding procedures simultaneously.

5.5 QUALIFICATION OF WELDING PROCEDURES

5.5.1 Prequalified Welding Procedures

5.5.1.1 Specified Limits

AWS Specifications allow all bridge steels except A242 or A618, grade I, to be welded with any process, other than multiple electrode processes using gas shielding or flux-cored wires and electroslag or electrogas, without requiring the welding procedure used with that process to be qualified by test. Testing procedures are performed by qualified welders or welding operators using joints, welding techniques, and levels of workmanship that are designated acceptable in Sections 2, 3, 4 and 9 of the AWS Specifications.

5.5.1.2 Significance of Steel, Joints, and Consumables

Unfortunately, some of the welding procedures that are prequalified may not be safe to use under certain conditions or with certain materials.

5.5.1.2.1 Effects of Steel Composition

For example, a welding procedure that is prequalified for use on A514 steel may have a heat input level that is satisfactory for use on grade F steels. When applied to grades A or H, however, it may reduce both the strength and toughness of the weld heat affected zone to the extent where the weldment is unsafe to use. Conversely, the levels and rates of weld heat input that produce a satisfactory weld on grades A or H may leave the heat affected zone at a weld on grade F steel so hard and brittle that it would be subject to brittle fracture. Similar problems arise when prequalified welding procedures are applied to welds on different grades of A588 steel or used interchangeably on bridge steels with minimum specified yields of 50 ksi, or less.

5.5.1.2.2 Effects of Joint Geometry and Thickness

Joint configurations and thicknesses influence the mechanical properties of the weld metal and heat affected zones of welded joints, independent of weld procedure. A prequalified welding procedure that is satisfactory for a 1/2" joint may produce brittle material in 2" joints.

5.5.1.2.3 Effects of Welding Consumables

The selection of welding consumables influences the properties of a weld joint depending on the thickness of the weld joint and the composition of the steel.

The requirements for prequalifying a weld procedure do not allow for the effects of variations in the chemical composition of either base metal or consumables and changes in cooling rate associated with different thicknesses and joint configuration. Consequently, the strengths, toughnesses and residual stress levels incorporated in prequalified welds can vary significantly and in ways that may not be fully anticipated in design.

5.5.1.3 Remedies

To combat this problem some State codes and, in particular, the AASHTO Code for fracture critical structures, limits the use of prequalified welding procedures to noncritical applications. One State, for instance, does not prequalify any welding procedure that is to be used on A588 or A514 steels nor any welding procedure made with semiautomatic or fully automatic welding processes on any steel.

5.5.2 Qualification of Weld Procedures by Test

5.5.2.1 Specified Limits vs Testing Significance

Even a successful weld procedure test may not ensure the suitability of an application if the test is not managed properly. Many codes allow welding procedure tests to be performed on test joints in thicknesses which are not representative of the joints used in construction nor on test joints of steels with different chemical compositions (i.e., of different specification, class, or grade) than those used in construction. A successful procedure test on a 1" test joint of A572 grade 50 steel will suffice to qualify the procedure for use on A36, A441, A572 grade 42, or any of the eight or more types of A588 steel in any thickness. Obviously, such a qualification is only slightly better than not using a prequalified welding procedure.

5.5.2.2 Circumvention of Specifications

Another problem also occurs when a weld procedure is qualified by test. Many fabricators use the procedure qualification test as a device to circumvent many of the requirements in a welding code. The AWS Code, for instance, lists the requirements that may be circumvented by procedure testing in Table E2, Appendix E, of the Code.

The combined effects of the kinds of code deficiencies described in the previous paragraphs can result in the qualification of welding procedures that may be totally incompatible with their intended use in the structure.

5.5.2.3 Remedies

Many codes attempt to counter these deficiencies by imposing more stringent limits on the joint thicknesses and the kinds of steel that may be qualified for use with the welding procedure on the basis of a single procedure qualification test. At least one State requires that procedures for use on A588 steel be qualified on the same type of A588 steel that will be used in the structure and, if possible, at the same heats. Such procedure qualification tests are performed on the maximum thickness to be used in construction even if it is four inches thick. In each case, such qualifications are extended downward to one-half the thickness of the test joint provided the joint configuration remains constant and provided the welding parameters remain within prescribed limits.

Qualification tests on A514 steels should be performed not only on the same grade, but also on one of the A514 heats used in the structure. The weldability of these steels can vary significantly with the source of the steel even when the grade is held constant. In these weld procedure tests on 100 ksi yield steels, the qualification is extended downward to 75 percent of the test plate thickness; a slightly more stringent limit than required by most codes.

5.5.3 Defining Qualification Requirements

5.5.3.1 Contractor's Responsibility

A contractor will generally formulate his procedure qualification test requirements in the process of preparing a bid for a contract. Thus, he usually prepares for the first prefabrication conference his proposed welding procedures which are submitted to the engineer for approval. If he does not, the engineer should advise him of this need at the time the contract plans are submitted for approval.

5.5.3.2 Engineer's Responsibility

When the engineer receives the contract plans for review, he should make sure all the weld joints are detailed somewhere in the plans and that each joint is matched with a prequalified, a previously qualified, and/or a "to be qualified" welding procedure. Possession of the contractor's documented list of proposed welding procedures simplifies this matching operation and enables the engineer to determine rapidly those weld procedures that will have to be established with weld procedure qualification tests before being applied to the work. If no list is submitted, it becomes the engineer's responsibility to notify the contractor that work cannot begin until he submits his proposed welding procedures. The engineer will have to itemize the weld joints in order to determine how many welding procedures are needed to satisfy all the weld joints.

5.5.4 Preparing the Procedure Qualification Weld

Once the welding procedures have been established and divided into prequalified, previously qualified, and to be qualified categories in accordance with the engineer's approval, the contractor prepares each of the necessary weld procedure qualification plates under the surveillance of the engineer's inspector. Where possible, the inspector measures and records the weld parameters (preheat, voltage, amperage, speed, etc.) used to make each test weld.

5.5.5 Testing the Procedure Qualification Weld

After each test weld has been completed, the contractor submits samples to the testing agency or laboratory of his choice. They are cut into specimens and tested in accordance with specified qualification requirements. Although not many specifications require it, such testing agencies should conform to the requirements of ASTM E329. This minimizes (but does not eliminate) the possibility of incompetent testing. The engineer's inspector should be present to witness the results of the tests.

5.5.6 Cautions

Since the contractor pays for testing, he is often reluctant to share information about those weld procedures that have failed. The engineer must be watchful lest he find that the weld procedure qualification test report submitted to him covers the last and only successful test following a series of unsuccessful tests performed on the same welding procedure. Some agencies attempt to circumvent this subterfuge by requiring that all tests performed on each procedure be reported, whether good or bad. This requirement is difficult to enforce, it represents one of the major problems for quality assurance.

5.6 SHOP FABRICATION, ASSEMBLY AND WELDING

At this stage of fabrication, tentative sequences of inspection during fabrication should be established, such as:

1. Prefabrication

   a. Review of specification documents

   b. Review of contract plans

   c. Hold prefab conference with fabricator

   d. Determine welding procedures necessary

   e. Verify welder qualification

   f. Verify wire, flux and electrode certifications

   g. Inspect storage of electrodes, wire and fluxes

      1. Electrode ovens

      2. Flux ovens

      3. Storage

   h. Verify credentials of fabricator's quality control personnel

   i. Inspect fabricator's testing agency for compliance with contract specifications

   j. Examine fabricator's nondestructive testing agency and personnel for compliance with contract specifications

2. Material

   a. Obtain mill order - mill test reports

   b. Obtain certificate of compliance for steel

   c. Check materials against mill test reports

   d. Sample any stock material (if required)

3. Cutting, preparing joints and welding

   a. Check burning (oxyacetylene cutting, etc.) of material (edge condition, roughness, etc.)

   b. Check transfer of heat or plate identification numbers on cut plates

   c. Check joint preparation and welding of flanges

      1. Procedure

      2. Electrodes, wire flux combination

      3. Preheat and/or postheat

      4. Discontinuities in material prepared for welding

      5. Plate edges for injurious defects

   d. Proper grinding for radiographic testing and ultrasonic testing

1. Examine transitions, etc.

2. Check specifications and plans for extent of nondestructive testing

e. Check all weld repairs

f. Check flanges for straightness

g. Check preparation and welding of bearing assemblies and cutting of stiffeners

4. Assembly

a. Check fitup of flanges to webs

b. Check preheat and/or postheat, if required

c. Check fillet, butt, and groove weld sequences and procedures

d. Record the results of NDT by heat number

1. Check for completeness and competence in the performance of nondestructive testing

e. Check fitup of stiffeners, gussets and bearing plates

f. Check webs and flanges for possible areas of deviations of camber and web flatness and depth tolerances

5. Final Shop Check

   a. Recheck web and flange after welding for deviation from web flatness and depth tolerances, flange sweep, tilt and camber

   b. Inspect for proper cleaning, grinding, pickup of nicks and gouges

   c. Inspect the welds and review the welding record to verify the acceptability of the welds

   d. Verify final camber and other necessary dimensions

   e. Inspect preparation of field weld joints and splice fitup

   f. Check bolted joints

6. Blast cleaning and painting (not included)--only check for cleanup of grinding and welding

7. Final Inspection

   a. Recheck to see that fabrication agrees with contract plans and approved drawings

   b. Check shipment for the presence of unauthorized welds used to attach clips, dogs and hold-downs for shipping purposes.

5.6.1 General Shop Fabrication, Welding and Materials

(1) Fitup for Welding

Two of the most significant factors in the welding of fabricated members are alignment and fitup of beams, flanges and web plates. Production of acceptable welded joints depends on the use of the proper weld joint. This involves the selection of the appropriate weld geometry, root opening, disposition of sound weld metal with full penetration of the root pass, and good welding techniques. These cannot be accomplished without proper alignment and fitup. Figures 5.13-5.16 show common fitup problems.

(2) Edge Preparation of the Weld Joint

The following aspects of joint preparation should be observed:

a. Beveling of the joint to see that there are no nicks or gouges that can interfere with the welding operation

b. Root opening and beveling

The root opening of the joint should be clean and ground to the prescribed dimensions. The current AWS tolerances for root openings may be too liberal for certain types of joints or for use with higher strength steels.

(3) Cleaning: Grinding and/or Blast Cleaning

a. Butt welds and/or groove welds after flange cutting should be ground smooth to ensure a clean welding surface

b. Fillet welds should be deposited on clean surfaces preferably blasted, cleaned of all rust, dirt, grease, oil, paint and mill scale to insure satisfactory welds.

Figure 5.13 Excessive Root Gap

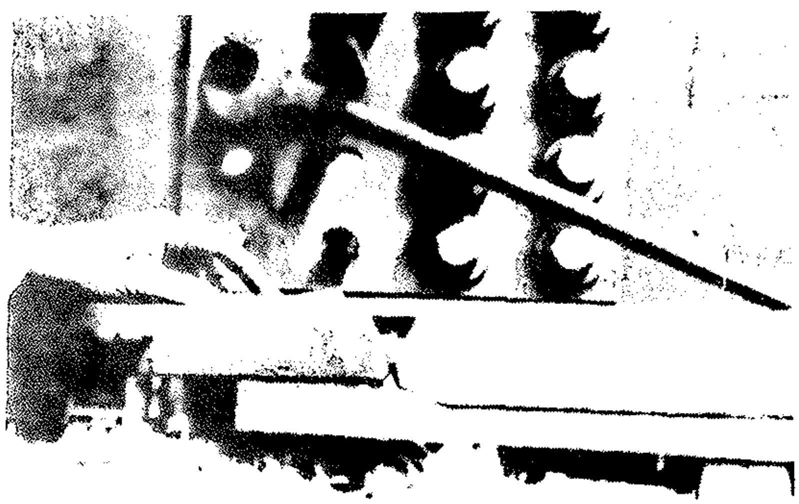


Figure 5.14 Proper Fit-Up

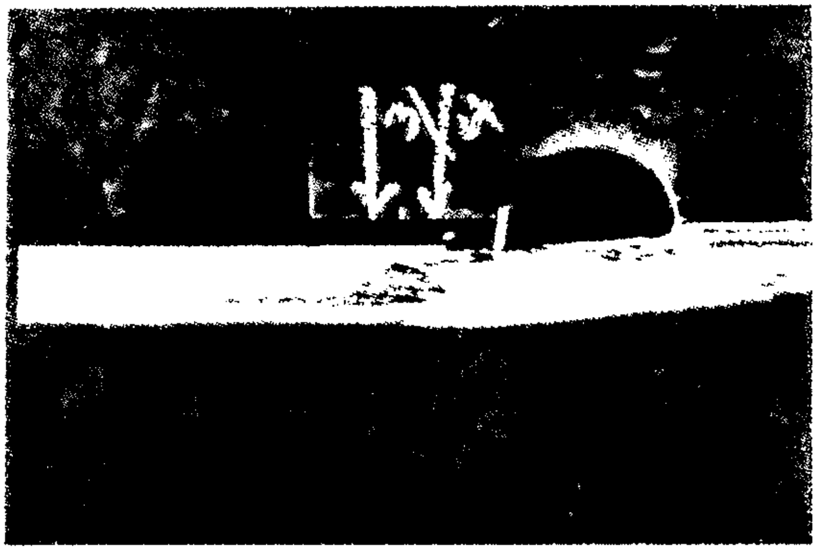


Figure 5.15 Excessive Web Trimming

Figure 5.16 Repair Made Necessary by Excessive Web Trimming

(4) Backing:

Backing bars should be permitted only where shown on the plans or approved shop drawings. Back-up bars have been a source of crack starters due to lack of weld penetration, poor fitup, lack of fusion, and poor workmanship. They are very difficult to test, nondestructively.

(5) Tack Welding

Tack welding can be a source of problems in the finish weld or joint in the form of cracking, lack of fusion, lack of penetration, slag inclusions and porosity. Tack welds should require the same quality and workmanship as the final weld. Tack welds which are incorporated into the final weld should be made with electrodes meeting the requirements of the final welds, and should be thoroughly cleaned. Large multiple pass tack welds should have cascaded ends in order for the finish weld or weld passes to tie into the cascaded ends.

Tack welds involve the application of comparatively little welding heat. Consequently, they cool very rapidly to, or near, the temperature of the plate.

Most workmen, welders and supervisors regard tack welds as minor items of little or no consequence or concern and often apply tack welding rather indiscriminately, with little or no preheat. Cracks frequently occur under the tack welds of heavy plate assemblies and particularly on high strength steel parts and assemblies.

When fabricating bridge girders, one of the most critical places to tack weld is at stiffeners, gussets, or connection (attachments) ends. The welding usually starts at a stiffener end on top of an existing tack weld. The tack weld may or may not be cracked, but the welding process may not remelt the tack weld until the process has developed full welding heat.

Highest stresses and stress concentrations usually exist at attachment ends, or corners, so starting or stopping main weld runs or weld passes on top of questionable or defective tack welds is not good welding practice. Tack welds should be located away from end or corner locations. This is especially important if the design called for vertical stiffeners welded to tension flanges.

Tack welding and welding of clips or "dogs" for fabrication must be controlled. There have been cases where these clips have been welded to fabricated parts, a weld crack started which propagated into the base metal. After the clip was burned off and the surface was ground smooth, the cracks in the base metal remained and were not detected until a later date. If this type of crack is allowed to remain, it could lead to serious trouble or failure.

All temporary welds should be removed unless otherwise permitted by the engineer. When these temporary welds are removed, the surface should be ground flush with the original surface. There should be no temporary welds in tension flanges unless approved by the engineer. All allowable welds should be shown on the approved shop drawings.

It is important that a tack welder be qualified before being allowed to tack weld on the job. A minimum test should be the welder qualification weld for limited thickness.

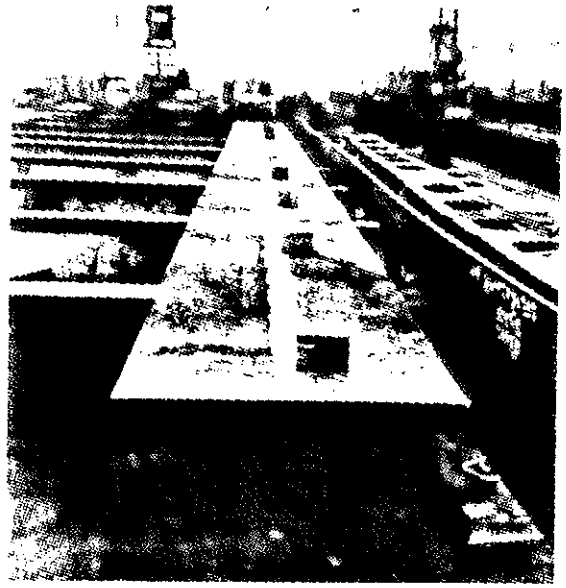


Figure 5.17 Welded Web Alignment Clips

Figure 5.17 shows an alloy steel flange that has been ground in the center in order to avoid incurring rollover due to the presence of mill scale. Welding web alignment clips to an alloy steel flange in order to simplify jigging is very poor practice. Transverse cracks in the flange have been found beneath such clip welds, and repairing cracks in alloy flanges is difficult without causing further damage.

(5) Runoff Tabs

Runoff tabs should be of similar materials as the weld joint material and the same weld joint or groove profile dimensions as illustrated in Figure 5.18.

Special attention should be given to runoff tabs, especially for flange splices at the weld joint end or flange edge where the runoff tabs extend for the weld runoff. After completion of the weld joint the runoff tabs are generally removed by flame cutting and the cut edge area ground flush with the flange sides and flange edges.

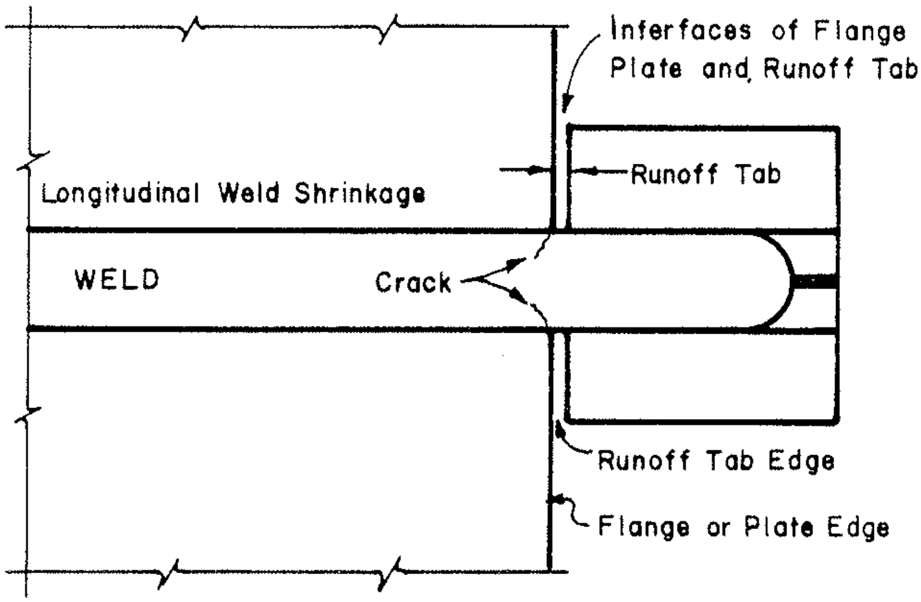


Figure 5.18 Runoff Tabs

Radiographic testing can be performed either before or after removal of the runoff tabs. Some prefer to radiographically test before the runoff tabs are removed, while others choose to remove the runoff tabs before testing. It has been demonstrated that radiographic and ultrasonic tests do not always show weld defects at the flange edge where the runoff tab is attached either before or after its removal. These edges should be examined by magnetic particle or dye penetrant in addition to RT and UT in order to detect small cracks that may exist due to longitudinal weld shrinkage in the presence of the flange edge to runoff tab connection interface. These runoff tab to plate edge cracks are often quite small and generally do not penetrate back into the weld material very deep; however, there have been exceptions where the crack penetrates to a depth of 3/8" to 1/2". A small crack can be ground out while deeper cracks need repair.

Welding defects such as cracks, incomplete penetration and lack of fusion are all dangerous and prevalent at the ends of welds at plate edges. Such defects can be traced to and apparently occur because runoff tabs are either too short or are not utilized for the full length of the runoff tab for start up. Welding conditions may not have stabilized to full welding heat over this reduced length.

When making up such a weld joint, there is a fitup interface between the runoff tab and the flange edge and any such interface is the equivalent of a crack potential facing onto the sides of the weld at the flange edge or edges.

(6) Backgouging of Weld Joints

Backgouging of weld joints and the shape of backgouged grooves is of major importance for good welding. Most of the backgouging of weld joints is done by air carbon-arc gouging, but flame gouging, grinding, or combinations of gouging and grinding are often performed. These methods are allowed by AWS Structural Welding Code, Paragraph 3.3.5, which states, "Grooves produced by gouging shall be in accordance with groove profile dimensions as specified in Figure 2.9.1 and 2.10.1." When backgouging is used, a welding procedure should be made qualifying the backgouging, shape of the weld joint, and cleanup grinding.

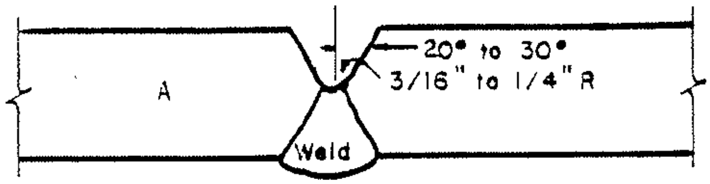
When backgouging a weld joint, the weld joint assumes a rounded bottom "U-groove" shape, which is proper. But what sort of "U" shape, what root radius, what minimum included angle will produce an ideal weld groove?

Some workers or welders who do the backgouging use relatively large carbon electrodes, and gouge or shave the sides of the groove with a fairly wide "U" shape and a favorable included angle. Of course, this is dependent on the thickness of the weld joint. Others will elect to use small carbon electrodes, shave or gouge little or none of the sides of the groove and produce backgouged groove with very narrow steep sides, and a small radius at the base of the groove which is not favorable for good welding. This small radius, narrow groove and burn-through are illustrated in Figure 5.19.

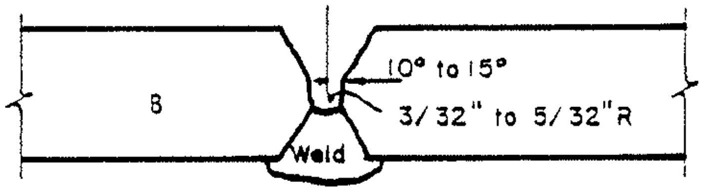
Figure 5.20 illustrates the kind of defect that can arise from a deep narrow backgouge.

Figure 5.21 shows an effective correction of a burn-through by proper backgouging and welding.

