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WELDING INFORMATION SYSTEM - DESIGN, OPERATIONS, METHODS

by

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ABSTRACT

This thesis encompasses a Welding Information System for Design, Operations, and Methods (WISDOM). In essence, the WISDOM project aimed at research and development of an innovative and comprehensive information collection on welding in close cooperation with industry utilizing modern micro-computer techniques of information review, retrieval, storage, updating, and transfer. Emphasis is placed on new and more efficient methods of knowledge communication, pursuing the idea of exploring knowledge (in contrast to rehearsing recipes). The development of this project concentrated on the following three main topics:

1. Background information on steel metallurgy and welding
2. Analysis and Design Methods for Welded Connections
3. Code Requirements of Welds

Preliminary research on the WISDOM project involved communications with industry to determine primary objectives. The need for an information system on welding was unquestionably apparent. The knowledge base is supplemented with graphic images and analysis and design programs. It is hoped that the information in this system will be delivered to the engineer in an efficient and useful manner. The priority here is to raise welding awareness while promoting efficient welded design. In addition, we hope that the WISDOM system will close the ever widening gap between the design engineer and the fabricator.

In the most general sense, the primary objective of the WISDOM project was to produce an integrated self-paced teaching tool for both engineering professional and student alike. The versatility of the system is maintained by presenting the information in modular form. Screen graphics oriented learning modules encourage the user to become more involved in the learning process than more conventional teaching methods allow. Analysis and design modules promote rapid and efficient connection design in a more consistent and professional manner. A smooth transition from the learning environment to engineering practice is envisioned by providing realistic design tools with a transparent background.

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1 Why W.I.S.D.O.M ?

The term "information age" is well known to most of us, however, the shear magnitude of its nature is not easily understood. Only in moments when we are buried under a flood of information, be it by periodical magazines or by large data-bases, such as the purchase orders of the last ten years, do we envision what the information age really means. Information can be printed, spoken, and drawn. It can help us to understand a concept from which we can create new methods which again may be put back into the information chain. We definitely need the information and we need even more the skill to manipulate that information.

The word processing program is an important tool for converting information into tangible form. The spreadsheet program is a more transitional tool. It allows one to transform data or formulae into figures or graphs, and permits the extrapolation of data to enable forecasting of events. A drawing program is a mere information handler which deals with the most compact form of information, the picture. An analysis program takes information from an input file through an established calculation procedure to create a new set of information, the response of a structure or process.

All of these programs act as stand-alone devices and are limited in application and scope. Relational data base programs, on the other hand, exist on a slightly higher plane. They allow data blocks to interact with each other and, better still, permit "links" between individual files and file segments. This type of database management software has been the cornerstone of information management on large computers and has more recently been adapted for use on personal computers.

The WISDOM project aims at research and development of an innovative and comprehensive information collection on welding in close cooperation with industry utilizing modern microcomputer techniques of information review, retrieval, storage, updating, and transfer. Emphasis is placed on new and more efficient methods of knowledge communication, pursuing the idea of exploring knowledge (in contrast to rehearsing recipes). The development of this pilot project concentrates on the following three main topics:

1. Background information on steel metallurgy and welding
2. Analysis and Design Methods for Welded Connections

3. Code Requirements of Welds

This includes the consideration of different types of welds and joints and a thorough review of welding history and basic welding processes.

1.1 WISDOM Objectives

Preliminary research on the WISDOM project involved communications with industry to determine primary objectives. The need for an information system on welding was unquestionably apparent. The content of the pilot project concentrates on basic welding information combined with state of the art design principles and code requirements. The knowledge base is supplemented with graphic images and analysis and design programs. It is hoped that the information in this system will be delivered to the engineer in an efficient and useful manner, so that we might raise welding awareness while promoting efficient welded design. In addition, we hope that the WISDOM system will close the ever widening gap between the design engineer and the fabricator.

In the most general sense, the primary objective of the WISDOM project was to produce an integrated self-paced teaching tool for both engineering professional and student alike. The versatility of the system is maintained by presenting the information in modular form. Screen graphics oriented learning modules encourage the user to become more involved in the learning process than more conventional teaching methods allow. Analysis and design modules promote rapid and efficient connection design in a more consistent and professional manner. A smooth transition from the learning environment to engineering practice is envisioned by providing realistic design tools with a transparent background. In particular, WISDOM focuses on three general educational aims which cannot be achieved by conventional textbook techniques:

1. Relations. It is important to facilitate the observation and creation of connections and relationships among ideas, concepts, events, and facts. WISDOM provides the software tools that will permit the user to annotate existing modules, create new information, and create new links between existing and upgraded information.

2. Visualization. Wisdom enhances the users ability to visualize and perceive complex and dynamic phenomena which might otherwise need extensive analytical calculations and graphical or motion picture representations. The full impact of this educational aim may not be realized until further additions are made to the pilot project.

3. Exploration. WISDOM encourages exploration of an information-rich environment such that students may discover ideas, themes, and facts on their own.

The following module types are typical of the WISDOM pilot project:

1. Background welding information modules
2. Specific welding design modules
3. Spreadsheet modules with code related requirements and design guides
4. Hypertext module with background information

Besides the above stated educational objectives, WISDOM has several technological aims:

1. Reusability. The software industry and the academic research community tolerate by far too much wasteful reinvention of code. A large portion of development time for each of the thousands of software products is spent writing code that performs functions already implemented in many other packages. While this competition is worthwhile for fine tuning special product areas, WISDOM uses large portions of commercially available code and remains accessible to future developers.

2. Consistency. An important feature of the WISDOM shell is to provide a consistent screen oriented user interface across modules, such that the selection of choices and commands, and the creation and following of links operate identically in all applications. This consistency is extended deep into the coding, such that even data structures and procedures can be easily understood by other developers.

3. Modularity. If a program system like WISDOM is developed by more than one person it is mandatory to be not only consistent, but modular. The modularity facilitates partial development and further extensions. Modules have to be created as stand-alone applications and one must by able to test them without interference with other developers. The modules must have the ability to be integrated into the system with little effort.

1.2 Hardware and Software Details

Hardware

The WISDOM environment was developed for use on IBM or compatible machines with 640k memory, hard-disk, and a MICROSOFT compatible mouse. For screen display, VGA is preferable but not required as EGA and CGA will work with minimal screen distortion. Although it is possible to use monochrome display, color is far more desirable. In addition, the software is compatible with UNIX type work stations (SUN, APPOLLO, NEXT).

Software Standards

The operating system for the WISDOM environment is PC-DOS. All software was developed in C and C++ in order to be compatible with further hardware development. C++ is essentially object-oriented C. Numeric files are in WK1-format with numbers in IEEE-format. Text files are in ASCII-format while intermediate and configuration files are in binary format. Software functions were purchased or licensed whenever possible and new modules, if fundamental, were contracted out.

User Interface Standard

WISDOM implements a window-like screen appearance with two-level menu choices that collapse after selection. Menus allow the user to examine all available commands and to then execute any one of them by making a choice with a mouse or other pointing device, or by typing the first letter of the menu choice. All menu titles are displayed at the top of the screen in an entity called the menubar. The windows are used to divide the screen into (overlapping) areas. The screen-mode switches from character-oriented to graphics-oriented whenever necessary.

Software Background

WISDOM utilizes a number of commercial programming tools. TURBO C by BORLAND International was used throughout the WISDOM project as the primary programming language. This dialect of C language is a structured, modular, compiled, general-purpose language traditionally used for systems programming. It is portable, so one can easily transfer application programmes from one system to another. TURBO C is relatively inexpensive, yet fast and powerful. It can be used through a so-called integrated environment, which combines editor, compiler and linker. However, larger programme modules must be handled in the command line mode. As the additional libraries, such as C-scape and CB-Tree, were rather voluminous, mostly the command line mode was used.

C-scape

C-scape is a collection of functions for controlling the user interface of C programmes. With this library of C routines one can create and modify virtually any type of text or data entry screen. The C-scape library is portable and available for many PC and mini-computer operating systems (DOS, OS/2, UNIX, etc.). Its unique object oriented approach to programming enables the user to achieve high efficiency during developmental work.

PowerCell

PowerCell is a commercially available collection of functions, similar to C-Scape, with which one can construct a spreadsheet programme. We aimed at LOTUS 1-2-3 emulation and, in fact, achieve this excluding the graphics, as the graphics part will be appended externally. Current work concentrates on implementation of engineering specific functions not existent in LOTUS. The previously used spreadsheet compiler was discarded, but can be revived for special tasks.

CB-Tree

CB-Tree is a collection of C functions which handle balanced binary tree data organization. The balanced binary tree is an improvement to the more classical binary tree which tended to be unbalanced when the sample size was small. The balanced tree permits optimal information storage and retrieval with the small number of records associated with the WISDOM system.

PCX Programmer's Toolkit

The PCX Programmer's Toolkit is a collection of C functions which handle the capture and output of bit-mapped screen images. The toolkit permits the generation of a window border, whose size is dependent on the image, to define the screen area used for PCX display.

1.3 Hypertext Development

The hypertext module is the focal point of the WISDOM development. After intensive market research and testing of existing hypertext programmes, it became necessary to pursue the development of a specialized hypertext system which met all our needs. Many parts are incorporated into the complete hypertext system, WISDOM.

The **database** is the backbone of the hypertext system. Most database functions stem from the CB-TREE function library. Therefore, fast access, retrieval, update, and storage is secured. The index is organized in a balanced tree which, for our application, is faster then the older binary tree approach. There are two basic files which make up the hypertext-database: the index file and the record file. The hypertext programme implements a search and retrieval mode that searches the index file for the desired record and then directly accesses the main data-file for its retrieval. Additions to the main data-file, as well as to the index file, are handled by the hypertext programme. The user does not recognize that he/she is basically manipulating a database when using the hypertext and, of course, you don't have to be a programmer to use the system.

Text display is one of two modes used by the hypertext system to present information. This mode allows the user to view the hypermedia text-base in text format while any figures require the PCX format. Furthermore, in the text display mode the user can establish and use links to other text, figures, and even programmes.

PCX format is required by the hypertext system to display scanned and or drawn pictures. This format is a widely spread file format used by popular programmes such as PC-Paintbrush. For most quality drawing packages there exists file conversion programmes to create the PCX format. As the pixel image of the screen is stored in these files, the picture is dependent on the graphics adapter used. Therefore, slight distortions of the images will be encountered when the pictures are created/scanned and shown on different adapters (i.e. origin VGA, show on EGA results in slightly stretched pictures). These problems are unavoidable but minor.

The C-scape based **text editor** is fully integrated in the hypertext system with mouse controlled menus and cursor. Most of the standard block commands have been implemented to maintain consistency with other text editors. In addition, it can import external ASCII type files into the WISDOM hypertext system to support the creation of new topics.

A simple **graphics editor** or paint programme is being conceptually developed. The motivation for having a custom graphics editor is that it can be used as a simple integrated sketching tool. This editor will be object oriented and will use for storage the PCX-file format. This module is not implemented yet.

The **interactive linker** allows the creation of active links to text portions, to screen images in PCX format, and to other stand-alone computer programmes. The linking feature represents the most important characteristic of the WISDOM system as it separates conventional data bases and text editors from an interactive hypertext system. With this feature one can create cross referenced, non-sequential, logically linked, innovative knowledge and skill bases. In future this type of "pre-digested" knowledge collection will probably rival conventional methods such as books, seminars, and videos.

The **script facility** allows for "hard-wired" training sessions. The user can just step through a predetermined path of text, figures, programmes, and exercises. The script facility will be very useful to educators and invaluable in training sessions. Another possible use of the script facility is the self-recording feature. The user can easily create demo sessions, either from an existing hypertext base or from his own text and programme material. This enables many new and exciting possibilities. The current WISDOM version does not yet contain the script facility, but the concept has been tested and optimized.

A new product on the software market will permit the rapid development of a **custom spreadsheet**. This spreadsheet will utilize a file format and user interface that is basically compatible with the industry standard, LOTUS 1-2-3. However, the implemented functions will be adjusted to the needs of the engineers in order to keep the spreadsheet compact. To improve the integration to the WISDOM package some features go beyond the LOTUS capabilities. For example one will be able to load the programme with specifying attributes, which means one can automatically retrieve an initial work template.

Most of the **analysis modules** are spreadsheet based. Using the formatted spreadsheet as introduced by Navin and Stiemer [NAVI89] the worksheets are LOTUS compatible but implemented in the custom spreadsheet. Other modules are written in TURBO C when iteration methods and heavy number crunching require a faster execution time.

1.4 WISDOM - A Quick Tour

A sample of the WISDOM environment is provided here to highlight some of its features.

The opening screen, as you must have seen to get to this point, lists available topics from the chosen data base. Any of the topics or menu choices may be pointed to and clicked on with the mouse. Choice of a topic will initiate a search and retrieval of the desired record. Choice of a menu will bring down a sub-menu providing further related options. As an example, you can click onto the SCRIPT selection from the menu-bar. The sub-menu shows all the new choices for generating a script.

Suppose we are interested in double angle connections. From the main menu we can click onto many topics for related information. Pointing and clicking onto the link provided here, @1[09.2.1.2 Double Angle Connections], brings up the main information module on double angle connection. If you clicked onto this link, you would have discovered a new screen with many new options to further the quest for knowledge. You could link to design considerations for information about designing the connection or go straight to the design module. Furthermore, at any time, you can edit the current screen or create new links to other components such as text modules, graphic images, or other external design modules. Pointing and clicking onto @1[FLEX1.PCX] illustrates the result of linking to a double angle connection image.

Choice of a design program, for example @1[MAPLCALC.EXE b_to_cf], would bring up a spreadsheet in the Maplcalc environment. The user may alter cell inputs to generate a connection design while in this environment. The user also has the power to completely alter the program, so be cautious. The spreadsheet environment may be exited in the usual manner and the WISDOM environment will then automatically reappear.

The above very brief tour should have just got you warmed up. You should now be ready to experiment with the system at your own convenience.

1.5 Target Group and Benefits of the Project

The main target groups of WISDOM are the practicing engineers and the university students, who wish to specialize in this area. Using such a tool which is based on explorational techniques of education does not require sitting in seminar sessions or attending rigidly scheduled classes.

A survey of representative consulting engineers and fabricators indicated a definite need for an up-to-date, self-paced, learning and application tool for welding. Trying to close the ever widening gap between consultant and fabricator seems to be a timely task.

Currently none of the Canadian universities offer an undergraduate civil engineering core course which contains welding as a single specialized topic. It is not expected that this type of material will ever be given at this level, and very little chance exists for an inclusion in the graduate curriculum. However, it would be an excellent opportunity to supply interested students with such a tool for a self-driven exploring type of studying.

The accessibility and understanding of current knowledge and expertise on welding will be greatly improved by the WISDOM project and will lead to more frequent application of welds with a higher degree of confidence. Education and training will be improved, and with this, the general quality of applications in welding. A higher level of confidence in the professional community will further the use of welding.

New areas of research might be identified and intensified. A general raising of the image of welding can be expected.

An improved educational preparation plus the continual updating of knowledge for key persons in the decision making process will further the applications of welding of steel structures, improve the quality of design, and increase the efficiency of application.

2 About Welded Construction

Over the last few decades, welding has played an important role in the evolution of the structural steel industry. The continuing progress made in developing welding equipment and electrodes, the cultivating of the art and science of designing for welding, and the growth in trust and acceptance of welding have all united to make welding an indispensable part of an expanding construction industry.

Although few buildings or bridges in Canada are composed entirely of all welded construction (due to the popularity of field bolting), many have incorperated the precepts of good welded design. The economies inherent in welding over bolting (or riveting) have helped to offset the ever increasing cost of material and labour. In addition, greater efficiencies in the steel fabrication shop, made possible by welding, help to accelerate new construction schedules.

Many architects, engineers, contractors, and their client-customers have seen the benefits of welded construction. The precept Designing for Welding has become increasingly important as more people realize a greater depth of knowledge and experience with it.

2.1 Generating Welding Awareness

The analysis of successes and failures, the substantial documentation of research findings, and the innovative accomplishments of more liberal engineers and builders, over the years, have all combined to heighten the awareness of welding as a safe and economical means of connecting structural components.

Although this widespread acceptance of welding was a considerable accomplishment, there still remains the task of educating the design and detailing personel on how to achieve maximum efficiency when applying welded design. In addition, further recognition by code writing bodies of the potential strength of properly designed and detailed welded joints is necessary as we continue to strive for efficiency.

The realization that designing for welding offers a more efficient use of material while shortening fabrication time has encouraged architects, engineers, and contractors to incorporate welded connections into their design schemes.

2.2 Advantages of Welded Construction

One of the most significant advantages brought about through welding is freedom of design. The architect and structural engineer are free to employ new and innovative concepts of form, proportion, and balance in an effort to achieve greater aesthetic value. Many of the design restrictions that plagued the minds of many an Engineer, have now been lifted because of welding. This ever growing freedom of design has already resulted in such novel ideas as open-web expanded beams, tapered beams, Vierendeel trusses, cellular floor construction, orthotropic bridge decks, composite floor construction, and tubular columns and trusses.

Other advantages of structural welding include the following:

1. Designing for welding decreases the weight and cost of structural members, which no longer need to be weakened by bolt holes. A reduced beam depth results because the gross section is effective in carrying loads. The weight of the structure may be greatly reduced with savings on column steel, walls and partitions, facia, and possibly even reduced foundation requirements. In addition, the savings in transportation, handling time, and erection are proportional to the weight savings.

2. The inherently more compact connections made possible by welding permit a savings in connection material. In addition, these compact joints offer the best method for creating rigid connections, making them well suited to plastic design.

3. A savings in the cost of fabricating, fitting, and assembling may be realized as operations such as punching, drilling, and countersinking are essentially eliminated.

4. The relatively quiet operations associated with welding provide the workers and any neighbouring individuals with a more pleasant environment. When field welding is adopted, structures may be erected in relative silence, a definite asset in building in downtown areas, near office buildings or hospitals.

5. A Welded structure conveniently facilitates alterations and additions with minimum need for removal of existing components.

6. The smooth surfaces provided by the continuous nature of welded construction generally prevent the build up of corrosive dust and other matter in the steel framing of industrial buildings, thereby reducing the cost of cleaning, painting, and maintenance of exposed steel work. In addition, the welds are usually more corrosion resistant than the base metal.

7. Welding provides a good seal for storage tanks, service piping, etc..

8. The quality of the weld metal is superior to the base metal and as such a properly welded joint is stronger than the material joined. Welding results in essentially one-piece construction creating a rigid structure that permits design assumptions to be realized more accurately. Welded joints are better for fatigue loads, impact loads, and severe vibration [BLODGETT66].

2.3 Weld Strength

Many engineers are unaware of the great reserve of strength that welds possess. In some instances, this lack of strength recognition has even found its way into the code writing bodies. Although "Matching Conditions" today maintain suitable bounds on the ultimate tensile strength level of the weld metal relative to that of the base metal, there still remains a significant difference in their respective tensile yield strengths. The tensile yield strength of the weld metal can be significantly higher than that of the base metal, reportedly as high as 75%.

There are numerous reasons why weld metal has higher strength than the corresponding plate. The two most important are:

1. The core wire used in the electrode is of premium steel, held to closer specifications than the plate.

2. There is complete shielding of the molten metal during welding. This, plus the scavenging and deoxidizing agents and other ingredients in the electrode coating, produces a uniformity of crystal structure with physical properties on par with electric furnace steel.

A properly deposited weld possesses a tremendous reserve of strength or factor of safety, usually far beyond what industry specifications recognize. Therefore, overwelding, "just to be sure", is certainly not necessary and in most cases will do more harm than good.

2.4 Weld Quality

Part of the expense involved in making a welded connection goes toward achieving a specified weld quality. Welding procedures and subsequent inspections as laid out by the governing standard must be applied to ensure the specified weld quality is achieved. Obtaining the weld quality specified is usually not a problem, however all too frequently the quality specified bears little or no relation to service requirements.

The following provisions, when met, wil result in welds that meet the actual service requirements at the least possible cost:

1) proper design of connections and joints,

2) good welding procedure,

3) good welder technique and workmanship, and

4) intelligent, responsible inspection.

Four examples of test specimens exhibit respectively undercut, undersize, lack of fusion, and porosity. These weld flaws are considered individually in the examples and in all cases, under steady tensile load, the weld tested stronger than the plate. These examples are not meant to show that weld quality standards should be lowered, but rather how easy it is to make full-strength welds that are stronger than the plate. Note: These examples are quoted from "Design of Welded Structures" by Omer W. Blodgett (June1966).

undercut

This term describes the melting away of the parent material during actual welding. The test samples, seen in @1[wcon1.pcx], were prepared to show the effect of undercut. The specimens were stressed in tension under a static load and in all cases failure occurred in the plate and not in the weld.

undersize

At the time of these tests one rule of thumb said fillet size should equal 3/4 plate thickness to develop full plate strength. Using this method, a 3/8" fillet weld on 1/2" plate should "beat the plate". But, as seen in @1[wcon2.pcx], so did 11/31" and 5/16" fillets. Not until fillet size was reduced to 1/4" did weld failure occur . . . at a stress of 12,300 lbs/linear inch, more than 5 times the AWS allowable at that time.

lack of fusion

As indicated in @1[wcon3.pcx], weld samples contained varying degrees of lack of fusion. Welds were machined flush before testing, and weld failure did not occur until the unpenetrated throat dimension had reached 31% of the total joint throat.

porosity

A porosity test was performed using two specimens, one with excessive porosity (proved by radiograph) and one that showed perfect. The porosity did not weaken the joint, but rather, in both cases the weld was stronger than the plate. Specimens failed in the plate at approximately 60,100 psi.

Porosity, if not excessive, is usually not a problem because the voids are spherical and therefore permit a smooth flow of stress. The voids or gas pockets are not indicative of a notch. Tests have shown that welds containing relatively large amounts of porosity show no material changes in the tensile strength, impact strength, or ductility of the weld. These voids, uniformly distributed, could fill up to 7% of the weld's cross-section without impairing the joint's performance.

Weld ductility

The welding process produces a unitized or one-piece construction that, in the case of welded plate, is exceptionally sound, strong, and ductile. In a properly detailed joint, the weld is so ductile that it can be easily bent around a small radius. This type of ductility would rarely be required in any type of structure.

2.5 Designing For Welding

In order to produce efficient and economical welded structures, a designer must be aware of the fundamental differences between welding and other assembly methods. A case in point involves a welded girder. If the flanges of the girder were constructed with multiple cover plates, the cost would be excessive. However, if instead the flanges were fabricated from single plates butt welded at reasonable locations where the plate thickness could be reduced, then the economies of welding would be realized along with improved fatigue resistance.

Many contemporary structures employ the look of exposed steel framing as part of the artistic scheme. The simplicity of form essential to sleek, modern architecture may be achieved with welded design. The decision to use welding as the primary connection element should be made at the preliminary design stage so as to incorporate proper welded design principles.

The full section properties of connecting steel members are only fully realized with welding. In addition, simply converting a bolted connection to a welded connection would lack some of the benefits of welded design. In fact, the full advantages of using steel in competition with other materials will only be realized when the structure is erected as a welded design, and when fabricators and erectors use modern techniques of welding, production scheduling, and materials handling.

2.6 Designing Welded Buildings

The advantages of welded design and the role of welding grow with the size of the building. This is seen through the shop fabrication of columns and other structurals as well as in any field welding that may be associated with erection.