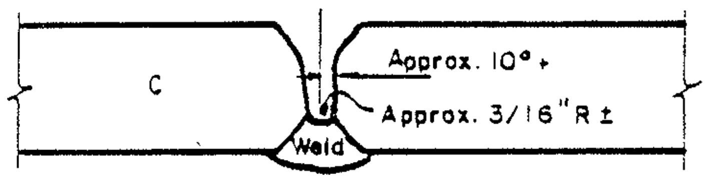
Figures 5.22 - 24 show the effective and proper use of backgouging to remove defects.



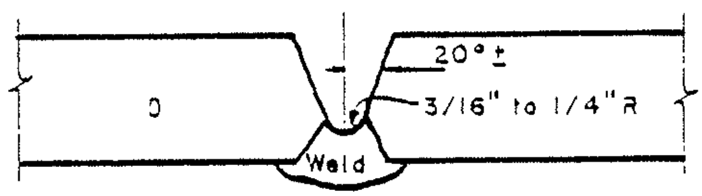
Backgouged groove with a good "U" shape radius and favorable included angle.



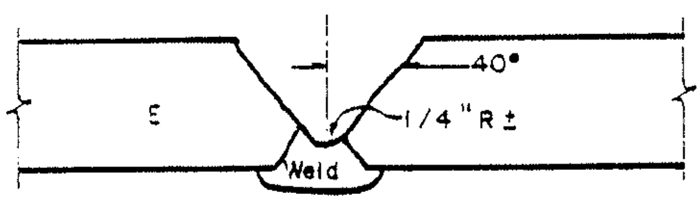
Backgouged groove made with a small carbon electrode. The small "U" shape radius and narrow groove are not favorable for good welding.



A backgouged groove with a small narrow "U" shape that is too deep is not favorable for good welding.



A backgouged groove with good "U" shape radius with a favorable included angle. However, the groove is too deep and will distort when welded.

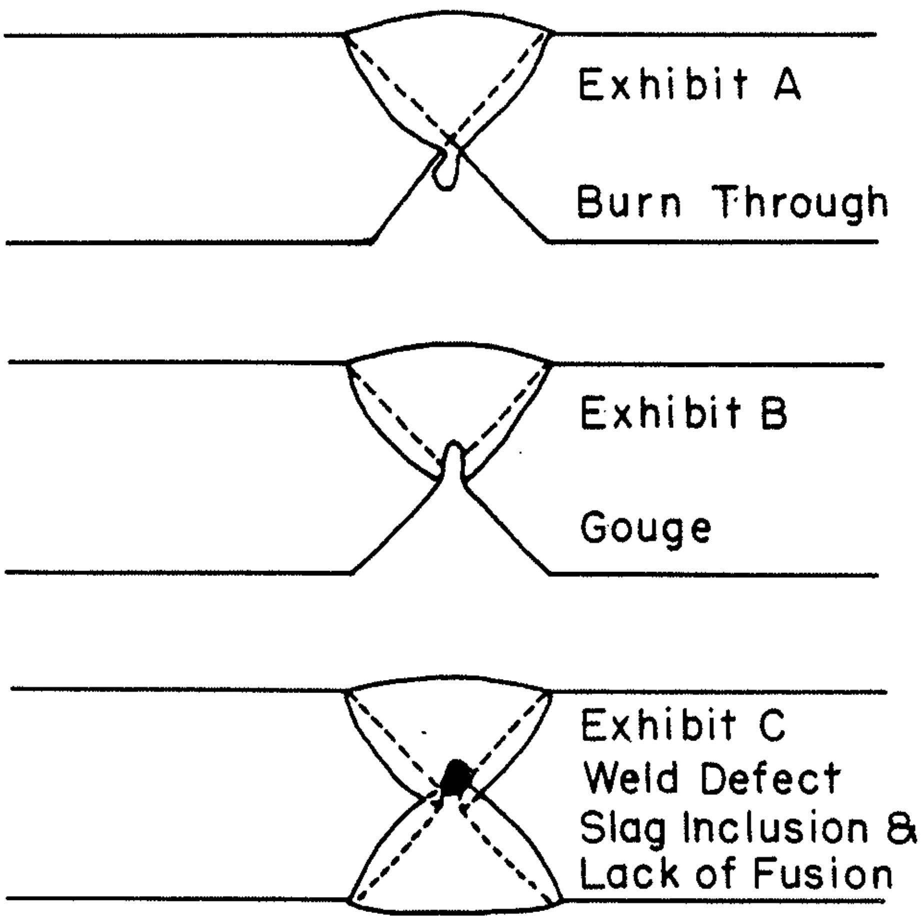


A groove backgouged to extreme. Sometimes it will burn through and distort when welded. This is not a good welding condition.

Backgouge groove sketches B, C, D & E are quite common if workmen are not supervised or instructed properly.

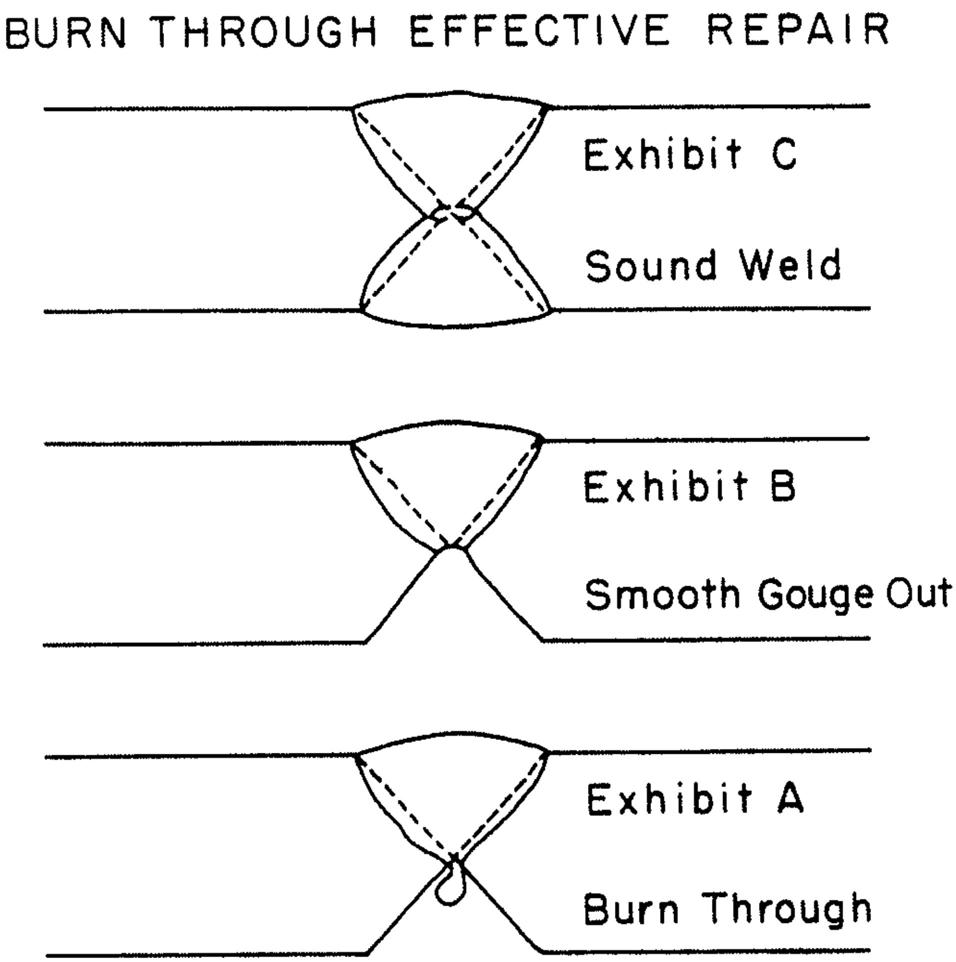
Figure 5.19 Examples of Backgouging

BURN THROUGH DEFECTIVE REPAIR



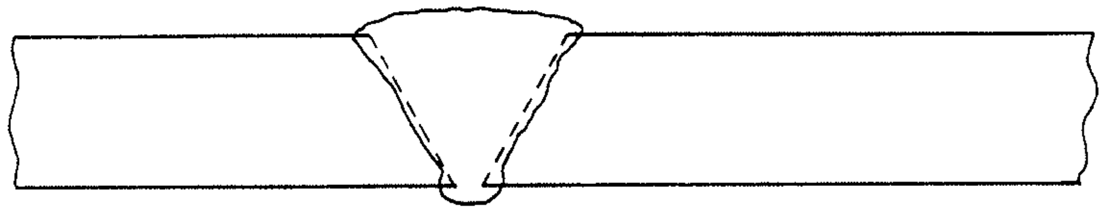
A defective repair may result if the backgouging operation is extended too far beyond the root of the weld and/or if care is not taken to open the gouge groove wide enough to insure penetration of the root pass reweld. Thus, the reweld may fill the groove with slag and bridge it so that the slag is left in the weld.

Figure 5.20 Defect Resulting From a Deep, Narrow Backgouge

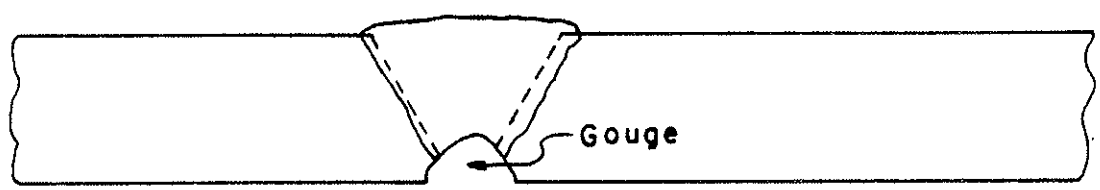


Repair is effective by gouging out and rewelding the first root pass or one side of the entire weld if necessary. Care must be taken to keep the gouge open and smooth so as to reduce the chance of a defective welding repair.

Figure 5.21 Effective Correction of a Burn Through



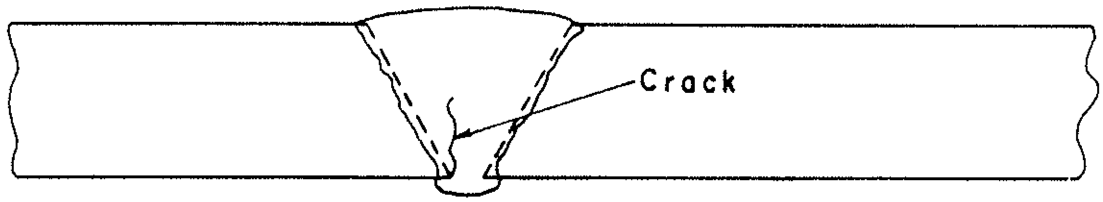
(a) Single-Vee Groove Weld



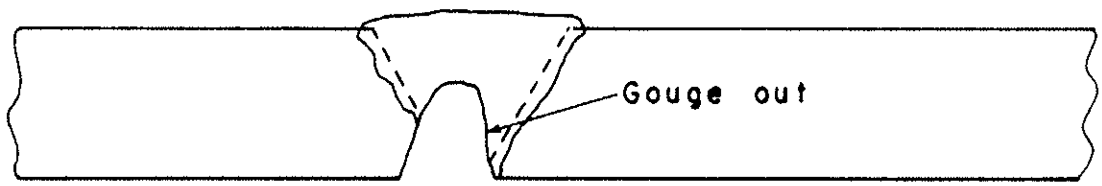
(b) Back gouge to sound weld metal

SINGLE-VEE GROOVE WELD

Figure 5.22 Proper Use of Backgouging--Single-Vee Groove Weld

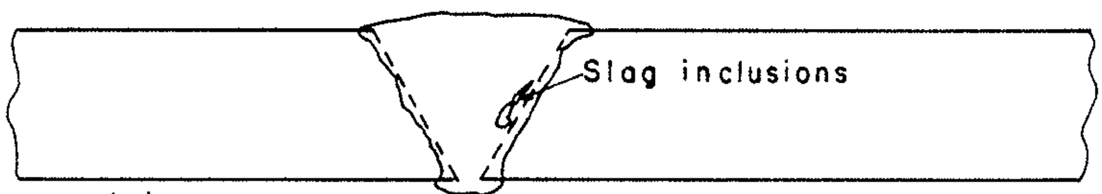


(a) Single-Vee Groove Weld

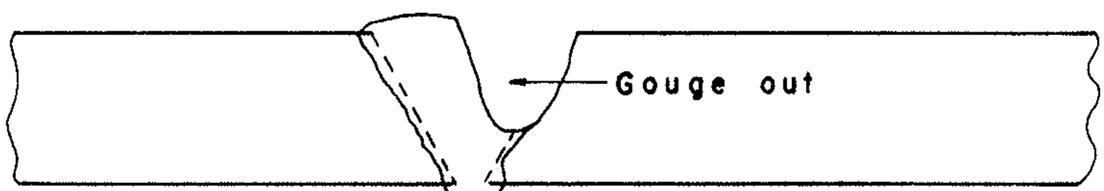


(b) Gouge out root crack and imperfect weld metal

BUTT WELD ROOT CRACK



(a)



(b) Gouge out excessive slag inclusions as shown in (a) and (b)

EXCESSIVE SLAG INCLUSIONS

Figure 5.23 Proper Use of Backgouging--
Root Crack and Slag Inclusions

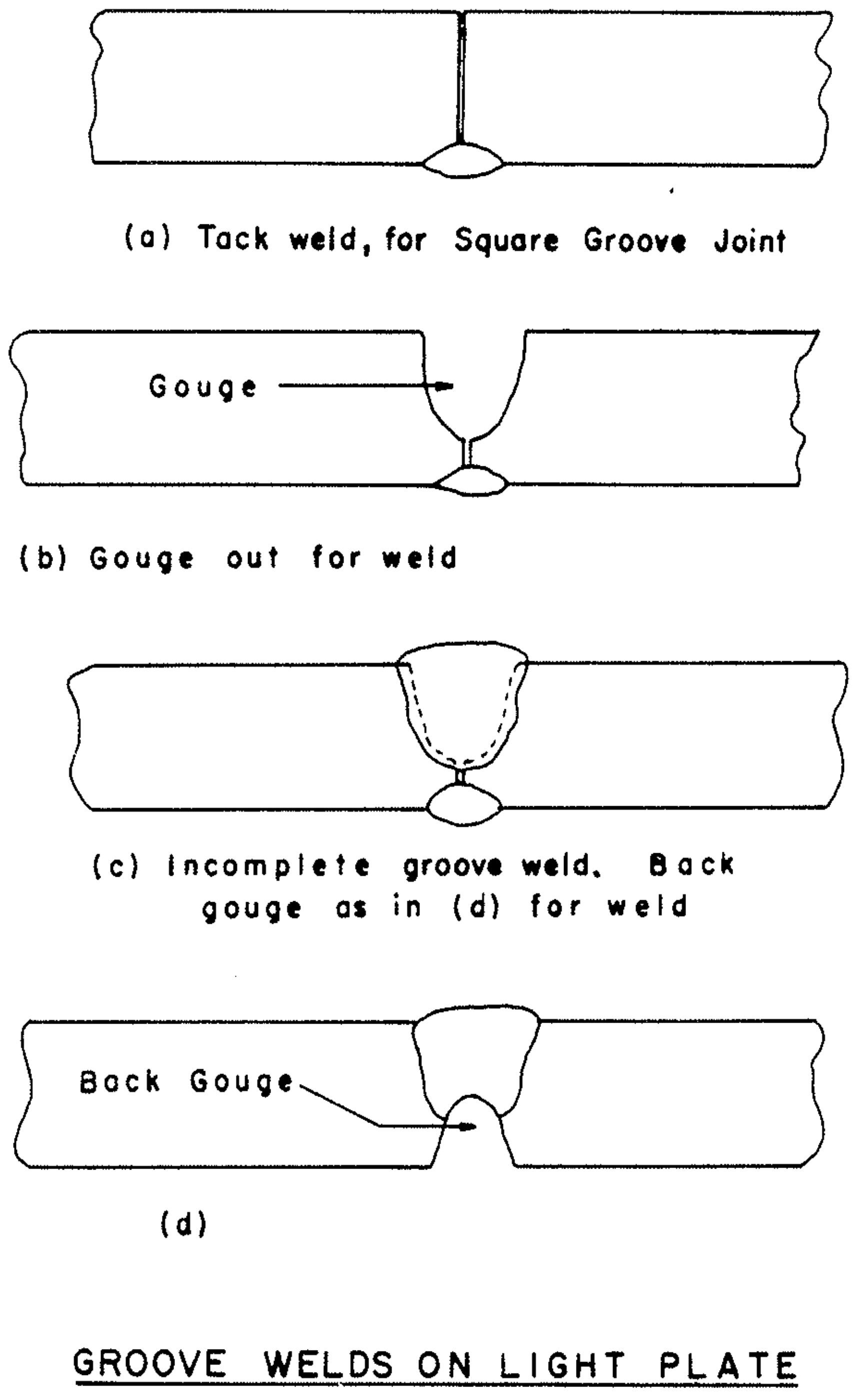


Figure 5.24 Proper Use of Backgouging--
Groove Welds on Light Plate

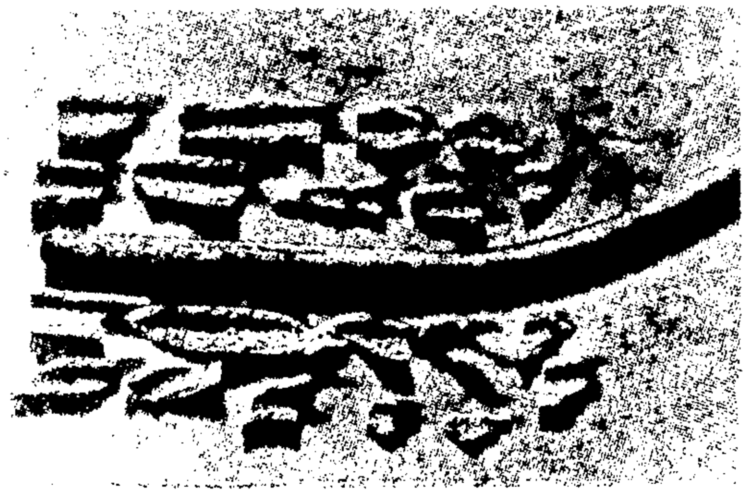
(7) Electrodes, Wires and Fluxes

It cannot be overemphasized that cleanliness, moisture control, and storage of welding materials to prevent moisture pickup is of the utmost importance.

Fabricators continue to experience the phenomenon of delayed cracking. This event is not confined to high-strength 100 ksi yield steels. It can and does occur in the lower yield steels, even in A36 steel. The important factors are the selection (strength) of electrodes and how the electrodes or consumables are stored or protected from moisture, rust, oils, and grease. Figures 5.25 and 5.26 show electrodes damaged by moisture. Figure 5.27 shows the defects that resulted from the use of these electrodes. The higher the strength of the weld deposit (and heat affected zone) the greater the inclination of the material to delayed cracking.

Delayed cracking of welds and the HAZ has extended into the parent metal and continued for three days or more. The fabricator's quality control personnel should be aware of this condition. The owner's quality assurance personnel should wait for 48 hours to 72 hours before performing ultrasonic tests on 100 ksi yield steel. Butt welds should be both radiographically and ultrasonically tested, with the radiographic testing performed before ultrasonic testing.

Radiographic testing prior to ultrasonic testing will minimize the time for quality control and quality assurance. Radiographic testing will locate severe defects (slag and porosity) less easily evaluated by ultrasonic testing. Ultrasonic testing is used for locating cracks, incomplete fusion and incomplete penetration, and tight defects (lack of volume).



E7018 electrodes showing rusty core wire. Rust contains 10-15% moisture. This moisture is chemically combined. It cannot be baked out at specified drying temperatures.

Figure 5.25 Electrode Damaged by Moisture

Figure 5.26 3/16" #7018 Electrode Showing Rust in Core Wire Seam and Around Core Wire

Figure 5.27 Fisheyes in Fractures of 3 Tensile Tests from a Procedure Test Joint Made With Moist Electrode

(8) Fillet Welds on Stiffeners

Fillet welds are hard, if not impossible, to inspect for subsurface defects such as can be seen in Figures 5.28 and 5.29. However, the fillet weld profile does tell one about the surface conditions. To inspect these critical welds in tension flanges, the welds should be ground as smooth as necessary to perform magnetic particle and dye penetrant inspection.

Longitudinal stiffeners that butt against or are fillet welded to a vertical stiffener on both sides so as to form a cruciform weld are another point of concern. These fillet welds may crack. However, this practice has become obsolete through changes in design that require the horizontal stiffeners to be cut back from the vertical stiffeners.

Figure 5.28 Fillet Weld Defect

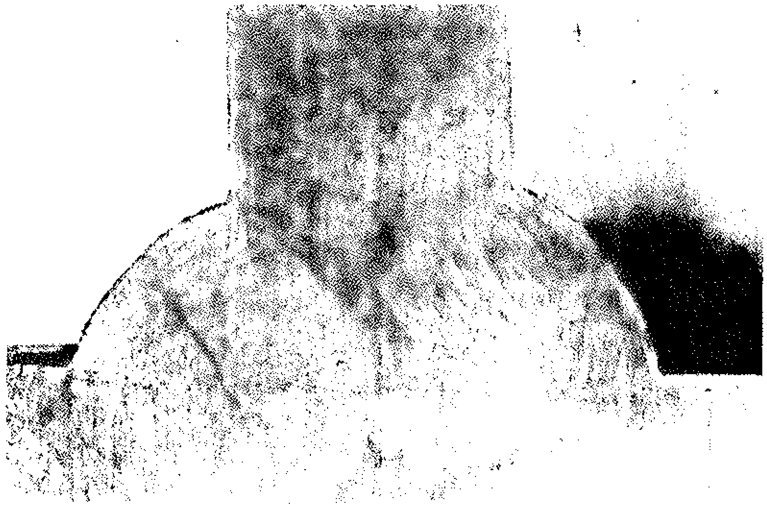


Figure 5.29 Fillet Weld Defect

(9) Arc Striking

Arc striking, or starting the arc, is also of utmost importance. Some welders strike the arc away from the weld area or groove and dragging the arc into the weld area to continue. The welding arc should never be started outside the weld area. This is especially important on low-alloy high strength steels. When the arc is struck outside the weld area and dragged into the weld groove, it leaves a trail of small arc pits which consist of minute deposits of extremely hard metal (quenched metal) embedded in the surface of the steel. These deposits are created when the arc strikes and breaks contact with the steel in a very short period of time without the benefit of proper flux or gas shield cover. These minute puddles of molten metal are quenched instantly by the mass of cold metal under them.

These particles of extremely hard material (up to 400 ksi yield strength) provide biaxial restraint at the interface between the particle and the base metal. This results in a sharp stress increase which may nucleate a crack at this point.

(10) Oxyacetylene Cutting

Cutting steel with an oxyacetylene torch is an art. Thousands of dollars are wasted in fabrication by careless cutting practices which require excessive grinding to repair the cut surfaces satisfactory.

Damage is also done to a structure or plate by slipshod cutting of re-entrant corners (radii), not following prescribed lines, improper beveling for weld joint, incorrect match of plates to be welded, irregular holes, and burning off items such as clips, dogs, and other attachments.

The cutting equipment for fabrication can be very elaborate. Equipment ranges from computerized automatic cutting equipment to automatic, portable cutting machines, and manual. Personnel who are to perform the burning or cutting should be qualified and tested by a qualification test designed to demonstrate his ability for flame cutting and operation of the equipment.

Figures 5.30 through 5.32 illustrate the cutting operation.

(11) Cutting-Flame Adjustments

To enable one to recognize and be familiar with the types of cutting flames, several different flame adjustments are shown in Figure 5.30. The type of flame adjustment for cutting steel is the neutral preheat heating flame as shown in photos 4 and 5.

1. Acetylene burning in air.

2. Strongly carburizing preheating flame without the cutting oxygen flow. The flame as shown in pictures No. 2 and No. 3 will cause the surface of the cut to be melted over. (Not satisfactory.)

3. Strongly carburizing preheating flame with cutting oxygen flow.

4. Good neutral preheat flame without the cutting oxygen flow. The neutral flame adjustment as shown in pictures No. 4 and No. 5 will give satisfactory flame cutting results.

5. Good neutral preheat flame with cutting oxygen flow.

6. Oxidizing preheating flame without the cutting oxygen flow. This type of flame, as shown in pictures No. 6 and No. 7 will cause the edge of the cut to be melted over and irregular. This is not a satisfactory cutting flame.

7. Oxidizing preheating flame with cutting oxygen flow. This type of flame will not give a satisfactory cut.

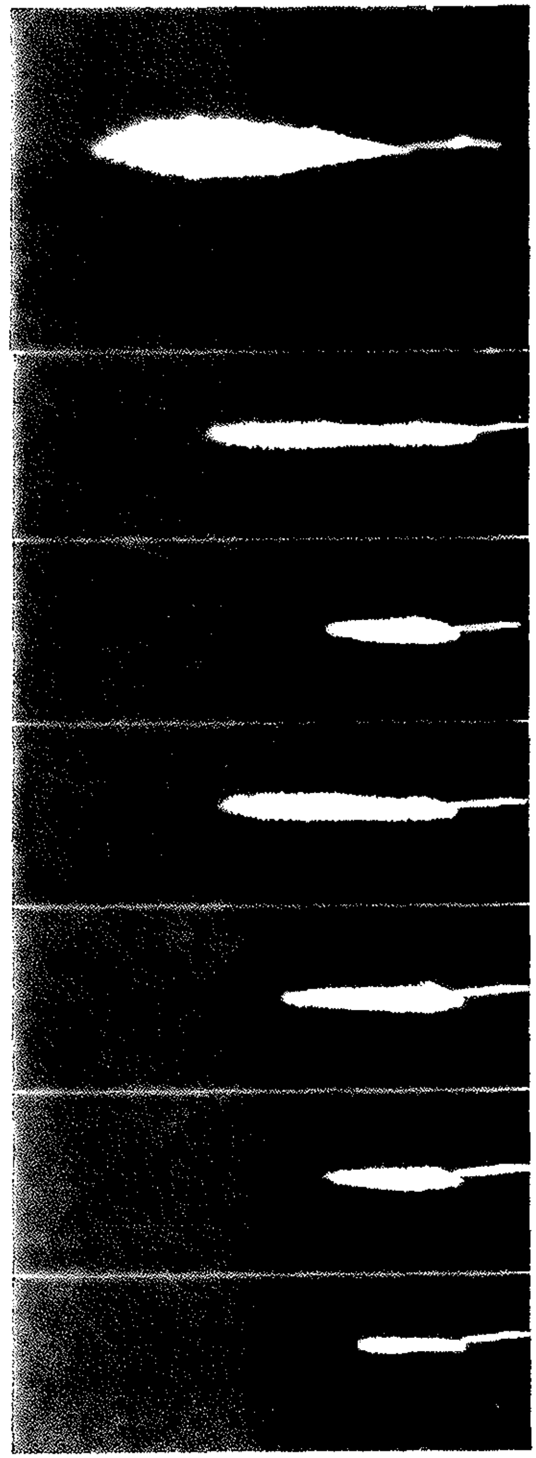


Figure 5.30 Oxyacetylene Gas Torch

(12) Oxyacetylene Automatic Cutting

The principles of automatic cutting are essentially the same as those involved in manual cutting.

Automatic cutting will be superior to manual cutting through producing greater accuracy, better quality and a finer degree of edge smoothness of the cut surface.

The strip cutting of structural flange plates should be done by automatic cutting when they are made from plate material.

Figure 5.31 illustrates the following:

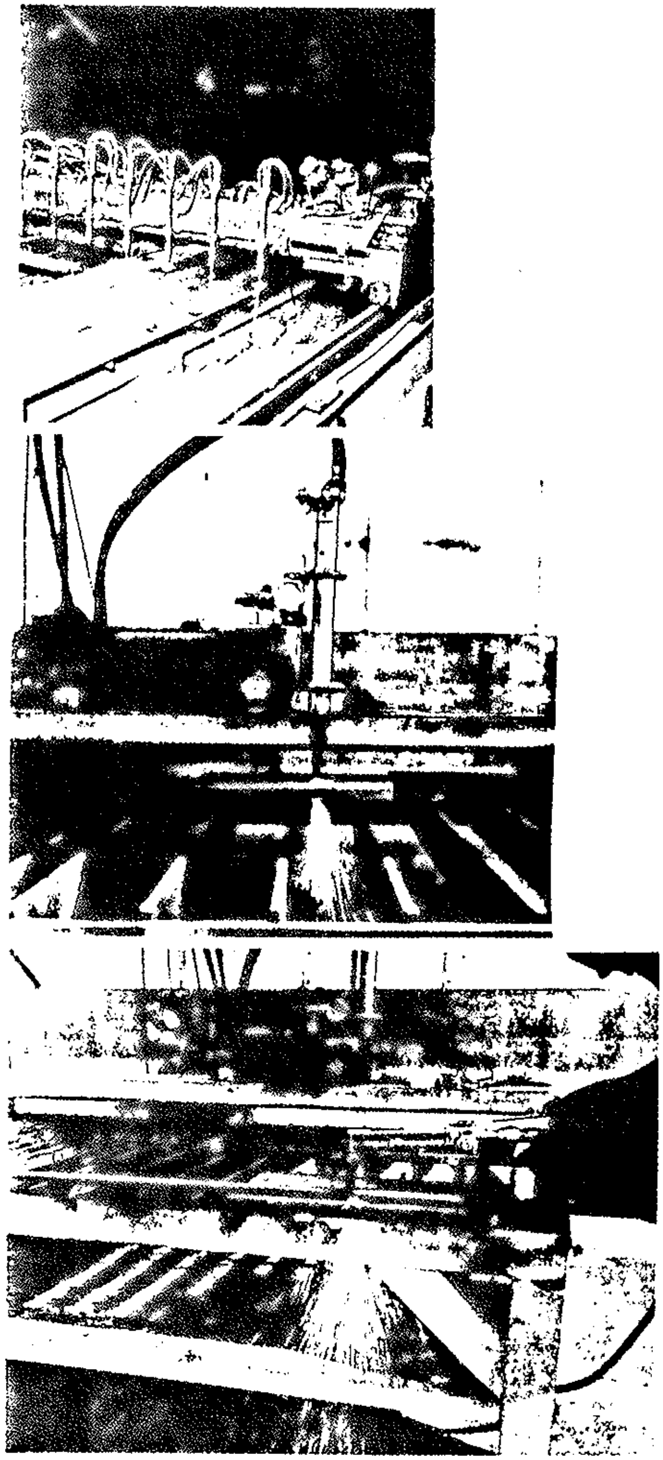
Photo 1. A typical cutting operation. Three flange plates are being cut from plate stock simultaneously.

Photo 2. A portable cutting machine. This machine can be used anywhere in the fabricating shop. Machines such as these do very accurate cutting and can be adapted to a wide variety of work, such as straight cuts, sweep cuts, circle cuts, square cuts, and bevel cuts.

Photo 3. Manual cutting a straight line by the use of a straight edge.

5.6.2 Welding Defects and Techniques

Figures 5.33 through 5.62 illustrate and describe some of the kinds of defects that must be anticipated and corrected by shop quality control operations. The Figures also show some good weld techniques and practices that must be pursued as an objective by quality control operations.

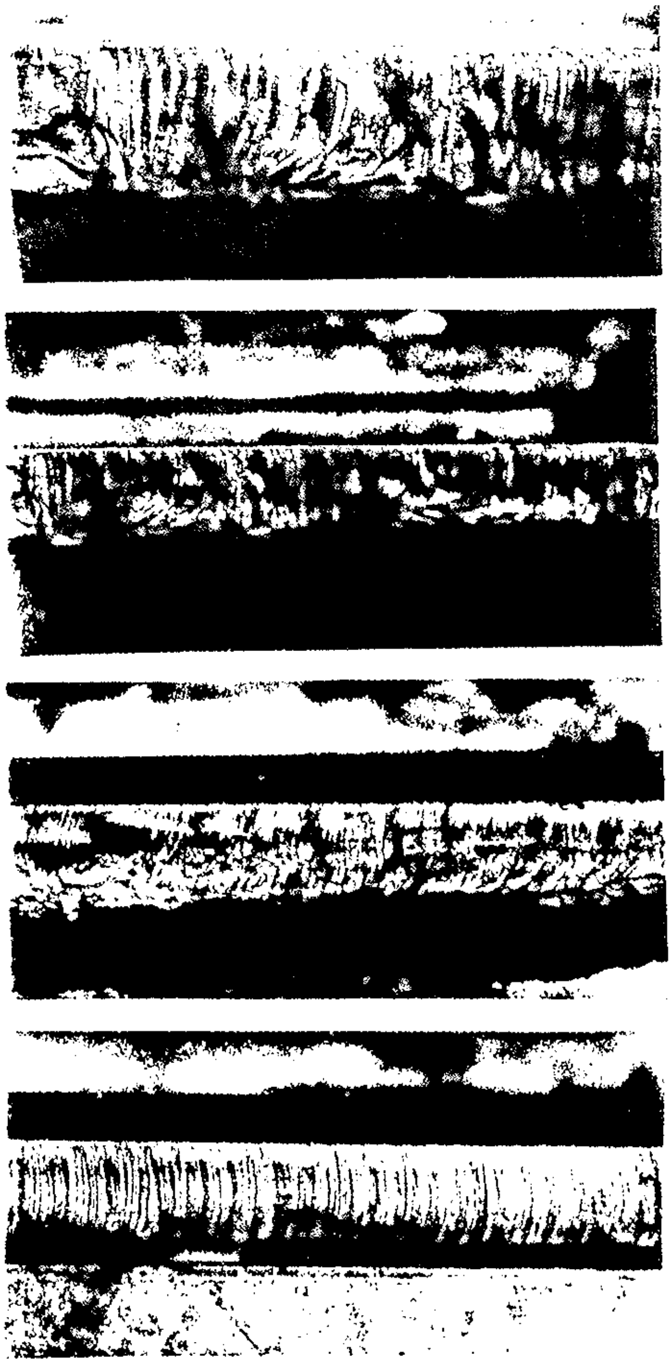


A. Automatic Multi-Tip Cutting Machine

B. Single-Tip Portable Cutting Machine

C. Manual

Figure 5.31 Oxyacetylene Cutting Equipment



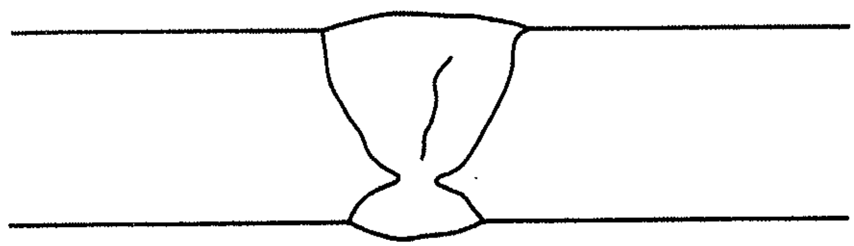
Unsatisfactory Cutting

Unsatisfactory Cutting

Unsatisfactory Cutting

Satisfactory Cutting

Figure 5.32 Oxyacetylene Cuts



CRACKING (LONGITUDINAL)

This type of defect is the most serious that can occur in a weld. It is not permitted in any degree. It is associated with welding at excessively high amperages, welding heavy material with insufficient preheat, and/or welding on restricted joints. Such restrictions occur on beam joints under the following conditions:

(1) Webs bolted or added together prior to welding of flanges.

(2) Web ends bearing against each other caused by thermal contraction of a partly completed flange weld.

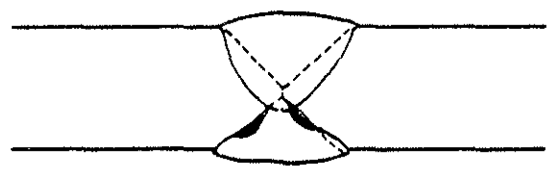
(3) Beam or flange under stress at time of welding.

(4) Failure to preheat and expand one completed flange weld on a joint in a beam while welding the other flange in the same joint.

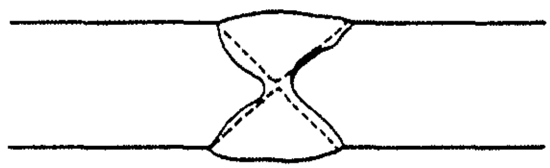
(5) Restricting the motion of the beam by any means during the welding.

Rectification may involve changing welding procedures and fabrication or erection methods.

Figure 5.33 Longitudinal Cracking

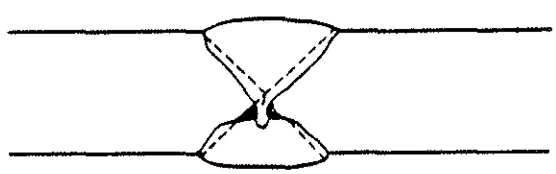


Lack of Fusion (Double)



Lack of Fusion (Single)

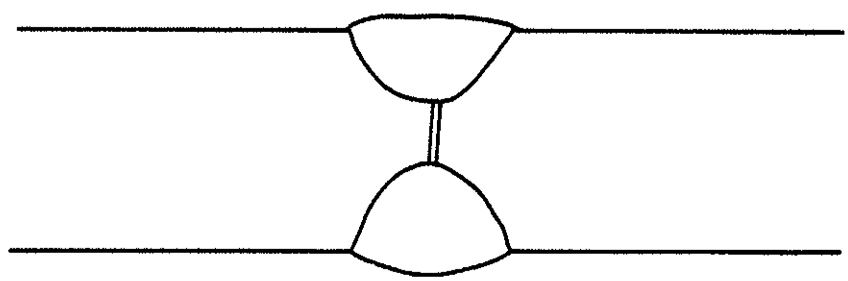
Lack of fusion is generally a product of poor welding techniques rather than faulty welding materials. It can be caused by welding at low amperage on metal which is too cold, by welding on dirty or scaly material, by welding with improper electrode size, by welding with improper electrode manipulation, and/or by welding at too great a speed. Lack of fusion also can be associated with improper joint preparation.



Burn Through Incomplete Fusion

This sort of defect is produced when the first root pass of a double "vee" butt weld is not gouged out. Burn through and slag have been trapped near the root of the weld under the backside passes.

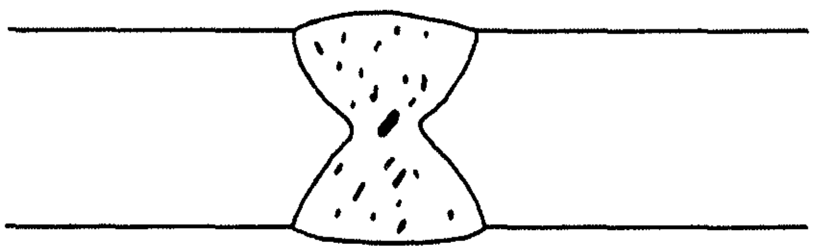
Figure 5.34 Lack of Fusion



Incomplete Penetration

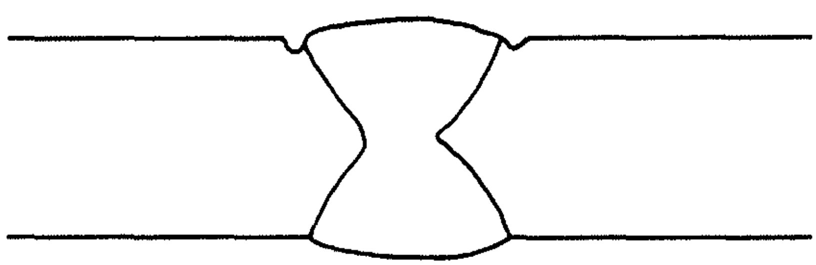
This type of defect is not permitted in any degree. Its occurrence is often associated with square butt joints and with "vee" joints welded from one side only. It is also associated with joints that have too wide a root face, too close a fitup, or too small a "vee" angle. It may occur on a joint that has not had the first root pass back gouged or scarfed out from the opposite side. It can also be caused by laying the root passes with too large an electrode, too low an amperage, and/or too fast a welding speed.

Figure 5.35 Incomplete Penetration



Porosity

Porosity is tolerated if the amount does not exceed that specified. However, it is symptomatic of poor welding technique and/or "sloppy" welding management--conditions which should be corrected. This type of defect can be caused by moisture or unstable oxides present on the joint or in the fluxes prior to welding, or it may be caused by holding a long arc and welding with a cold puddle, or welding at excessive speed. Better joint preparation and a little preheat will generally correct the first condition, better flux or rod storage and drying will generally correct the second condition, and better welders the third condition.



Undercut

Undercut should be rejected from visual inspection of the surface of the weld rather than by radiographic inspection, although it should be rejected in any case.

Figure 5.36 Porosity and Undercutting

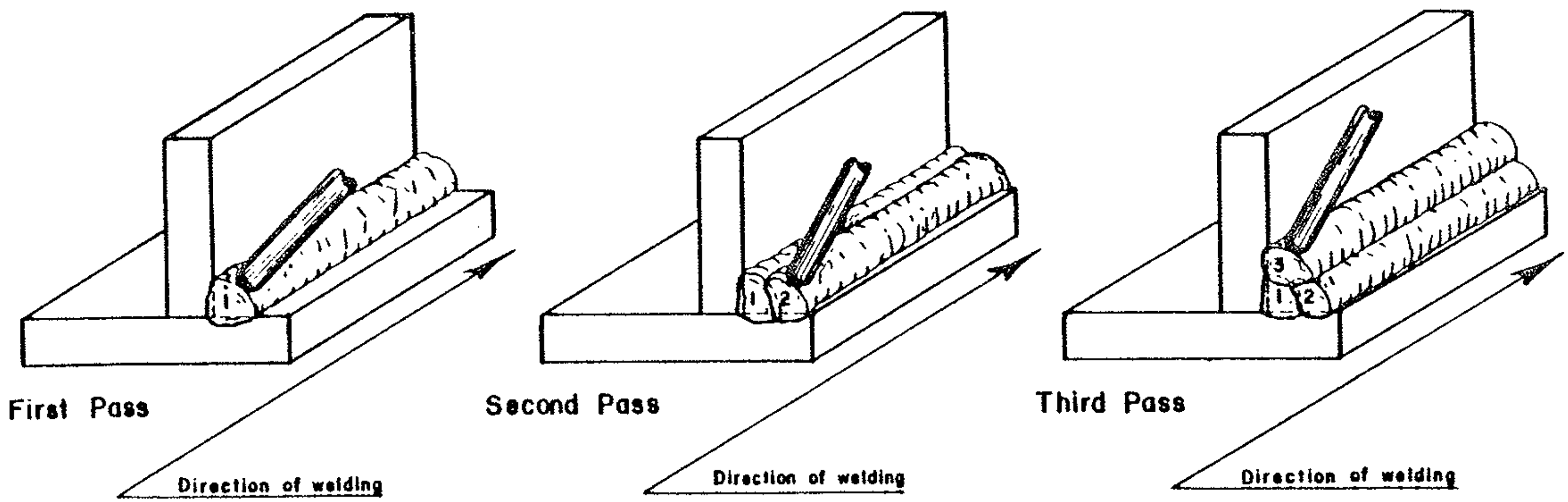
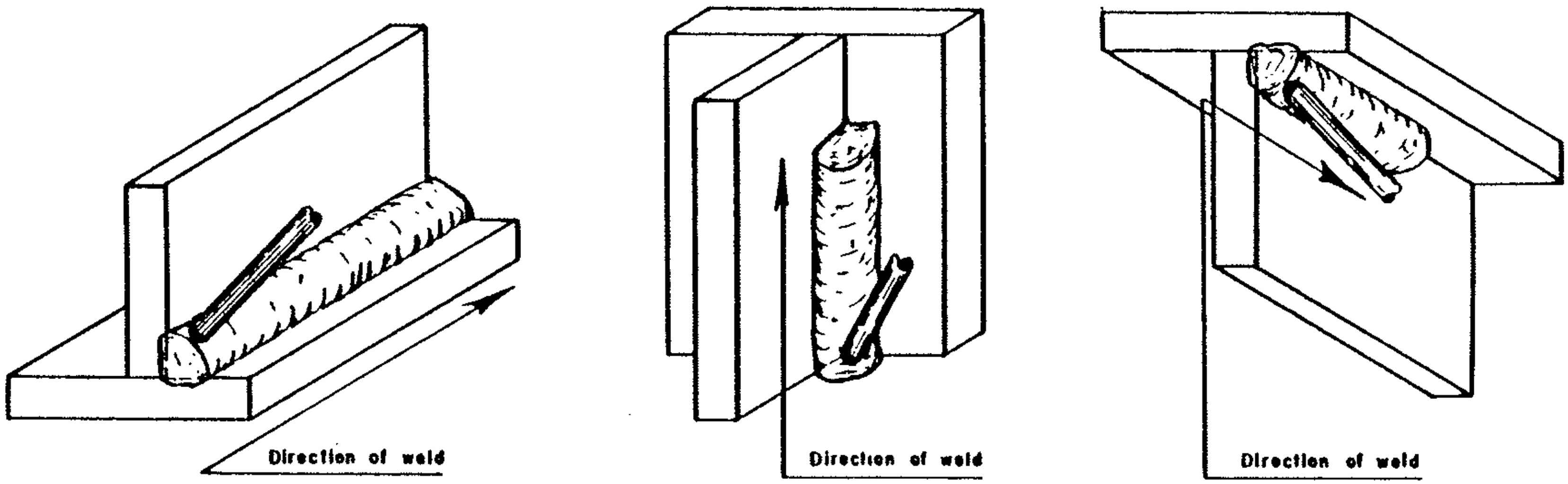
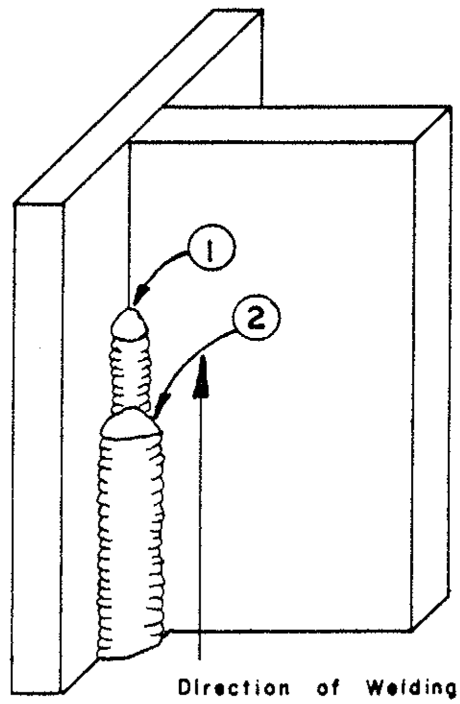


Figure 5.37 Three Pass Horizontal Fillet Weld



HORIZONTAL 5/16" FILLET WELD   VERTICAL 5/16" FILLET WELD   OVERHEAD 5/16" FILLET WELD

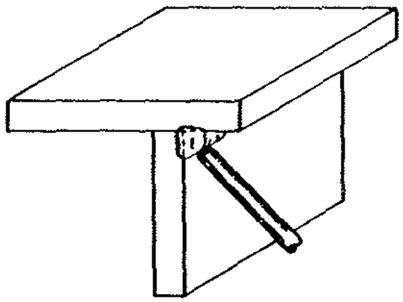
Figure 5.38 Single Pass 5/16" Fillet Weld



3/16" First Fillet Pass
3/8 Finished Fillet Weld

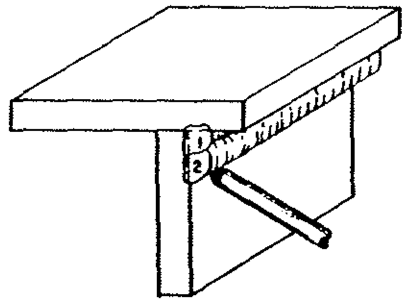
All Vertical Welding Shall Start At The Bottom And Progress In An Upward Direction

Figure 5.39 Two Pass 3/8" Vertical Fillet Weld



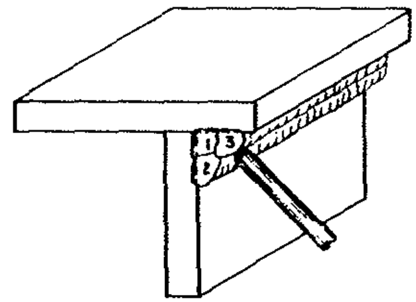
First Pass

Start the first fillet pass at the left corner.