Many buildings (and bridges) have seen great savings through the use of expanded open-web beams and girders, fabricated by cutting and welding standard rolled beams. Significant savings in weight (as high as 50%) have been realized with open-web girders designed to have the required moment of inertia. The overall height of multistory buildings can be substantially reduced by running utility supply lines through these beams and girders rather than suspending them below. The resulting savings in material costs for columns, facia, stairs, etc. can be quite significant.

Tapered beams and girders can be easily fabricated from standard rolled beams permiting an endless variety of savings in building design. Tapered spandrel beams, made deep enough at the column end, may reduce the bending force and eliminate the need for column stiffeners. Shop welding the spandrel beam to the column before shipping to the site provides for low cost with production efficiency.

Optimal use of building space may be achieved by welding specially built-up columns to provide column-free interiors.

The clean trim lines which compliment the look of exposed steel can only be achieved by welding. Three-dimensional truss systems or space frames are optimally produced with properly designed welded connections. A welded design reduces or entirely eliminates any extraneous material in the multiplicity of connections that would otherwise be present with other assembly methods. An efficient erection procedure would have the space frame shop-fabricated in sections with final assembly on the ground at the site before lifting into place.

Unlike other design methods, plastic design uses the calculated ultimate load-carrying capacity of the structure. In the case of rigid framing, plastic design requires less engineering time than the more conventional elastic design and, in many cases, also results in a significant savings in steel. A plastically designed structure demands full plastic moments with sufficient strength, adequate rotational ability, and proper stiffness from its members. Designing for welding is the most sensible way to achieve an efficient plastic design.

2.7 Designing Welded Bridges

Many suspension, arch, truss, or plate and box girder bridges have been constructed from steel because of its inherent strength, reliability, and permanence. With the versatility of welding, bridge engineers can turn innovative ideas into enduring reality. This freedom of design has enabled some rather unusual and unique bridges over the past few decades. All this experience has shown that substantial savings may be realized through properly designed welded bridges. Variable depth bridge girders allow the placement of metal where it is needed and the removal of metal where it is not. This process can effectively save tons of steel.

In many instances, site requirements will place restrictions on the shape of the bridge. Welded design lends itself well to these limitaions on shape. For example, a site incorporating both a vertical grade and a horizontal curve will require super-elevation. The use of welding permits a more flexible design leading to a significant savings in steel while maintaining simplicity in fabrication.

The more work that can be performed under controlled shop conditions, the better. Welding lends itself well to bridge construction as large sections are shop-fabricated, shipped to the site, and merely lifted into position. This procedure can lower erection costs while compressing the project timetable.

The use of welded shear connectors in composite floor construction has provided large savings on many building and bridge projects of the past.

2.8 Welding Quickens Erection Time

The structures of today are erected quickly because of welding. Structures are built on a sub assembly basis with as much work as possible being performed under ideal shop conditions so that mass-production techniques can be fully exploited.

The progress made in recent years in automatic and semi-automatic welding equipment and in positioners and manipulators has made shop fabrication of special girders, knees, and built-up columns extremely attractive. In many cases, the ingenious designer can make tremendous savings through the design of special structural members. This includes members having complex cross-sectional configuration and hybrid members that are a mix of steels having different analyses. Modern structural fabricating shops have fixtures for assembling plates into columns and girders, manipulators for welding automatically, and positioners for supporting members so that attaching plates may be welded in the flat position.

Welding developments over the past few decades have improved welding speeds, while assuring high quality welds. In submerged-arc welding the use of multiple arcs, with two and three welding heads has tremendously increased welding speeds. Continuous wire processes for semi-mechanized welding for both shop and field applications have substantially increased productivity. Much progress has been made in automatic manipulators, enabling the welding head to be put into proper alignment with the joint of the member in a matter of seconds. This alignment is automatically maintained along the length of the joint during welding. These manipulators represent a major cost reduction possibility. As the size of the structure increases, the total arc time on a welded job becomes a decreasingly smaller percentage of the total fabricating time. Thus savings in handling time and increasing manufacturing cycle efficiency are the major potentials for cost reduction.

3 Metallurgy and Steel

The @1[3.1 Production of Steel] is overviewed briefly to provide some background on the origin of one of the most versatile construction materials in the structural field, steel.

@1[3.2 The Micro-structure of Iron and Steel] should provide an understanding of micro-structural transformations as they relate to the heating and cooling of steel.

@1[3.3 Welding Metallurgy in Structural Steels] covers certain characteristics of the weld metal and the heat affected zone.

3.1 Production of Steel

The production of iron and steel for industrial use is actually quite a simple process. A @1[3.1.01 Blast Furnace] charged with coke, limestone, and iron ore produces molten iron containing approximately 4 percent carbon as well as other impurities such as sulphur, phosphorous, and silicon. This molten iron is called pig iron because it was formerly cast into bars called pigs. Because of the high percentage of carbon, controlled cooling of pig iron (in a copula furnace) will produce @1[3.1.02 Cast Iron].

Steel may be defined as iron containing 0.1 to 1.7 percent carbon. In order to produce steel, the carbon in the iron must be removed in the oxidizing atmosphere of a steel-making furnace. Several types of steel-making furnaces are in use today and most of them reduce pig iron in combination with scrap steel. These steel producing furnaces are:

@1[3.1.03 Basic Oxygen Furnace]

@1[3.1.04 Open Hearth Furnace]

@1[3.1.05 Electric Furnace]

@1[3.1.06 Crucible Furnace]

@1[3.1.07 Induction furnace]

@1[3.1.08 Vacuum Furnace]

These steel producing furnaces create an oxidizing atmosphere to burn off and eliminate the carbon and other impurities in the pig iron. With the impurities and carbon reduced to a minimum, controlled amounts of carbon and other alloying elements may be added to the iron to produce the desired steel grade. The molten steel is then formed into rough stock shapes such as ingots, slabs, blooms, or billets. The @1[3.1.09 First Solid Forms of Steel] may later be formed into more exact shapes and products.

There are many methods used when @1[3.1.10 Processing Metals] into the finished product.

3.1.1 Blast Furnace

The modern blast furnace consists of a cylindrical steel vessel lined with firebrick. The average size is approximately 30 meters high and 8 meters in diameter. Holes in the bottom of the furnace exist to permit hot air to be blown in from hot blast stoves. A vent in the top permits gases to escape. Iron ore, coke, and limestone are carried to the top of the furnace via conveyor or skip hoist and dumped into the furnace through a bell shaped housing (hopper). The melted iron collects at the bottom of the furnace and is drawn off when sufficient quantity has accumulated.

A little about the materials .....

Iron constitutes approximately 4.7 percent of the earth's crust and is second in abundance only to aluminum among the metals. Because iron seldom exists freely in nature it is mined from the earth in the form of IRON ORE or IRON OXIDES mixed with impurities in the form of clay, sand, and rock. The most important types of iron ore are: Hematite (70% iron), Magnetite (72.4% iron), Limonite (63% iron), and Siderite (48.3% iron).

Limestone is the most common flux used in the reduction of iron ore. In the blast furnace the limestone melts and combines with the impurities from the iron ore. This combined form of the flux (SLAG) floats on the surface of the molten iron and may be drained off when desired.

COKE, which is practically pure carbon, is produced by heating soft coal in a closed container until the gases and impurities are driven off. Coke is one of the best fuels for the blast furnace because it is low in impurities such as sulphur and phosphorous and it furnishes enough heat to reduce the iron ore. Some modern furnaces use gas injection and solid fuel injection.

More about the process ......

The blast furnace has five major operations to perform: 1. Deoxidize the iron ore, 2. Melt the slag, 3. Melt the iron, 4. Carbonize the iron, and 5. Separate the slag from the iron.

Once the iron ore, coke, and limestone are charged into the top of the furnace they encounter the super hot air blasts coming up from the bottom of the furnace. The coke begins to burn and produces heat and gas to remove oxygen from the ore. Deoxidation of the iron oxides in the ore occurs at various temperatures as it drops down the furnace until the final reaction occurs which frees up the iron. At this point there is sufficient heat to melt the iron. Excess carbon from the coke unites with the iron and lowers its melting temperature. The molten iron then forms at the bottom of the furnace and is drawn off when a sufficient quantity has been collected. Meanwhile the limestone has melted and absorbed most of the impurities and is floating on top of the molten iron as slag. This slag will be drawn off before the melted iron is removed.

The blast furnace operation is continuous with the right proportions of coke, limestone, and iron ore being dumped in when necessary. Every 3 to 5 hours the molten iron is drawn off as a white hot stream of liquid iron called pig iron.

3.1.2 Cast Iron

Cast iron is usually produced by melting and oxidizing pig iron in a COPULA FURNACE. Here again coke is used as fuel to heat the furnace and limestone is used as flux. Scrap cast iron and steel may be combined with the pig iron in the furnace. The copula furnace eliminates the excess carbon and impurities as the metal and flux melt. After a sufficient quantity of molten metal has formed the molten slag is drawn off and the copula furnace is ready to be tapped. Tapping is the term used to describe the drawing off of molten metal from the furnace. Once removed the liquid iron is ready to be cast and for obvious reasons the product is called cast iron.

GREY CAST IRON is the most common form of cast iron and is produced by allowing the casting to cool slowly. This action permits some of the carbon to separate and form into free graphite (carbon) flakes which in turn gives the grey appearance. Grey cast iron can be machined.

WHITE CAST IRON is made by cooling the casting quickly. White cast iron is very hard and brittle and is difficult to machine.

Malleability may be defined as the ability to be pounded into shape. The cast irons produced from the procedures discussed so far lack malleability. In parts demanding malleability, or resistance to shock, MALLEABLE IRON may be produced by ANNEALING white cast iron at a temperature of approximately 900 degrees Celsius. In other words, the white cast iron is heated to a temperature of 900 degrees Celsius and held there for about 50 hours (in this instance) after this time it is allowed to cool slowly. The heating of the casting allows the carbon to diffuse within the structure. This moving carbon gathers and forms into small rosette shapes. The remaining iron is low in carbon which results in good ductility and toughness.

Recall that grey cast iron contains carbon flakes which make the material brittle. If magnesium were added to the liquid iron it would attract the carbon and form graphite spheres upon cooling. This would leave the remaining iron low in carbon and therefore very ductile. DUCTILE CAST IRON is produced using this process and has good strength, hardness, and ductility.

Although no longer produced in large quantities, WROUGHT IRON is still ideal for ornamental work as it is rust resisting, easily shaped, and easily welded. Wrought iron contains the lowest percentage of carbon (0.003%) of any of the ferrous metals used commercially and as such is soft, tough, and malleable. It is produced by melting pig iron on the hearth of a reverberatory or puddling furnace which is lined with iron oxide. This process almost completely removes all of the carbon, manganese, and silicon from the pig iron. The lower carbon level causes the fusion temperature of the iron to rise and when the iron becomes pasty it can be rolled up into balls and removed from the furnace. Any excess slag is removed by squeezing the balls of iron through rollers. The wrought iron is then rolled into muck bars and finally into commercial forms.

3.1.3 Basic Oxygen Furnace

The basic oxygen furnace consumes large amounts of oxygen to support the combustion of unwanted elements for the purpose of elimination. The furnace is charged with a combination of scrap steel (approx. 25%) and molten metal from the blast furnace (approx. 75%). Oxygen is blown into the furnace through a water cooled lance and the products of combustion are exhausted into a pollution control system. During the oxygen blow, lime and other materials are added as fluxes to help carry off the oxidized impurities as a floating layer of slag. After refinement, the molten steel is poured into a ladle where alloys may be added to give the steel the precise chemistry desired. Approximately 80 tons of steel can be produced in one hour.

3.1.4 Open Hearth Furnace

The open hearth furnace consists primarily of a large shallow basin to be charged with scrap steel (up to 50%), both molten and solid pig iron, and fluxes (primarily limestone). The basin, once charged, will be exposed to a sweep of open flames of burning air and fuel gas. Here again the high temperature reactions cause several unwanted elements and impurities to oxidize or burn off. These unwanted particles then either exit with the exhaust gases or combine with the limestone to form slag for later removal. At either end of the open hearth is a preheating stove constructed of fire bricks arranged in a checker board pattern. The air and fuel gas are heated as they enter the furnace through the preheating stove at one end thereby resulting in higher temperatures inside the open hearth furnace. The heat from the exhaust is used to maintain the temperature of the preheating stove and the direction of flow of air and fuel gas is occasionally switched to take full advantage of this regenerative process of heating. Once the steel has achieved the specified chemistry it is released through a tap hole into a ladle and the surface slag overflows into a slag thimble. Alloy additions are made to the molten steel in the ladle. For the open hearth furnace, a typical 350 ton volume of steel requires 5 to 8 hours.

In order to speed up production, recent advances have led to the use of lance fed oxygen into the roof of the hearth. This action effectively raises the flame temperature and speeds up the melting process.

The open hearth process is expected to produce a cleaner steel containing fewer oxides with better control over the alloying elements being maintained. Larger batches of steel may be produced at one time and pig iron and scrap considered unsuitable for the basic oxygen furnace made be made into steel by the open hearth method. Also, steel manufactured by the basic oxygen furnace may be further refined using the open hearth furnace.

3.1.5 Electric Furnace

In the electric furnace, the heat required to melt the combination of scrap steel and pig iron is produced by an electric arc. Limestone and other fluxes are added to the charged metal after it becomes molten. Three carbon electrodes inserted through the top of the furnace, which is closed to the atmosphere, are slowly consumed as they provide the electric arc necessary for the melting of the metal. As the electrodes are consumed they move down to maintain a specified distance above the molten metal. Here again the fluxes combine with the unwanted impurities to be later discarded as slag. Test samples may be removed from the furnace through small inspection ports to be analyzed and alloying elements may be added in the form of ferro-alloys as necessary. When the desired steel chemistry has been achieved most of the slag is poured off however some remains with the molten metal so that when poured into the ladle it will act as an insulating layer. Electric furnaces vary in capacity from 5 to 50 tons requiring 3 to 6 hours for heat completion.

Electric furnaces have traditionally been used to produce alloy steel, stainless steel, tool steel, and specialty steels. However more recently they have become high tonneage producers of carbon steel.

3.1.6 Crucible Furnace

One of the oldest methods of refining steel is the crucible furnace. The crucible or covered pot is made of ceramics and graphite. The charge consists of wrought iron or wrought iron and scrap steel plus the correct amounts of carbon and other alloying elements. The charged crucible is covered and sealed and then placed into the furnace where it is heated by hot gases. The size of the charge may vary between a few pounds and several tons. The quality of the steel produced in the crucible furnace is generally considered to be higher than that produced in an electric furnace, but the process is much slower and more expensive.

3.1.7 Induction Furnace

In an induction furnace, the metal to be heated is contained in a vessel and electrical conductors are wound around the vessel to form a coil. Alternating current is passed through the coil resulting in an alternating magnetic field being set up throughout the metal. When magnetic materials such as iron and steel are placed within an alternating magnetic field, they are heated. Therefore, induction heating of the vessel and the metal in the vessel results from this alternating magnetic field. By accurately controlling the frequency and amperage of the alternating current passing through the induction coil, it is possible to accurately control the temperature of the metal being heated. The design of the furnace and its low frequency induction coil sets up a controlled stirring action of the molten metal in the furnace thus producing a more homogeneous metal.

3.1.8 Vacuum Furnace

One of the drawbacks of the more conventional steel making furnaces is the absorption of gases (oxygen, nitrogen, and hydrogen) by the molten steel leading to porosity and inclusions in the metal when it solidifies. With the melting of steel in a vacuum furnace most of the gases are removed with the vacuum pumps leaving only a small amount of gases present in the furnace. The absence of these gases improves many of the qualities of the steel including ductility, magnetic properties, impact strength, and fatigue strength. These qualities would be otherwise unattainable. Two main types of vacuum furnaces used today are the @1[3.1.08.1 Vacuum Arc Furnace] and the @1[3.1.08.2 Vacuum Induction-Type Furnace].

Although the resulting steel is not quite as pure, vacuum degassers are an alternative to the vacuum furnace and involve subjecting the molten steel from a more conventional steel making furnace to a vacuum. Two types of vacuum degassing systems are @1[3.1.08.3 Vacuum Stream dagassing] and @1[3.1.08.4 Ladle Degassing].

3.1.8.1 Vacuum Arc Furnace

The metal charge for this furnace comes from huge consumable electrodes that were formed from the steel product of a more conventional steel making furnace. These long cylindrical metal electrodes are fed into the vacuum arc furnace at a controlled rate of speed to control the arc length. As the metal melts and drops from the end of the electrode, air and contaminating gases are constantly being pumped out of the furnace by vacuum pumps. The molten metal falls into a water-cooled steel crucible below which is the grounded part of the electrical circuit. This process is frequently used when the uniformity and purity of the metal is very important. No provision is made in this furnace for the adding of alloying elements.

3.1.8.2 Vacuum Induction-Type Furnace

The vacuum induction process involves charging an electric vacuum induction furnace with scrap or molten steel after most of the atmosphere has been evacuated. The induction furnace is airtight and attached to vacuum pumps so that most of the contaminating gases may be constantly removed. Provisions are made to allow alloying materials to be added to the furnace without destroying the vacuum. The heating of the metal, the pouring of the metal into ingots, and the cooling of the ingots are all done under vacuum conditions to prevent any contamination. The vacuum induction furnace is used when close control of the chemistry of the metal is of prime importance.

3.1.8.3 Vacuum Stream Degassing

Vacuum stream degassing utilizes a ladle of molten steel from a conventional furnace. The molten steel is separated into droplets before being exposed to the vacuum in the degassing chamber. Most of the undesirable gases are drawn away from the droplets before they fall and solidify into an ingot mold below which is also in the vacuum chamber.

3.1.8.4 Ladle degassing

Ladle degassing also utilizes a ladle of molten steel from a conventional furnace. A vacuum vessel is placed over the ladle with a funnel like end placed into the molten metal. With the vacuum pumps engaged, atmospheric pressure forces molten steel up into the vacuum chamber where unwanted gasses may be removed. After a sufficient period of time, the vacuum chamber is raised so that gravity will force the cleansed molten steel back into the ladle. This process must be performed several times to ensure all of the steel in the ladle gets degassed.

3.1.9 First Solid Forms of Steel

Molten steel from the basic steel making furnaces follow one of two major routes to the rolling mills where most of the industries finished products are formed.

1. INGOTS

The ingot molds are filled with the molten steel which then begins to cool and solidify immediately from the outside inwards. When the steel has solidified enough for the ingots to be removed from their molds they are taken to a furnace called a soaking pit. The ingots remain there soaking in heat until they reach uniform temperature throughout. The reheated ingots are then taken from the soaking pit to the roughing mill where they are shaped into semi-finished steel (usually blooms, billets, or slabs). Some roughing mills are the first in a series of continuous mills, feeding sequences of finishing rolls.

2. CAST SLABS (Strand Casting)

A continuous casting process is utilized to turn the liquid steel into semi-finished steel products. The liquid steel is poured from the ladle into a reservoir or TUNDISH where it is then permitted to flow vertically into a water cooled copper mold. The liquid metal in contact with the cool mold solidifies quickly and shrinks away from the side of the mold. A shell has now formed around the molten metal in the center of the mold. Further down the line the column of steel is supported by a system of withdrawing rolls. Meanwhile, a system of cooling water jets spray the steel as it emerges from the mold until solidification is complete. The continuous casting process generally produces steel in the form of blooms, billets, or slabs. BLOOMS are large and mostly square in cross section and are used in the manufacture of beams and columns. BILLETS are smaller and generally square in cross section and are used in the production of bars, pipes, wire, and wire products. SLABS tend to be wider in cross section and are used to make sheets, strips, plates, and other flat rolled sheet products.

3.1.10 Processing Metals

Molten metal from the furnace is originally cast into ingots or molds for later processing. In preparation for final processing, these castings are reheated to a definite temperature depending on the metal. The metal is then formed into a finished or semi-finished product by one of the following methods: 1. Casting, 2. Rolling (hot or cold), 3. Forging, 4. Extruding, or 5. Drawing.

CASTING is a common method used to produce objects with intricate shapes. The metal is cast into a mold that may be stationary or spinning (centrifugal).

ROLLING is used to form the molded rough stock into more usable shapes while improving the physical properties of the metal. Large powerful rollers in a rolling mill are used to produce rails, T beams, I beams, angles, bar stock, etc. Numerous operations may be required to form some of the more complicated shapes.

FORGING may be either drop or press and is used to obtain shapes stronger than castings. Generally these are shapes which are not easily rolled. Forge hammers and/or forming dies are used to pound the metal into the desired shape.

EXTRUDING metal is a process by which metal normally in its plastic state is pushed with great force through dies which are cut in the shape of the desired cross section.

DRAWING is a process of pulling metal through dies to form wires, tubing, and moldings.

In addition to the above methods, many intricate shapes are now being produced by the POWDERED METALS PROCESS. The metal to be used is reduced to a powdered texture and is forced under pressure into a heated steel mold. The properties of the finished part approximate those of the original solid metal. Most of these parts may be used without any additional machining or grinding as they are produced to very close tolerances.

Metal may also be reduced to actual fibers. The fibers are then laid down or woven to form mats. Mats of fibrous metal are finding increasing use in resistance welding where the mats are placed between parts to be welded.

3.2 The Micro-Structure of Iron and Steel

Iron and steel possess a crystalline structure when in the solid state. This simply means that they possess a regular geometric arrangement of atoms. The symmetry of crystalline structures may be explained in terms of the unit cell. The unit cell is a common arrangement of atoms which when repeated successively in three dimensions will form the crystal structure. The most common unit cell for most metals is cubic and may be either body-centered or face centered. The **Body-Centered Cube** is constructed with one atom at each corner and one located at the center of the cube. The **Face-Centered Cube** contains an atom at each corner and one at the center of each face. Iron is unique among the common structural metals because it changes from a face-centered cubic to a body-centered cubic during slow cooling.

If more than one element is present, the unit cells may change slightly or considerably depending on the identity of the atoms present. The crystal lattice, or atomic pattern, will have to accommodate the new atoms either substitutionally or interstitially. In a substitutional solid solution the atoms of the additional element have simply replaced the atoms of the parent metal in the crystal lattice. If the atoms of the additional element are quite small, however, they will tend to locate themselves at in-between points, or interstices, in the lattice. This latter arrangement is referred to as an interstitial solid solution. Nickel will form a substitutional solid solution in iron while elements such as carbon, hydrogen, and nitrogen form interstitial solid solutions in iron. Interstitial solutions are usually limited to small but important values of solubility.

3.2.1 Formation of the Solid Phase

The initial solidification from liquid form will occur at cooler regions of the liquid (container walls) and at locations where accidental groupings of atoms favour solidification. The first tiny crystals to form are called nuclei and they act as seeds for crystal growth. In a cubic lattice the growth proceeds preferentially in directions perpendicular to the cube faces. This type of growth leads to the formation of tree like skeletons (called dendrites) with branches and subbranches at right angles to each other. Each growth forms what is termed a dendritic crystal, or grain. These dendritic crystals develop independently and randomly and as such exhibit independent and random orientation. Therefore grain boundaries ensue as the individual crystals grow into mutual contact. The completely solid metal then is composed of individual crystals, or grains, held together by atomic attractive forces at the interface boundaries.

To put things into perspective, a piece of metal the size of a sugar cube may contain thousands of these grains. In addition, the size of the grains may have a considerable influence on the steel's physical properties and behavior during heat treatment.