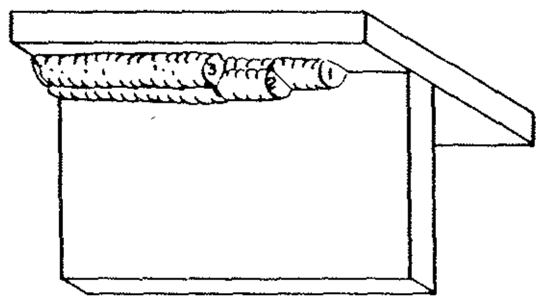
Second Pass

Weld second fillet puss in same direction with arc centered on the lower edge of the first bead.



Third Pass

Weld third fillet pass in the same direction with arc centered on the upper edge of the second bead.



Complete sequence of weld passes with 3/8" finished fillet weld.

Figure 5.40 Three Pass 3/8" Overhead Fillet Weld

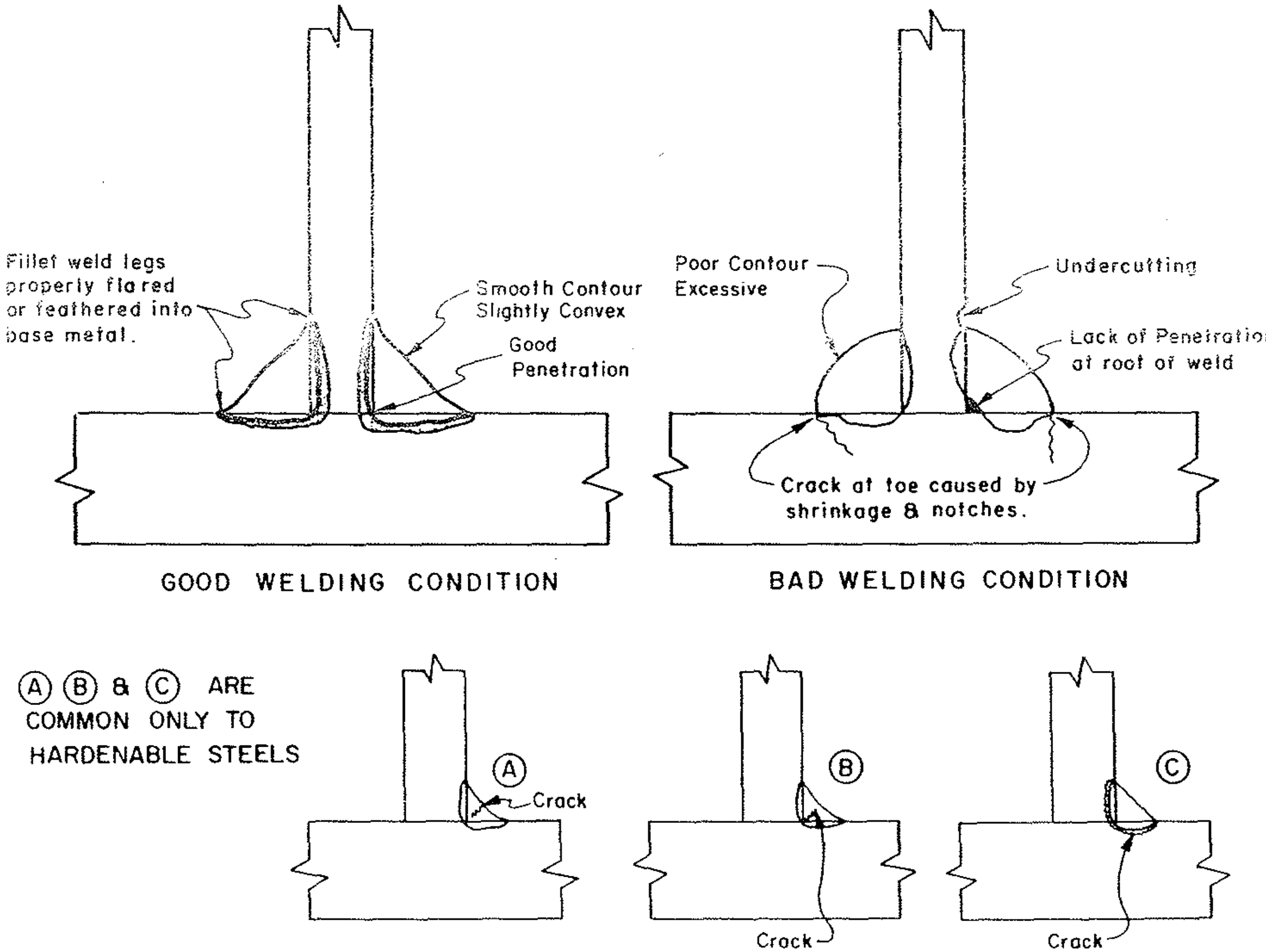
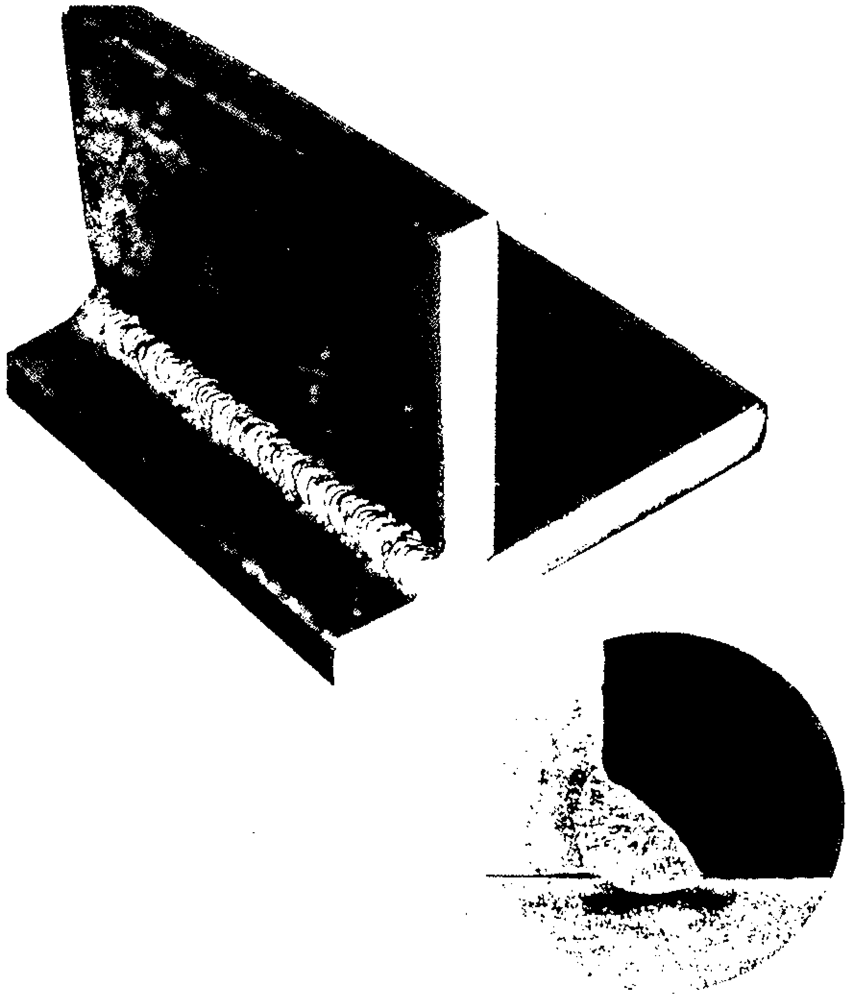


Figure 5.41 Fillet Welds

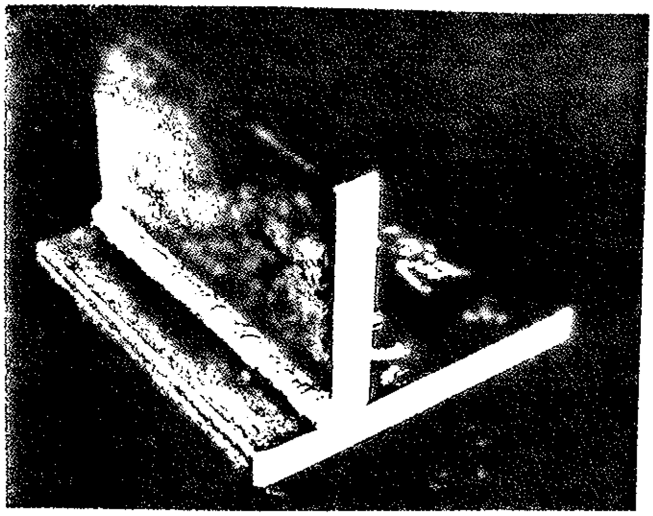


(a) Shows relatively smooth weld with adequate bead size and a slightly convex surface. No undercut, overlap, or unequal leg size is visible.

(b) Etched section shows adequate penetration and a good bead contour with sound weld metal.

(c) Break would show the penetration along the edges of the plate is complete.

Figure 5.42 Single Pass Horizontal Fillet Weld



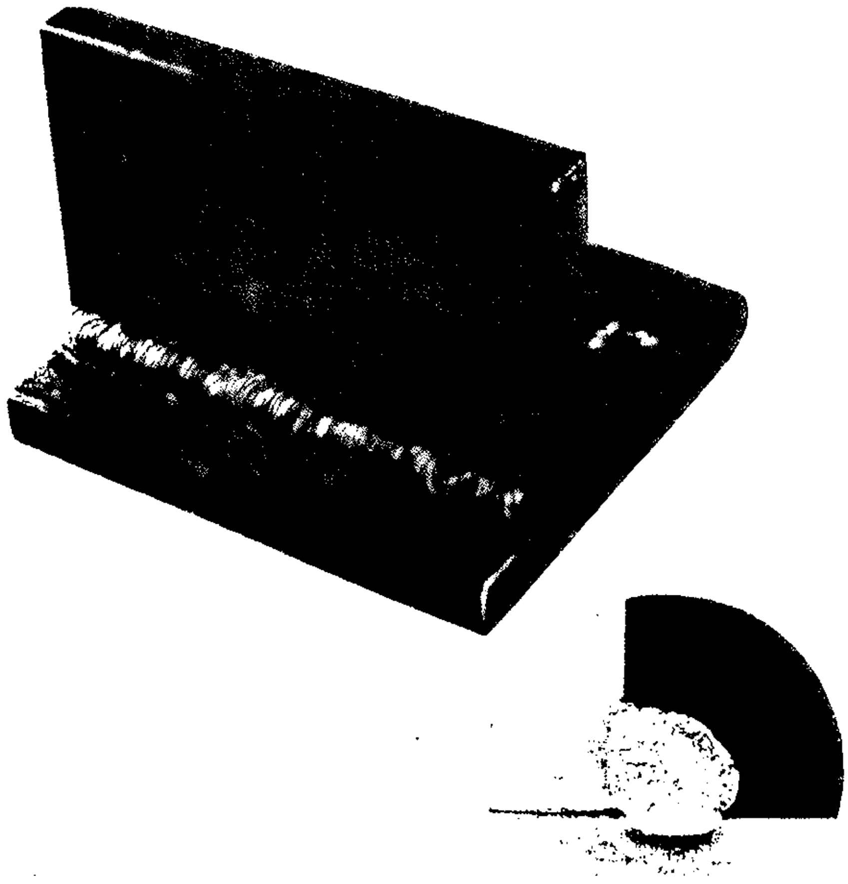
Defect: Lack of fusion
Quality: Not acceptable
Appearance:

(a) The fillet weld bead is not fused properly to the plates. This can be seen from the surface of the bottom plate. The weid bead surface should merge smoothly into the plate surface with no signs of undercutting or overlapping.

(b) Etched section shows a lack of fusion at the weld root and at the toe of the fillet on the bottom plate.

(c) Fillet weld test specimen when broken open would show the small amount of weld metal that was fused to the base plate.

Figure 5.43 Single Pass Fillet Weld



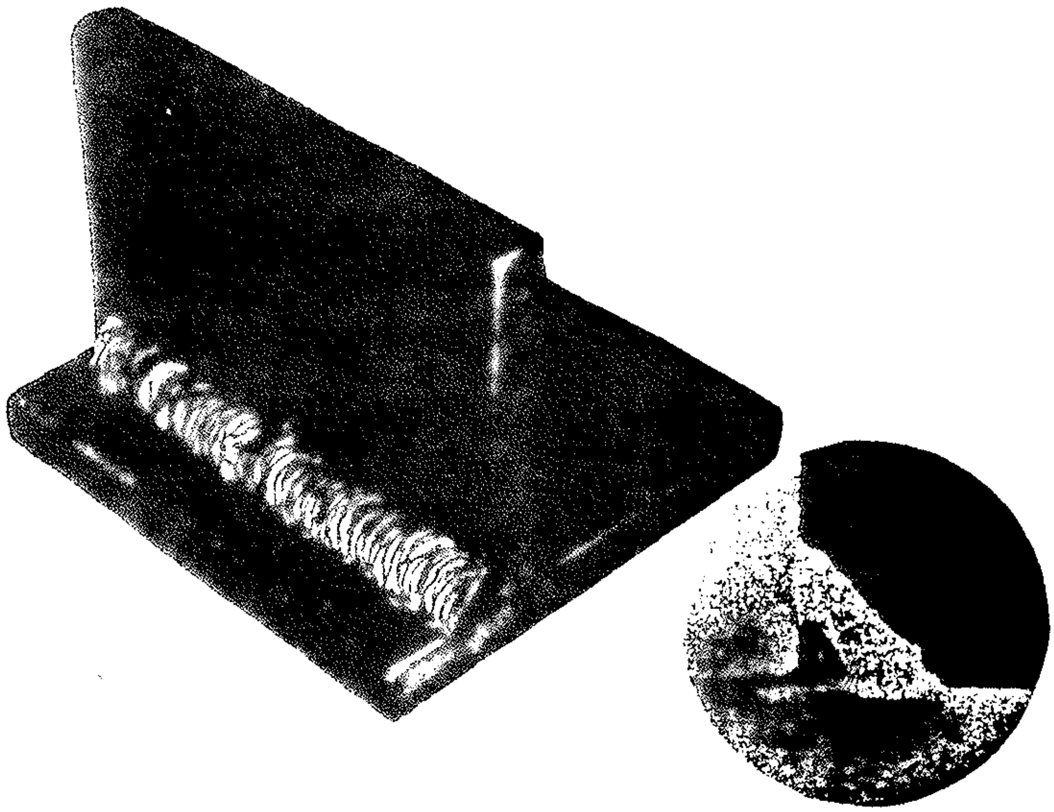
Defect: Excessive convexity
Quality: Not acceptable
Appearance: Excessive weld metal reinforcement: this can be seen in the etched section.

This defective weld can be caused by improper welding technique and/or insufficient welding current.

Poor surface contour welds with this type of defect will not be acceptable. This defect is associated with lack of fusion and roll over. This can be seen in the etched section.

Excessive convexity tends to produce harmful notch effects. For multi-pass fillet welds, lack of fusion and/or slag inclusions will occur.

Figure 5.44 Single Pass Fillet Weld



Defect:	Incomplete penetration, oversize single pass fillet weld made by bridging
Quality:	Not acceptable
Appearance:	

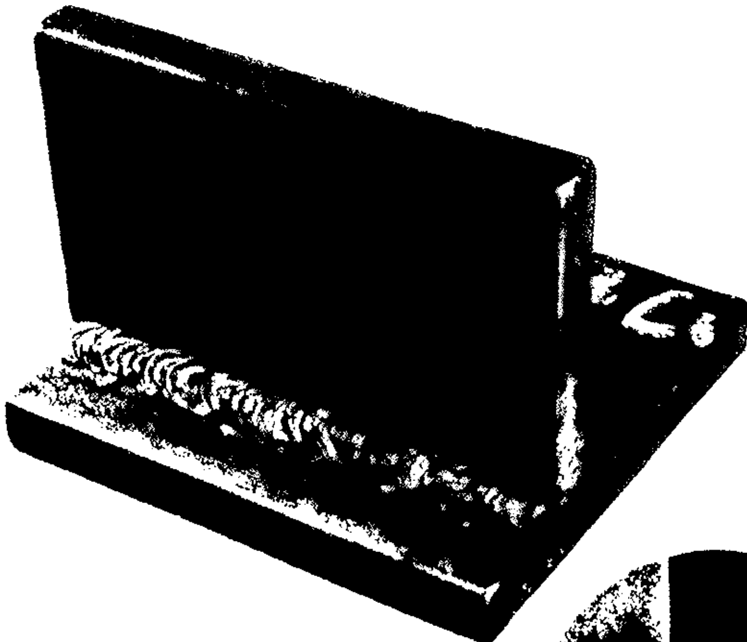
The fillet weld surface is fairly uniform without undercutting or roll over. However, when a weld of this size is made in a single pass it is very likely that bridging has taken place. This can be seen in the etched section.

Incomplete penetration. This type of defect is caused by making too large a weld in one pass and/or the use of too large an electrode, insufficient welding current, or a high rate of travel in welding.

The weld is too large. Made in one pass. On test plates, a break test will confirm this incomplete penetration.

This oversize single pass manual weld is associated with making too large a fillet weld in a single pass, which causes lack of penetration or bridging.

Figure 5.45 Single Pass Horizontal Fillet Weld



Defect: Undercut
Quality: Not acceptable
Appearance:

Note furrow in the surface of parent metal along the toe of weld. A rough weld bead is often related to this defect.

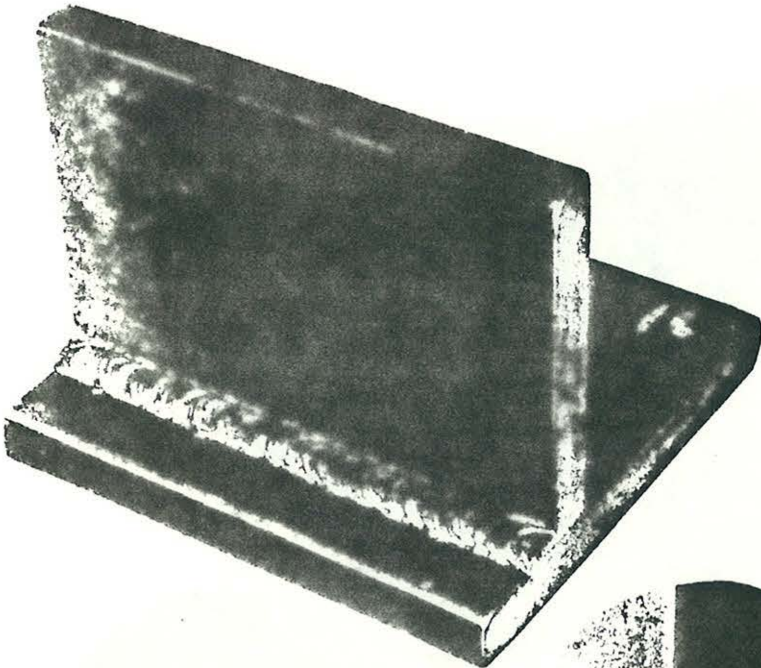
(1) Improper electrode angle and/or manipulation.

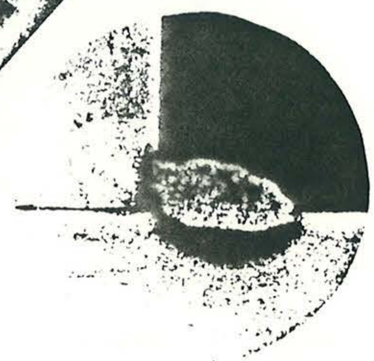
(2) Improper welding amperage and/or electrodes.

Welds with this type of surface defect should be rejected.

Undercutting is a common but serious surface defect in welding. It reduces the strength of the welded piece by reducing its cross-section and by creating a stress-raising notch at the margin of the weld. It is very easy to identify and avoid.

Figure 5.46 Single Pass Horizontal Fillet Weld





Defect: Insufficient leg
Quality: Not acceptable
Appearance:

(a) Most of the weld deposited bead is on the bottom plate, with a very small part of the bead welded to the web, so that the difference between the legs of the weld exceeds 1/8".

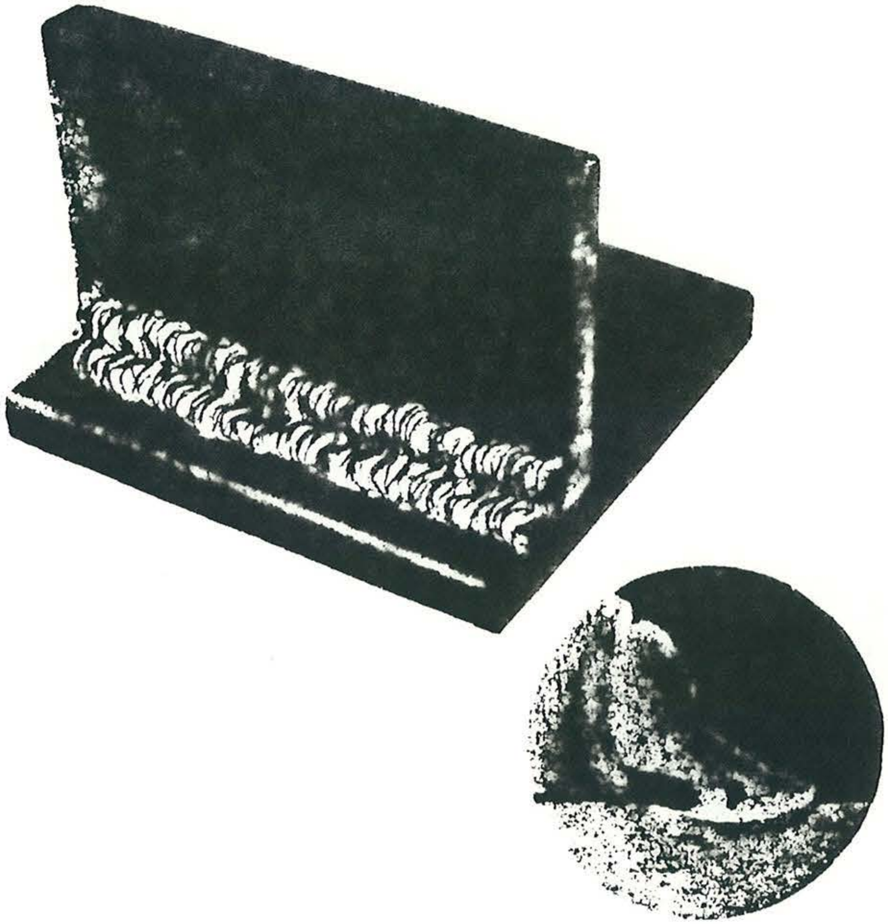
(b) Etched section shows uneven weld deposit, uneven legs, and insufficient throat for the required strength of the fillet.

(c) Fillet weld break test would show that only a small amount of weld metal was deposited on the vertical plate.

This type of defect is caused by the use of poor technique in manipulating and aiming the electrode.

This defect is associated with a lack of welding knowledge and experience.

Figure 5.47 Single Pass Fillet Weld



Defect: Lack of fusion, slag inclusions, defective profile, and excess concavity.

Quality: Not acceptable

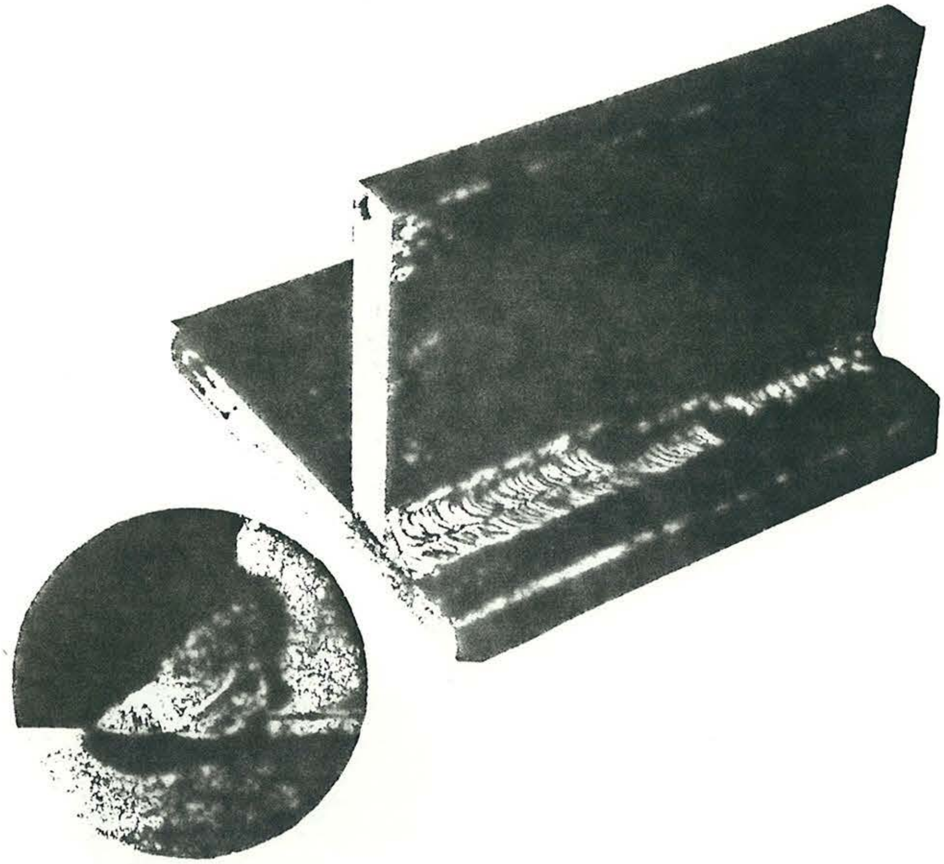
Appearance:

Weld profile defective and irregular between the second and third weld passes: this also is an indication of slag inclusions and lack of penetration.

Poor surface contour and irregular.

The fillet weld, as seen here, should prove to the inspector that the ability of the welder is not satisfactory.

Figure 5.48 Triple Pass 3/8" Horizontal Fillet Weld



Defect: No defects
Quality: Acceptable
Appearance:

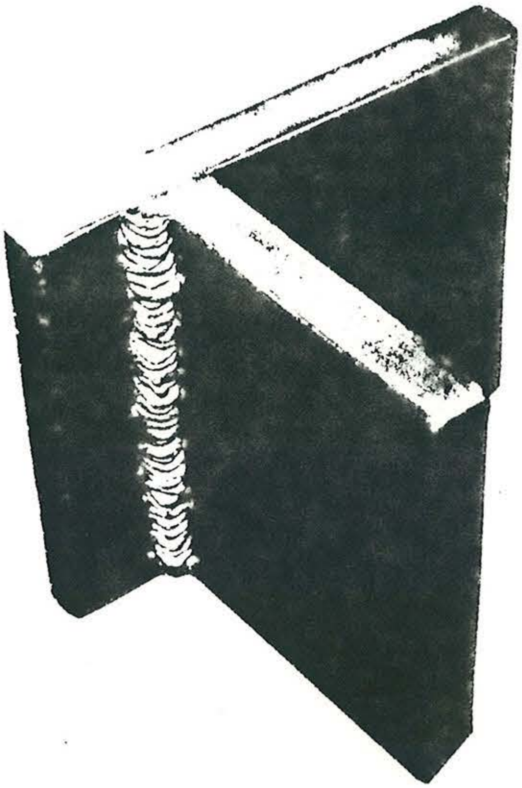
The fillet weld surface is uniform with all three weld passes blending into each other, with no undercutting, overlapping or excessive weld metal contour.

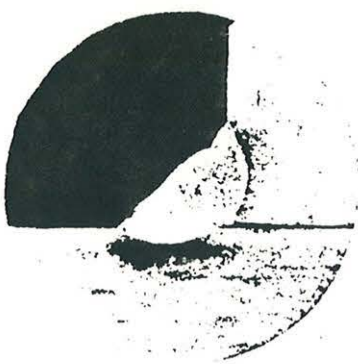
This acceptable weld was made by a welder using proper welding techniques and electrical conditions (voltage, amperage, polarity).

This three pass horizontal fillet weld indicates to the inspector that the procedure and welding techniques used by the welder are satisfactory.

Carefully inspect the root pass for defects. It is most important that the root of a multiple pass weld be made properly to insure complete fusion and a crack-free weld nugget.

Figure 5.49 Triple Pass 3/8" Horizontal Fillet Weld





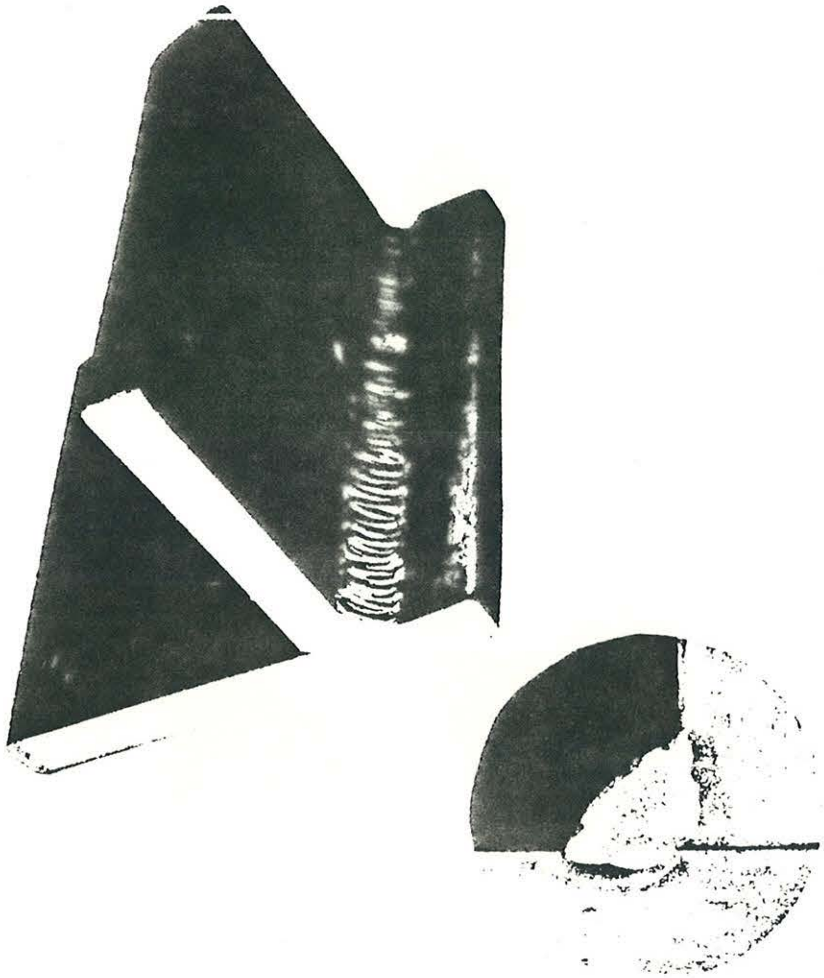
Defect: No visible defects
Quality: Acceptable
Appearance: Good, with acceptable, uniform surface contour.

Made with proper welding techniques and electrical conditions.

See that the fillet weld is uniform, without excessive weld metal contour, undercutting or overlapping, and has no slag or gas inclusions.

This weld indicates to the inspector that the procedure and welding techniques used by the welder are satisfactory.

Figure 5.50 Single Pass Vertical Fillet Weld



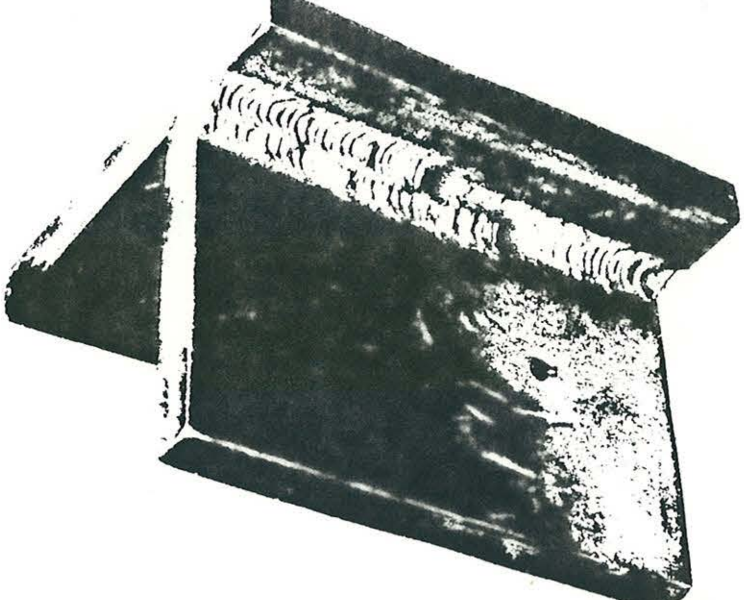
Defect: No visible defects
Quality: Acceptable
Appearance: Good appearing, uniform surface contour.

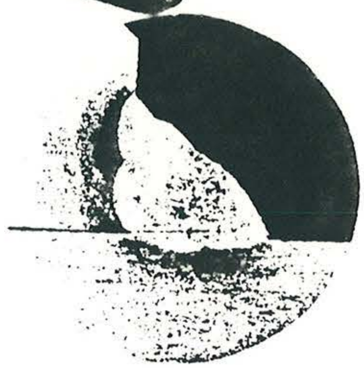
Made with proper welding techniques and electrical conditions.

See that the fillet weld has good surface appearance, is uniform, and has no excessive weld metal contour, undercutting, overlapping, or slag or gas inclusions.

This weld indicates to the inspector that the procedures and welding techniques used by the welder are satisfactory.

Figure 5.51 Double Pass 3/8" Vertical Fillet Weld





Defect: No visible defects
Quality: Acceptable
Appearance:

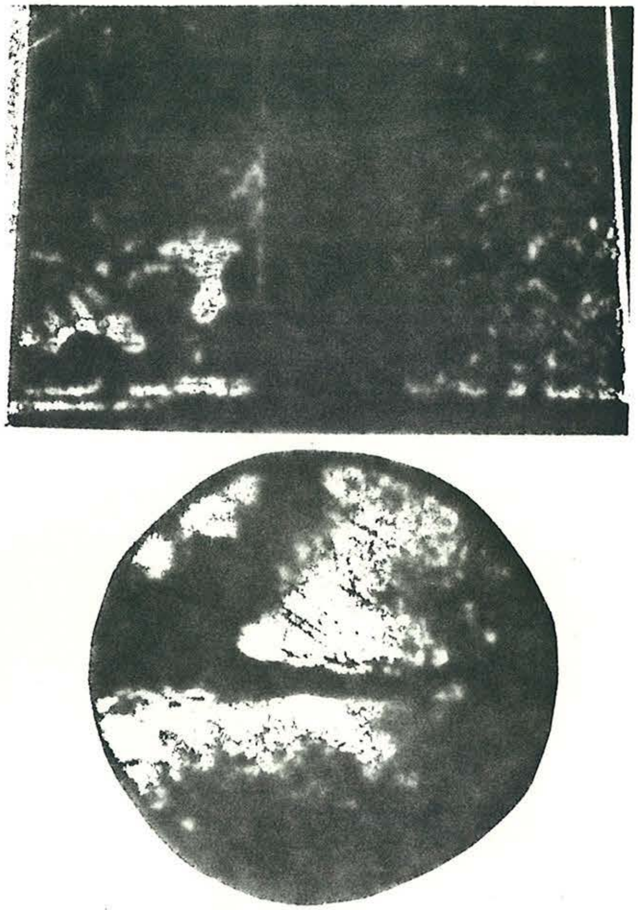
Good appearing, uniform surface contour with all three weld passes blending into each other, with no undercutting, overlapping or excessive weld metal.

Made with proper welding techniques and electrical conditions.

See that the fillet weld has good surface appearance, is uniform, and has no excessive weld metal, undercutting, overlapping, slag or gas inclusions.

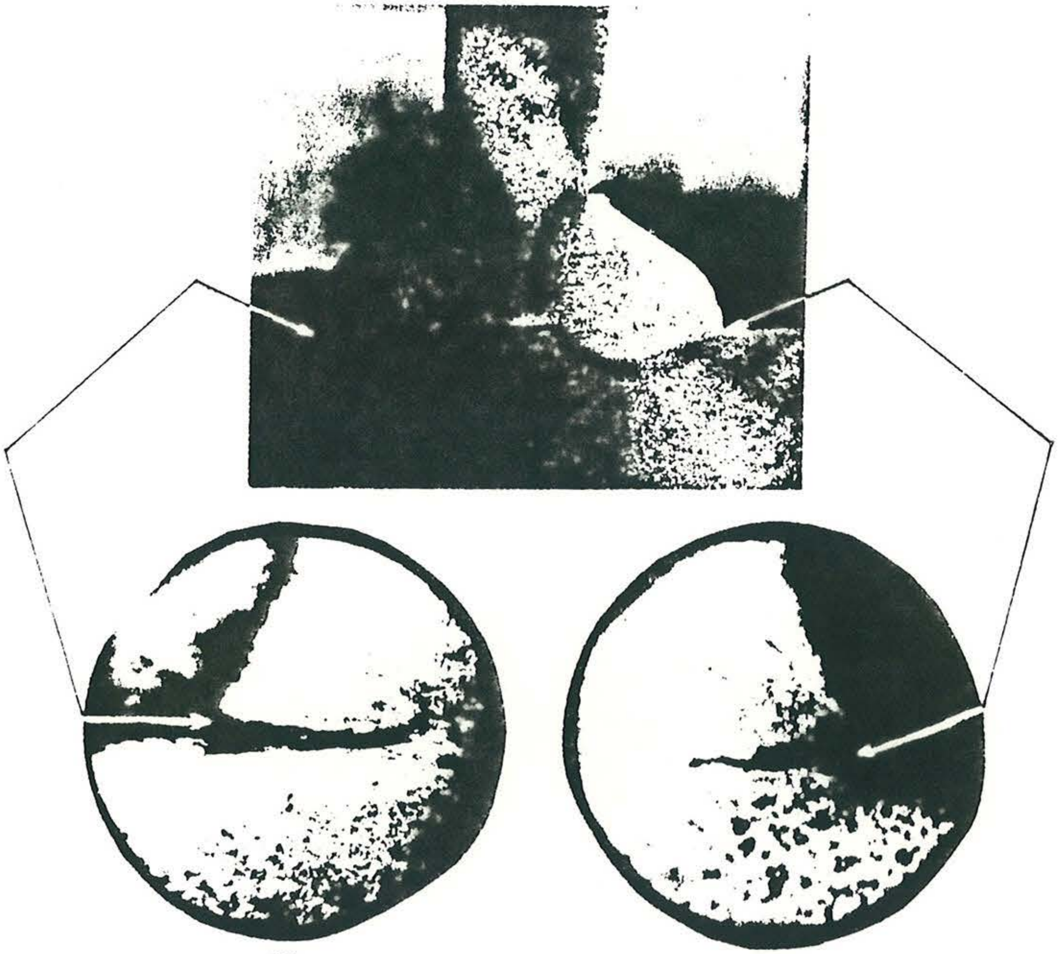
This weld indicates to the inspector that good procedures and welding techniques were used by the welder.

Figure 5.52 Triple Pass 3/8" Overhead Fillet Weld



These pictures show mill scale and the consequences of placing a weld over the heavier type shown at the top. The lower picture shows that the oxide has not been washed out by the flux, and consequently it has prevented the weld metal from wetting the plate beneath. The sharp reentrant created is a very likely place for longitudinal cracking to start. This type of overlap defect is often invisible to the eye.

Figure 5.53 Weld Over Mill Scale



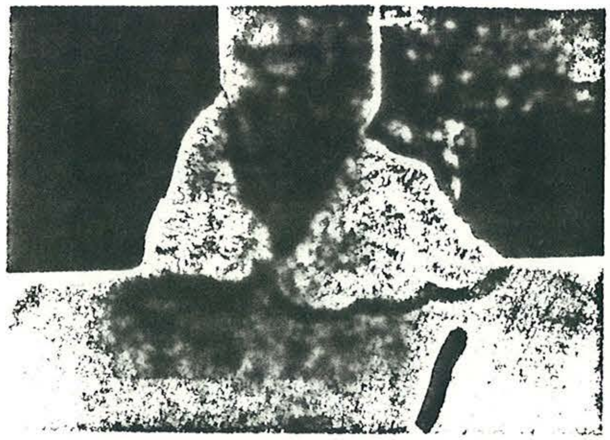
(1) (2)

This section is taken from a defective submerged arc fillet weld (excessive convexity), placed on a rolled surface with heavy mill scale.

Heavy mill scale is detrimental to submerged arc welding for long continuous fillet welds. The heavy mill scale is not only detrimental to weld metal, but obstructs the wetting or feathering at the toe of the fillet weld to the base metal causing a lack of fusion as can be seen. This defect can also be associated with manual welding using electrodes of iron powder or similar types.

Heavy mill scale should be removed before welding.

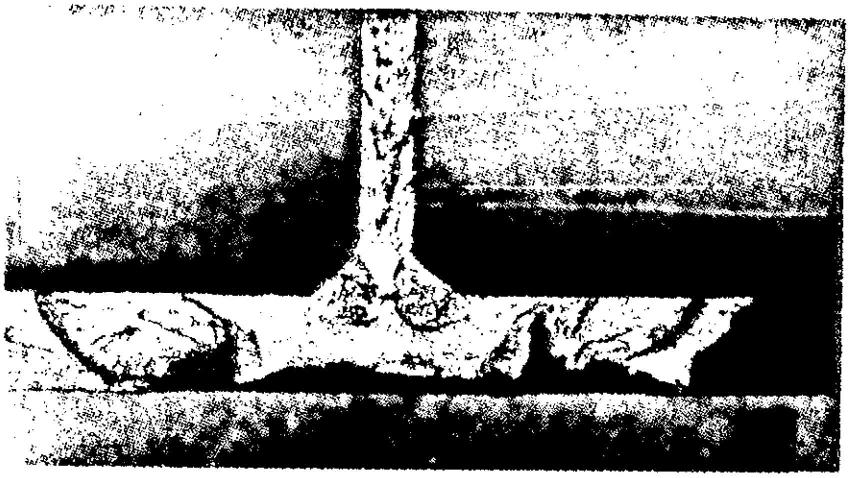
Figure 5.54 Effects of Mill Scale on Fillet Welds



A cross-section through a partially gouged multipass web to flange fillet weld on A514 steel showing lamellar tearing along the fusion line resulting from inadequate preheat, high electrode moisture and the use of overmatched electrodes.