Ingot Grain Growth and **Weld Metal Grain Growth** clearly show expected grain orientation. The growth of nuclei into large crystals will tend to follow a direction opposite to the temperature gradient. In the ingot, the nuclei form first on the vessel walls and then grow inward with little opportunity to spread sideways because of adjacent crystals. The surface of the weld metal is analogous to the vessel wall. In both cases, the crystals form as elongated grains, perpendicular to the walls or weld surface resulting in a columnar structure. A crystal structure of this nature may result in planes of weakness parallel to the grain growth which may tear or crack during cooling or during subsequent forming operations.

3.2.2 Iron, Carbon, and Steel

Commercially pure iron is moderately weak and plastic and is not suitable for use as a construction material where strength is required. However, the addition of less than 1% carbon turns the iron into one of the most valuable engineering materials in existence today, steel. Carbon steel is the combination of iron and controlled amounts of carbon. In addition, New and desirable properties may be achieved if carbon steel is then combined with controlled amounts of other metals such as chromium, manganese, molybdenum, nickel, tungsten, vanadium, etc.. These are referred to as **Alloy Steels**.

Table_1 shows the percent of carbon present in the iron and steel classes.

Table_1: Carbon Percentages in Iron and Steel	
Metal Class	Percent Carbon
Wrought Iron	0.003
Low Carbon Steel	0.01 - 0.30
Medium Carbon Steel	0.30 - 0.55
High carbon Steel	0.55 - 0.80
Very High Carbon Steel	0.80 - 1.80
Cast Iron	1.80 - 4.30

@1[3.2.2.1 The Crystal Structure of Iron] is interesting because it changes depending on the temperature.

3.2.2.1 The Crystal Structure of Iron

Iron possesses a rather unusual ability in that its crystal structure changes when it is heated or cooled. At room temperature, iron has a body centered cubic (BCC) crystal lattice, known to metallurgists as alpha iron. If the iron is slowly heated it will gradually expand. Once 910 degrees Celsius is reached, the structure will transform into a face centered cubic (FCC) crystal lattice, called gamma iron, with a simultaneous contraction of about 1% by volume. Upon continued heating, gamma iron persists until 1400 degrees Celsius is reached. At this temperature, the FCC crystal lattice transforms back to a BCC crystal lattice called delta iron. Delta iron melts at 1540 degrees Celsius to give liquid iron.

3.2.2.2 The Phase Diagram

The physical properties of iron and steel are profoundly affected by the internal structure of the material. Temperature, composition, and prior mechanical and/or thermal treatment are the major factors affecting the material's atomic structure. An understanding of the crystal structure, and therefore many key properties of the metal, may be gained through the use of phase diagrams. Basic information concerning the equilibrium temperature and the composition limits of phase fields and phase reactions in an alloy system are best presented in the phase diagram.

It may be recalled that a pure metal always freezes at a single temperature whereas most alloys freeze over a range of temperatures. Most of the information on the phase diagram must be determined experimentally, the freezing and melting temperatures being one of the easier tasks. The simplistic Copper and Nickel Phase Diagram, @1[metal1.pcx], indicates how the diagram would look for solid solution, binary alloys. A solid solution alloy will show only one type of crystal structure in the solid state no matter what the mixing proportion. The three phases (liquid, liquid plus solid, and solid) are separated by the liquidus and solidus lines. The liquidus line indicates where freezing begins and the solidus line indicates where freezing is complete. Note that if the changes in the alloy are reversible, as they are with most phase diagrams, then equilibrium conditions are being maintained. As a result, phase diagrams are commonly called equilibrium diagrams.

The Copper-Silver Phase Diagram, @1[metal2.pcx], is indicative of an alloy in which the component metals have partial or limited solid solubility. In this case, the metals, when combined in certain proportions, will form alloys in which there are two separate phases in the solid state. As can be seen on the copper-silver phase diagram, there exists one alloy that freezes at a single temperature which is lower than all the other alloys in the series. Although for many of the other alloys freezing is complete at this temperature, it begins at a considerably higher temperature. The lowest melting alloy in the series is called a eutectic, and a diagram containing such an alloy is called a eutectic diagram. A look at the copper-silver phase diagram shows that the eutectic type of structure will only form in the alloys containing between 9 and 92 percent copper. Alloys with less than 9% copper will solidify as single phase alloys containing crystals of a solid solution of copper in silver, called alpha phase. Alloys with more than 92% copper also contain a single phase, a solid solution of silver in copper, called beta phase. It may be observed that alloys with compositions permitting passage through the alpha or beta phase will freeze in a manner analogous to a solid solution system.

The Silver Rich Portion of the Copper-Silver Phase Diagram, @1[metal3.pcx] may be used to indicate the changes in a eutectic alloy during cooling. Point A1 of the liquidus line on the diagram indicates where the 15% copper alloy would begin forming its primary crystals. The exact composition of those crystals is indicated by point B1 on the solidus line. As cooling continues, the composition of each new crystal is determined by the solidus line with the liquid composition determined by the liquidus line. A look a points A2 and B2 indicates how the progression is moving through the liquid plus solid phase. By the time the cooling has proceeded to the eutectic freezing temperature, the crystals will have the composition shown by point B3 with the liquid now at the eutectic alloy composition. The remainder of the freezing must now take place through simultaneous solidification of alpha and beta solid solutions. This simultaneous freezing will form a structure composed of either thin alternating plates of the two phases or a "salt and pepper" type of intermingling. The final solid alloy of 15% copper will contain the larger primary crystals of alpha formed first at higher temperatures, together with a eutectic mixture of the alpha and beta.

Although other alloys may involve more complicated arrangements between the components, almost all phase diagrams may be broken up into the two basic types described above.

3.2.2.3 The Iron-Carbon Phase Diagram

The Iron-Carbon Equilibrium Diagram, @1[metal4.pcx], is important for producing, forming, welding, and heat treating steel and cast iron. From this diagram, the phases forming in alloys of iron and carbon may be determined through known values of carbon content and sustained temperature. Note that the diagram is cut off at approximately 5% carbon because the rest is of little interest. Weldable steels typically have carbon contents less than 0.5%.

Starting with the molten alloy (with composition between 0 and 4.3 percent carbon), the first phase encountered upon cooling is the liquid plus austenite region. The complicated region involving delta iron may be ignored. Austenite is a solid solution of gamma iron containing up to 2.11% dissolved carbon. Like pure gamma iron, austenite too possesses the face centered cubic crystal structure. Austenite is soft and ductile and is a very important region of the diagram when heat treating steel. If the molten alloy contained between 4.3 and 6.67 percent carbon then the first phase encountered would be the cementite plus liquid phase, only a portion of which is shown on the diagram. Cementite is also known as iron carbide and its molecular formula is Fe_3C. Cementite is hard and brittle and is present in all steel and cast iron at room temperature. Although it is not well shown in the diagram, austenite forms a eutectic with cementite. The pure eutectic alloy contains 4.3 % carbon and melts at 1130 degrees Celsius, the lowest melting point of any alloy in the system. Note that any phase changes taking place below the eutectic temperature (1130 degrees Celsius) will necessarily involve changes in solid metal.

Consider now the portion of the diagram representing the iron/carbon alloys which fall under the classification of steels. Recall that plain carbon steels contain between 0.01 and 1.80 percent carbon. It can be seen that the lines below the austenite region resemble those of a eutectic system. However, because only solid phases are present, the area is designated eutectoid. In other words, a solid solution of carbon in alpha iron, or ferrite, forms a eutectoid with iron carbide, or cementite. The pure eutectoid alloy, called pearlite, contains 0.77% carbon and melts at 727 degrees Celsius.

The transformation of steels with less than 0.77 % carbon results in primary or proeutectoid ferrite (ferrite formed above the eutectoid temperature) and pearlite (eutectoid). 0.77 % carbon steels transform wholly to pearlite. Steels with greater than 0.77 % carbon form proeutectoid cementite and pearlite. The relative amounts of proeutectoid and eutectoid crystals that form from the austenite depend on the carbon content of the steel.

The manner in which proeutectoid crystals of ferrite or cementite separate from the austenite is illustrated in @1[metal9.pcx]. Part a.) of the figure shows austenite of a 0.4 % carbon steel possessing a granular structure. As the steel cools below the upper transformation temperature, ferrite will precipitate at the grain boundaries of the austenite and gradually increase in volume. Refer to part b.) of the figure. The ferrite will have completely surrounded islands of austenite by the time the lower transformation temperature has been reached, part c.) of the figure. Now, at this temperature and at the eutectoid level of carbon content, the remaining austenite will transform to pearlite resulting in the final structure shown in part d.) of the figure. Of course, all of the above phase changes require equilibrium cooling conditions.

Relatively slow - equilibrium cooling conditions:

Under equilibrium cooling conditions in the composite range of interest (less than 0.8 % Carbon), austenite transforms to ferrite and pearlite. Ferrite possesses gamma iron's body centered cubic structure and will dissolve up to 0.025 percent carbon. It is soft, ductile, and present in all steel and cast iron. Pearlite is composed of alternating platelet layers of ferrite and cementite. Pearlite possesses properties in between those of ferrite and cementite. In carbon steels under equilibrium conditions; austenite, ferrite, and cementite are the only three phases that may be present in the solid state.

The conversion from austenite to pearlite is relatively slow because the crystal structure has to change (fcc/bcc) and the surplus carbon needs time to diffuse to the cementite plates.

Relatively fast - nonequilibrium cooling conditions:

If insufficient time is allowed for the austenite/pearlite transformation then, depending on the chemical composition and cooling rate, other structures such as fine-pearlite, martensite, or bainite may be formed. The prevention of normal transformations in the steel is essential to developing its ability to harden.

Fine pearlite is a much harder phase than the regular slow cooling pearlite. In addition, the plate widths are significantly smaller.

If the cooling rate is greatly accelerated then the transformation from austenite to pearlite will be suppressed. The surplus carbon simply cannot diffuse out in time. Instead, it becomes trapped in a supersaturated solution which takes up a body-centered-tetragonal structure, martensite. The martensite phase is hard, brittle, and highly stressed internally. Martensite formation begins at the "Ms" temperature which is usually around 350 degrees Celsius. The "Mf" temperature marks the end of the austenite to martensite transformation, below which no further transformation of austenite takes place.

The "critical cooling velocity" defines the cooling rate necessary to suppress the austenite/pearlite transformation. This critical rate is reduced with the presence of alloying elements.

Bainite is formed if the critical cooling rate or better is utilized down to the "Ms" temperature but below that the material is no longer quenched. The properties of bainite are in between those of martensite and pearlite.

@1[metal5.pcx] summarizes the crystal structure transformations.

3.2.3 Time-Temperature-Transformation (TTT) Curves

It should be clear by now that final phase formation depends primarily on time and temperature. The time required for transformation to the various phases at constant temperature has been studied experimentally for quite some time. One of the more useful outcomes of this research has been the development of the isothermal transformation diagram. These diagrams display time-temperature-transformation curves that may be used to predetermine likely transformations. @1[metal6.pcx] illustrates the curves. The following conclusions can be drawn regarding the curves:

1. The nose of the curve defines the shortest time for austenite transformation. Rapid cooling rates will bypass the nose, resulting in a lower temperature transformation. This action generally leads to a harder final phase.
2. The final structure, of course, depends on the transformation temperature.
3. Carbon and alloy contents influence the shape of the curve as well as its position on the time axis. Carbon, manganese, and nickel will shift the curve bodily to the right without significantly changing its shape. On the other hand, chromium and molybdenum will alter the shape by forcing the austenite/pearlite transformation farther to the right than the austenite/bainite transformation.
4. If it were possible to displace the curve bodily to the right than the martensite phase would form even during slow cooling.
5. The temperature required for the complete conversion to martensite (Mf) might be below ambient temperature.

One point to be noted about isothermal diagrams is that they cannot be used directly to obtain quantitative information about phase changes in continuous cooling; nor can the data be considered exact when applied to a different heat of the same grade of steel. The diagrams are, however, beneficial in showing the products of austenite decomposition at different temperature levels and the effects of various factors on them. In addition, they give a clue to the response of a steel to the cooling rates following welding.

3.2.4 Continuous Cooling Transformation (CCT) Diagrams

The constant temperature representation of the TTT curves is somewhat unrepresentative of welding where rapid continuous cooling takes place. More relevant information may be obtained from a CCT diagram, @1[metal7.pcx], in which phase changes are tracked for a variety of cooling rates. Production of these diagrams for weld heating and cooling requires considerable experimental effort. For construction of the diagram shown, special apparatus was used to simulate HAZ cooling, and the transformation points were derived from both dimensional and temperature changes in the specimen. There exists few diagrams of this type.

3.2.5 HAZ Microstructures in Steel Welds

Grain growth will thrive when a steel is held at the upper austenite range for a significant length of time. This growth proceeds through grain boundary migration and coalescence. In general, coarse grained steels possess poor fracture toughness. During welding coarse grain zones are produced, the size of which depends on the welding process and parameters. Notch toughness may be improved by using normalizing procedures to reduce the grain size. This procedure involves uniformly heating the complete fabrication to a point just above the T1 temperature, see @1[metal8.pcx], and then allowing it to cool naturally in air. At this time, the practical difference between normalizing and stress-relieving will be noted. The required furnace temperature for stress-relieving is much higher than that for normalizing, 950 as compared to 650 degrees Celsius, respectively. At this higher temperature, the structure may not possess the strength required to support its own weight.

For material heated in the lower austenite range, T1 to T2 in @1[metal18.pcx], a much finer grained microstructure will be produced. The upper temperature limit T2 is controlled largely by grain refining agents such as aluminum, niobium, vanadium, and nitrogen. These elements form carbides and nitrides which prevent grain boundary migration. Unfortunately, as the temperature is increased they lose their effectiveness because they go into solution.

The remaining portion of the HAZ, seen in the figure between the T0 and T1 temperatures, permits pearlite from the original parent metal to transform to austenite. It is possible to form martensite in this region if the material is high in carbon and other alloying elements, and if subsequent cooling is rapid enough.

The bottom of the figure shows the microstructure which varies continuously across the HAZ. Unaffected parent metal merges into fine and then coarse grain phases to the fusion line which defines the boundaries of the cast weld structure.

In multipass welds, each weld bead is partially reheated and tempered by subsequent passes. This treatment greatly improves toughness, as carbon diffuses out of any martensite previously formed, leading to a tough microstructure consisting of ferrite and spheroidised carbides. It becomes clear now that the welding engineer must make a choice between multipass welds which give excellent metallurgical properties and single pass welds which give inferior properties but are more economical.

3.3 Welding Metallurgy in Structural Steels

Modern structural steels are comprised mainly of Carbon-manganese (C-Mn) or low alloy steels. C-Mn steels with up to 0.5 percent carbon and 1.7 percent manganese are extensively used to build structures and machinery. These steels will additionally contain up to 0.6% Silicon and occasionally a small amount of aluminum for grain size refinement. Although chrome, nickel, molybdenum, copper and columbium can be present as residual elements, no other deliberate alloying elements are added.

Low alloy steels incorporate deliberate additions of one or more elements to modify their strength and/or toughness. These steels contain less than 5 percent by weight of total alloying additions, excluding manganese and silicon. The alloying additions are generally comprised of chromium, nickel, molybdenum, and vanadium. Microalloyed steels are relatively new and contain less than 1 percent by weight of alloying elements, again excluding manganese and silicon. The alloy additions here are some combination of molybdenum, columbium, vanadium and aluminum.

The microstructure of a simple C-Mn steel, cooled slowly from high temperatures can be described with the Iron-Carbon equilibrium diagram, @1[metal4.pcx]. Consider a typical medium carbon steel which has a composition less than the eutectoid composition (0.77 %). Under slow cooling, where the diffusion of carbon is relatively uninhibited, certain portions of austenite (stable at high temperature) will decompose into ferrite (which has a very low solubility for carbon,) while remaining portions become carbon enriched. The carbon level in this remaining austenite will approach the eutectoid composition, through continued slow cooling, where it will then be transformed into pearlite below 723 degrees Celsius. @1[metal9.pcx] illustrates the structural changes from austenite at 870 degrees Celsius to pearlite and ferrite at room temperature. If the cooling rate is increased, there is less time for diffusion to occur and the result is a finer pearlite with improved strength and toughness. Since the transformation starts at austenite grain boundaries, a finer austenite grain size will also improve strength and toughness, primarily by providing a better relative distribution of the pearlite.

Still further increases in cooling rate will minimize the time for carbon diffusion and subsequently lower the nucleation temperature. Once the nucleation temperature drops below 550 degrees Celsius, bainite will form at the nucleation sites. Bainite may be further classified into upper and lower bainite with the latter forming at lower nucleation temperatures. Lower bainite contains very fine carbide particles in a matrix of ferrite with more carbon than the equilibrium limit. This residual carbon distorts the body centered cubic ferrite structure, which has less room for interstitial atoms than the more open faced austenite cubic. At very high cooling rates, carbon diffusion is nearly totally suppressed, the transformation temperature drops below 250 degrees C, and a shear transformation to martensite occurs. In this structure, the austenite must transform to a ferritic structure, but still retain the carbon in solution. The interstitial carbon greatly distorts the lattice, producing a hard, brittle material. The hardness (and brittleness) rise directly with the carbon level in the steel, so that higher carbon levels produce more distortion, higher hardness and lower toughness.

The carbon content is the key ingredient for determining the maximum achievable hardness associated with a severe quench (rapid cooling rate). Whether or not the steel achieves that maximum hardness depends on the cooling rate and the alloy content. Most alloying elements tend to diminish the rate of carbon diffusion thereby promoting lower temperature transformation products and increased hardenability. Therefore maximum hardness in a section should be relatively easy to achieve. The Jominy end-quench hardness test is often used to assess this behavior. A jet of cold water is sprayed on the end of a hot bar of steel, and the hardness measured along the length. Maximum hardness, based on carbon content, is achieved on the end of the bar adjacent to the cooling water where the quenching effect is most severe. The steel further along the bar experiences a slower cooling rate and therefore increased carbon diffusion. This results in a softer microstructure. The addition of certain alloying elements can delay this behavior and increase the hardenability through an extension of maximum hardness along the length of the bar. A correlation of Jominy data with an Isothermal Transformation diagram will show which transformation products will occur for a steel cooled rapidly from about 900 degrees C to a specified lower temperature, which is then held until a transformation occurs. Unfortunately, it does not happen this way in practise, especially in welding, where cooling takes place starting from close to the melting point and where cooling is continuous, not isothermal. Instead, in these conditions, continuous cooling transformation diagrams may used to determine transformation behavior, @1[metal12.pcx].

If we can determine the transformation behavior, then welding problems can be approached with a higher degree of confidence. Two important regions of the problem are the weld metal and the HAZ. Solidification cracking is one common problem associated with the weld metal. It occurs because low-melting point constituents, primarily associated with sulphur, segregate. With high restraint comes an increase in strain as the metal solidifies. The last material to freeze may be called upon to fill a larger void than its volume permits, the result, of course, is a hot crack. High welding speeds and small weld beads tend to produce tear drop shaped welds, see part a.) of @1[metal13.pcx]. This shape tends to concentrate segregates at the centerline and promote cracking. A superior pool shape is illustrated in part b.) of the same figure. It tends to direct segregates in to the molten weld pool (at the ends of the dendrites) thus delaying or preventing centerline cracks. Manganese may be used to tie up sulphur in a sulphide with less inclination to cause cracking than iron sulphides. Because sulphur segregation is increased by higher carbon levels, the Manganese to Sulphur ratio must also increase, @1[metal14.pcx]. Centerline cracking is a larger concern with deep penetration narrow welds, see @1[metal15]. Multipass welding may be used to control this problem.

The principal affliction in the heat affected zone (HAZ) is cold cracking. @1[metal16.pcx] illustrates its many forms. Although the weld metal is also susceptible, it is to a lesser extent. The simultaneous occurrence of four events is necessary to cause cold cracking:

1) Tensile stresses.
2) Low temperature (below 200 degrees C)
3) Hydrogen levels above 5-10ml/100gm metal.
4) A hardened (martensitic or bainitic) microstructure susceptible to hydrogen cracking.

We essentially have no control over the first two events and so we will confine our attention to the latter criteria. It appears that diffusible hydrogen atoms induce cracking by generating low stress fractures in distorted ferrite structures by acting as interstitial atoms on particular crystal planes. Of course, the best way to control this cracking is to avoid the existence of hydrogen in susceptible areas. @1[metal17] illustrates that this is possible with the use of clean, dry consumables in some arc welding processes, but not all. Foresight is necessary in the selection of welding consumables and processes so that hydrogen levels may be controlled. Selecting low hydrogen electrodes and ensuring they are known to be clean and dry before use is a necessary precaution in susceptible areas. The use of preheat to minimize moisture levels and promote hydrogen diffusion out of the weld zone is another suitable, and often necessary precaution.

If low hydrogen procedures are not thought to be completely reliable or if the material is highly susceptible, then manipulation of microstructure and hardness may be required to control hydrogen cracking. Three types of control exist, Figure 11. In general, for carbon contents less than 0.11%, only minimal precautions are necessary. This is because the lattice distortion and hardness are relatively low even when alloy levels (and hardenability) are very high. These steels are located in Zone I of @1[metal18]. Hardenability is described by the carbon equivalent formula on the X axis. This is because the carbon equivalent formula empirically relates the alloy level to an equivalent amount of carbon which would give an identical hardenability and maximum hardness is determined by actual carbon levels. Low hardenability steels in Zone II may be hardness controlled by welding procedure variables such as preheat temperature and heat input (voltage times current divided by welding speed). The preheat and heat input are established such that the cooling rate in the HAZ results in a maximum hardness that is representative of a microstructure which is not susceptible to hydrogen cracking. Often a maximum hardness of 350Hv is taken. Although hardnesses of up to 450Hv are acceptable, provided hydrogen levels can be controlled. Zone III of the figure represents steels with higher carbon and alloy levels which are highly hardenable and cannot be limited in hardness via preheat and procedure control alone. These steels need to post-weld heat treated (tempered) in addition to the controls utilized on zone II steels (hydrogen level, preheat, and heat input) if cracking is to be avoided.

@1[metal20.pcx] illustrates the variation of thermal cycle experienced by the weld zone from process to process. Short, rapid thermal cycles experienced at low level heat input will promote hardened structures that tend to retain hydrogen. Extended thermal cycles, on the other hand, will minimize hardening and promote the diffusion of hydrogen away from the weld zone. Unfortunately, they also promote very large grain sizes and often produce low fracture toughness and high ductile to brittle transition temperatures. @1[metal21.pcx] illustrates how variations in hydrogen levels, chemical composition and micro-structures will produce highly varying toughness in structural steels.

4 Welding Basics

A @1[4.1 History of Welding] module is provided for general interest purposes.

The @1[4.2 Basic Welding Processes] are described in brief detail to permit an understanding of the joining of steels.

@1[4.3 Basic Welding Types] are primarily those used in arc welded construction.

@1[4.4 Terminology in Welded Joints] illustrates the finer joint details used in fabrication.

@1[4.5 Welding Positions] illustrate standard methods of depositing weld metal in the joint.

@1[Welding Standards] are provided for reference.

4.1 History of Welding

Joining of metal pieces by heating to a plastic or fluid state, with or without pressure, is known as welding. It has been known for as long as metals have been known - several thousand years. Table_2 gives an overview of the history of welding.