A second cross-section through the weld shown above showing lamellar tearing induced in the flange at the toe of the fillet weld by the high residual stresses developed by the weld at that point.

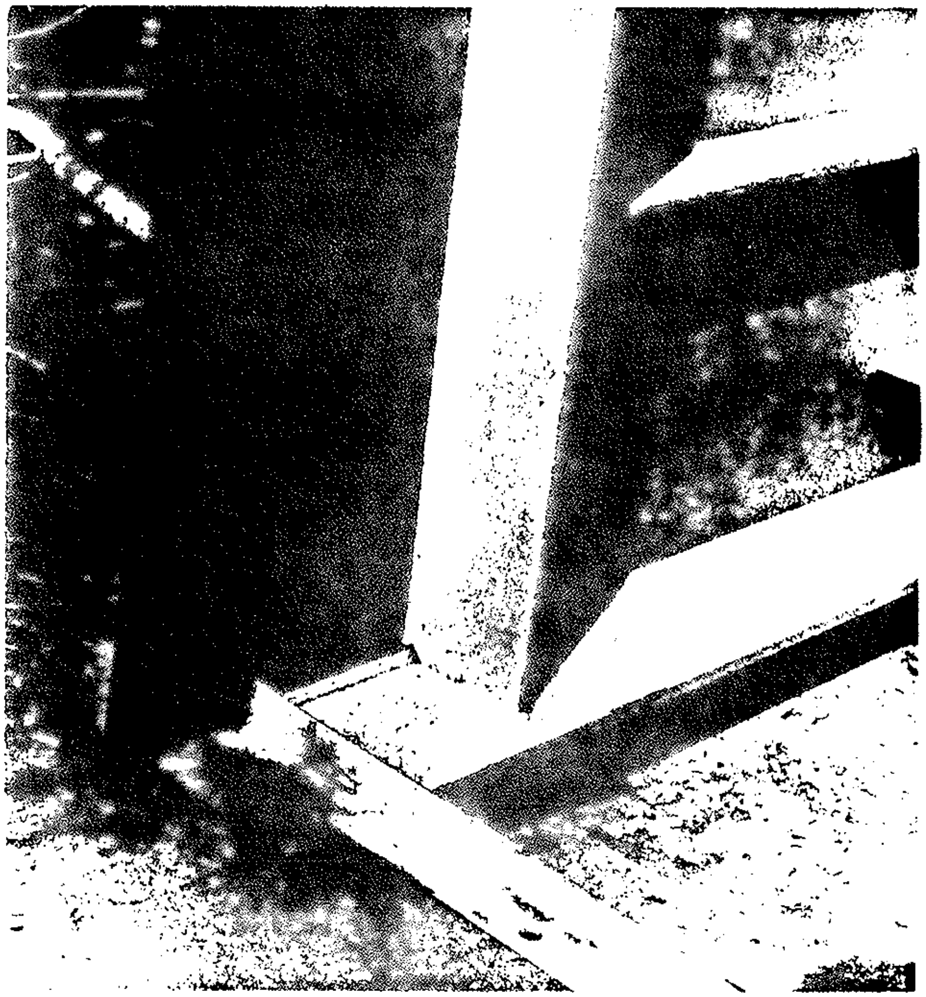
Figure 5.55 Lamellar Tearing



Fractured girder section showing the initiating crack formed at the toe of the transverse lap weld across the end of the cover plate. This crack viewed transversely resembles the lamellar tear crack shown in Figure 5.55. The blue-black coloration indicates the crack formed when the metal was still at a temperature of 400° to 500° from the welding heat.

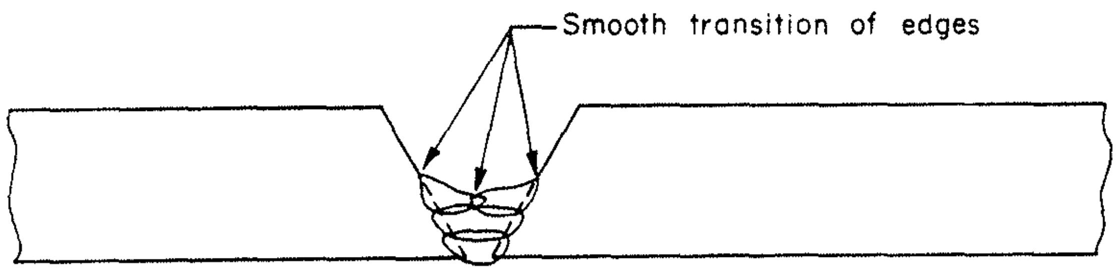
Second side of fracture shown above. The paint in the crack indicates that the girder was painted after the initiating crack was formed. This raises some question as to why this crack remained undetected and points out the great danger associated with the use of welded cover plates.

Figure 5.56 Fractured Girder Section

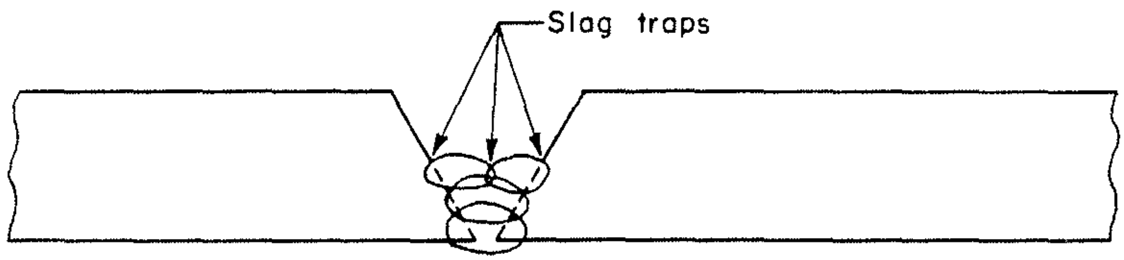


This is a picture of an attachment welded to the flange of a girder to hold it in place during shipping. This kind of careless welding can be the cause of disastrous cracks. Notice the similarity between the orientation of this weld and the cover plate weld that initiated the crack shown in Figure 5.56.

Figure 5.57 Welded Attachment



Good

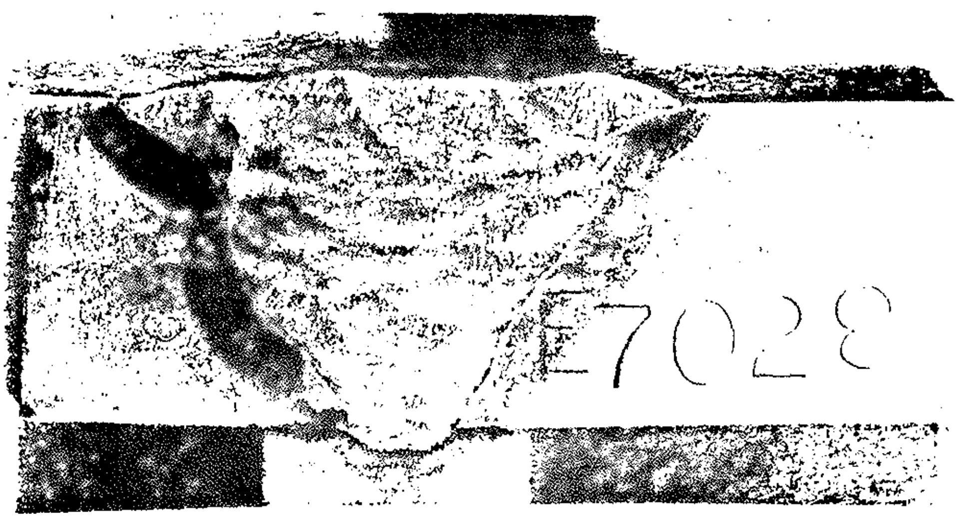


Bad

In multiple layer welding, the preceding weld pass should be clean and free from fused welding flux before depositing the next weld pass. Slag removal or cleaning of the weld requires use of a slagging pick or pneumatic scaling tools, followed by vigorous wire brushing. The intermediate weld passes should have a smooth transition of their edges so that the next pass can fuse properly.

Figure 5.58 V Welds

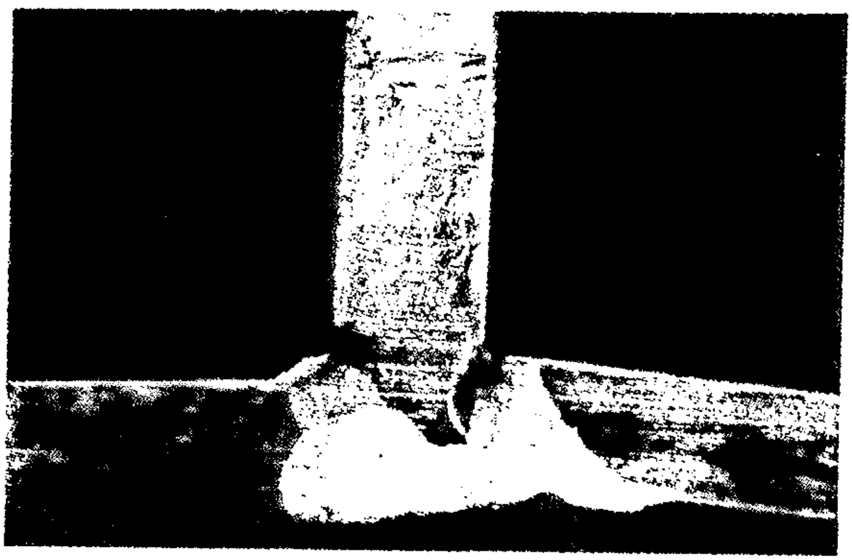
This cross-section of a multipass single V weld shows a lack of penetration of the right hand side of the weld root hidden by the backup strip. This illustrates one of the more important reasons for the removal of backup strips.



This is a cross-section of the same multipass weld shown above at a different point showing good penetration. Nevertheless, the reentrants between the backup strip and the plate can nucleate cracking at the fusion line.

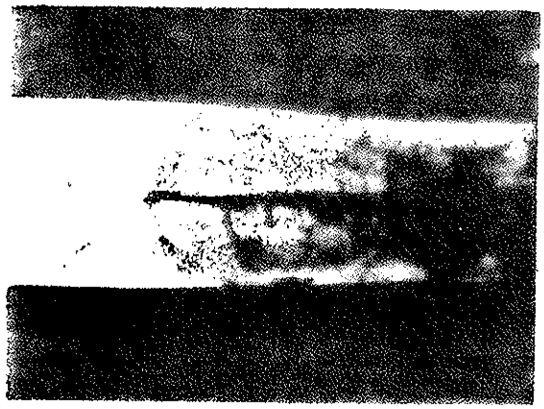
Figure 5.59 V Weld

This is a transverse cross-section through multipass fillet welds joining the center web to the flange of a double box bridge tower leg of A514 steel. The lamellar weakness of the flange coupled with residual stresses developed by the weld have combined to cause this extreme lamellar tear. This failure extended almost the full length of the section.

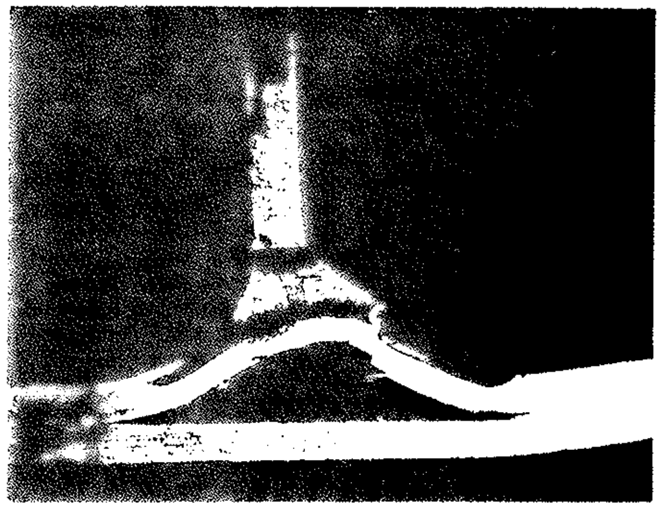


The full penetration weld joining the right hand leg of this section to the stem shows lack of fusion and a penetration defect caused by the difference in the heat capacities of the sections joined by the weld. This difference has made it difficult for the unskilled welder to bring the left side of the joint to welding heat without over-melting the right hand side of the joint.

Figure 5.60 Lamellar Tearing and Lack of Fusion



Laminated Section



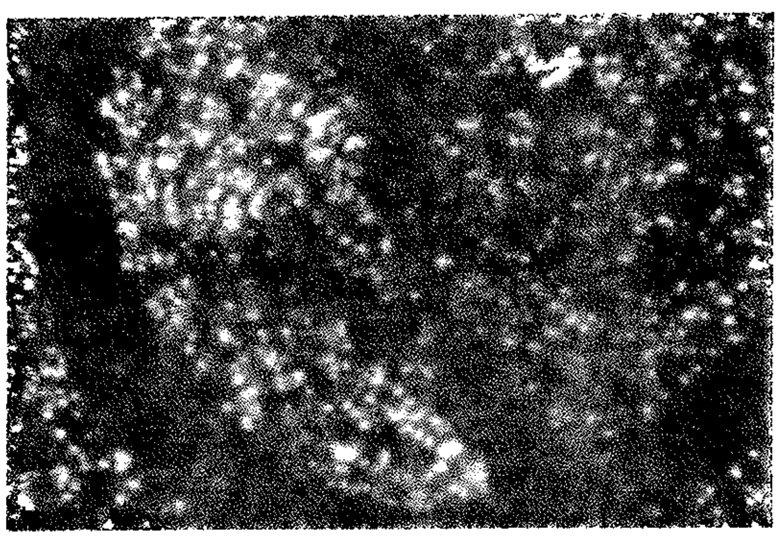
Laminated section showing characteristic weakness in the short transverse direction

Ingot mold design and ingot pouring practices are designed to reduce or confine the formation of pipes to the upper control portion of the ingot, where they can be cropped off before the ingot is rolled. However, occasional pouring errors may cause cavities to extend further into or even the full length of the center of the ingot. If the surfaces of these cavities become oxidized while red hot they may not fuse together to form a solid ingot when rolled, hence they will form laminations in any plates rolled from ingot which contain them. Such laminations will be located in the center of the width and the thickness of the plate.

Thus, they are usually found near the center of the flange cross-section and near the center of the web cross-section about half way between the upper and lower flange. Figure 5.61 shows examples of such laminations. When the plate on one side of a lamination is being cut away with a torch, the plate on the other side will usually remain intact because of the thermal barrier formed by the lamination. This provides a means of determining the extent of the lamination. While incurred plate laminations will be parallel to the applied stresses on fabricated girders, they reduce the buckling strength of webs and may cause failures by extending to the plate surface through heat affected zones of welds made on laminated sections.

Figure 5.61 Laminated Section

Weld joint cross-section showing the weld used to block off non-metallic inclusion intersecting the joint. The circled area is enlarged below.



Enlarged view of the circled portion of the heat affected zone of the blocking weld shown above. This view shows the transverse crack that has formed off these inclusions.

Figure 5.62 Non-metallic Inclusions

5.6.3 Scope of Quality Control

Quality control tasks performed during shop fabrication should include checking conformance to welding procedure specifications including tolerances, dimensional checks, prevention of damaging and unnecessary surface repairs (those which would preferably be ground out rather than covered by weld metal), efforts to minimize damage caused by material handling, and demanding conformance to each aspect of the specifications. Nondestructive inspection, as a part of the contract specifications, is a quality control (QC) function. Problems arise in all phases of inspection. Occasionally, too much reliance is placed on NDI to discover faulty welding. Good visual inspection and supervision prior to and during welding operations will prevent 90 percent of the rejectable weld defects before they occur. It is not enough to rely on NDI to discover weld defects--every effort should be made to prevent them.

5.6.4 Shop Inspection Personnel

Concern for quality begins with the welders themselves. A well-trained and conscientious welder is an enviable asset. Shop inspection personnel should be required to be certified by AWS and must have as much authority as possible over the actual fabrication. Oversight is difficult in a shop operation, and many times there is an overlap of responsibilities in an organization. Quality control personnel do not want to delay production and production personnel are usually content with only enough quality to meet minimum specification requirements.

It is important that the duties of quality control personnel be strictly limited. On any large project, this is easier to enforce than on small projects with few supervisory personnel, but someone should be in charge of quality control with full responsibility.

5.6.5 NDT Scheduling

Quality control personnel are often forced to assume a minor role in production and scheduling. Most fabrication plants do not realize that properly applied quality control will save many manhours by reducing weld defects, eliminating errors in cutting of material, and repair of damages caused by improper material handling.

By timely scheduling of nondestructive testing (as part of quality control as opposed to quality assurance), bottlenecks in production do not occur. Shop time must be scheduled to allow for weld repairs. When both radiographic and ultrasonic inspection is required, additional time must be allotted. Again, it is highly recommended that UT be performed after RT has been completed (welds cleared radiographically).

Many times NDI is scheduled for the swing or graveyard shift because it interfers less with production operations. This makes it difficult to monitor NDI technicians as inspections are being performed unless quality assurance personnel are notified of all NDI to be performed and are permitted to witness tests. It is difficult to say how much NDI should be witnessed by quality assurance technicians. Some sort of judgment has to be made depending on the type of structure and the amount of NDI actually performed by QA personnel.

5.7 QUALITY ASSURANCE

The quality assurance program for shop fabrications entails basically inspection and record keeping. The customer's own inspection and records are usually relied upon to add validity to quality control inspections performed and many times are used to determine the status of a particular member if the quality control records are inaccurate or not up to date.

5.7.1 Quality Assurance Program--Shop

Quality assurance inspection entails strict enforcement of specification requirements, especially at the outset of a project. Once fabrication is under way, it is difficult to change a particular fabrication procedure and an experienced quality assurance inspector who is unafraid to correct mistakes before they cause defective welds is essential. The QA inspector can also fill any gaps or weaknesses in the quality control program.

5.7.2 Nondestructive Inspection

Nondestructive inspection for quality assurance must also be performed. Many times this is the only way to verify the competence of quality control NDI personnel. NDI must be performed on some of the same welds examined by QC and also performed on some welds not examined by QC.

A large part of quality assurance must entail verification of test results of NDI. This may involve extra radiography as a spot check of weld quality or radiographic results already reported by QC. As far as UT is concerned, the most effective way to verify test results is to perform some additional UT on welds reported by QC as acceptable and on some welds with defects reported but not of rejectable severity.

The reporting of defects not of a rejectable magnitude should be a specification requirement. This allows for QA checks of defect severities; it allows for later checks on any growth of flaws; it provides constant verification of the abilities of the QC technicians, and it helps the QC personnel maintain control. Another instance to consider would be a case of an entire weld containing marginally acceptable defects. Quality assurance inspectors must have support within the specifications to require repair of a weld that is found to be marginal.

5.7.3 Records

Quality assurance records must be formulated such that all pertinent information can be included on one form for a particular member. This information should include heat numbers of all plates used to make up that member, plate thicknesses, dates of all NDI performed on that member, and possible dates of acceptance of the member. In addition to this one form, there may be many report forms for individual inspections, but these can easily be referenced by date from the master record form. A systematized chart showing progress of fabrication and NDI is highly recommended. A chart such as this can be mounted on a wall or in large booklets for quick reference and can be updated on a day-to-day basis.

Figures 5.63 through 5.66 present suggested charts to record fabrication progress and NDI.

5.8 FIELD ERECTION

5.8.1 Erection Conference

Field erection must be considered in the same manner as the shop fabrication. Details of erection must be scrutinized closely and subject to approval of the owner.