Table_2: The History of Welding		
Date	Person	State of Art
5500 B.C.	Egyptians	pressure welding of copper pipes by hammering
3000 B.C.	Egyptians	gold foil welded to base copper by hammering, **forge welding**
		Damascus sword consisting of several layers of iron with different properties
		Romans named welding god (the god of fire and metalworking) VULCAN
1885	E. Thompson	patent for **resistance welding**
1885	Zerner	**carbon welding process**
1877	Elihu Thompson	reversal of polarity of transformer coils

1887	American Institute Fair	butt welding machine
1887	Thomas Fletcher	use of blowpipe burning hydrogen and oxygen to cut or melt metal
1888	N.G. Slavinoff	use of **metal arc welding process** using uncoated, bare electrodes
1889	A.P. Strohmeyer	use of **metal arc welding process with coated metal electrodes**
1889	Coffin	patent for **flash-butt welding**
1892	Coffin	U.S. patent for metal arc welding process
1901-03	Fouche & Picard	use of acetylene torches to cut or melt metal **(oxyacetylene welding)**
1920		use of copper-tungsten alloy electrodes for **spot welding**

4.2 Basic Weld Processes

The welding process is defined by the Welding Handbook as "**a materials joining process which produces coalescence of materials by heating them to suitable temperatures, with or without the application of pressure alone, and with or without the use of filler metal.**" The sources of heat can be varying: chemical, electrical, mechanical, and optical.

All modern processes use heat to melt the base metal, and the filler material, if filler is present. Welding produces a simpler, more compact connection than bolting, and is more economical for shop fabrication. During the welding process two pieces of steel are fused together by melting them at the joint. After solidifying, the welds will transfer shear, tensile and compressive forces. They are used occasionally to stitch components together or to seal edges of surfaces against moisture. During shop fabrication small tack welds may be applied to hold pieces in place prior to final bolting or welding.

We distinguish mainly between four types of welding processes for structural steel:

1. **Electric Arc Welding** - fusion process with heat from electric arc.
2. **Resistance Welding** - pressure process with heat from flow of current.
3. **Oxyacetylene Gas Welding** - fusion process with heat from acetylene burning in the presence of oxygen.
4. **Electroslag Welding** - automatic machine welding process for primarily vertical position welding.

Other types of welding (induction-type welding, thermit welding, welding with optical or mechanical energy, and solid state bonding) are not commonly used in structural steel fabrication.

4.2.1 Electric Arc Welding

Electric arc welding is the most important welding process, both in the shop and in the field. It is performed manually (hand or stick welding) with a coated metal electrode (stick), or automatically with a continuous bare wire electrode. @1[welbas1.pcx] illustrates the arc welded circuit.

The **manual process** is referred to as **shielded metal arc welding (SMAW).** @1[welbas2.pcx] illustrates this process. An electric arc is produced between the end of the electrode and the steel components to be welded. The arc heats the metal parts until they melt. As the arc is moved along the line to be welded, the molten material solidifies and bonds the components together. The electrode coating contains flux which purifies the molten metal and produces a shielding gas to protect the molten metal from oxidization. Sometimes the shielding contains other material, such as deoxidizers, to refine the grain structure of the weld material. The coating remains on the weld surface in the form of a blanket, to protect the molten pool and the solidified weld from oxygen and nitrogen in the air, and to retard cooling. This is the so called slag, which must be removed before painting, by chipping or scraping.

This is the oldest, simplest, and perhaps most versatile type for welding structural steel. Often one refers to SMAW as **manual stick electrode process.** The stick coating is a clay-like mixture of silicate binders and powdered material, such as fluorides, carbonates, oxides, metal alloys, and cellulose. The mixture is extruded and baked to produce a dry, hard, concentric coating.

An **automatic process**, known as **submerged arc welding (SAW)**, is generally used in the fabricating shop. @1[welbas3.pcx] illustrates this process. It is similar to the shielded arc welding. The arc is not visible because it is covered by a blanket of granular, fusible flux while the bare electrode is continuously fed by a machine and consumed as the filler material. Welds of consistently high quality and deeper penetration can be produced at a faster rate than would be possible manually. Sometimes machines are set up to apply welds at several locations between the components at the same time in order to avoid distortion owing to shrinkage during the cooling process. Both manual and automatic processes can be modified by shielding the weld with a special gas, usually carbon dioxide. Depending on the type of gas and electrode polarity, a wide range of materials can be welded and many different weld penetrations are possible.

The flux covers the arc and prevents spatter, sparks, or smoke. It also protects the weld pool from the atmosphere. The flux cleans and modifies the chemical composition of the weld metal. Welds produced by this process usually have good ductility, high impact strength, small grain size, and good corrosion resistance. The weld tensile strength is at least as good as that of the base metal.

Gas metal arc welding process:

A more expensive and sophisticated method is the **gas metal arc welding process (GMAW)**. @1[welbas4.pcx] illustrates this welding process. Here a continuous wire is supplied from a coil through an electrode holder, the gun. The shielding is supplied as a gas, mostly carbon dioxide (CO_2). It was developed as a method needing inert gases only, and got the name MIG (metal inert gas). However, due to the high costs, mixtures of gases have become common. The mixtures consist of argon, helium, and carbon dioxide, with ratios depending on the steel quality. Low alloy steels can be welded with 3/4 argon and 1/4 carbon dioxide. When toughness of the weld is required, a high percentage of helium (about 65 %), less argon (about 30%), and little carbon dioxide (5%) is required.

The shielding gas has great control over the arc size, the metal transfer, the penetration, the width of fusion, and the shape of the weld region. Furthermore it affects the speed of welding and determines the undercutting. An experienced welder and/or exact weld recommendations are extremely important in order to achieve high quality GMAW welds.

The affect of adding an inert gas to the inexpensive carbon dioxide is that the arc is made more stable and the spatter during the metal transfer can be reduced. The higher the carbon dioxide content, the lower the cost for the shielding gas, the higher the welding speed, and the better the joint penetration.

Flux cored arc welding process:

A similar process is the **flux cored arc welding process (FCAW).** The continuously supplied wire is here a thin tube, containing the flux material within its core. An "inverted" stick electrode process is used here because, with the automatic supply of the wire, if the coating were on the outside of the electrode it would peel off. Therefore, it is placed inside the tube. In certain cases, additional CO_2 is provided as shielding gas because it does not add considerably to the welding costs, but does have the previously stated beneficial affects.

4.2.2 Resistance Welding

In resistance welding the heat is produced by resistance of the parts to an electric current. When the components reach their melting temperature at the contact surfaces they fuse, and bond on cooling. This type of welding is usually used in the fabrication of lighter steel components, such as open-web steel joists, or for the application of steel cladding (spot welds).

4.2.3 Oxyacetylene Gas Welding

Gas welding is popular for minor repair work of difficult-to-reach structural components, because it requires only relatively light equipment and no electric power source, unlike the methods described above.

Generally, oxyacetylene torches are used to cut holes and to cut edges. When they are carefully guided or automatically controlled they cut clean contours along straight or curved lines by following templates or numerically input geometries. Great skill is needed to produce satisfactory results, i.e. to avoid irregular surfaces, when the cutting is done by hand.

4.2.4 Electroslag Welding

Two wide, thick plates are best joined by an electroslag welding process (ESW). The weld is made vertically from bottom to top in the weld cavity, to form a butt joint. Water-cooled copper slides confine the molten slag and weld metal on each side of the joint as the weld is formed. The heat is supplied by electric arcs and the weld electrodes are automatically fed in as the weld progresses. The Arc can be extinguished once the process is under way so that the welding is done by the heat produced through the resistance of the slag to the flow of current. Since resistance heating is used for all but the initial heat source, the ESW process can't be counted as an arc welding process.

This type of welding is preferred for applications requiring minimum distortion of the joined members.

4.2.5 Stud Welding

The most commonly process of welding a metal stud to a base material is known as arc stud welding, a fully automatic process which is similar to the SMAW process. The stud works as the electrode and an electric arc is stretched from the end of the stud to the plate. A ceramic ferrule around the end of the stud in the gun shields the arc. The setting gun is positioned and after a short time of firing the arc, the gun pushes the stud into the molten pool. When the weld is completed a small fillet has formed around the shank of the stud. The total time for setting one stud can be less than a second.

4.3 Basic Welding Types

Primary attention is given here to the types of welds used predominantly in arc welded construction where the heat comes from an electric arc and no pressure is used.

There are three basic types of welds used to achieve an acceptable joint. These weld types are termed GROOVE, FILLET and PLUG (or SLOT). Welds are composed of one or more beads. A bead is a single run or pass of weld metal. The diagram, @1[welbas5.pcx], shows the basic weld types used in various applications. In preparation for the plug and slot welds, holes or slots have been made in the upper plate. On relatively thinner material such welds can be made without preparation and are called arc spot and arc seam welds.

4.3.1 Groove Welds

Welds are distinguished by their cross section. Approximately 20% of all structural welds are **groove welds.** They are used when ends, edges and/or surfaces of two parts must be joined together. The components to be joined are usually prepared by cutting or machining them to provide square, vee, bevel, U-, or J-shaped grooves which are filled in by the weld material. We further distinguish between complete and partial penetration welds, depending on the extent of fusion throughout the depth of the joint.

As the name implies, the joints for such welds are in some manner prepared to form a groove or crucible to receive the weld bead or beads. There are seven types defined in terms of shape: square groove, V-groove, bevel groove, U-groove, J-groove, flare bevel and flare Vee.

@1[welbas6.pcx] shows various types of groove welds as applied to butt joints. Some or all of these welds are applicable also to corner joints, Tee joints, lap joints and edge joints. Also shown are the flare bevel and flare Vee groove welds both of which tend to involve more specialized applications. Both single groove and double groove joints can be prepared. Double grooves are used for thick material.

Groove welds are classified in terms of two types of efficiency. One is a complete joint penetration groove weld (CJPG) and the other is a partial joint penetration groove weld (PJPG). CJPG welds are considered to be the most efficient type of weld. For matching "electrode - base metal" conditions they readily develop the full capacity of the material.

4.3.2 Complete Joint Penetration Grooves

A complete joint penetration groove weld is defined as one welded 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. If welded from one side, steel backing must be used and full fusion into it obtained. If welded from both sides, backgouging to sound metal is required before welding to second side. See @1[welbas7.pcx].

The complete joint penetration groove weld status can also be claimed for joints welded from one side without backing, or with backing other than steel, as well as those welded from both sides without backgouging if successfully qualified in a procedure test. The structural codes provide for a great number of prequalified joint (geometries) details for CJPG welds for designated welding processes.

In recognizing the special importance of these types of grooves and the related significance of full development of the total cross-sectional area of the joint, the use of extension bars or run-off plates is stipulated, when satisfactory termination at the ends by other means cannot be obtained. Refer to @1[welbas8.pcx].

4.3.3 Partial Joint Penetration Grooves

A partial joint penetration groove weld is defined primarily as one having joint penetration less than complete. However, prequalified partial joint penetration grooves shall also include joints:

(a) Welded from one side without steel backing;

(b) Welded from both sides without back-gouging with very few exceptions related to either their thicknesses or a deep penetrating welding process.

@1[welbas9.pcx] illustrates the partial joint penetration groove weld.

It should be noted that all flare groove welds are officially designated as PJPG welds.

The allowable working strength of joints welded from one side only is restricted as compared to that allowed for joints welded from both sides, or with a suitable backing bar. Moreover, a single partial joint penetration groove weld shall not be subject to tension normal to the weld or to bending about the longitudinal axis of the weld if it produces tension at the root of the weld. See @1[welbas10.pcx]. Other restrictions applicable to PJPG welds can be found in CSA W59 - M1989.

The effective size of PJPG welds, other than flare groove types, is a function of the angle at the root of the joint and the root opening. For angles measured at the root equal to:

(1) 60 degrees or greater, the size is designated as equal to the depth of preparation.

(2) less than 60 degrees, but not less than 45 degrees, the size is designated as the depth of preparation minus 3 mm (1/8 inch), when in both cases the root opening is zero.

4.3.4 Fillet Welds

Fillet welds are the most common type of weld (nearly 80% of all welds) because they require little or no preparatory work. Their typical cross section is triangular. The leg of the triangle designates the weld size. The root of a weld is at the intersection of the legs. A line perpendicular to the weld face and passing through the root is called the weld throat, and the length of this line is the throat size.

With root openings greater than zero deeper penetration is obtainable for the grooves with smaller angles at the root.

This term is given to welds making lap, Tee joints or inside corner joints. Such welds may consist of one or more beads or layers and are approximately triangular in cross section; they join two surfaces, usually but not always, at right angles to each other in the above mentioned joints. @1[welbas11.pcx] shows the ideal fillet weld to be one where the joint faces are at 90 degrees, the weld has equal legs, the face is flat or slightly convex and the toes merge smoothly with the surfaces of the joint members.

To determine the effective cross-sectional area of a fillet weld, a triangle is inscribed into its profile. For an equal-legged, right-angled fillet, the triangle will be isosceles while an unequal-legged fillet will be any triangle. These inscribed triangles are used for the determination of effective throats and effective leg sizes. Refer to @1[welbas12.pcx].

Generally, the limitations on maximum size of fillet will be based on considerations of good welding practice or balanced design. Minimum size fillets, on the other hand, are governed primarily by strength but additionally by minimum thickness of material considerations as specified in governing standards (CSA W59).

4.3.5 Plug Weld

Plug or slot welds are usually used in lap joints to transmit shear loads or to prevent buckling. They are similar to fillet welds but their maximum shear capacity is limited by the projected slot area. @1[welbas13.pcx] illustrates this weld type.

A descriptive term for this weld is rivet weld. They are employed chiefly for lap joints but may be used for Tee joints on thick plate where a fillet weld is not adequate or is not possible due to inaccessibility. When a slot is made rather than a circular hole the weld is called a slot weld. It is a form of plug weld.

Usually a hole or slot is punched, drilled or flame cut in one plate and a weld made through it to the underlying plate joining the two by a fillet weld around the periphery of the hole or slot. The hole or slot may or may not be entirely filled in with weld metal. On relatively thin material a hole or slot may not be necessary.

It should be emphasized that fillet welds in holes or slots are not to be confused with plug and slot welds.

4.4 Terminology in Welded Joints

@1[welbas14.pcx] shows a single V-groove weld in a butt joint to illustrate the correct terms used in identifying the salient aspects of all welded joints. Appendix D of W59-M89 contains information on welding symbols.

4.5 Welding Positions

A weld is said to be made in the flat position, horizontal position, vertical position or overhead position depending on the position of the joint in relation to the floor. Welding techniques for the four positions of welding vary according to the ease of depositing metal. It is possible to deposit weld layers of considerable volume in the flat and vertical positions but stringer beads become necessary for horizontal and overhead welding. These positions are better illustrated in @1[welbas15.pcx].

4.6 Welding Standards

1. CSA Standard W59-M89, Welded Steel Construction (Metal-Arc Welding).
2. CAN3-S16.1-M89, Steel Structures for Buildings, Limit States Design.
3. CAN3-G40.20-M87, General Requirements for Rolled or Welded Structural Quality Steel.
5. CAN3-G40.21-M87, Structural Quality Steels.
6. W47.1 - 1983 Certification of Companies for Fusion Welding of Steel Structures.
7. W48.1 - M1980, Mild Steel Covered Arc Welding Electrodes.
8. W48.3 - M1982, Low-Alloy Steel Covered Arc Welding Electrodes.
9. W48.4 - M1980, Solid Mild Steel Filler Metals for Gas Shielded Arc Welding.
10. W48.5 - M1982, Mild Steel Electrodes for Flux Cored Arc Welding.
11. W48.6 - M1980, Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding.
12. W48.7 - M1977, Diffusible Hydrogen in Mild Steel and Low-Alloy Steel Weld Metals: Test Method.
13. W55.3 - 1965, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings.
14. W178 - 1973, Qualification Code for Welding Inspection Organizations.

5 Weldability of Steels

The prevention of cracks during welding or in service is first and foremost in maintaining structural integrity. Special welding procedures are not usually required for a correctly designed joint and properly made weld. However, there is a need for special procedures with heavy plate structural members and steels having great amounts of alloying elements in their chemistry.

Weldability is defined in @1[5.1 Weldability].

5.1 Weldability

Weldability describes the relative ease of producing effective, sound and crack-free joints. Most steels may be commonly arc welded to produce sound, secure welded joints. A steel is thought to be ideally weldable if the required welded joint can be produced with relative easy and minimal cost.

Although not necessary with the more common mild steels, preheating or some other measure may be required when welding thick plates or higher-strength alloys.

Steel chemistry becomes more important with the use of high-speed welding. Even though the composition of the electrode core wire is accurately controlled to produce good welds, the plate metal becomes part of the weld and control of the plate chemistry is also important. Higher currents may be used to achieve higher welding speeds, however, more of the plate metal will now mix with the weld. If possible, select an easily welded steel that does not require special electrodes or elaborate welding procedures. Table_3 gives a range of carbon steel composition for maximum welding speed. This table was produced in 1966 and, although the numbers are approximately correct, today's standards should be consulted to check for accuracy.

TABLE_3: Preferred Analysis of Carbon Steel for Good Weldability			
Element		Normal Range, %	Steel Exceeding Any one of the Following Percentages Will Probably Require Extra Care
Carbon	C	.06-.25	.35
Manganese	Mn	.35-.80	1.40
Silicon	Si	.10 max	.30
Sulphur	S	.035 max	.050
Phosphorus	P	.030 max	.040

The commonly used mild steels fall within the preferred analysis listed. Sulphur content of these steels is usually below 0.035%, although the specification limits permit as much as 0.050%.

The range of Weldability is continually being broadened with new developments in the composition of steel, electrodes, and fluxes. New and innovative welding processes are providing improved levels of compatibility.

The basic construction steels usually do not require special precautions or special procedures. However, the increased rigidity and restraint associated with the welding of thicker plates where the quench effect is quite severe, makes the use of a proper welding procedure vitally important. In addition, thick plates usually have higher carbon content.

The increase in the use of higher strength low alloy steels and heat treated very high yield strength steels, where chemical analysis indicates excessive elements with respect to the ideal composition, requires special precautions and procedures to achieve high welding speeds.

Pre-planned and proven welding procedures may be required to assure the production of crack-free welds when joining thicker plates or the alloy steels. These procedures usually call for one or all of the following:

1. Proper bead shape and joint configuration.
2. Minimized penetration to prevent dilution of the weld metal with the alloy elements in the plate.

3. Preheating, controlled interpass temperature, and sometimes even controlled heat input from the welding procedure to retard the cooling rate and reduce shrinkage stresses.

5.2 Base Procedure on Actual Composition

Published standard production welding procedures generally apply to normal welding conditions and the more common "preferred composition" mild steels.

If the specification composition of the steel being welded falls outside the preferred composition, the user may attempt to adopt a special welding procedure based on the **extremes** of the material's chemical content "allowed" by the steel's specification. However, this action may prove superfluous because the actual chemistry of a specific heat of steel may run far below the top limit of the "allowables", and a special procedure may not be required, or may require only a slight change from standard procedures.

For optimum economy and quality, under either favorable or adverse conditions, the welding procedure for joining any type of steel should be based on the steel's **actual** chemistry rather than the **maximum** alloy content allowed by the specification. This is because a mill's average production normally runs considerably under the maximum limits set by the specification.

Usually a Mill Test Report is available which gives the **specific** analysis of any given heat of steel. Once this information is obtained, a welding procedure can be set that will assure the production of crack-free welds at the lowest possible cost.

5.3 Weld Quality

The primary objective of any welding procedure is to fasten the pieces as required using the most efficient weld possible while incurring minimal cost. "As required" means the weld's size and quality must be consistent with service requirements. Extreme precautions to procure unnecessary quality, beyond that needed to meet service requirements, only serve to increase costs.

Because it greatly increases cost without any benefit, inspection should not subsequently demand the correction of nominal undercut or insignificant radiographic defects (Eg. limited scattered porosity, slag inclusions) unless thorough study shows such defects cannot be tolerated because of specific service requirements.

Weld quality is governed by W59-M89 under clause 11.5.4 Quality of Welds and clause 12.5.4 Quality of Welds for statically and dynamically loaded structures, respectively.

5.4 Weld Cracks and Their Prevention

A crack in a weld is never minor and cannot be condoned. Good design and proper welding procedure will prevent the following cracking problems:

1. weld cracks occurring during welding.
2. cracking in the heat affected zone of the base metal,
3. welded joints failing in service.

5.4.1 Factors that Affect Weld Cracking During Welding

The following factors affect weld cracking during welding:

1. Joint Restraints that cause high stresses in the weld.

2. Bead Shape of the deposited weld. As the hot weld cools, it tends to shrink. A convex bead has sufficient material in the throat to satisfy the demands of the biaxial pull. However, a concave bead may result in high tensile stresses across the weld surface from toe to toe. These stresses frequently are high enough to rupture the surface of the weld causing a longitudinal crack.

An excessively penetrated weld with its depth greater than its width under conditions of high restraint may cause internal cracks.

Both of these types of cracking are greatly aggravated by high sulphur or phosphorus content in the base plate.

3. Carbon and Alloy Content of the base metal. The higher the carbon and alloy content of the base metal, the greater the possible reduction in ductility of the weld metal through admixture. This contributes appreciably to weld cracking.

4. Hydrogen Pickup in the weld deposit from the electrode coating, moisture in the joint, and contaminants on the surface of the base metal.

5. Rapid Cooling Rate which increases the effect of items 3 and 4.

5.4.2 Factors that Affect Cracking in the Heat-Affected Zone

The following factors affect weld cracking in the heat affected zone:

1. High Carbon or Alloy Content which increases hardenability and loss of ductility in the heat-affected zone. (Underbead cracking does not occur in non-hardenable steel.)

2. Hydrogen Embrittlement of the fusion zone through migration of hydrogen liberated from the weld metal.

3. Rate of Cooling which controls items 1 and 2.

5.4.3 Factors that Affect Welded Joints Failing in Service

We generally refer to welds that break in service as a result of insufficient sizing rather than welds that crack because welds do not usually crack in service. Two other factors might be:

1. Notch toughness, which would affect the breaking of the weld or the plate when subjected to high impact loading at extremely low temperatures.

2. Fatigue cracking caused by a notch effect from poor joint geometry. This occurs under service conditions of unusually severe stress reversals.

5.4.4 Items to Control Weld Cracking

1. **Bead Shape**. Proper bead surface (i.e. slightly convex) and proper width-to-depth ratio are two important parameters to control when laying down a bead. This is not critical with single pass welds or with the root pass of a multiple pass weld.

2. **Joint Restraint**. Design weldments and structures to keep restraint problems to a minimum.

3. **Carbon and Alloy Content**. Select the correct grade and quality of steel for a given application, through familiarity with the mill analysis and the cost of welding. Suppress the percentage content of elements like sulphur and phosphorus, whose presence adversely affect weld quality.

Avoid excessive admixture by reducing penetration. Procedural changes such as using different electrodes, lowering currents, changing polarity, or improving joint design such as replacing a square edge butt weld with a bevel joint, will all effectively reduce penetration.

4. **Hydrogen Pickup**. Select low-hydrogen welding materials.

5. **Heat Input**. Total heat input can be controlled by monitoring preheat, welding heat, heating between weld passes to control interpass temperature and post heating to control cooling rate. Control of heat input lowers the shrinkage stresses and retards the cooling rate. This will help to prevent excessive hardening in the heat-affected zone.

5.5 Tack Welds

The use of tack welds is covered in clause 5.4.7 Tack Welds of W59-M89. In general, tack welds that are incorporated into the final weld shall be subject to the same quality requirements of the final weld, except that preheat is not necessary for single pass welds where it can be proven that the tack weld will be totally remelted and incorporated into the final weld. See @1[weldab1.pcx]. Tack welds not incorporated into the final weld must be removed in dynamically loaded structures and need only be removed in statically loaded structures at the request of the Engineer.

5.6 Welding Thin Plates

Weld cracking is rarely a factor when welding thinner plates and only becomes a factor when the plates possess unusually high carbon or other alloy contents. The heat of welding on the low mass thinner plate generates a relatively slow cooling rate. In addition, the reduced internal stresses resulting from a good weld throat to plate thickness ratio and the fact that the thinner plate is less rigid and can flex as the weld cools and shrinks, helps to control the factors that induce cracking.

5.7 Welding Thick Plates

In the steel mill, all steel plates and rolled sections undergo a rather slow rate of cooling after being rolled while red hot. The red hot thick sections, because of their greater mass, cool at a slower rate than the thin sections. For a given carbon and alloy content, slower cooling from the critical temperature results in slightly lower strength.

For the normal thicknesses, the mill has little difficulty in meeting minimum yield strength requirements. However, in extremely thick mill sections, because of their slower cooling rate, the carbon or alloy content might have to be increased nominally in order to meet the required yield strength.

Since a weld cools faster on a thick plate than on a thinner plate, and since the thicker plate will probably have a slightly higher carbon or alloy content, welds on thick plate (because of admixture and fast cooling) will have higher strengths but lower ductility than those made on thinner plate. Special welding procedures as well as preheating may be necessary for the joining of thick plate, especially for the first or root pass. If we can decrease the weld's rate of cooling we can increase its ductility.

In addition to improving ductility, the preheating of thick plates tends to reduce the shrinkage stresses that develop from excessive restraint. However, because of its expense, preheating should be selectively specified. For example, fillet welds joining a thin web to a thick flange plate may require less preheat than a butt weld joining two highly restrained thick plates.

If metal-to-metal contact exists prior to welding and there is no possibility of plate movement then as the welds cool and contract, all the shrinkage stress must be taken up in the weld, part (a) of @1[weldab2.pcx]. If restraint is severe then the welds may crack, especially in the first pass on either side of the plate. If, however we allow a small gap between the plates, then they can move in slightly as the weld shrinks thereby reducing the transverse stresses in the weld. See parts (b) and (c) in the figure. Heavy plates should always have a minimum of 1/32" gap between them, if possible 1/16".

This small gap can be obtained by means of:

1. Insertion of soft steel wire spacers between the plates. The soft wire will flatten out as the weld shrinks. If copper wire is used, care should be taken that it does not mix with the weld metal.
2. A deliberately rough flame-cut edge. The small peaks of the cut edge keep the plates apart, yet can squash out as the weld shrinks.
3. Upsetting the edge of the plate with a heavy center punch. This acts similar to the rough flame-cut edge.

Generally the plates will have closed up after the weld has cooled.

5.7.1 Fillet Welds on Thick Plates

The discussion in @1[5.7 Welding thick Plates] on metal-to-metal contact and shrinkage stresses especially applies to fillet welds. A slight gap between plates will help to ensure crack-free fillet welds.

Bead shape is another important factor that affects fillet weld cracking. Freezing of the molten weld, part (a) of @1[weldab3.pcx], due to the quenching effect of the plates commences along the sides of the joint, part (b) of the figure, where the cold mass of the heavy plate instantly draws the heat out of the molten weld metal and subsequently progresses uniformly inward, part (c), until the weld is completely solid, part (d). Notice that the last material to freeze lies in a plane along the centerline of the weld.

The concave fillet weld was originally favoured by designers because it seemed to offer a smoother path for the flow of stress. However, experience has shown that single-pass fillet welds of this shape have a greater tendency to crack upon cooling. This fact generally outweighs the effect of improved stress distribution, especially with steels that require special welding procedures.
When a concave fillet weld cools and shrinks, its outer face is stressed in tension, @1[weldab4.pcx]. A surface shrinkage crack, should it occur, is better avoided by the use of convex fillet welds. As seen in part (b) of the figure, the weld can shrink, while cooling, without stressing the outer face significantly in tension and subsequently, should not crack. For multiple-pass fillet welds, the convex bead shape theory usually applies only to the first pass.
If concave welds are desired for special design considerations, such as stress flow, they should be made in two or more passes - the first slightly convex, and the other passes built-up to form a concave fillet weld.

5.7.2 Groove Welds on Thick Plates

It is usually the first (or root) pass of a groove weld on heavy plate that requires special precautions. This is especially true of the root weld on the back side of a double Vee joint because of the added restraint from the weld on the front side. The weld tends to shrink in all directions as it cools, but is restrained by the plate. The tensile shrinkage stresses set up within the weld frequently undergo plastic yielding to accommodate the shrinkage.
@1[weldab5.pcx] illustrates the reason for possible locked-in stresses and plastic flow of the weld. The dotted lines indicate free shrinkage of the weld and surrounding plate. The solid lines indicate the plates pulled back to the original position that they would normally be in during and after welding. Stretching of the weld results. Most of this yielding takes place while the weld is still hot and has lower strength and ductility. If the internal stress exceeds the physical properties of the weld, at the elevated temperature, then a crack will occur, usually down the centerline of the weld.
This problem is enhanced by the fact that the root bead usually picks up additional carbon or alloy by admixture with the base metal. The root bead is therefore less ductile than subsequent beads.

A concave bead surface in a groove weld creates the same tendency for surface cracking as described for fillet welds, see @1[weldab4.pcx]. This tendency is further increased with lower ductility, as found in the root bead. See @1[weldab6.pcx].

Using electrodes or procedures that develop a convex bead shape in the root pass will increase its throat dimension and help to prevent cracking. The use of low hydrogen welding materials might also prove beneficial. If preheat is specified, it should have been adopted only as a last resort because it will incur the greatest increase in weld cost.

Figure @1[weldab7.pcx] illustrates how the problem of centerline cracking can also occur in succeeding passes of a multiple pass weld, if the passes are excessively wide or concave. Corrective measures call for a procedure that specifies a narrower and slightly convex bead shape, making the completed weld two or more beads wide.

5.8 Internal Cracking in Welds

In some instances a crack may arise that is sub-surface, internal to the weld. This type of crack is usually the result of poor joint design or the misuse of a welding process that can achieve deep penetration.

The freezing action for butt and groove welds is the same as that illustrated for fillet welds, @1[weldab3.pcx]. Freezing starts along the weld surface adjacent to the cold base metal and finishes at the centerline of the weld. If, however, the weld depth of fusion is much greater than width of the face, the weld's surface may freeze in advance of its center. Now the shrinkage forces will act on the still hot center or core of the bead and could result in a centerline crack along the length of the weld. This crack may never extend to the face of the weld. Refer to @1[weldab8.pcx].

Part (b) of the figure illustrates how internal cracks can result with improper joint design or preparation. Here, a thick plate has been combined with a deep penetrating welding process and only a 45 degree included angle.

Part (c) shows that a small bevel on the second pass side of the double-V-groove weld combined with arc gouging a groove too deep for its width can lead to the internal crack illustrated.

Internal cracks have also been found on fillet welds where the depth of fusion is sufficiently greater than the face width of the bead, part (d).

Internal cracks should give cause for concern because they cannot be detected with visual inspection methods, however, a few preventive measures can assure their elimination. Limiting the penetration and the volume of weld metal deposited per pass through speed and amperage control is one positive control method. Using a joint design which sets reasonable depth of fusion requirements is another.

The most important factor to help control internal cracking is the ratio of weld width to depth. Experience has shown that a weld width to depth of fusion ratio in the range from a minimum of 1:1 to a maximum of 1.4:1 proves most satisfactory.

5.9 Underbead Cracking

Underbead cracking is not a problem with the controlled composition, low carbon steels. When it does occur, it shows up in the heat-affected zone of the base metal. Higher carbon or alloy contents in thick plates provide a suitable environment for this type of cracking. For example, construction alloy steels which have over 100,000 psi tensile strength and are heat treated before welding may experience underbead cracking in thick plates. Underbead cracking is generally only important on hardenable steels.

One cause of underbead cracking is hydrogen embrittlement in the heat-affected zone. As a result, low-hydrogen processes should be used to join these materials. Hydrogen in the welding arc, either from the electrode coating or from wet or dirty plate surfaces, will tend to be partially absorbed into the droplets of weld metal before deposit and absorption into the molten metal beneath the arc.

Recently deposited and solidified hot weld metal and the adjacent now heated base metal are both austenitic at this elevated temperature, and therefore have a high solubility for hydrogen. Although, a considerable portion of hydrogen escapes through the weld's surface into the air, a small amount may diffuse back through the weld into the adjacent base metal. The rate of hydrogen diffusion decreases with decreasing temperature.

Beyond the boundary of the heat-affected zone, the base metal is in the form of ferrite, which has practically no solubility for hydrogen. This low solubility ferrite boundary becomes an imaginary barrier and the hydrogen tends to pile up here with no where else to go. See @1[weldab9.pcx].

Upon further cooling, the once austenitic area transforms back to ferrite with almost no solubility for hydrogen. The existing hydrogen now tends to separate out between the crystal lattice and build up pressure. This pressure, when combined with shrinkage stresses and any hardening effect of the steel's chemistry, may cause tiny cracks. Because the weld metal generally contains less carbon than the base plate, the problem arises just beyond the weld along the once existing austenite-ferrite boundary and is called "underbead cracking". See @1[weldab10.pcx]. If these cracks reach the plate surface adjacent to the weld, they are called "toe cracks".

Reduced welding speeds and the application of preheat both tend to slow down the cooling process thereby permitting more time for the hydrogen to escape. The reduction in hydrogen through the process just described or, even better, the use of low-hydrogen welding materials will essentially eliminate the problem of underbead cracking.

5.10 Reasons to Preheat

Preheating, while not always necessary, is used for one of the following reasons:

1. To reduce shrinkage stresses in the weld and adjacent base metal; especially important in highly restrained joints.
2. To provide a slower rate of cooling through the critical temperature range (about 980 to 720 degrees Celsius) preventing excessive hardening and lowered ductility in both weld metal and heat-affected area of the base plate.
3. To provide a slower rate of cooling through the 200 degree Celsius range, allowing more time for any hydrogen that is present to diffuse away from both the weld and adjacent plate and as a result, avoid underbead cracking.
4. To increase the allowable critical rate of cooling below which there will be no underbead cracking. Thus, with the welding procedure held constant, a higher initial plate temperature increases the maximum safe rate of cooling while slowing down the actual rate of cooling. This tends to make the heat input from the welding process less critical.
5. To increase the notch toughness in the weld zone.
6. To lower the transition temperature of the weld and adjacent base metal.

Normally, not much preheat is required to prevent underbead cracking and even less is required when low-hydrogen welding materials are used. Higher preheat temperatures, however, may occasionally be required for some other reason, such as, a highly restrained joint between very thick plates, or high alloy content in the base plate.

Preheating makes other factors less critical, but since it invariably increases the cost of welding, it should not be indulged in unnecessarily.

6 Maintaining Control Over Welding

This local section of the hypertext base discusses,

@1[06.1 Residual Stresses]

@1[06.2 Shrinkage and Distortion]

@1[06.3 Summary and Check List]

@1[06.4 Lamellar Tearing]

6.1 Residual Stresses

Residual stresses are those stresses that remain in a body after all external loads have been removed. The following technical terms have been used to denote residual stresses:

Internal stresses

Initial stresses

Inherent stresses

Reaction stresses

Locked-in stresses

In addition, the residual stresses remaining in a body subjected to a non-uniform temperature change are called thermal stresses. Thermal stresses arise when temperature change is accompanied by restraint.

Occurrence of Residual Stresses:

Unfortunately, the manufacturing of metal structures permits the creation of residual stresses in many ways. Rolling, casting, or forging of materials such as plates, bars, and sections can produce significant residual stresses. In addition, shearing, bending, machining, and grinding during the forming and shaping of metal parts may also produce substantial stresses. Another way to produce residual stresses is through fabrication processes such as welding. The manufacturing process may involve heat treatments such as quenching which produces residual stress or stress-relieving which alleviates residual stress.

The complex temperature changes associated with the welding process cause transient thermal stresses in the weldment thereby producing non-elastic or incompatible strains in regions near the weld. It is these strains that generate residual stresses upon completion of welding. Residual stresses produced from the welding process may be classified into two categories according to the mechanisms which created them:

1. Reaction stresses - The restraining effect comes from external sources. These stresses extend well away from the weld with magnitude determined mainly by the degree of external restraint. Reaction stresses might more informatively be described as 'suppression of distortion' stresses.

2. Residual stresses - The restraining effect comes from the colder metal adjacent to the weld. These stresses are in equilibrium over a relatively small range and are more difficult to control.

@1[06.1.2 Residual and Reaction Stresses in a Butt Weld] illustrate the distribution of longitudinal and transverse stresses.

6.1.1 Discussion of Residual Stresses

An unrestrained, uniformly heated material will expand uniformly, without the production of thermal stress. The generation of thermal stress requires non-uniform heating and expansion of a material. Further, if transient thermal stresses are completely elastic and no incompatible strains are formed, no residual stresses will remain. However, non-elastic or incompatible strains are produced during welding and once the weld is completed, residual stresses remain. Residual stresses may also result from unevenly distributed non-elastic strains such as plastic strains.

Residual stresses are generated in structural members as a consequence of localized plastic deformation. In other words, the failure of the member, or part of the member, to relax to an unstressed condition after experiencing plastic deformation results in residual stress. For example, the material contractions that occur in rolled structural shapes as they cool from rolling temperature down to ambient room temperature will result in plastic deformations because some parts of the shape cool much faster than others. Inelastic deformations are predominant in the slower cooling portions.

Similar analogies may be shown for welded members and heat-treated bars that are not furnace-cooled. Also, cold-straightened members contain residual stresses as a result of the applied plastic bending deformations

In general, without plastic deformation during the life of the material there can be no residual stresses. Furthermore, for elements experiencing a temperature gradient, the part to cool last will usually be in a state of tensile residual stress.

Equilibrium Condition:

Residual stresses exist without any applied external forces. Therefore, there must be no resultant force or resultant moment produced by the residual stresses.

§ s*dA = 0 on any plane section

§ dM = 0

6.1.2 Residual and Reaction Stresses in a Butt Weld

Three diagrams will be used to illustrate residual stresses in a butt welded plate. @1[resid30.pcx] shows the butt weld orientation and stress directions. @1[resid31.pcx] and @1[resid32.pcx] show the most important residual stress distributions, stress parallel to the weld direction and stress transverse to it, respectively.

In @1[resid31.pcx], the distribution of longitudinal residual stress shown indicates large magnitude tensile stress in the region near the weld tapering off rapidly to compressive stresses several weld widths away. This distribution may be characterized by the maximum tensile stress at the weld, s, and the width of the tension zone, b. In low carbon steel weldments, the maximum residual tensile stress, s, is usually as high as the yield stress of the weld metal.

According to Masubuchi and Martin, the following equation may be used to approximate the longitudinal residual stress.

s (y) = s {1 - (y/b)2} e EXP(-½(y/b)2)

Curve 1 of @1[resid32.pcx] illustrates the distribution of transverse residual stress along the length of the weld. Low magnitude tensile stresses are shown along the middle section of the joint with compression stresses at the ends of the joint.

Curve 2 of the same diagram represents the reaction stress distribution that would exist if the lateral contraction of the joint was restrained by an external constraint. Note that the added tensile stresses are approximately uniform along the length of the weld.

As an added note, an external constraint would have little influence on the distribution of longitudinal stress.

6.1.3 Residual Stress in Columns

The measurement of residual stresses in WF shapes of ASTM-A7 designation was found to be, on average, of about 13 ksi compression at the tips of the flanges and about 5 ksi tension at the center.

With regard to WF shapes welded from universal mill plates, a significantly high increase in the residual tensile stress at the flange-to-web junction generates correspondingly higher residual compressive stresses at the extremes of the flanges. The tensile residual stresses will frequently approach the yield stress of the material, while the residual compressive stresses reach about two-thirds of this value.

A typical distribution of residual stresses in two of the more common welded column sections, H-shape and Box-shape, is shown in the following figures. Refer to @1[resid2.pcx] and @1[resid3.pcx].

The existence of high compressive residual stresses at critical locations on the section may reduce the buckling strength of a column considerably. Therefore, the magnitude and distribution of residual stresses in the component parts of the built-up shape, as the result of welding, are important.

Beedle and Galambos wrote a paper on the influence of residual stress on concentrically loaded columns. The following three points summarize the research on this topic:

1. Residual stresses due to: cooling after rolling, welding, cold bending, and various heat treatment processes, have a pronounced effect on the stress-strain curve, and generally decrease the load carrying capacity of columns.
2. Residual stresses in rolled shapes do not appear to be influenced by the level of yield stress. Thus, the higher strength steel columns show less reduction in strength because of residual stress.
3. Residual stresses due to full strength welds are usually greater than those set up by cooling after rolling. As a result, the column strength of welded "H" shapes is less than that of rolled WP shapes for columns of intermediate lengths. However, since geometry plays an important role, tests show that box shapes give higher strength. Further research on the influence of geometry and of weld size will shed more light on the problem.

Recent studies have revealed that flame cutting of plates to be used in compound WF sections can favorably affect the residual stress distribution. This trend is illustrated in @1[resid4.pcx] and @1[resid5.pcx]. High tensile residual stresses induced into the cut edge of the flange plate prior to welding of the web flange connection will considerably reduced the residual compressive stresses that would otherwise be produced in the flange tips, if not totally eliminate them. Increased strengths of columns have been reported to be 5 to 10 percent.

Furthermore, flame cutting of the flange plates after the welding of the web plate, which may be less practical but feasible, is reported to improve still further the stress distribution in a column, @1[resid6.pcx]. This procedure may result in a strengths slightly higher than those of rolled sections.

Another simple method to obtain a favorable residual stress pattern in welded WF columns is to apply a weld bead to the toes of the flanges during shop fabrication, or on site, as shown in @1[resid9.pcx].

Reinforcement of welded columns already in service, by the welding on of cover plates, is another way to improve their strength. The side welds on the cover plates reverse the unfavorable residual stress distribution at the toes of the flanges, as shown in @1[resid8.pcx].

@1[resid7.pcx] shows the standard stress distribution of a medium sized built up H-shape before any stress reduction techniques have been applied.

6.1.4 Residual Stress in Flexural Members

The distribution of residual stresses associated with the welded fabrication of built-up flexural members, in general, has been found to depend on the following factors:

1. The geometry of the plates
2. The type and heat of welding
3. The speed of welding
4. The rate of cooling

Of these four, geometry clearly proves to have the greatest effect.

The results of an experimental investigation into the magnitude and distribution of longitudinal residual stresses arising in plates due to edge and center welding, as summarized by Nagaraja and Tall, are:

1. The distribution of residual stresses in all plates (ASTM-A7, 4 in. to 20 in. wide and 1/4 in. to 1 in. thick), whether due to cooling from rolling in the unwelded plates or due to cooling from welding in the welded plates, is approximately parabolic in shape, except at the weld itself. This is true for welded plates up to about 16 in. in width.

2. The first pass causes the major portion of the residual stress to occur, and there is no great variation of residual stress in welded plates between subsequent passes. At the edge, however, the effect of succeeding passes of welds is quite marked.

3. Where the welding conditions are uniform along the plate, the residual stresses are also uniform along the length of the plate (except at the start and termination of the weld).

4. Welding changes the properties of the plate material only in the vicinity of the weld. The most important change is that the yield stress level is increased by about 50%. Because of higher residual tensile stresses, this requires a proportionate increase in the compressive residual stresses somewhere in the distribution.

5. Plates of less than 1/2 in. thickness show the same longitudinal residual stresses on both faces.

In the case of plate girders, the bending strength is provided mainly by flanges. Since it is the yielding in the compression flange which determines its own individual strength, the residual compressive stresses in that flange due to welding, which influences this yielding, may be taken as affecting to some extent the bending strength of the girder.

However, whether it is a case of residual compressive stresses in the compression flange, or of residual tensile stresses in the web-to-flange joint, the effects of residual stresses on the flexural strength of welded members is generally considered negligible.

Residual stresses do not influence the strength of welded connections, with the exception of bracing or stiffening requirements for compression elements.

6.2 Shrinkage and Distortion

Residual stresses, shrinkage, and distortion are closely related phenomena. During welding, the heating and cooling cycle produces thermal strains both in the weld metal and base metal regions near the weld. These heating strains result in plastic upsetting of the material. The stresses generated from those strains combine and react to produce internal forces that cause bending, buckling, and rotation. These weldment deformations are called distortions.

6.2.1 Welding Factors That Cause Movement

The complex heating and cooling cycle is an unavoidable aspect of welding that must be controlled and designed for. This temperature cycle will always cause shrinkage in both base metal and weld metal. The internal forces set up as a result of this shrinkage can cause varying degrees of distortion. Designers and engineers must anticipate and provide control of this shrinkage to achieve the full economies of arc-welded steel construction.

The enormous temperature differential in the arc area creates a non-uniform distribution of heat in the weldment. As temperature increases, there is a corresponding increase in the coefficient of thermal expansion and the specific heat, while properties such as yield strength, the modulus of elasticity, and the thermal conductivity decrease. Anticipation of material movement from a straight forward analysis of heat is difficult at best.

Other factors to be considered are restraint from external clamping, internal restraint due to mass, and the stiffness of the steel plate itself. All these factors have a definite influence on the degree of movement.

Finally it is necessary to consider the factor of time as it affects the rapidly changing conditions. The period of time during which a specific condition is in effect controls the importance of that condition.

Further influence of shrinkage and distortion arises from the different welding procedures. The type and size of electrode, welding current, speed of travel, joint design, and preheating and cooling rates all bear significantly on the problem.

It is obvious that distortion cannot be analyzed by viewing each one of these factors separately. A solution based on correcting the combined effect is the only practicable approach.

6.2.2 Weld Metal Shrinkage

The weld metal is in its maximum expanded state, or occupies the greatest volume it can occupy, right at the point in time where it solidifies and fuses with the base metal. During the process of cooling the weld metal would like to occupy a smaller volume, however it is prevented from doing so by the adjacent base metal. This restraining effect leads to the development of stresses within the weld which eventually reach the yield strength of the weld metal. The weld is then forced to stretch, or yield, and thin out as it attempts to adjust to the volume requirements of the joint being welded. Only the stresses which exceed the yield strength of the weld metal will be relieved by this stretching effect. Upon reaching room temperature, and assuming the base plates are fully restrained, there will exist locked-in tensile stresses equal in magnitude to the yield strength of the weld metal. If the restraints are then released, the locked-in stresses will find partial relief through movement of the base metal thus deforming or distorting the weldment. @1[resid10.pcx] illustrates the residual longitudinal and transverse stresses that arise with the creation of a fillet weld. To picture how these stresses get into the weld, it is helpful to imagine the situation depicted in @1[resid11.pcx]. Here the fillet welds are considered independently of the base plates. The weld metal is shown shrunk down to the volume it would occupy at room temperature. It is both restraint and stress free. You can further imagine what has to be done to the weld in order to attach it to the base plates. This action would require stretching in both the longitudinal and transverse directions thereby generating stresses equal to the yield strength of the weld material.

6.2.3 Base Metal Shrinkage

The build up of stresses that lead to distortion is further influenced by shrinkage in the base metal adjacent to the weld. The heat of welding causes the base metal in the vicinity of the weld to reach melting temperatures. However, only a few inches away the metal is substantially cooler. This sharp temperature differential will cause non-uniform expansion followed by base metal movement, or metal displacement if the parts being joined are restrained. Upon cooling the base metal will shrink and any restraint from surrounding metal will lead to internal stress build up. The tendency to distort is then increased as these stresses act in combination with those developed in the weld metal.