A conference covering erection procedures may be held in conjunction with the prefabrication conference mentioned earlier. It is preferably held at another time than the prefabrication conference for all medium to large size structures, since different personnel are usually involved in the two phases of contruction. The topics of

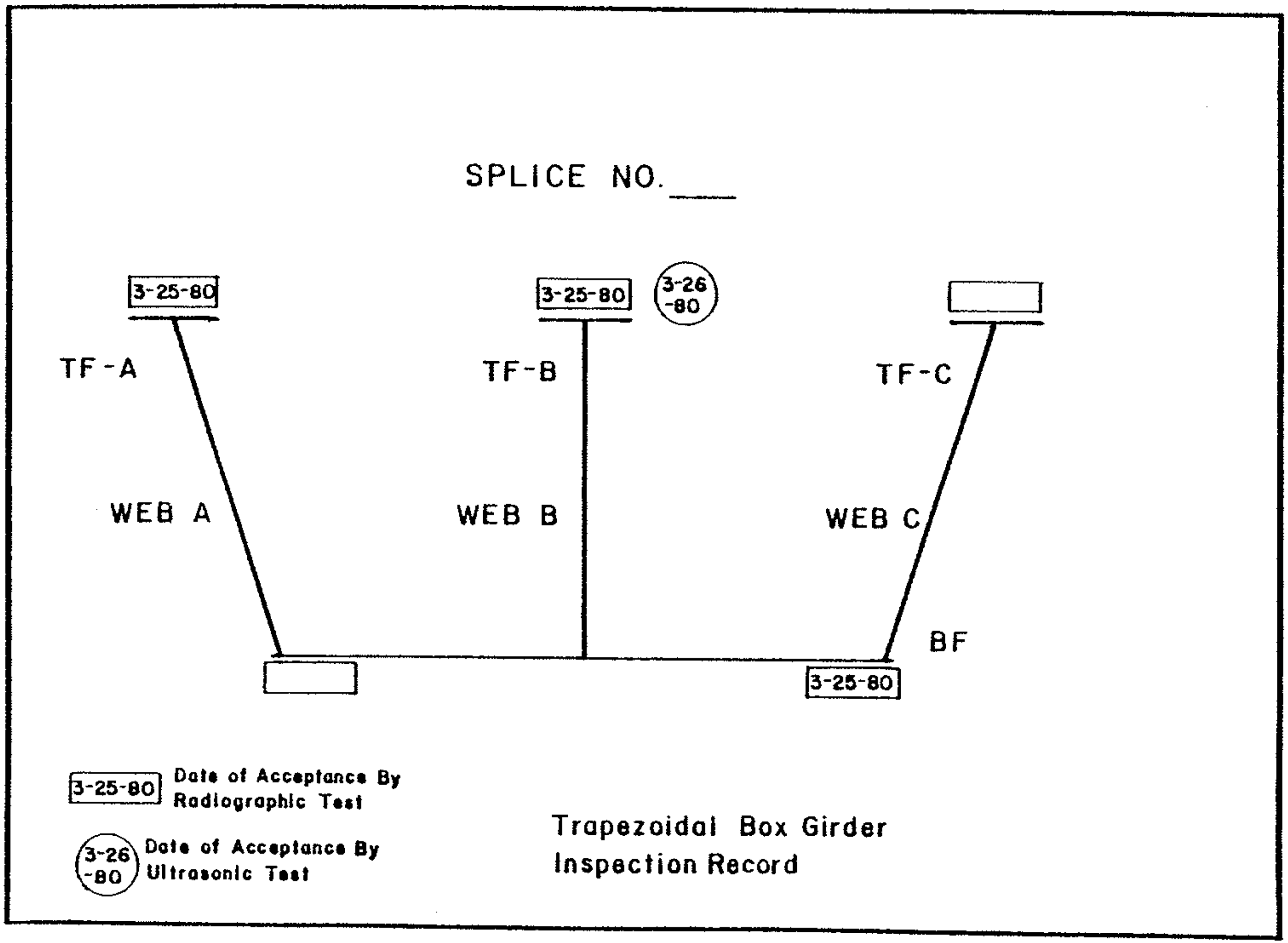


Figure 5.63 Sample Inspection Record

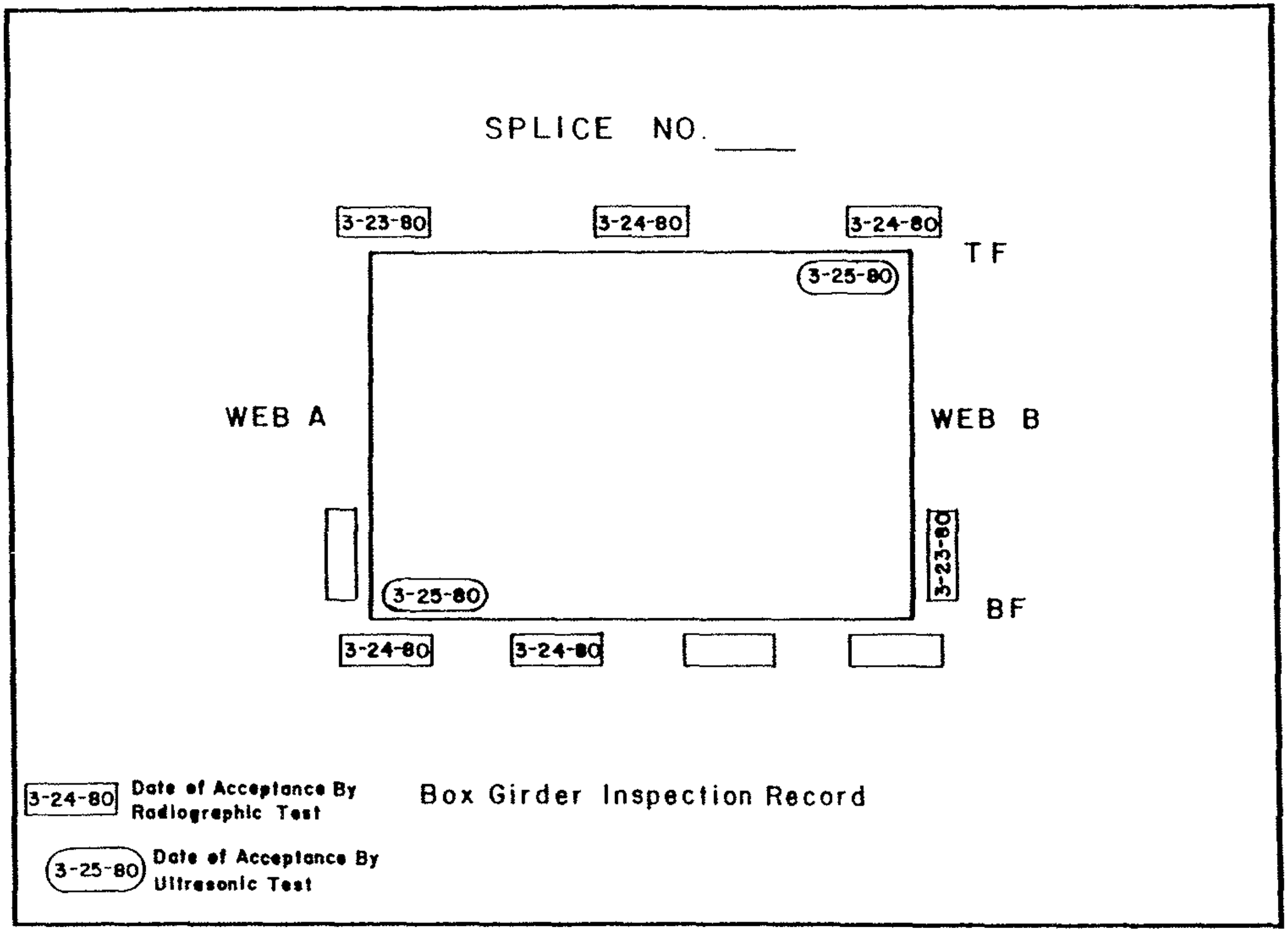


Figure 5.64 Sample Inspection Record

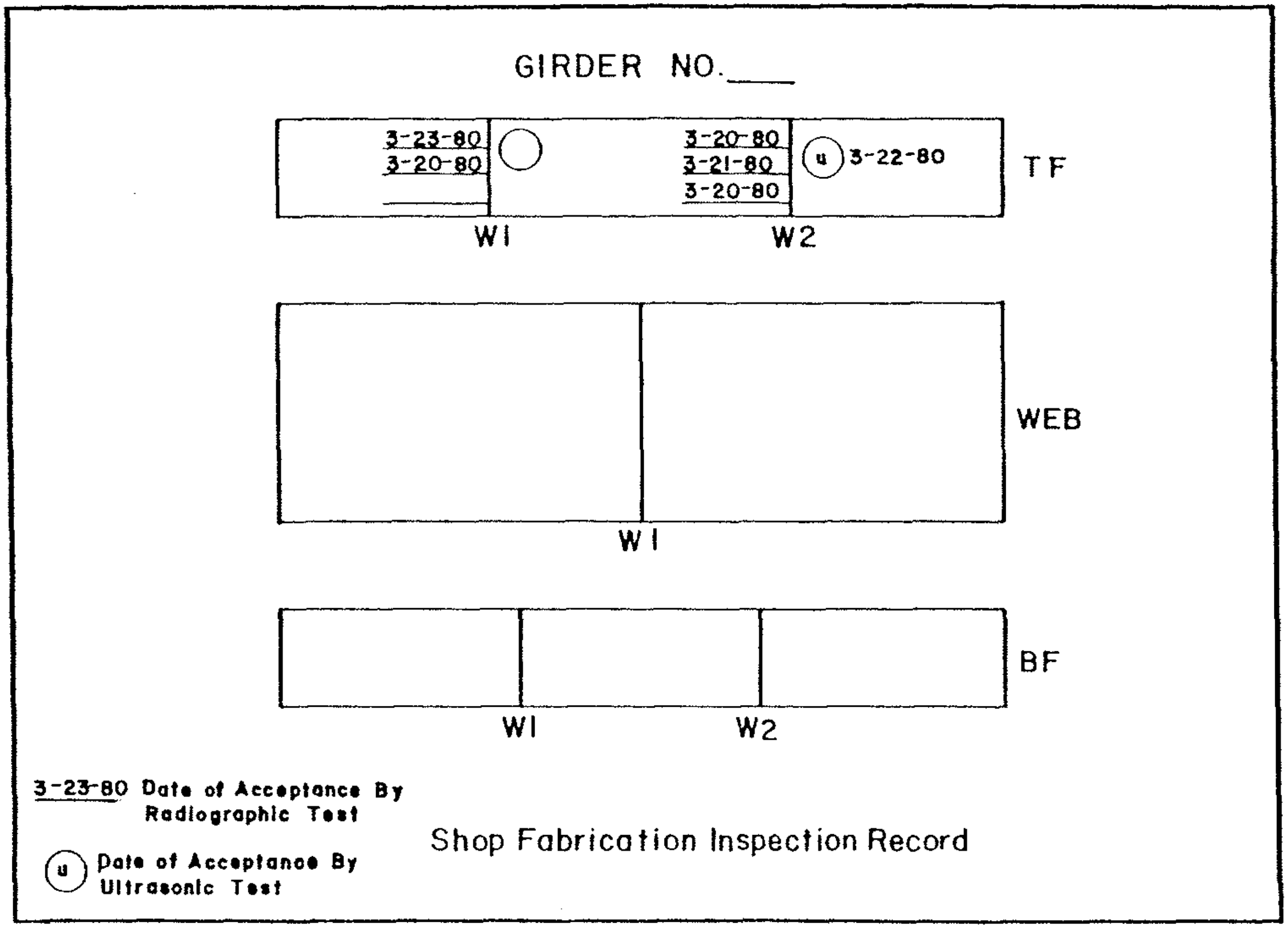


Figure 5.65 Sample Inspection Record

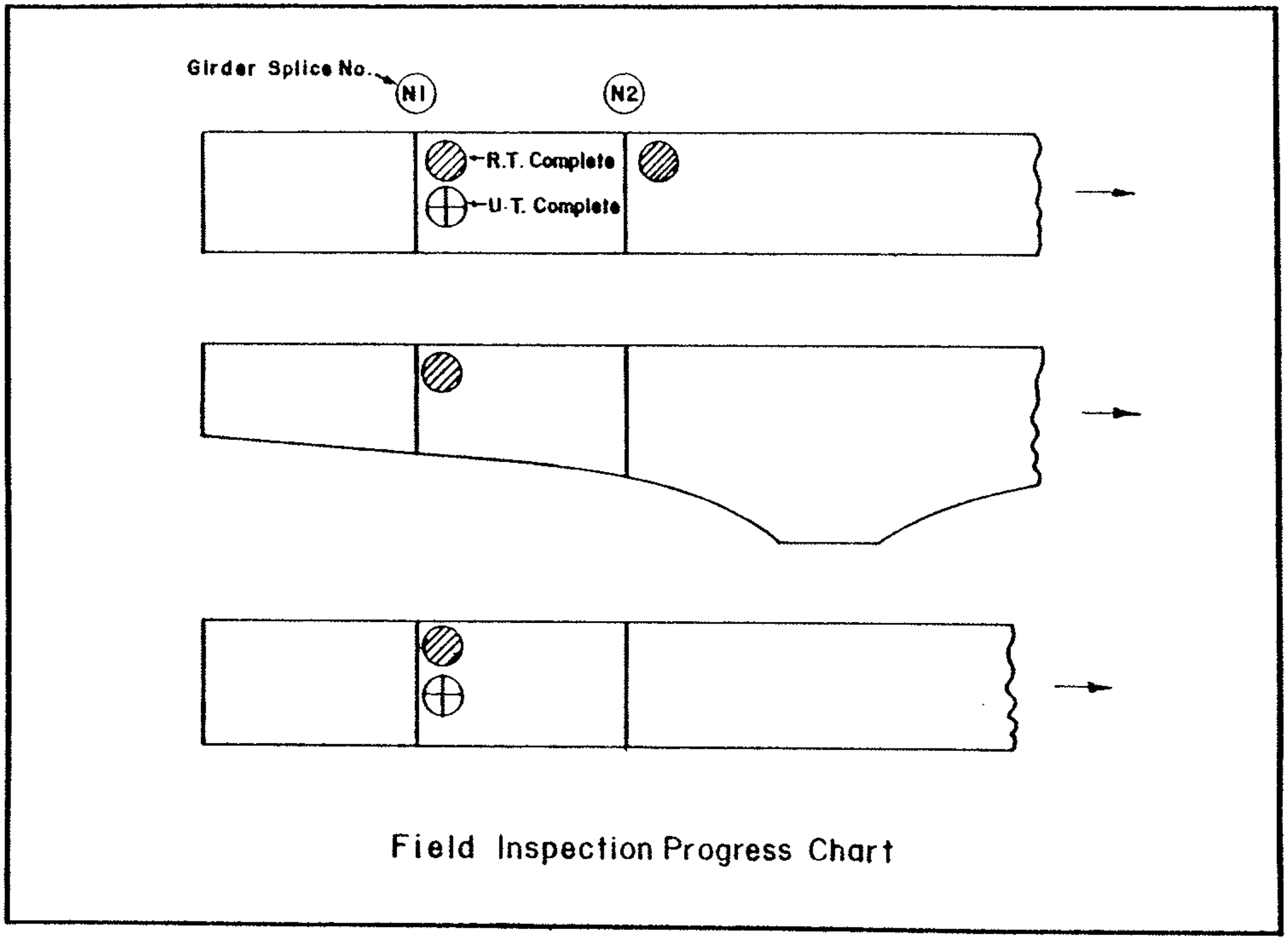


Figure 5.66 Sample Inspection Record

concern at the erection conference include erection drawings, field erection sequence, and welding specifications as they apply to field joints and girder splices. These are all in addition to general field erection information, plans, specifications, quality control, etc.

5.8.2 Review of Erection Drawings

Erection drawings must be subject to review and approval by the owner. Of particular interest should be lifting and jacking lugs and clamps, locations of any dogs used for alignment, details of maintaining girder position during weld splicing, and details of any temporary welds on the structure.

All work should conform to the approved erection drawings since these become contract documents as soon as they are approved and returned. Owner personnel who should be included in review of the erection drawings are the design engineer, the welding engineer, and the chief quality assurance inspector.

The reason that lifting lugs and other erection aids are of particular interest is primarily that they are temporary as far as the structure is concerned and upon removal, the areas at which they were attached are usually ignored by the contractor (except for cosmetic grinding). Problems which may go unnoticed without close inspection include underbead cracking, toe cracking, crater cracking, and lamellar tearing.

5.8.3 Review of Erection Welding Sequence, Procedures, Welders Qualifications, etc.

Erection welding sequences and procedures must be closely reviewed for conformance to specification requirements. Welder qualifications must also be reviewed.

Because of the great amount of out of position welding inherent in field erection, extra attention must be directed to welding procedures and welding sequences and their enforcement by QC and QA.

5.8.4 Review of Quality Control Plans

The quality control plan for field erection or fabrication generally requires provisions for appreciably more inspection of fitup and dimensional checks than for shop fabrication. Field erection is complicated by the need for false work (substantial in many cases) which must maintain girder segments or smaller members in a stable condition for completion of welding.

Access during all phases of field erection is necessary. Provisions for leaving access facilities in place until all necessary quality control and quality assurance inspections are carried out should be included as part of the quality control plan. The amount of time allowed for QC and QA should be determined early in the project and can be noted with comments on the quality control plan submittal. These comments should be developed by the quality assurance inspection department; the review of falsework and other devices such as jacking lugs would naturally be carried out by the owner's design department and the welding engineering department.

5.8.5 Scheduling of Nondestructive Testing

Nondestructive testing performed at field sites generally requires more planning and more time. The logistics problems alone at a field site are an obvious time consuming and frustrating factor to consider in the overall construction schedule.

Preparation, access and the NDI itself must all be considered to a great extent in the scheduling of field erection. As part of the preparation, it should be noted that NDI should not be performed on welds until they meet all other requirements of the specifications. Welds not made in accordance with the procedure specifications (i.e.,not preheated properly, with improper joint configurations, or outside the limitations of variables) should be located prior to NDI and replaced or otherwise evaluated and documented prior to NDI. For critical structures especially, total reliance on NDI to determine weld integrity is a mistake.

5.9 QUALITY ASSURANCE PROGRAM--FIELD

The field quality assurance program must rely heavily on well-trained and experienced personnel. This is more important than in a shop fabrication situation because very often inspection must be carried out at considerable heights and in awkward positions. Usually, time cannot be allowed for inexperienced QA personnel to gain experience.

5.9.1 Inspection

As mentioned above, considerably more inspection of fitup and alignment and elevations is necessary for field erection than for shop fabrication. Engineering or surveying personnel should usually be relied on for much of this work on large and/or critical structures. Fitup can be checked easily by a competent inspector, but most inspectors should not be expected to check alignment, camber, or elevations.

5.9.2 Nondestructive Inspection

Quality assurance NDI must be performed to a greater extent in field erection situations primarily because of difficulties in witnessing enough QC inspections for adequate assurance that it is done correctly. Even if enough time is spent by QA personnel to witness all QC work, some spot checking of test results submitted by UC technicians must be done.

5.9.3 Records

Records of NDI by both QC and QA technicians must be kept to an even greater extent for field operations. This is especially true for projects conducted with more than one location of erection being carried out, such as on top of two or more piers of a multiple span bridge or on two or more spans. Reliance on these records is usually greater for field operations to verify progress and status of members than in shop operations because of the multiple locations of erections that must be overseen by the contractor's supervisory personnel.

TOPIC 6
FRACTURE CONTROL

OBJECTIVES:

1. *To summarize and compare salient features of the AASHTO and the FHWA fracture control plans.*

2. *To describe the application of these plans to:*

   a. *Design*

   b. *Materials*

   c. *Construction*

   d. *Inspection*

6.0 INTRODUCTION--FRACTURE CONTROL PLANS

When Topic 6 was prepared, there were currently two documents that set forth fracture control plans. One was the AASHTO "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members" (September 1978). The other was a proposed FHWA plan comprised of three volumns entitled, "A Proposed Fracture Control Plan for New Bridges with Fracture Critical Members" (June 1978).

Both of these documents specifically addressed fracture critical members (FCMs) whose failure may result in collapse of the bridge. By definition, the connecting welds to any attachment joined to a tension component of a FCM are considered as an integral part of the tension component and, therefore, are considered fracture critical.

it is the author's opinion that all primary members should be treated alike except for material toughness and quality assurance.

6.1 DESIGN

The AASHTO Plan does not deal directly with design and detailing. According to the commentary, the current AASHTO Design Specifications are considered adequate to meet fracture control needs. This, apparently, is based on the assumption that the fatigue criteria will achieve the desired results.

Some states, as well as many individual designers, do not agree that meeting the AASHTO fatigue criteria will always minimize fracture; consequently, they prohibit details they consider potential problems even though such details are permitted by the AASHTO Design Specifications. Details not permitted include, but are not limited to, partial length cover plates, fillet welding across tension flanges, welded connections to tension flanges, back-up bars and partial penetration welds.

Generally, this prohibition pertains to all primary members whether they are fracture critical or not.

The authors agree with prohibiting details that may be potential problems and also agree that such decisions apply to all primary members.

The proposed FHWA Plan, unlike the AASHTO Plan, contained design requirements including prohibition of some details that are permitted by the AASHTO Design Specifications. The design requirements were a minor part of the Plan in comparison to the construction requirements, yet the Plan required "that the designer have overall responsibility for implementation of this fracture control plan, both in fabrication and in erection."

It is the author's opinion that this Plan did not recognize the usual roles of the designer engineer and the construction engineer. We agree that it is the designer's responsibility for implementation of a fracture control plan through design and specifications; however, once the project is under contract, the designer assumes the role of a consultant to the construction engineer.

The FHWA Plan, as does the AASHTO Plan, referred only to fracture critical members; thus, the prohibition of certain details was less effective than requirements of some states and individual designers who apply such criteria to all primary members.

6.2 MATERIALS

Both the AASHTO and FHWA Plans included toughness requirements for the steel used in fracture critical members; however, they differed. The two Plans differed as to what temperature the steel should be tested to obtain a reliable toughness value as determined by Charpy Vee-Notch (CVN) tests. AASHTO specifies CVNs for various types of steels and thicknesses at temperatures 70° above the Lowest Anticipated Service Temperature (LAST) for three temperature zones. The FHWA Plan specified CVNs for various yield strengths and thicknesses at a temperature equal to the Lowest Anticipated Service Temperature.

Research indicates a 70° or greater temperature shift exists for most specimens, but not all.

The authors believe that fracture control should apply to all primary members with increased attention to fracture critical members. This increased attention can be achieved partially through testing for toughness at the Lowest Anticipated Service Temperature.

6.3 CONSTRUCTION

The term construction as used here refers to fabrication and erection which includes both shop and field welding. The AASHTO and FHWA Plans emphasized construction requirements such as fabricator's qualifications and welding requirements. The FHWA Plan was more detailed.

Both Plans pertain, as was intended, to fracture critical members with no concern for other primary members.

The authors firmly believe that the same fabrication and erection requirements should apply equally to all primary members involving tension, whether fracture critical or not.

6.4 INSPECTION

Quality control and quality assurance involves inspections using various types of tests and to various degrees. Both the AASHTO and the FHWA Plans include numerous requirements for inspection such as welding inspector qualifications, nondestructive testing personnel qualifications, test methods, techniques, etc.

In addition to the two documents referred to as the AASHTO and FHWA Plans, the FHWA on November 27, 1979, issued Technical Advisory T 5140.11 on the subject of "Quality Control and Quality Assurance Inspections on Welded-Steel Fracture Critical Members."

As stated in the title, this Advisory pertains to fracture critical members; however, the following is included:

4. Discussion

    a. While this TA is written primarily for fracture-critical members, it must be remembered that most of these recommended practices are equally applicable to the inspection of redundant members. The engineer in charge should utilize judgment on the application of the specific requirement to individual members depending on the degree of sensitivity involved.

b. While criticality in terms of safety is not so severe for redundant members, the cost implications still exist in such cases.

The authors are basically in agreement with the above quotation, but would go further to declare that a minimum quality control requirement should apply equally to redundant and non-redundant members. Quality assurance inspection should be more demanding for non-redundant members. This implies that better quality and quality assurance control should be required during fabrication.

Current construction specifications, including those published by AWS, may be deficient on quality control; however, this deficiency is not limited to inspection of fracture critical members. The quality required by the contract plans and specifications should be achieved through fabricators and erectors who have quality control programs that meet this need. Adequate quality control should be a specification requirement for all bridge members irrespective of whether or not they are fracture critical.

In addition to the quality control, the owner or his consultant should provide a dependable quality assurance program. Quality control and quality assurance should be two distinctly different programs under separate administrations.

TOPIC 7
SUMMARY

OBJECTIVES:

1. *To summarize the training course and the author's views on, and experiences in, bridge design and construction with special references to*

   a. *Fracture control*

   b. *The design team*

   c. *Specifications*

   d. *Quality Control*

   e. *Quality Assurance*

7.0 INTRODUCTION

Fracture control as presented in this course requires considerations by both design and construction that will minimize fatigue and fracture problems.

The term fracture control, as currently used, generally refers to fracture critical members of a non-redundant system. It is the author's opinions that (1) fracture control should apply equally to both redundant and non-redundant members, (2) added emphasis should be placed on non-redundant members through assurance of steel toughness by testing at the Lowest Anticipated Service Temperature, and (3) assurance of quality fabrication and erection can be achieved through more demanding quality assurance testing.

7.1 SUMMARY

Fatigue and fracture problems do exist. Information and data on some current or recent problems were presented under Topic 1. The problems are not limited to fracture critical members, a specific type of bridge, or type of member, or to any one particular strength of steel. Most problems can be attributed to the selection of poor design details and/or to poor fabrication and erection practices combined with inadequate quality control and quality assurance.