Welding procedures are often used to control the volume of adjacent base metal contributing to the distortion. The amount of adjacent base metal that is affected by the heat of the arc can be reduced by using welding processes that achieve higher welding speeds. Less affected base metal means less distortion.

According to studies on the mechanisms of transverse shrinkage in butt welds, the major portion of the shrinkage is due to contraction of the base plate. Shrinkage of the weld metal itself only accounts for about 10 percent of the actual shrinkage.

6.2.4 Fundamental Modes of Distortion

The welding process generates dimensional changes that lead to distortion in fabricated structures. Three of the more fundamental dimensional changes are:

1. Transverse shrinkage perpendicular to the weld line.
2. Longitudinal shrinkage parallel to the weld line.
3. Angular distortion (rotation around the weld line).

The appearance of the dimensional change gives rise 6 classifications:

(a) Transverse shrinkage. Shrinkage perpendicular to the weld line. @1[resid12.pcx]

(b) Angular change (transverse distortion). A non-uniform thermal distribution in the thickness direction causes distortion (angular change) close to the weld line. @1[resid13.pcx]

(c) Rotational distortion. Angular distortion in the plane of the plate due to thermal expansion. @1[resid14.pcx]

(d) Longitudinal shrinkage. Shrinkage in the direction of the weld line. @1[resid15.pcx]

(e) Longitudinal bending distortion. Distortion in a plane through the weld line and perpendicular to the plate. @1[resid16.pcx]

(f) Buckling distortion. Thermal compressive stresses cause instability when the plates are thin. @1[resid17.pcx]

6.2.4.1 Transverse Shrinkage of Butt Welds

Typical transverse shrinkage can be seen in @1[resid12.pcx]. This diagram shows uniform shrinkage along the length of the weld, however, this is not usually the case. The real weldment will exhibit complex transverse shrinkage which, for butt welds, is primarily dependent on rotational distortion and restraint.

Rotational distortion will occur as a result of progressive welding from one end of the joint to the other. The reason being that the unwelded portion of the joint will move during this procedure. See @1[resid14.pcx]. Both the welding heat input and the location of tack welds effect rotational distortion. Tack welds must be large enough to withstand the stresses caused by the rotational distortion. The separating force caused by the rotational distortion can be large enough to fracture the tack welds and crack portions of the weld metal. The largest amount of distortion will be seen during the first pass when the unwelded portions of the joint are relatively free.

The degree of restraint applied to the weld joint will affect the magnitude of transverse shrinkage. We can represent the external restraint by a system of springs whose rigidity describes the degree of restraint. @1[resid18.pcx] illustrates this simple principle. As the degree of restraint increases, the amount of shrinkage decreases, and vice versa. The degree of restraint is not always uniform along the length of the weld. For example, in a slit weld, the transverse shrinkage is restrained by the bare metal surrounding the weld. The ends of the slit provide a high degree of restraint resulting in minimal transverse shrinkage. The center of the slit, however, imparts a significantly lower degree of restraint which yields much higher transverse shrinkage.

Consider a single pass butt weld in a free joint. Almost immediately after welding, the heat of the weld metal is transmitted into the base metal. This transferral of heat results in the contraction of the weld metal and the expansion of the base metal. This volume expansion of the base metal is accommodated mostly through a change in thickness. So far, no sectional movement of the base plate has occurred. In fact, it is not until the weld metal begins to resist the thermal deformation of the base metal, time t = t(s), that the plate sections begin to move in response. In other words, the contraction of the heated base metal, through cooling, begins to be resisted by the weld metal, which is now developing tensile stresses, and therefore base plate sections must move in. Subsequent contraction in the weldment commences as the heat leaves the base plates. Ignoring the mathematical determination of the various thermal deformations of both base plate and weld metal, we will jump to the conclusion. The final transverse shrinkage depends on the thermal expansion of the base metal at time t(s) and on the thermal contraction of the weld metal. Studies have shown weld metal shrinkage to account for only about 10 percent of the actual shrinkage. Therefore, the most important factor in the final shrinkage of a single pass butt weld in a free joint is the expansion of the base metal at time t(s).

The charts in @1[resid19.pcx] and @1[resid20.pcx] throw some light on transverse shrinkage. In the latter chart, transverse shrinkage for a given plate thickness is seen to vary directly with the cross-sectional area of the weld. The large included angles only help to illustrate this relationship and do not represent common practice. The relative effects of single and double V-joints are seen in the former chart. Both charts assume no unusual restraint of the plates against transverse movement. Calculations show that transverse shrinkage is about 10% of the average width of the cross-section of the weld area.

shrinkage = S = 0.10 * (Aw) / (t)

Although the above formula has been used for many years, alternative formulas have been proposed. For example, Spraragen and Ettinger, after examining the shrinkage data obtained from several earlier investigators, developed the following:

shrinkage = S = 0.2 * (Aw) / (t) + 0.05 * (d)

where: S = transverse shrinkage (in.)

Aw = cross-sectional area of weld (in.2)

t = thickness of plate (in.)

d = free distance or root opening (in.)

The moral, of course, is that transverse weld shrinkage calculation is not an exact science. However, it is a well known fact that transverse shrinkage decreases when a joint is restrained. Kihara and Masubuchi have studied multiple pass welding of butt joints in carbon steel. Their results show that transverse shrinkage increases during multiple pass welding. This result is illustrated in @1[resid21.pcx]. It was found that shrinkage diminished with subsequent weld passes because the resistance against shrinkage increased as the weld got larger. A linear relationship was then discovered between the transverse shrinkage, s, and the logarithm of the weight of the weld metal, w. See @1[resid22.pcx].

s = s(0) + b * (log w - log w(0))

where: s = transverse shrinkage

s(0) = shrinkage after the first pass

w = total weight of weld metal per unit length

w(0) = weight of weld metal per unit length after the first pass

b = a coefficient

The most important procedural factors that affect transverse shrinkage are listed below:

1. Transverse shrinkage increases with weld volume.

2. Transverse shrinkage increases with increasing root gap.

3. Transverse shrinkage increases with total heat input.

4. Transverse shrinkage decreases with block welding.

6.2.4.2 Transverse Shrinkage of Fillet Welds

Fillet welds undergo less transverse shrinkage than butt welds. Although the amount of study on this topic is limited, Spraragen and Ettinger have suggested the following simple formula:

1. Tee-joints with continuous fillets

   shrinkage = [(leg of fillet) / (thickness of plate)] * 0.04 (in.)

2. Intermittent welds - apply correction factor of proportional length of fillet to total length

3. Fillets in lap joints (2 welds)

   shrinkage = [(leg of fillet) / (thickness of plate)] * 0.06 (in.)

6.2.4.3 Longitudinal Shrinkage

Longitudinal shrinkage in butt welds:

Generally, longitudinal shrinkage from a butt weld amounts to approximately one one-thousandth of the weld length. This is much less than the transverse shrinkage. Although research is limited, King proposed the following formula:

shrinkage = s = [0.12 * I * L] / [100000 * t] .... (inches)

where: I = current (amps)

L = weld length (in.)

t = plate thickness (in.)

Longitudinal shrinkage of fillet welds:

Extensive research has been conducted by Guyot on the longitudinal shrinkage of fillet welds in carbon steel. Guyot discovered that the total cross-section of the joints involved will primarily govern the magnitude of longitudinal shrinkage. Thicker and wider plates will provide more effective restraint. Therefore, Guyot decided to call the total cross-section of the resisting plate in the transverse section the "resisting cross-section".

Guyot's results are illustrated in @1[resid23.pcx]. Cross shaped assemblies are used to maintain symmetry and thereby keep longitudinal deflection to a minimum. The expressed shrinkage values are a function of the cross-sectional area of the weld metal, Aw, and the resisting cross-sectional area, Ap. For ratios of Ap to Aw less than 20, the following formula may be used:

longitudinal shrinkage per mm of weld = (Aw / Ap) * 25

6.2.4.4 Longitudinal Bending Distortion

Usually distortion appears as a shortening of the weld area. When longitudinal shrinkage acts along a weld line that is not coincident with the neutral axis of the member, longitudinal bending distortion results in the form of bowing or cambering. This type of distortion is most important in the fabrication of T-bars and I-beams.

Distortion results when a condition of non-uniform expansion and contraction is created. Distortion can be anticipated by evaluating the following factors:

1. The weld along with some adjacent metal contracts on cooling, producing a shrinkage force, F.

2. The shrinkage force acts about the neutral axis of a member. The distance between the center of gravity of the weld area and this neutral axis represents the moment arm, d.

3. The moment of inertia of the section, I, resists this contraction. The I of a section also resists straightening, should it be necessary.

Distortion or bending of longitudinal members results from the development of a shrinkage force applied at some distance from the neutral axis of the member. The amount of distortion is directly controlled by the magnitude of the shrinkage moment and the member's resistance to bending as indicated by its moment of inertia.

Assuming no unusual initial stresses, the following formula indicates the amount of distortion or bending displacement that will result from any longitudinal welding on a given member:

displacement = 0.005 * [Aw * d * L2] / I

Actual measured distortion corresponds well with calculated distortion, using the formula given. In some instances when equal welds are positioned symmetrically around the neutral axis of a member, a certain amount of distortion still occurs even though the magnitudes of the shrinkage moments are equal and opposite. It is believed some plastic flow or upset occurs in the compressive area next to the weld area after the first weld is made. Because of this upset, the initial distortion, from the first weld, is not quite offset by the second weld on the opposite side. Where multiple-pass welding is involved, this condition can be corrected by controlling the groove-weld sequence. Frequently this problem is of no major importance since the sections to be welded are large enough in respect to the size of the weld to prevent the occurrence of this upsetting.

In cases where the welds are not symmetrically balanced about the neutral axis of the section, advantage may be taken of this difference in distortion by first completing the joint nearest the neutral axis (it has the shorter moment arm) and then welding the joint on the side farthest from the neutral axis (taking advantage of its greater moment arm). Similarly, weld sizes may be varied to help balance forces.

Many long slender members are made by welding together two light-gauge formed sections. For example, long, thin box sections are often welded up from two channels. Waiting until the first weld has cooled before making the second weld on the opposite side will usually result in some final bowing since the second weld may not quite pull the member back. The heating of the top side of the member by the first weld initially causes some expansion and bowing upward. Turning the member over quickly while it is still in this shape and depositing the second weld, increases the shrinking effect of the second weld deposit and the member is usually straight after cooling to room temperature.

6.2.4.5 Angular Distortion

Angular changes of butt welds:

Transverse shrinkage is not always uniform in the thickness direction and, as a result, angular change often occurs. Procedural parameters associated with the welding process, such as groove shape and degree of restraint, will affect the angular change in butt welds.

@1[resid24.pcx] represents some of the work performed by the Shipbuilding Research Association of Japan on angular change in butt welds. The figure shows the groove shape that most successfully minimized angular change in butt welds of various thicknesses. The two curves represent situations with and without strongbacks. The diagram illustrates how angular distortion can be controlled through appropriate joint design. The use of double Vee joints not only balances the distortion but also uses less weld metal which will demand less transverse shrinkage.

Angular changes of fillet welds:

The two types of simple fillet welded structures, shown in @1[resid25.pcx], illustrate one-dimensional out-of-plane distortion. Fillet welded joints, free from external constraints, will produce bending at the joints in the manner illustrated in the top of the figure. However, if the stiffeners are fixed to a rigid beam, then a different type of distortion is produced. The angular changes of the fillet welds will result in a wavy distortion of the plate strip. This type of distortion can be seen in the bottom of the figure.

The effect of preheating T-shaped weldments has been studied by Watanabi and Satoh. They looked at the angular distortion of fillet welds in low carbon steel. Results clearly showed that preheating will reduce angular distortion and that preheating the back of the plate is more effective than preheating the front. @1[resid26.pcx] illustrates this effect.

There are two well known methods of reducing angular distortion:

1. Plastic prebending .... top of @1[resid27.pcx]
2. Elastic prestraining .... bottom of figure "resid27" above

It is potentially possible to create a fillet weld with no angular distortion by simply applying the exact amount of plastic prebending. If only it were so simple. Unfortunately the exact amount of prebending is difficult to achieve as it depends on parameters such as plate thickness and welding conditions. To create further difficulty, the plastic prebending line must exactly match the weld line.

Elastic prestraining may also be used to create zero angular distortion, provided the proper amount is used. As seen in figure "resid27" above, this method uses a restraining jig. The general belief is that elastic prestraining is more reliable than plastic prebending. This is because, with a clamped weldment, the angular distortion is always significantly less than if it were free and even if an error is made, the distortion will still be reduced. However An error made with plastic prebending could prove deleterious.

Flange warpage calculation:

@1[resid28.pcx] illustrates angular distortion in a flange due to the flange to web fillet welds. The formula for calculating this warpage is:

delta = * = 0.02 * W * w^(1.3)

Close agreement between calculated and actual values has verified the use of this formula.

6.2.4.6 Buckling Distortion

The welding of thin plates causes residual compressive stresses to occur in areas away from the weld. These compressive stresses can lead to buckling. Buckling distortion occurs when the specimen length exceeds the critical length for a given thickness in a given size specimen. An important consideration when studying weld distortion in thin plated structures is whether the cause is buckling or bending. Buckling distortion differs from bending distortion in the following two ways:

1. There is more than one stable deformed shape.
2. The amount of deformation in buckling distortion is much greater.

Proper selection of such structural parameters as plate thickness, stiffener spacing, and welding parameters is the best way to avoid the large distortions associated with buckling.

6.2.5 Control of Weld Shrinkage and Distortion

Although shrinkage cannot be prevented, it can be controlled. Strategies incorporated at the design stage and implemented through good shop practices will effectively control the distortion caused by heating and cooling cycles. Some of the more important strategies are listed below.

@1[06.2.5.01 Do Not Overweld]

@1[06.2.5.02 Use as Few Weld Passes as Possible]

@1[06.2.5.03 Place welds Near the Neutral Axis]

@1[06.2.5.04.Balance Welds Around the Neutral Axis]

@1[06.2.5.05 Welding technique]

@1[06.2.5.06 Make Shrinkage Forces Work in the Desired Direction]

@1[06.2.5.07 Balance Shrinkage Forces With Opposing Forces]

@1[06.2.5.08 Welding Sequnce]

@1[06.2.5.09 Removal of Shrinkage Forces During or After Welding]

@1[06.2.5.10 Reduce the Welding Time]

@1[06.2.5.11 Effect of High Welding Speeds]

@1[06.2.5.12 Type and Size of Electrode]

@1[06.2.5.13 Degree of Restraint]

6.2.5.1 Do Not Overweld

More weld metal means higher shrinkage forces. Distortion can be controlled by correctly sizing the weld for the service requirements of the joint. Remember, only the effective throat of a conventional fillet is used in the determination of the strength of the weld. A highly convex bead will only add to the shrinkage force developed. In a butt joint, proper edge preparation, fitup, and reinforcement will minimize the volume of weld metal used.

Use intermittent fillets, where permissible, with emphasis on small size rather than short length (however, the minimum size must be in agreement with specifications).

6.2.5.2 Use as Few Weld Passes as Possible

A large portion of the transverse shrinkage that takes place in butt welds occurs during the first and the second passes. See @1[resid21.pcx]. Much work has been done on the question of how various procedure parameters affect transverse shrinkage. One important discovery has been that the use of larger diameter electrodes will result in less shrinkage.

@1[resid24.pcx] suggests that a butt weld, free of angular distortion can be acquired through proper selection of joint design and welding sequence. In many cases, fewer passes with large electrodes are preferable to many passes with small electrodes when distortion could be a problem.

6.2.5.3 Place Welds Near The Neutral Axis

The smaller the lever arm, the less likely that plates will be distorted out of alignment. Here is where both the design and welding sequence can be used to control distortion.

Frequently the neutral axis of the member is below the center of gravity of the welds. By making the welds with the submerged-arc automatic welding process, the deep penetration characteristic of this process further lowers the center of gravity of the weld deposit and reduces the moment arm, thereby reducing the shrinkage moment.

6.2.5.4 Balance Welds Around the Neutral Axis

This action will cause the weld shrinkage forces to work against each other to obtain minimal distortion. Here again, both the design and welding sequence are used to control distortion.

6.2.5.5 Welding Technique

The types of welding technique used on multipass weldments sometimes have considerable influence on distortion.

In multipass welding, each subsequent pass has an annealing effect on the pass already deposited and progressively adds less to the distortion already produced. This is especially noticeable on the first and second passes.

During manual welding the length of electrode limits the length of weld which necessitates occasional breaks in the welding operation for the purpose of replacing the electrode. If the welding time is intentionally broken and the welding arranged in different steps, then the magnitude and orientation of the residual stresses may be affected considerably.

The most frequently used methods for interrupting a welding pass are known as the "back-step" and "wandering" sequences. The general principle behind both of them is the same. In each case, the order of welding has been arranged so that one increment has reached the stage of cooling and contracting by the time the adjacent increment is deposited. This constant repetition of adjacent zones of expansion and contraction results in alternate zones of tension and compression that tend to cancel out and localize the heat effect, thus maintaining a more effective balance of residual stresses within a narrower range of amplitudes.

Frequently, in multipass welding one specific technique may be adopted for the root passes and an entirely different one for any subsequent layer.

6.2.5.6 Make Shrinkage Forces Work in the Desired Direction

Shrinkage forces can be utilized constructively by presetting parts out of position, the correct amount, prior to welding. Plastic prebending and elastic prestraining the parts to be welded are two methods commonly used to mechanically control shrinkage forces. Elastic prestraining is preferred because it is more forgiving if an error is made. Refer to @1[resid27.pcx].

6.2.5.7 Balance Shrinkage Forces With Opposing Forces

These opposing forces might be: (a) other shrinkage forces; (b) restraining forces imposed by clamps, jigs, and fixtures; (c) restraining forces arising from the arrangement of members in the assembly and; (d) the counterforce from the sag in the member produced by the force of gravity. One common practice is to position identical weldments back-to-back and clamp them tightly together. The welds are completed on both assemblies and allowed to cool before the clamps are released. Wedges may be inserted at suitable locations between the parts to incorporate prestraining if necessary.

Clamps, jigs, and fixtures are probably the most commonly used means of controlling distortion in small assemblies or component parts. These restraining devices hold the parts in the desired position until the welding process is finished (including cooling). A fixture considerably reduces the distortion because the strain associated with the elastic locked-in stresses is very small compared to the movement that would occur with no restraint during welding

6.2.5.8 Welding Sequence

Investigators have found that welding sequence affects transverse distortion. Block-welding sequences were generally found to cause less shrinkage than multi-layer sequences.

The welding sequence can be used to your advantage by balancing the shrinkage forces of subsequent weld passes with the previous ones. For example, in making a butt weld, a well planned sequence would suggest alternate welding on both sides of the neutral axis.

6.2.5.9 Removal of Shrinkage Forces During or After Welding

Although post-weld straightening should be kept to a minimum, especially with heat-treated materials, distortion that exceeds the acceptable limits does occur. This distortion often occurs during fabrication, but can also occur during service, by overload or collision, for example. When this happens, the distortion must be removed economically and with a minimum amount of damage to the structure.

Peening, although considered questionable practice, is one method that has been used to remove shrinkage forces. It is a mechanical method that applies a force to the weld to make it thinner thereby making it longer and relieving residual stresses. Peening, if properly applied, tends to expand the weld area, however this expansion occurs mainly near the surface. The root bead is never peened because of the danger of either creating or concealing a crack. Also, the final pass is rarely peened because of the possibility of work-hardening, interfering with inspection, or covering a crack. Although there have been instances where peening between passes was the only solution to a distortion or cracking problem, it must be recognized that its use is limited and should have engineering approval before execution.

Upsetting or expansion of the weld metal by peening is most effective at higher temperatures where the yield strength of the metal is rather low. Unfortunately, most of the distortion occurs later at the lower temperatures after the yield strength has been restored to its higher value. For this reason, peening does not accomplish the desired results. An additional disadvantage of peening is that it work-hardens the surface of the metal and uses up some of the available ductility. Stress relief is also seen through controlled heating and cooling of the weldment. The flame heating of a plate at selected spots or along a certain line followed by cooling with water is one common technique. Occasionally two weldments are placed back-to-back, clamped together, welded, and then stress relieved while held in this straight condition. The residual stresses that would have attempted to distort the structure are thereby removed.

Flame shrinkage or flame straightening are methods of correcting distortion through localized heating with a torch. The heat causes the metal in the area to expand, and this expansion is restrained in all directions by the surrounding cooler metal. As a result, this area of the metal expands abnormally through its thickness and upon cooling tends to become shorter in all directions. The section so treated will become shorter and stressed in tension with each successive application of heat.

The bending of a member by welding and its straightening by flame shrinking is analogous to the case of a stool which will tilt to one side when the legs on one side are shortened but will again become erect when the opposite legs are also shortened the same amount.

6.2.5.10 Reduce the Welding Time

Generally speaking, it is desirable to finish the weld as quickly as possible so as to minimize the heat transfer to the surrounding metal. The welding process used, the type and size of electrode, welding current, and speed of travel all affect the welding time and therefore the amount of shrinkage and distortion. In addition, iron-powdered manual electrodes and mechanized welding equipment can be used to reduce the time of welding.

6.2.5.11 Effect of High Welding Speeds

The volume of adjacent base metal which contributes to the distortion can be controlled by welding procedures. Higher welding speeds through the use of powdered-iron-type manual electrodes, semi-automatic and fully automatic submerged-arc welding equipment, or vapour-shielded automatic welding equipment reduces the amount of adjacent material affected by the heat of the arc and progressively decreases distortion.

Consider two procedures that produce approximately the same size weld but vary in weld speed. The important difference lies in the fact that the higher-speed welding technique produces a slightly narrower isotherm, measured outward from the edge of the molten pool. The width of this isotherm can be used to indicate the amount of base metal contributing to the shrinkage along with the weld, and therefore the amount of distortion. This helps to explain why, in general, faster welding speeds result in less distortion. This slight difference is also evident in a comparison of the quantity of welding heat applied to the plate.

The welding speed is not the only factor that controls the amount of distortion, the type and size of electrode also play a significant role.

6.2.5.12 Type and Size of Electrode

The essential effect of the type and size of electrode on the magnitude of distortion is dependent on the quantity of heat required to deposit a certain volume of weld metal.

With reference to types of electrodes of different classifications, only minor variations exist in the optimum heat energy requirements, and as such the variations are mostly due to varying thicknesses of the coating.

The size of the electrode within a classification also has negligible influence on the total amount of heat required to melt a unit volume (or weight) of weld metals. However, because of higher deposition rates, allowing greater welding speeds, a bigger diameter electrode will be preferred. In the case of single-pass welds, the influence of a larger electrode on the magnitude of distortion may generally be considered as small. However, on multiple-pass joints the larger electrodes show significant advantage, and this mostly because of the lesser number of passes required.

Only the width of the heat zone is affected by the size of the electrode, hence, if a welded member has restrained edges, a bigger diameter electrode will produce higher transverse residual stresses. On the other hand, the use of a smaller diameter electrode may result in too steep of a cooling rate and subsequently increase the chance of cracking.

The above considerations serve to point out that a compromise must be reached between welding procedures favouring control of distortion and those that are better for control of cracking.

It should also be noted that the interpass temperature has a direct and important bearing on the effects under discussion.

6.2.5.13 Degree of Restraint

Engineers involved in welding fabrication want to know how to reduce distortion without increasing residual stress. It is commonly believed that when a weld is made under restraint, distortion will decrease and residual stress increase. The type of welding fixtures used in shop work affect the final deformation or distortion. In many cases, the residual stresses present upon removal from a restraining jig are no greater than those in weldments made without additional restraint.

Fully restrained members in which no freedom of movement is allowed in any direction do not deform during the welding operation. However, when the member is allowed to cool down to some degree in the restrained condition before being taken out of the fixture, greater plastic deformation will occur in the welding zone than is the case when two loose plates are welded. Although high residual stresses in loose welded plates will result in considerable distortion.

Still greater deformation will take place if the work is removed from the fixture immediately after welding, with no cooling allowed under the restrained condition. During welding of fully restrained members, the heated zones, having no freedom to expand, show an upsetting effect as plastic deformation is set up from the developed compression. During solidification, however, tensile stresses are produced which to some degree reduce the extent of plastic deformation previously established. If the work in question is then released from the fixture right after welding, the full original amount of plastic deformation is free to impart its influence on the work as a whole.

For the above reasons, a complete and rigid restraint on the member to be welded is sometimes not desirable. A proper procedure with rigid fixtures is to preset the work so as to apply an amount of deformation of the same magnitude as, but opposite in orientation to, the one expected to result from welding.

It is considered good practice to weld first the joints that will produce the greatest distortion.

The division of work into sub-assemblies may be the only means of economically and effectively reducing the distortion down to tolerable levels.

It is largely shop experience that counts in the fabrication of reasonably straight, finished products.

6.3 Summary and Check List

Access to four sections containing notes in point form regarding the control of shrinkage in common scenarios provides a quick reference for the user.

@1[06.3.1 Notes on Transverse Shrinkage and Distortion]

@1[06.3.2 Notes on Reducing Angular distortion]

@1[06.3.3 Notes on Bending of Longitudinal Members]

@1[06.3.4 Notes on Assembly procedures to Control Distortion]

6.3.1 Notes on Transverse Shrinkage and Distortion

The following points summarize a few of the more important factors that effect transverse shrinkage and distortion.

1. Transverse shrinkage and distortion depends on restraint.
2. Transverse shrinkage, assuming no unusual restraint, is equal to about 10 percent of the average width of the weld area.
3. Transverse shrinkage increases with the weld area for constant plate thickness.
4. Transverse shrinkage increases with the root opening and the included angle.
5. Transverse shrinkage is directly proportional to the welding heat input per inch, that is, Joules per inch.

6.3.2 Notes on Reducing Angular distortion

The following points summarize a few important considerations necessary for reducing angular distortion:

1. The use of double bevel, V, J, or U preparation for butt joints is preferred.
2. Angular distortion may be controlled by alternating weld passes from side to side.
3. Beveling the web of a T-joint will reduce the moment arm of the weld and reduce the angular movement.
4. Use the smallest practical leg size for fillet welds because the distortion varies approximately with the 1.3 power of the leg size of such a weld.
5. Use thicker flanges where possible because distortion varies approximately inversely with the square of the flange thickness.

6.3.3 Notes on Bending of Longitudinal Members

The following points summarize a few of the important considerations necessary for reducing distortion from longitudinal welds in bending members.

1. Balance welds about the neutral axis of the member by:

(a) making welds of the same size at the same distance on the opposite side of the neutral axis of the member.

(b) making the welds that are farther away smaller. Welds of different sizes should optimally be at different distances from the neutral axis of the member.

2. If the welding is not symmetrical, reduce potential distortion by:

(a) prebending the member.

(b) supporting the member in the middle and letting the ends sag, and for the opposite effect, by supporting the member at the ends and letting the middle sag.

(c) breaking the member into sub-assemblies so that each part is welded about its own neutral axis.

3. Deflection is directly proportional to the shrinkage moment of the welds (weld area times its distance from the neutral axis of the member) and inversely proportional to the moment of inertial of the member. Although a high moment of inertia for the member is desired to resist bending it also makes the member more difficult to straighten once it has become distorted. Flame shrinking may be applied to the longer side if welding has bent the member.

6.3.4 Notes on Assembly procedures to Control Distortion

The following points provide some important considerations for the fabrication of weldments:

1. Clamp the member in position and hold during welding.
2. Preset the joint to offset expected contraction.
3. Prebend the member to offset expected distortion.
4. Before welding, clamp two similar members back to back with some prebending.
5. If stress-relieving is required, weld two similar members back to back and keep fastened until after stress has been relieved.
6. Use strong-backs for elastic prestraining.
7. Use jigs and fixtures to maintain proper fit-up and alignment during welding.

8. Make allowances for contraction when a joint is assembled.

9. Arrange the erection, fitting, and welding sequence so that parts will have freedom to move in one or more directions as long as possible.

10. Use subassemblies and complete the welding in each before final assembly.

11. If possible break the member into proper sections, so that the welding of each section is balanced about its own neutral axis.

12. Weld the more flexible sections together first, so that they can be easily straightened before final assembly.

6.4 Lamellar Tearing

Although lamellar tearing is rarely the cause of service failures in the welded construction industry, it is an important phenomenon that must be understood so that its occurrence may be minimized. Lamellar tearing develops in susceptible material as a result of high through thickness strains. These strains are the result of normal weld metal shrinkage, especially in restrained joints. Lamellar tears may be found in the base metal in planes generally parallel to the rolling direction of the plate. The location of the tear may be subsurface just outside the heat affected zone and it may or may not propagate to the toe of the weld to allow visual detection. If the tear is not visible than ultrasonic testing is the most effective method of detection. Refer to [lamtear1.pcx]. It is the anisotropic nature of rolled shapes and plates, inherent in their manufacturing process, which renders the steel less ductile in the through thickness direction thereby leaving the parent metal susceptible to lamellar tearing. Generally, the type, shape and distribution of nonmetalic inclusions such as sulfides, silicates and aluminum oxides control this reduction in ductility. The inclusion content depends upon many factors including the steel making process, deoxidation and final sulphur controls, ingot pouring, and rolling practices. The extent of inclusions, which have been elongated during the rolling process, close to the surface determines the susceptibility of the material to lamellar tearing.

For tearing to occur the following three conditions must be satisfied:

1. the parent metal must have poor through thickness ductility,

2. the weld orientation must be such that strains are directed through the plate thickness, and

3. strains due to weld metal shrinkage must be developed and potentially enhanced due to restraint.

If the above three conditions are satisfied then tears may develop through a process of decohesion originating in an area containing string-like nonmetalic inclusions. In their mature state, lamellar tears exhibit a unique step-like appearance through the joining of long terraces (cracks linking together individual separations) parallel to the rolling direction with short transverse shear walls. See [lamtear2.pcx]. The tear surface is fibrous or woody in appearance, characteristic of a low ductility fracture. The tear may run along a joint for a considerable distance, and have a width approximately equal to the size of the weld.

The risk of lamellar tearing could be virtually eliminated with the production of steel employing isotropic properties however the expense involved presently in accomplishing this task makes this an unrealistic solution. As an alternative the designer could specify steel with improved through thickness ductility in critical regions of the structure or judiciously select a structural arrangement which is not conducive to tensile stresses in the through thickness direction. Another alternative would be to replace welded tee joints with forged monolithic elements or electroslag splicing. Unfortunately, these latter two solutions would also prove quite costly.

The fabricator may utilize the following techniques to reduce the risk of lamellar tearing:

1. select a structurally acceptable, more ductile welding consumable.

2. butter the joint with a ductile layer using lower strength electrodes. See [lamtear4.pcx].

3. remove susceptible material in parent metal and replace with ductile weld metal. See [lamtear7.pcx].

4. utilize fillet or partial penetration welds in preference to complete penetration welds. See [lamtear8.pcx].

5. symmetrically deposit individual weld passes to balance double sided joints and minimize strains. See [lamtear3.pcx].

6. select a joint geometry to minimize through thickness strains. See [lamtear5.pcx] and [lamtear6.pcx].

7. select a fabrication sequence to minimize strains due to restraint at critical locations.

8. judiciously supply preheat temperatures higher than normal provided additional restraint is not created.

9. where possible, chamfer the thickness elements exposed to the through thickness stress transfer.

10. use soft wire spacers to reduce shrinkage stresses during weld cooling.

The occurrence of lamellar tearing is predominant in tee and corner joints. Design emphasis in these instances should be placed on providing optimal component flexibility. On the other hand, fabrication emphasis should be placed on providing minimal shrinkage stresses.

Additional control of lamellar tearing is provided through UT inspections both before welding and after weld cooling.

7 Designing For Welding

@1[7.1 Steel Grade Selection] can be an important factor in a welded design. A few of the more important parameters are considered in this module.

@1[7.2 Design Considerations for Improved Service Behavior] are also studied.

@1[7.3 Detailing to Achieve Practical Welded Fabrication] provides many important design considerations.

7.1 Steel Grade Selection

The following main factors must be considered when choosing the grade of steel for a particular project:

(a) Availability

(b) Strength (yield and ultimate)

(c) Ductility

(d) Weldability

(e) Service conditions (static vs. dynamic load)

(f) Environmental conditions (corrosion resistance)

(g) Cost

Each of the above will be examined with the observation that the one over-riding concern is probably cost.

7.1.1 Steel Grade Availability

The following items must be evaluated in connection with steel grade availability:

(a) **Amount of steel needed:** If large, the fabricator may order directly from mill. Given some time, most structural grades will be available.

(b) **Time factor:** When is the material needed? If the construction time frame is very short, mill delivery should be avoided. It is an advantage to deal directly with the local steel service centres.

(c) **Grade, shapes and sizes:** Are the required shapes and sizes available in the steel grade(s) wanted? Note, for example, that Q & T high-strength steel (Fy = 700 MPa (100 ksi)) is only made in plates with thicknesses less than 50 mm (2"). If welded shapes are specified, are they available in the requisite grade?

(d) **Cost factor:** Can money be saved by changing grade of steel, choices of steel shapes, etc.? The many premiums that apply due to length, size, trimming, rolled vs. welded, special grade requirements, etc., must be considered.

7.1.2 Steel Grade Strength

In most cases for civil engineering structures, 250, 300 or 350 MPa (36, 44 or 50 ksi) yield strength steel will be the best choice. Consider using different grades of steel for different structural elements: Higher strength steel for beams than for columns is often justified. Naturally, this requires that the amount saved in steel tonnage will at least offset the higher price of higher strength steel. Will a reduced structural dead weight be of importance, for example, with regard to foundations? Note that higher strength steel may not be justified if deflection criteria govern the design.

7.1.3 Steel Grade Ductility

Material ductility, or its ability to deform with neither local nor total failure, is one of the primary advantages of steel construction from a behavior viewpoint. This is particularly true in regions where seismic performance is important. Similarly, operating temperatures must be considered: Material ductility **and toughness** are important factors when fitness-for-purpose evaluations are made.

What type of structure is to be built? Depending on members and connections, what appears as satisfactory ductility under uniaxial service conditions may not be so if biaxial or triaxial states of stress are considered. This is especially important in the design of connections, where degrees of restraint will impose severe ductility limitations. Many welded joints demand high deformability of the parent material.

7.1.4 Steel Grade Weldability

There are few steels that cannot be welded, but a number can only be welded by specially developed, expensive processes. This is particularly true in the case of the steel grades that were used in older (pre-1930, approximately) structures. For renovations work on older buildings or bridges it is therefore better to avoid welding altogether.

A good measure of the weldability of a steel is the **carbon equivalent**, CE:

$$CE = C + \frac{Mn}{6} + \frac{Si}{6} + \frac{Cr}{5} + \frac{Mo}{5} + \frac{V}{5} + \frac{Ni}{15} + \frac{Cu}{15} < 0.48$$

where C = carbon content (%), Mn = manganese content (%), Si = silicon content (%), Cr = chromium content (%), Mo = molybdenum content (%), V = vanadium content (%), Ni = nickel content (%), and Cu = copper content (%). It is recommended that CE be less than 0.48, to assure good weldability with any of the commonly used welding methods.

Most structural steels satisfy the weldability requirement. Only the general construction purpose grades (designation G) are not considered weldable in the above sense.

7.1.5 Steel Grade Service Conditions

It is very important to consider the anticipated service conditions of the structure. Will it be subjected to static (or essentially static) loads only, or can it be expected to undergo fatigue-type (cyclic) loading? The requirements to a steel structure will be quite different under such conditions. Are service temperatures expected to stay within a relatively narrow range, or can large fluctuations occur? It is especially important to know of any low temperature conditions; the demands of the material in terms of fracture toughness are crucial to the successful performance of the structure. Naturally, fracture tough steel grades are more costly than others.

In welded structures, the types and sizes of possible defects must be considered in the steel grade selection. This goes hand in hand with the service conditions (load, temperature), as well as the fact that welded joints often pose two- or three-dimensional degrees of restraint on the surrounding material. Table_4 below gives critical defect size as a function of type of steel and loading condition. It relates steel groups and critical stress intensity factor, Klc, to the size of a critical defect. Also indicated are the types of loading conditions for which these data are applicable. Note, that in general, increasing Klc/Fy indicates a steel with increasing ductility.

Table_4: Critical Defect Size Chart					
Material Group	Typical Steel Grades	Klc/Fy			
		0.5	1.0	2.0	3.0+
C-Mn	ASTM A36 G40.21-44W	-	*0.3"	**1.2"	***2.8"
C-MN + Alloy (HSLA)	ASTM A572 ASTM A588 G40.21-50A	-	**0.3"	**1.2"	***2.8"
Q & T High Strength	ASTM A514 G40.21-100Q	-	**0.3"	***1.2"	-
Q & T Very High Strength	Fy = 200 ksi	**0.1"	**0.3"	-	-

\+ Essentially the static load range when Klc/fy = 3.0

* Dynamic loads

** Static and dynamic loads

*** Static loads.

7.1.6 Steel Grade Environmental Conditions

The service conditions of a structure also should consider environmental factors. In particular, if the structure will exist in a highly corrosive atmosphere (steel mill, chemical or petrochemical plant, some mining equipment, etc.), a high corrosion resistance steel grade may be chosen. Is the cost of maintenance high? Again, a high corrosion resistance steel may serve well: The weathering steels are particularly useful under such conditions.

Will the structure serve under particularly elevated temperatures (certain steel mill operations, cement factories, etc.), or will a potential fire be especially serious? Both of these criteria have an impact on the strength and ductility of the material, as well as long-term metallurgical effects. It is advisable to contact a metallurgist when such factors are considered.

7.2 Design Considerations for Improved Service Behavior

This section of the hyper-text system covers some of the basic in service considerations as they relate to the design stage.

@1[7.2.1 Design - Service Conditions] briefly covers potential in service problems

@1[7.2.2 Improvement of Service Conditions] discusses points to consider at the design stage.

@1[7.2.3 Weld Strength Computations] covers the basics of proper consideration of weld strength.

7.2.1 Design - Service Conditions

Structure design under static loading presents relatively minor problems, however service temperatures may be important. What might be considered a static load at 20 degrees C may not be treated as such at -20 degrees C.

The design of dynamically loaded structures must incorporate the fatigue characteristics of both the material and the structure.

For service conditions it is important to note the relative influence of weld defects. Weld defects influence fatigue life in the following **descending** order of severity:

(1) Weld cracks

(2) Undercut

(3) Lack of fusion; poor fit-up

(4) Lack of penetration

(5) Slag inclusions

(6) Porosity

Structural strength generally remains relatively unaffected, until the fatigue crack has grown to substantial proportions (recall the critical defect size chart under [steel grade service conditions]). However, by this stage the crack may be growing very rapidly, thus becoming a serious strength and integrity problem for the structure.

Lamellar tears are much more serious for fatigue loading than for static loading. However, the joint and member should always be analyzed to determine stress and strain conditions in the vicinity of the tear.

Base metal laminations may be analyzed as a lamellar tears where structural strength and integrity are concerned.

7.2.2 Improvement of Service Conditions

The following points should be considered when selecting a material:

(1) Higher fracture toughness leads to **some** improvement in fatigue life.

(2) Higher fracture toughness leads to **clear** improvement for fracture conditions.

(3) Semi or fully killed steel reduces susceptibility to lamellar tearing, but is generally not sufficient to **guarantee** no lamellar tearing.

Structural service conditions may be improved if the following fabrication and welding process considerations are made:

(1) Better quality control leads to a reduction of the size of flaw that is detected. Hence, fatigue life is increased.

(2) Choice of appropriate welding technique for the job at hand will improve quality.

(3) Careful control of pre-heat, interpass, and post-heat conditions will reduce the risk of obtaining unsatisfactory welds.

(4) Pre-qualified joints **do not** guarantee that the weld will be flawless. It only means that the welder will be able to make the weld properly.

(5) The proper sequence of weld passes, and of different welds in the assembly, is a significant factor in achieving an improved product.

In the design stage it is important to note that improved service behavior is not only a function of stress, but also of stress range. Both deserve ample consideration because the former influences fracture conditions and the latter controls fatigue behavior. The following criteria should be incorporated into the structural evaluation:

(a) Lower stress improves **fracture** behavior.

(b) Lower stress range improves **fatigue** behavior.

(c) Static service load is not normally sufficient to propagate a crack.

(d) Joint geometry that produces a condition of high restraint (to weld metal contraction) should be avoided. It is therefore important to obey the following rules:

(1) Do not overmatch weld metal and base metal.

(2) Do not use intermittent (stitch) welds in a dynamically loaded structure.

(3) Avoid intersecting welds.

(4) Use stiffeners **only** if the analysis indicates that they are needed.

(5) Specify an erection or construction procedure, if such can be instrumental in reducing high restraint conditions for welds.

(6) Orient welds such that contraction strains are imposed on the base metal in a longitudinal direction.

(7) Be aware of service conditions for the structure. Localized effects that have little or no importance in the structural design may produce fatigue cracks. (Example: Flexible web of a bridge girder to which a transverse beam has been attached.)

(e) A lamellar tear develops when weld metal contraction is restrained. Service loads will not produce such tears.

7.2.3 Weld Strength Computations

The behavior of different types of welds under load can be just as important to the design stage as weld stresses and capacities. Cost will also play an important role at this level, since some weld types are clearly more costly than others and the selection of a particular weld profile will influence the layout of a joint. For example, partial or full penetration welds are expensive. Although these welds are sometimes necessary to develop the full capacity of the base material, such as full-strength flange splices of a plate girder, there are many applications where these costly welds are simply not needed. As an example, it is noted that some designers prefer to call for full penetration welds between bearing stiffeners and the girder to which they are attached. Fillet welds would serve this purpose at least as well.

Weld orientation can also be important. The required **strength** of a fillet weld is obtained whether the weld is placed parallel or perpendicular to the primary load direction. Its **behavior**, on the other hand, it much improved when the weld axis parallels that of the load. In particular, the ductility of such a welded joint is significantly enhanced. Note that although it is generally agreed that fillet welds are 30 to 40 percent stronger when loaded in the transverse direction, the governing codes do not recognize this increase in strength. This may be partly attributed to the fact that with increased strength generally comes decreased ductility. As a general rule, therefore, it is normally preferable that the axis of a weld be oriented parallel to the main direction of force. This also provides for a better force transfer in the connection.

Strength of Butt Welds

Assuming matching conditions, the strength of butt welds is relatively easy to determine. Whether the weld is of the full or partial penetration type, if the joint is loaded in compression perpendicular to the effective throat, the strength is based on that of the base metal. That is, in no instance can the welded joint be ascribed a strength higher than the connected base metal.

Full penetration welds loaded with tension normal to the effective throat are also limited in strength to that of the base metal. The strength of partial joint penetration groove welds, however, is governed by the smaller of the weld metal and base metal capacity. This is similar to the governing criterion for design for shear. Certain restrictions apply to the use of partial joint penetration groove welds in a dynamically loaded structure.

A special note must be made with regard to butt welds subjected to tension or compression parallel to the weld axis. It is implied that no load transfer takes place across the weld; the joint simply acts to hold two components together to make them act as a unit. In this case the weld strength is governed by that of the base metal. Any force transferred across the tension or compression loaded weld will be carried by shear.

The factored shear strength of a butt weld is computed as the smallest of:

(a) **Base metal strength:**

Vr = 0.67 [phi] Am Fy (1)

and

(b) **Weld metal strength:**

Vr = 0.67 [phiw] Aw Xu (2)

All of the variables are defined in @1[W_11.3 Design Provisions] for statically loaded structures or in @1[W_12.3 Design Provisions] for dynamically loaded structures.

Equation (1) depicts the strength of the fusion or interface between weld metal and base metal. [phi] = 0.9 is the performance factor for the base metal. The constant of 0.67 in Eq. (1) represents the multiplier that transforms tensile yield strength into shear yield strength, thus:

Shear yield strength = (tensile yield strength) / sqrt(3) = 0.57 Fy

It is further increased by approximately 10%, to 0.67 Fy, to account for strain hardening and other material factors.

Equation (2) depicts the shear strength of the weld metal itself. [phiw] = 0.67 is the performance factor for the weld metal.

It is seen that since Aw = Am **for butt welds**, the governing equation of (1) and (2) can be found by determining the smaller of 0.60 Fy and 0.45 Xu.

The constant in Equation (2) accounts for the relationship between the ultimate tensile and shear "yield" strengths of the weld metal, as well as a modifier that is incorporated to ensure that the weld will not fail before the member itself.

Strength of Fillet Welds

The strength of fillet welds loaded in tension parallel to their longitudinal axis is controlled by the base metal strength. However, no load is to be transferred across the weld itself. If the weld has to transmit load, it will be carried in shear.

The shear strength of a fillet weld is governed by the smaller of the capacities computed from Equations (1) and (2). However, the definitions of fusion area and effective area of a fillet weld are different from those of a butt weld. Refer to @1[W_11.3 Design Provisions] and @1[W_12.3 Design Provisions].

The capacity of the fusion face is governed by the shear yield strength of the base metal, including an allowance for strain hardening and local plastification.

The weld metal shear strength of a fillet weld is governed by Equation (2), but where the effective throat area, Aw, is governed by the root measure of the weld. With a fillet weld leg length of a, the root measure is traditionally computed as

$$a(\text{root}) = a \cos 45 = 0.707\, a \qquad (3)$$

Naturally, this assumes that the weld has been placed correctly, with equal leg length on both faces, and no concavity of the weld surface.

It should be noted that Equation (3) gives the root measure of a manually placed fillet weld (SMAW, e.g.). For automatic welding such as submerged arc (SAW), using {0.707 a} will be conservative, since the depth of penetration of an automatic process is significantly larger than a manual weld. This increase in penetration is acknowledged in @1[W_4.3.2 Fillet Welds].

7.3 Detailing to Achieve Practical Welded Fabrication

Few designers or fabricators are completely up-to-date on modern welding technology or welded structural design. As a result, many of the errors made by engineers today are caused by a lack of awareness of state-of-the-art design principles and technological advances. An important point of concern with regard to the above is: "Anything done five years ago in welding design or fabrication could be obsolete" [Blodgett1980]. Therefore, any welded detail should be re-evaluated to determine:

(1) if it incorporates new knowledge about how to handle forces;

(2) if it takes advantage of new provisions in governing codes;

(3) if it is compatible with new advances in welding equipment, electrodes, processes, and procedures; and

(4) if it offers opportunity to minimize costs through the use of more precise determinations made possible by new and more powerful software.

Many weld failures over the years have been the result of certain oversights. The following design principle violations describe, with example, the results of these oversights:

@1[07.3.01 Failure to provide a path for transverse force] to enter parts of the member or section that lies parallel to it.

@1[07.3.02 Failure to provide a component when a force changes direction]

@1[07.3.03 Failure to recognize a major force resulting from a load]

@1[07.3.04 Failure to alter detailing when welding replaces riveting]

@1[07.3.05 Failure to recognize strain compatibility]

@1[07.3.06 There are no secondary members in welded design]

@1[07.3.07 Do not rely on welder skill in place of proper design] to provide strength and performance of the weldment.