The bridge designer is responsible for preparation of contract plans and specifications that include considerations that may minimize fatigue and fracture problems. Appropriate consideration can only be made if the designer can distinguish between good and bad details, and can recognize the fact that current design specifications do not always provide answers as to which details are preferable. The designer is not expected to know all the answers; thus, it is of utmost importance that input from other specialists be available. Generally, designers have limited knowledge and experience in specialities other than design. Designers have seldom been in a steel mill, a fabricating plant or at a construction site during erection and field welding.

The designer is the principal specialist during preparation of contract plans and specifications. He is assisted by material and welding engineers and, to some extent, by the construction engineer. Once a project is under construction, the resident engineer--one who is a specialist on construction and contract administration--becomes the responsible person. He can call upon the special expertise of the materials engineer, the welding engineer, and the designer for assistance. Fracture control must be a team effort.

The theme, presented for design considerations, is focused on clean-cut members and selection of splices, connections and attachments that are likely to give the least amount of problems.

The design specifications have been developed from laboratory research and contain many choices but do not always provide assistance in the selection of the most desirable details as related to fabrication and welding. Some details permitted by the specifications should not be allowed except under the very best quality control and quality assurance programs.

A designer diligently attempts to avoid defective welds and members regardless of whether they are secondary members, primary members or fracture critical members. Secondary members have been treated somewhat lightly in design and construction. Some current problems stem from the failure of designers and construction engineers to recognize a secondary member. Any attachment welded to a primary member will reduce the allowable stress range of the primary member.

Longitudinal stiffeners for the tension flange of a box girder are definitely a part of the primary member; yet they have been known to be classed as secondary members.

It has been accepted practice to require quality control inspection for 100 percent of the splices in primary tension members and flanges. Quality assurance testing has varied depending on the type of steel, thickness, difficulty of fabrication, etc. Construction, fabrication and erection of fracture critical members need not be treated in a different manner than other primary tension members; however, it remains critical to provide a greater factor of safety for fracture critical members. The authors believe this can be provided through touchness testing and more demanding quality assurance testing.

Some engineers assume that the current fatigue specifications provide a greater factor of safety through reducing the stress range. It has been shown that for many structures, the more stringent fatigue specifications have no effect.

The current AASHTO Specifications do not address the problem of fracture control specifically. Instead, a piece-meal fracture control plan exists in various specifications such as AASHTO, AWS, and ASTM and the Contract Special Provisions. The proposed FHWA Plan was a more complex fracture control plan for fracture critical members.

Application of current specifications with supplemental Special Provisions are generally considered adequate; however, some states do not concur and have written their own welding specifications.

The authors believe there is a particular weakness in quality assurance programs. There is no recognized or accepted document or guide that adequately covers the topic of quality assurance.

In summary, fracture control can best be achieved by designing clean-cut bridge members with good clean attachments, selecting steels that are workable and have adequate toughness, and by enforcing quality control through adequate quality assurance.

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GLOSSARY

TERMS AND DEFINITIONS

A

Air-Arc Gouging, Arc-Air Gouging or Air-Carbon-Arc Gouging: A process for metal removal where the metal is melted by an electric arc and blown clear of the removal area by compressed air.

All-Weld-Metal Test Specimen: A test specimen wherein the portion being tested is composed wholly of weld metal.

Amplitude (U.T.): The vertical height of the trace deflection on the cathode ray tube of the ultrasonic flaw detector.

Amplitude Length Rejection Level (U.T.): The length of defect permitted for various "Decibel Ratings" as associated with throat thickness.

Angle of Bevel: See preferred term Bevel Angle.

As-Welded: The condition of weld metal, welded joints, and weldments after welding prior to any subsequent aging, thermal, mechanical or chemical treatments.

Attenuation (U.T.): The absorption of sound energy by the test material. In the ultrasonic test method of inspection specified by this manual the attenuation factor is at the rate of 2 db per inch of sound travel after the first inch. (Attenuator Factor "c").

Automatic Welding: Welding with equipment which performs the entire welding operation without constant observation and adjustment of the controls by a welding operator. The equipment may or may not perform the loading and unloading of the work. See Machine Welding.

Axis of a Weld: A line through the length of a weld, perpendicular to the cross section at its center of gravity.

B

Back Gouging: The forming of a bevel or groove on the other side of a partially welded joint to assure complete joint penetration upon subsequent welding from that side.

Backing: Material (metal, weld metal, asbestos, carbon, granular flux, gas, etc.) backing up the joint during welding.

Backing Pass: A pass made to deposit a backing weld.

Backing Strip: Backing in the form of a strip.

Backing Weld: Backing in the form of a weld.

Back Weld: A weld deposited at the back of a single-groove weld.

Base Metal: The metal to be welded or cut.

Bevel Angle: The angle formed between the prepared edge of a member and a plane perpendicular to the surface of the member.

Boxing: The operation of continuing a fillet weld around a corner of a member as an extension of the principal weld.

Butt Joint: A joint between two members lying approximately in the same plane.

Butt Weld: A weld in a butt joint.

C

Complete Fusion: Fusion which has occurred over the entire base-metal surfaces exposed for welding, and between all layers and passes.

Complete Joint Penetration Groove Weld: See Full Penetration Groove Weld.

Complete Joint Penetration: Joint penetration which extends completely through the joint.

Complete Penetration: See preferred term Complete Joint Penetration.

Concavity: The maximum distance from the face of a concave fillet weld perpendicular to a line joining the toes.

Consumable Guide Electroslag Welding: See Electroslag Welding.

Continuous Weld: A weld which extends continuously from one end of a joint to the other. Where the joint is essentially circular, it extends completely around the joint.

Convexity: The maximum distance from the face of a convex fillet weld perpendicular to a line joining the toes.

Corner Joint: A joint between two members located approximately at right angles to each other in the form of an L.

CO^2 Welding: See preferred term Flux Cored Arc Welding with External Shielding Gas.

Couplant (U.T.): A material used between the face of the ultrasonic search unit (transducer) and the test surface to permit or improve the transmission of the ultrasound between the search unit and the material under test.

Crater: In arc welding a depression at the termination of a weld bead or in the weld pool beneath the electrode.

D

Decibel (U.T.): A measurable unit of sound amplitude.

Decibel Rating (db) (U.T.): A value of amplitude of signal varying up or down from the standard reference gain setting, and corrected for distance attenuation.

Defect: Weld or base metal discontinuity discovered and evaluated by visual or nondestructive tests that is of rejectable size. This includes all dimensional discrepancies that exceed the allowable tolerances of these specifications and defective properties of the weld metal or base metal.

Defect Level (U.T.): The calibrated gain control or attentuation control reading obtained from a discontinuity.

Defect Rating (U.T.): The decibel reading in relation to the zero reference level after being corrected for distance attenuation.

Density: A method for measuring the degree of exposure of radiographs. The density is equal to the logarithm of the ratio of the light intensity incident on the film to the light intensity transmitted. (Sometimes referred to as Hurter and Driffield Density.)

Depth of Fusion: The distance that fusion extends into the base metal or previous pass from the surface melted during welding.

Discontinuity: Any internal or surface interruption of the continuity of the metal. This includes porosity, cracks, slag inclusions, inclusions of other metals or nonmetals, incomplete fusion, undercut, laminations and any other phenomenon or material that interrupts the metal. Minor changes in microstructure are not included.

E

Effective Length of Weld: The length of weld throughout which the correctly proportioned cross section exists. In a curved weld, it shall be measured along the centerline of the throat.

Electrogas Welding: A method of Gas Metal-Arc Welding or Flux Cored Arc Welding with Carbon Dioxide Shielding wherein molding shoes confine the molten weld metal for vertical position welding.

Electroslag Welding (EW): A welding process wherein coalescence is produced by molten slag which melts the filler metal and the surfaces of the work to be welded. The weld pool is shielded by this slag which moves along the full cross section of the joint as welding progresses. The conductive slag is maintained molten by its resistance to electric current passing between the electrode and the work.

Electroslag Welding (Consumable Guide): A method of electroslag welding wherein filler metal is supplied by an electrode and its guiding member.

F

Faying Surface: That surface of a member which is in contact or in close proximity with another member to which it is to be joined.

Filler Metal: The metal to be added in making a welded, brazed, or soldered joint.

Flat Position: The position of welding wherein welding is performed on the upper side of the joint and the face of the weld is approximately horizontal.

Flux Cored Arc Welding with External Shielding Gas (FCAW): An arc welding process wherein coalescence is produced by heating with an arc, between a continuous filler metal (consumable) electrode and the work. Shielding is obtained from a flux contained within the electrode and from an externally supplied carbon dioxide gas or gas mixture.

Full Penetration Groove Weld: A groove weld which has been made from both sides or from one side on a backing having complete penetration and fusion of weld and base metal throughout the depth of the joint.

Fusion-type Defect (Also referred to as Fusion Defect): Signifies slag inclusions, incomplete fusion, inadequate penetration and similar generally elongated defects in weld fusion.

Fusion: The melting together of filler metal and base metal which results in coalescence. See Depth of Fusion.

Fusion Boundary: The interface between the weld metal (consisting of filler metal and melted base metal) and the unmelted base metal as observed visually or by metallographic tests.

Fusion Zone: The area of base metal melted as determined on the cross section of a weld.

G

Gas Metal-Arc Welding (GMAW): An arc welding process wherein coalescence is produced by heating with an arc between a continuous filler metal (consumable) electrode and the work. Shielding is obtained entirely from an externally supplied gas, or gas mixture. Some methods of this process are called MIG or CO2 welding.

Gas Pocket: A cavity caused by entrapped gas.

Gouging: The forming of a bevel or groove by material removal. See also Back Gouging and Air-Arc Gouging.

Groove Angle: The total included angle of the groove between parts to be joined by a groove weld.

Groove Face: That surface of a member included in the groove.

Groove Weld: A weld made in the groove between two members to be jointed.

H

Heat-Affected Zone: That portion of the base metal which has not been melted, but whose mechanical properties or microstructure have been altered by the heat of welding or cutting.

Heat-Shrink: A procedure for curving, straightening or cambering plates, beams, girders and other pieces or fabricated members by the controlled application of heat to specific locations in the piece. The dimensional change of the material results from the upset shortening of the steel in the heated areas.

Horizontal Position: Fillet Weld - The position of welding wherein welding is performed on the upper side of an approximately horizontal surface and against an approximately vertical surface.

Groove Weld - The position of welding wherein the axis of the weld lies in an approximately horizontal plane and the face of the weld lies in an approximately vertical plane.

Horizontal Reference Line (U.T.): A horizontal line near the center of the ultrasonic test instrument scope to which all echoes are adjusted for db reading.

I

Incomplete Fusion: The failure to fuse together adjacent layers of weld metal or adjacent weld metal and base metal. This failure to obtain fusion may occur at any location in the weld deposit. This type of defect can result from the following:

a. Failure to raise the surface of the metal adjacent to the weld metal being deposited to its melting temperature through improper manipulation of the heat source.

b. Failure to remove mill scale, oxides, or other foreign material from the surfaces to which the deposited weld metal must fuse.

c. Failure to remove all traces of slag formed during the deposition of a previous weld bead. In such cases, where slag particles or films of slag are entrapped at the interface, the defect is called a "slag inclusion."

Intermittent Weld: A weld wherein the continuity of the weld is broken by recurring unwelded spaces.

Interpass Temperature: In a multiple-pass weld, the temperature (minimum or maximum as specified) of the deposited weld metal and adjacent base metal before the next pass is started.

J

Joint: The location where two or more members are to be joined.

Joint Penetration: The minimum depth a groove weld extends from its face into a joint, exclusive of reinforcement.

Joint Welding Procedure: The materials, detailed methods and practices employed in the welding of a particular joint.

L

Lack of Fusion: See Incomplete Fusion.

Lap Joint: A joint between two overlapping members.

Layer: A stratum of weld metal, consisting of one or more weld beads.

Leg of a Fillet Weld: The distance from the root of the joint to the toe of the fillet weld.

Longitudinal Weld Discontinuity: A weld discontinuity whose major dimension is in a direction parallel to the weld axis.

M

Machine Welding: Welding with equipment which performs the welding operation under the constant observation and control of an operator. The equipment may or may not perform the loading and unloading of the work. See Automatic Welding.

Manual Welding: Welding wherein the entire welding operation is performed and controlled by hand. See Automatic Welding and Machine Welding.

N

Node (U.T.): The distance the shear wave travels in a straight line before being reflected by the surface of the material being tested.

O

Overhead Position: The position of welding wherein welding is performed from the underside of the joint.

Overlap: Protrusion of weld metal beyond the toe or root of the weld. A notch defect resulting from excessive convexity and failure to fuse at the toe of the weld.

Oxygen Cutting (OC): A group of cutting processes wherein the severing or removing of metals is effected by means of the chemical reaction of oxygen with the base metal at elevated temperatures. In the case of oxygen-resistant metals the reaction is facilitated by the use of a chemical flux or metal powder.

P

Pass: A single longitudinal progression of welding operation along a joint or weld deposit. The result of a pass is a weld bead.

Peening: The mechanical working of metals by means of impact blows.

Penetrameter: A radiographic quality indicator.

Piping Porosity: Pinholes that are included in a plane passing through the root of a weld approximately normal to the weld surface whose depths are greater than their diameter.

Plug Weld: A circular weld made through a hole in one member of a lap or tee joint joining that member to the other. The walls of the hole may or may not be parallel and the hole may be partially or completely filled with weld metal. (A fillet-welded hole or a spot weld should not be construed as conforming to this definition.)

Porosity: Gas pockets and any similar generally globular type voids.

Positioned Weld: A weld made in a joint which has been so placed as to facilitate making the weld.

Postheating: The application of heat to an assembly after a welding or cutting operation.

Preheating: The application of heat to the base metal immediately before welding or cutting.

Preheat Temperature: The temperature specified that the base metal must attain in the welding or cutting area immediately before these operations are performed.

Procedure Qualification: The demonstration that welds made by a specific procedure can meet prescribed standards.

Q

Qualification: See preferred terms, Welder Qualification and Procedure Qualification.

R

Random Sequence: See preferred term Wandering Sequence.

Reference Level (U.T.): The decibel reading attained from a horizontal reference line height indication of a reference reflector.

Reference Reflector (U.T.): The standard reflector contained in the IIW reference block or other approved blocks.

Reinforcement of Weld: Weld metal in excess of the specified weld throat.

Rejectable Discontinuity (Defect)(U.T.): A reflector of sufficient size to produce a signal (Decibel Rating) equal to or greater than the reject values specified in Table 704.54.

Any discontinuity or weld flaw not permitted under the weld quality requirements of the specifications.

Resolution (U.T.): The ability to distinguish separate trace deflections from closely spaced reflecting surfaces.

Root Face: That portion of the groove face adjacent to the root of the joint.

Root Gap: See preferred term Root Opening.

Root of Joint: That portion of a joint to be welded where the members approach closest to each other. In cross section the root of the joint may be either a point, a line or an area.

Root of Weld: The points, as shown in cross section, at which the back of the weld intersects the base metal surfaces.

Root Opening: The separation between the members to be joined at the root of the joint.

S

Scanning Level (U.T.): The db setting during scanning.

Semiautomatic Arc Welding: Arc welding with equipment which controls only the filler metal feed. The advance of the welding is manually controlled.

Shielded Metal Arc Welding (SMAW): An arc-welding process wherein coalescence is produced by heating with an arc between a covered metal electrode and the work. Shielding is obtained from decomposition of the electrode covering. Pressure is not used and filler metal is obtained from the electrode.

Size of Weld:

Groove Weld - The joint penetration (depth of chamfering plus the root penetration when specified).

Fillet Weld - For equal leg fillet welds, the leg length of the larges isosceles right-triangle which can be inscribed within the fillet-weld cross section.

Slag Inclusion: Oxides and other nonmetallic solids entrapped in weld metal or between weld metal and base metal. Slag inclusions generally result from the failure to remove the slag between beads and layers of multipass welds, from improper manipulation of the electrode or from failure to provide a proper contour on which each weld bead is deposited.

Slot Weld: A weld made in an elongated hole in one member of a lap or tee joint joining that member to that portion of the surface of the other member which is exposed through the hole. The hole may be open at one end and may be partially or completely filled with weld metal (A fillet-welded slot should not be construed as conforming to this definition.)

Sound Beam Distance (U.T.): The distance between the search unit sound index point at the steel interface and the reflector (as calibrated).

Spatter: In arc and gas welding, the metal particles expelled during welding and which do not form a part of the weld.

Stringer Bead: A type of weld bead made without appreciable transverse oscillation.

Stud Base: The stud tip at the welding end, including flux and container, and 1/8 in. of the body of the stud adjacent to the tip.

Stud Welding (SW): An arc-welding process wherein coalescence is produced by heating with an arc drawn between a metal stud, or similar part, and the other work part until the surfaces to be joined are properly heated, when they are brought together under pressure. Partial shielding may be obtained by the use of a ceramic ferrule surrounding the stud. Shielding gas or flux may or may not be used.

Submerged Arc Welding (SAW): An arc-welding process wherein coalescence is produced by heating with an arc or arcs between a bare metal electrode or electrodes and the work. The arc is shielded by a blanket of granular, fusible material on the work. Pressure is not used and filler metal is obtained from the electrode and sometimes from a supplementary welding rod.

a. Single electrodes - means one power source which may consist of one or more power units.

b. Parallel electrode - means two electrodes connected electrically in parallel exclusively to the same power source. Both electrodes are usually fed by means of a single electrode feeder. Welding current, when specified, is the total for the two electrodes.

c. Tandem Electrode - refers to the geometrical arrangement of the electrodes in which a line through the arcs is parallel to the direction of welding. Separate power sources are used for each electrode. It is common to use direct current reverse polarity in the lead electrode and alternating current in the following electrode.

T

Tack Weld: A weld made to hold parts of a weldment in proper alignment until the final welds are made.

Tacker: One who, under the direction of a fitter, or is a fitter, tack welds parts of a weldment to hold them in proper alignment until the final welds are made.

Tee Joint: A joint between two members located approximately at right angles to each other in the form of a T.

Temporary Weld: A weld made to attach a piece or pieces to a weldment for temporary use in handling, shipping or working on the weldment.

Throat of a fillet weld:

Theoretical - The distance from the beginning of the root of the joint perpendicular to the hypotenuse of the largest right-triangle that can be inscribed within the fillet-weld cross section.

Actual - The shortest distance from the root of a fillet weld to its face.

Throat of a Groove Weld: See preferred term Size of Weld.

Toe of Weld: The junction between the face of a weld and the base metal.

Transverse Discontinuity: A weld discontinuity whose major dimension is in a direction perpendicular to the weld axis.

U

Undercut: A groove melted into the base metal adjacent to the toe or root of a weld and left unfilled by weld metal.

V

Vertical Position: The position of welding wherein the axis of the weld is approximately vertical.

W

Wandering Sequence: A longitudinal sequence wherein the weld bead increments are deposited at random.

Weave Bead: A type of weld bead made with transverse oscillation.

Weld: A localized coalescence of metal wherein coalescence is produced either by heating to suitable temperatures, with or without the application of pressure, or by the application of pressure along, and with or without the use of filler metal. The filler metal either has a melting point approximately the same as the base metals or has a melting point below that of the base metals but above 800° F. (427° C.).

Weld Bead: A weld deposit resulting from a pass. See Stringer Bead and Weave Bead.

Weldability: The capacity of a metal to be welded under the fabrication conditions imposed into a specific, suitably designed structure and to perform satisfactorily in the intended service.

Welder: One who is capable of performing a manual or semiautomatic welding operation. (Sometimes erroneously to denote a welding machine.

Welder Certification: Certification in writing that a welder has produced welds meeting prescribed standards.

Welder Qualification: The demonstration of a welder's ability to produce welds meeting prescribed standards.

Welding (Noun): The metal joining process used in making welds.

Welding Machine: Equipment used to perform the welding operation. For example, spot-welding machine, arc-welding machine, seam-welding machine, etc.

Welding Operator: One who operates machine or automatic welding equipment.

Welding Procedure: The detailed methods and practices including all joint welding procedures in the production of a weldment. See Joint Welding Procedure.

Welding Sequence: The order of making the welds in a weldment.

Welding Technique: The details of a welding operation which, within the limitations of the prescribed joint welding procedure, are controlled by the welder or welding operator.

Weldment: An assembly whose component parts are joined by welding.

English to Metric System (SI) of Measurement

Quanity	English unit	Multiply by	To get metric equivalent
Length	inches (in)or(")	25.40 .02540	millimetres (mm) metres (m)
	feet (ft)or(')	.3048	metres (m)
	miles (mi)	1.609	kilometres (km)
Area	square inches (in^2) square feet (ft^2) acres	6.432×10^{-4} .09290 .4047	square metres (m^2) square metres (m^2) hectares (ha)
Volume	gallons (gal) cubic feet (ft^3) cubic yards (yd^3)	3.785 .02832 .7646	litres (l) cubic metres (m^3) cubic metres (m^3)
Volume/Time (Flow)	cubic feet per second (ft^3/s)	28.317	litres per second (l/s)
	gallons per minute (gal/min)	.06309	litres per second (l/s)
Mass	pounds (lb)	.4536	kilograms (kg)
Velocity	miles per hour(mph) feet per second(fps)	.4470 .3048	metres per second (m/s) metres per second (m/s)
Acceleration	feet per second squared (ft/s^2)	.3048	metres per second squared (m/s^2)
	acceleration due to force of gravity(G)	9.807	metres per second squared (m/s^2)
Weight Density	pounds per cubic (lb/ft^3)	16.02	kilograms per cubic metre (kg/m^2)
Force	pounds (lbs) kips (1000 lbs)	4.448 4.448	newtons(N) newtons (N)
Thermal Energy	British thermal unit (BTU)	1055	joules (J)
Mechanical Energy	foot-pounds(ft-lb) foot-kips (ft-k)	1.356 1.356	joules (J) joules (J)
Bending Moment or Torque	inch-pounds(ft-lbs) foot-pounds(ft-lbs)	.1130 1.356	newton-metres (Nm) newton-metres (Nm)
Pressure	pounds per square inch (psi)	6895	pascals (Pa)
	pounds per square foot (psf)	47.88	pascals (Pa)
Stress Intensity	kips per square inch square root inch (ksi $\sqrt{in}$)	1.0988	mega pascals $\sqrt{metre}$ (MPa $\sqrt{m}$)
	pounds per square inch square root inch (psi $\sqrt{in}$)	1.0988	kilo pascals $\sqrt{metre}$ (KPa $\sqrt{m}$)
Plane Angle	degrees (°)	0.0175	radians (rad)
Temperature	degrees fahrenheit (F)	$\frac{tF - 32}{1.8} = tC$	degrees celsius (°C)