@1[07.3.08 Do not place welds in regions of bending]

@1[07.3.09 Failure by compressive fatigue load] in a region containing residual tensile stress.

@1[07.3.10 Failure to design against lamellar tearing]

@1[07.3.11 Redundancy with diagonal stiffeners] can be costly and unnecessary.

@1[07.3.12 Failure to consider not-in-service loads]

@1[07.3.13 Failure to understand matching electrode conditions]

7.3.1 Failure to Provide a Path for Transverse Forces

Example 1:

Three figures, linked to in the text below, help to illustrate a very simplistic principle. In order to transfer a force into the load carrying portion of a member or structure, a suitable path must be provided. @1[detail1.pcx] shows a lug welded onto the bottom flange, in line with the web, to transfer the load into the web. @1[detail2.pcx], on the other hand, has the lug mounted perpendicular to the web and therefore web stiffeners are needed to transfer the load into the web. Without the stiffeners, an applied load would result in an uneven distribution of stresses in the weld because of the more flexible flange. Note that the stiffeners are not welded to the top flange. Finally, @1[detail3.pcx] shows a lug welded onto the flanges which, in this instance, are the load carrying sections. The lug is not welded to the web here because that would serve no purpose in transferring the force. While it seems so elementary it is hardly worthy of mention, failure to provide such a path possibly leads to more structural design problems than any other cause.

Further examples of force transfer show lugs welded to box sections. @1[detail4.pcx] shows an internal diaphram which, of course, is only realistic for welded box sections. The diaphram would be welded in before welding on the top plate. @1[detail5.pcx] illustrates a lug shaped like a sling and welded directly to the sides of the box section. Note that the lug is not welded to the bottom flange of the box. In @1[detail6.pcx], the lug is designed to transfer the force directly into the two webs from below. This method is very efficient.

Two methods of attaching lugs to circular members are illustrated here. @1[detail7.pcx] demonstrates how a sling shaped lug might be attached to a circular tube. The lug is welded onto the tub in the area most parallel to the force, the two side quadrants of the circular section.

Theory suggests that these two side quadrants carry 82 percent of the shear capacity of the tube. As a note of interest, the top and bottom quadrants are expected to carry 82 percent of the moment capacity. [detail8] illustrates an alternative lug that is somewhat simpler to fabricate.

Example 2:

@1[detail9.pcx] and @1[detail10.pcx] show an actual problem that resulted from welding an attachment to the center sill of a piggyback railroad car. Unfortunately, because the designer overlooked the necessity of providing a proper path for the transfer of forces, cracks developed in the web. This example applies directly to attachments on structural box girders.

The attachment here is a bracket, or two, used to carry a 500-pound air compressor unit. Note that there are no interior diaphragms. The vertical force from the weight of the tank is transferred as moment into the bracket, creating out-of-plane bending at the web. As illustrated, there are no ready pathways to transfer the horizontal bending forces into the flanges of the open box section. As a result, the flexible web soon developed fatigue cracks, @1[detail10.pcx].

Two possible means are suggested for correcting the faulty design. One solution, @1[detail11.pcx], is to shape the bracket so that it runs the full height of the web and can therefore be welded directly to the center sill flanges. Alternatively, @1[detail12.pcx], an internal diaphragm could be welded into the assembly thereby stiffening the flexible web where necessary. The diaphragm is welded to both flanges and to the web in question. There are now paths for the bending forces to get to the flanges.

Example 3:

@1[detail13.pcx] illustrates a floor beam framing into a main girder on a railroad bridge. A bracket is welded to the top of the floor beam and bolted to the transverse stiffener of the girder. The forces should eventually end up in the girder flanges but there are no suitable paths because the stiffener is not welded to the flanges of the girder. The tensile force is taken by the bracket, and then by the stiffener, after which only the web of the girder can provide reaction. The out-of-plane bending force on the web is too great, and it cracks under fatigue loading.

7.3.2 Failure to Provide a Component When a Force Changes Direction

Another important consideration occasionally overlooked by structural designers is the need to provide a component when a force changes direction.

Example1:

@1[detail14.pcx] shows a knee, such as is used in building frames, with no provision made for the component force, Fc. In this instance, the component force is high at the web and low, almost zero, at the outer edge of the flange thereby producing an uneven distribution of force in the flange. The limitation here on the component force (ie. only carried by the web) also limits the flange force thereby limiting the capacity of the overall connection. Now consider @1[detail15.pcx]. The diagonal stiffener provides the necessary component force to handle the change in direction of the top and bottom flange forces. The forces are transmitted and balanced through the welds joining the diagonal to the flanges. The diagonal does not have to be welded to the web for the transfer of forces, however, if there is a concern about buckling, the stiffener should be designed as a column with appropriately spaced intermittent welds. Additionally, if the knee is exposed, such as in a bridge, a continuous seal weld might be needed to keep out water. Note that the size of the force balloon in each figure gives an indication of the capacity of the knee.

Example 2:

This example illustrates how a crack developed in a part of a structural frame for a cantilevered balcony. Unfortunately, during fabrication of the girder, no thought was given to the component force and no stiffener was specified. To compound the error, when the welder placed the intermittent welds, it just happened that no weld was placed at the point of change in direction of the flange. The top of @1[detail16.pcx] illustrates the problem portion of the frame. From observation, we can see how large tensile forces might be introduced into the web during service conditions and how, subsequently, a crack might soon develop. The bottom of [detail16] shows how brackets should have been welded on where the flange changes direction and component forces are developed.

Example 3:

Structural designers should always be aware of the different requirements for curved sections subject to moments.

The large radius of flange curvature in the structural knee of @1[detail17.pcx] will transfer much smaller forces into the flange-to-web weld than the small radius of flange curvature in the mechanical press of @1[detail18.pcx]. In the structural knee, the unit radial forces, which are developed from the curvature, are compressive and therefore push the inner flange against the web. The welds attaching the inner flange to the web transfer little or no force at all; they merely hold the parts together. On the other hand, the unit radial forces in the mechanical press are relatively high because of the lesser radius of curvature, and are in the opposite direction. With the inner flange now in tension rather than compression, the flange-to-web weld must now bear the full radial force throughout the length of curvature. Occasionally these welds break because they are undersized.

Three methods of handling component forces in a press frame are shown in @1[detail19.pcx]. In all three illustrations, the inner flange develops component forces that must be dealt with. The press frame on the left of the figure uses a diagonal to accomplish this while the middle frame and the one on the right use diaphrams.

Example 4:

@1[detail29.pcx] shows a portion of an arm used on large concrete hauling trucks to carry a third axle which was put down when the truck was loaded. This example illustrates an actual problem that occurred as a result of transferring radial force.

The arm was made from 1/4 inch plate and welded with 1/4 inch fillets. Under service conditions, cracks were discovered in the fillet welds. Further analysis demonstrated that the applied bending moment was subjecting the inner flange to tension. This axial tension was generating a radial force directed toward the center of curvature of the arm. This radial force, acting across the width of the flange, was only resisted by the two webs. Therefore, the flange went into curvature and imposed bending moments on the web-to-flange fillet welds. These bending moments put the root of the fillet weld into tension which subsequently caused the weld to crack.

One solution to the problem was to place a second fillet weld along the inside of the web-to-flange connection for the full length of the arc and past the point of tangency. The induced bending would then put the face of the inside fillet weld into tension which is quite acceptable for reasonable loads. This, of course, means that the top flange would have to be welded last.

7.3.3 Failure to Recognize a Major Force Resulting From a Load

Example 1:

The large trailer illustrated in @1[detail30.pcx] will serve as a case in point with regard to not recognizing a major force resulting from a load. The trailer is composed of two long beams joined with cross beams, and two tongue assemblies tied into the main end cross beams.

The problem was that in the original design of the tongues, no attention was given to the effects of torsion. As indicated in @1[detail31.pcx], the welds broke where the main end cross beams tie into the longitudinal beams. With the open sections and the considerable length of the cross members, torsion caused the beams to bend horizontally (ie. in the plane of the deck) at the weld ends. The solution, of course, was to increase the torsional stiffness of the sections where the torque is applied. This meant boxing in the two main end cross beams to produce a closed section thereby eliminating the bending action on the end welds. @1[detail33.pcx] illustrates the resulting solution.

Example 2:

This second example involves the tailgate of a garbage truck. @1[detail32] shows the frame of the tailgate.

Two hydraulic cylinders are used to pull the gate forward into the truck and compress the garbage. Under uniform operating conditions the design of the tailgait was satisfactory, however, when a solid object jammed up one side of the mechanism while the other side continued to move, the torsional effects induced into the tailgate proved the design to be faulty. The primary structural element contributing to the faulty design was the 6 inch diameter tube running along the length of the tailgate. The tube was essentially the only member providing torsional stiffness to the frame and by itself was not enough to resist the torque induced by one of the 30,000 pound hydraulic cylinders. In service, the welds between the tube and the plate broke repeatedly under an 80,000 psi shear stress developed by the twisting action. Because the frame without the tube was sufficiently flexible, the easiest solution to this problem was to simply remove the tube.

7.3.4 Failure to Alter Detailing When Welding Replaces Riveting

It is a well known fact that the continuous nature of a welded assembly facilitates crack propagation. Many riveted or bolted details of the past were sufficient because these types of connections act as "crack arresters". However, simply replacing the riveted fastener with a weld fastener, without altering the detail, will most likely result in weld and plate cracking.

The experience acquired when designers switched from riveted to welded ship construction during World War II, involving brittle fracture which occasionally resulted in the ship actually splitting in two, drove home the need to change detailing concepts and to follow carefully planned welding procedures. @1[detail20.pcx] shows both a riveted and a welded connection between the ship's deck and its side shell. If a crack should develop in the deck plate of a riveted ship, it would be stopped at the first riveted joint and would not proceed through the gunnel angle and into the side shell. However, if the connection was welded, there would be a path for propagation from one plate to the other. This discovery lead to the realization that a riveted joint was more "forgiving" when the design was poor. For example, riveted square corner hatch openings, shown on the left in @1[detail21.pcx], had to be converted to round corner openings with welded design to prevent stress raisers and the initiation of cracks that might propagate the length of the deck. Corner stress raisers were of no major consequence with riveted decks; the crack would stop at the first riveted joint. The same considerations apply when changing joining methods in bridge and building construction.

7.3.5 Failure to Recognize Strain Compatibility

Unrecognized strain compatibility can lead to serious problems.

@1[detail22.pcx] illustrates a stringer being run through and supported by a box girder bent. A slot has been cut into the web of the box girder to receive the lower flange of the stringer, and the designer has decided to weld this to the web of the box girder at the slot. This is a mistake because the web of the box girder is stressed from bending and will elastically strain. The lower flange of the stringer will resist this strain and induce high stresses into the weld.

@1[detail23.pcx] gives a perspective on the potential stresses that could arise if the stringer flange is welded to the box girder web. Assume that the web in the region of the slot is stressed to 20 ksi, then the strain, assuming an elastic modulus of 30000 ksi, will be 0.000667 in./in.. Under this strain, a 20 inch slot in the box girder will elongate approximately 0.0133 inches. If welded, the flange of the stringer must also elongate this much. The question then becomes: What tensile force will be required to elongate the flange 0.0133 in.? Referring to @1[detail23,pcx], we can see that the resulting tensile force depends on what width of flange is taken perpendicular to the box girder web. A 1-inch wide strip would induce 20 ksi into the weld, a 3-inch strip ... 60 ksi, and a 6-inch strip ... 120 ksi. Carrying this procedure further would obviously impose forces and stresses that are unrealistic. It would be impossible for any weld to withstand the force when the web of the box section is stressed just 20 ksi in service. If the weld did not break, it would pull out of the web plate.

The solution to the problem would be to let the bottom flange of the stringer ride unattached to the web in a smooth flame-cut and ground slot, or to use an attachment as shown in @1[detail24.pcx], where a wide plate cut to a radius of 24 ins. is welded between the web of the box and the flange of the stringer.

7.3.6 There are No Secondary Members in a Welded Design

Both the designer and the fabricator must be aware that there are no secondary members in welded design. It is quite feasible for an interrupted backing bar to cause a main member to crack.

Example 1:

The codes today specify that backing bars must be continuous for their full length (W59-M1989 clause: 5.4.9 Continuity of Backing). Despite what the code says, many situations have arisen where the designer felt continuity was not required. @1[Detail25.pcx] shows an orthotropic deck for a bridge. The deck was welded to the stringers first and then short pieces of backing bars were tacked in between to back the groove weld that would be placed the width of the deck. It was soon discovered, during field welding, that transverse shrinkage cracks were occasionally occurring in the groove weld over the interrupted backing bars. The detail was subsequently changed to provide a continuous backing bar laid in slots cut into the stringer webs.

Example 2:

@1[detail26.pcx] illustrates a detail near the lower flange of a deep bridge girder. A horizontal attachment was to be placed across an existing transverse stiffener and then welded as shown. One problem with the location of the attachment was that there would be high tensile bending stresses in the web near the weld. A second problem was that, because there was no room for overhead welding, backing bars would be needed to weld the attachment to the web. The real problem now arises because the backing bar will have to be interupted at the transverse stiffener. Under fatigue loading, the notches resulting from the interuption will cause the weld to crack and subsequently propagate into the web and then down into the bottom flange. An analogous scenario to this detail would be to make a 1/4 inch saw cut directly into the tension flange of the girder.

@1[detail25.pcx] illustrated how transverse shrinkage cracks could result from interrupted backing bars, and @1[detail26.pcx] showed how fatigue cracking could develop. Brittle fracture is another consequence of welding over the notches created by interrupted backing bars. This is discussed in example 3.

Example 3:

In this example, the combination of notches and cold temperatures led to the brittle fracture of a box beam boom for a large earth moving machine. The box beam was fabricated from two bent plates joined together with groove welds using backing bars. See @1[detail27.pcx]. Unfortunately, the continuity of the backing bar was lost when someone in the shop felt it was sufficient to merely use pieces of backing bar without welding them together. Brittle fracture of the boom occurred on a cold February morning during transportation down Michigan Avenue in Chicago. The boom, under its own weight, snapped in two. A notch, plus low temperature, had led to brittle fracture. Had the pieces of scrap backing bar been welded together and ground before use, there would have been no notches, and thus no problem.

Example 4:

The cautious designer will always check for undesirable side effects which may result from the addition of a new member to a previously designed structure. @1[detail28.pcx] illustrates a case in point. The problem was to design a structural system to carry two large pipes between two buildings. The designer selected a fabricated girder as the load-carrying member. Its bending stress under load was calculated to be about 10 ksi. He then chose to weld 1/4" x 60" x 48" panels to the girder, as shown in the figure, to enclose the structure. Unfortunately, the designer neglected to include the panel additions as part of the load-carrying member and subsequently neglected to recalculate for bending stress. A simple recalculation would have shown the stress to be lowered from 10 ksi to 5 ksi, and further calculation would have found the critical buckling stress of the 1/4" x 60" x 48" panel to be 1.5 ksi. As would be expected, the panels buckled during erection. The structure had adequate strength - didn't collapse - but the buckled plates are a constant reminder that there are no secondary members in a weld design.

Example 5:

A box member used as the central part of a so called "log roller" for washing gravel is depicted in @1[detail34.pcx]. The box is fabricated by welding four angles together. During operation, the corners of this assembly are subjected to high rates of wear as the box rolls in the gravel. To protect the corners of the angles against this wear, it was suggested to weld on steel reinforcing bars. This idea worked well when the bars were continuous for the full length of the box member however, problems arose when random lengths were butted together and an intermittent weld was placed across the joint. The notch between the reinforcing bars acted as a stress raiser resulting in weld cracking during operation. Here again, ignoring the importance of a secondary member destroyed the integrity of a good design.

7.3.7 Do Not Rely on Welder Skill in Place of Proper Design

Too often the designer specifies a joint detail where the skill of the welder determines the strength and performance of the weldment. The designer must depend upon the skill of the welder for integrity of the assembly, but he should not detail in such a manner that it is the welder's skill that determines the strength and performance of the structure. At times it may even be necessary for the designer to tell the welder "this way, and only this way, it must be done."

Example 1:

@1[detail35.pcx] shows a cover plate that is wider than the beam flange. This detail contains a point of weakness, especially under fatigue loading, where the two separate planes of fillet welding may have to be joined. Note that W59-M89 Clause 11.4.7 Lap Joints (Statically Loaded Structures) only permits the joining of opposite side fillets when an all around seal is required and

Clause 12.4.7 Lap Joints (Dynamically Loaded Structures) does not permit the joining at all. This point of weakness demands the welder to properly join the longitudinal and transverse fillets. The problems that would arise as a result of producing a crater, concave end, or notch at the point of weakness are far too important to be gambling on such a detail. Instead, the detail in @1[detail36.pcx] should be used where the cover plate is narrower than the flange and the fillet weld is continuous in one plane. This detail will help to eliminate the question of welder performance.

Example 2:

@1[detail37.pcx] illustrates a gearbox used in mining machinery. You can see many points of weakness where welds on opposite sides of a plane are tied together. The assembly did not last long in service before it began to break up. Many of the points of weakness have been eliminated in the preferred detail shown in @1[detail38.pcx].

Example 3:

@1[detail47.pcx] illustrates a detail which leaves much to the welder. @1[detail48.pcx] is a substantial improvement in detail because the performance of the structure does not hinge on the knowledge, skill, and convenience of the welder.

The detail is the support bracket for a short beam section. A two-span continuous plate girder cantilevers out beyond a pier to support the short suspended beam. The bracket will be loaded in bending with its top outer fibre in tension. The outer fibre in @1[detail47.pcx] is comprised of the edge of a flame-cut plate and either the start or stop of the fillet weld. There is an even chance that the weld will terminate with a crater at the point of high bending stress. Perhaps of minor importance is that the detail provides no horizontal stiffness for the bracket.

@1[detail48.pcx] requires a little more material and work, but eliminates the chance of design failure. The outer fibre in bending now becomes the top smooth surface of the flange plate. There is no flame-cut edge of a plate or weld crater to affect performance and there is good horizontal stiffness for the bracket.

7.3.8 Do Not Place Welds in Regions of Bending

Example 1:

Placing welds in regions of bending is another mistake often made by designers. @1[detail39.pcx] illustrates a case where welds were placed in a region of high bending. Two welded angles were used to support a hydraulic motor. The motor was fastened into the outstanding legs while the hold down legs were bolted to the base. An uplift force on the pinion of the motor put the hold down bolts in tension which, because of the eccentricity, demanded a high moment from the welded angle corners. With this detail, there was no way of eliminating the moment and the welds broke in service.

To correct the problem, the detail attaching the motor to its base had to be modified. One solution was to have the base metal plate run the full length of the motor assembly, as shown in @1[detail40.pcx]. Most of the moment developed from the eccentricity of the hold down bolts is now taken by the base plate. The support welds are now subjected to tensile uplift instead of bending.

Example 2:

This example makes use of a principle which is well known in the structural field, namely, one can avoid problems with bending moments by placing welds at points of inflection (ie. zero bending moment).

The middle of @1[detail41.pcx] illustrates the moment diagram from internal pressure in the pressure vessel. Welding at the points of inflection would mean placing the welds as shown in the top of the Figure. This method of fabricating will produce a stronger vessel than one made with only two welds, seen in the bottom of the figure. If the internal pressure is high, the cost of twice as much welding may be money well spent.

7.3.9 Failure by Compressive Fatigue Load

There have been a few rare instances where a designer has inadvertently imposed a compressive fatigue load onto a region of residual tensile stress. A weld zone will remain in a state of residual tensile stress providing it has not undergone any form of stress relief. If repeated compressive loads are then imparted onto this region, it will bounce back and forth between a state of stress relief and a state of residual tensile stress. If the compressive loads are severe enough, the weld will eventually break by tensile fatigue.

Example 1:

This first example involves a welded, shrink-fitted gear which failed in service because of compressive fatigue loading. As illustrated in @1[detail42.pcx], the rim was first shrink-fitted to the disc and then fillet welded on both sides all the way around the disc. This resulted in very high tensile residual stresses in the welds.

Unfortunately, during service loading, the rim fell off the disc. Subsequent investigation showed excessive wear at the root of the teeth which suggested that the meshing gears had been operating too closely together. This observation led to the cause of, and the solution to, the problem. During operation, the fillet welds received repeated compressive loading because the gears were out of alignment. Because these welds were already in a state of tensile residual stress, the repeated loading induced tensile fatigue and failure eventually occurred at angles of 10 and 20 degrees from the rim. See @1[detail43.pcx]. The angle of the failure plane further reenforces the theory of tensile fatigue because the maximum tensile stress under transverse loading occurs on the 22.5 degree plane as opposed to the maximum shear stress which occurs on the 67.5 degree plane. See @1[detail44.pcx]. Furthermore, if the weld had failed as a result of torsion, the failure plane would be closer to 45 degrees.

The solution to the problem was to eliminate the shrink fitting so as to reduce the locked in residual stresses in the welds and to ensure proper alignment of the gears.

Example 2:

A frame for a hydraulic lift is shown in [detail45]. Here again compressive loading has caused a fatigue crack by working against a residual tensile stress. @1[detail46.pcx] shows more closely what is actually going on. The failure here occurs as a crack in the plate adjacent to the weld.

7.3.10 Failure to Design Against Lamellar Tearing

Any reduction in weld metal used usually means a reduction in cost. Additional benefits are a decrease in weld distortion and a reduction in locked in stresses that might lead to lamellar tearing. In general, the designer should consider alternative joint designs and use the one which requires the least amount of weld metal.

In addition to weld metal reduction, a change in detail may also aid in reducing the probability of lamellar tearing. @1[detail49.pcx] illustrates how transverse residual stress can act on an inclusion to create a lamellar tear. The tendency for lamellar tearing is reduced in @1[detail50.pcx] by skewing the critical section and passing the weld through many lines of inclusion.

Example:

@1[detail51.pcx] illustrates a detail that was originally included in the plans for the construction of a stadium. The cantilevered beam was to be welded to a column where stiffeners were required. Design consultants on the job felt the detail lacked any sort of safeguard against lamellar tearing, which had been encountered recently in other welded structures. Should striations exist in the rolled plate forming the flange of the column and, further, should they pull apart, there would be nothing to prevent the beam and column from separating, and no other means to avoid collapse.

@1[detail52.pcx] illustrates the preferred detail that was recommended and used in the construction of the stadium. Note that the possibility of lamellar tearing at the tension region has been virtually eliminated. Although the bottom flange of the beam is still welded to the outside of the column plate, lamellar tearing should not be a problem because the bottom flange is in compression.

7.3.11 Redundancy With Diagonal Stiffeners

Redundancy serves no useful purpose in a well designed ground structure. The addition of unnecessary parts merely runs up costs and may actually weaken the member. In certain instances, "just to be safe", a designer may place double diagonal stiffeners in the column framing of a building. This suggests that the designer was uncertain about the performance of diagonals.

@1[Detail54.pcx] and @1[Detail55.pcx] show two ways to use diagonal stiffeners in transferring a moment from a beam into a column. The former sketch shows the diagonal in compression while the latter shows it in tension. Each method of placement transfers the moment effectively. Of the two details, welding is less critical when the welds transfer compression.

The fitting and welding of a double diagonal brace greatly increases the connection cost. If the designer felt that one diagonal was inadequate for the moment, the logical step would have been merely to increase the thickness of that single diagonal. Note that in welding diagonals it is only necessary to weld where force enters and leaves the diagonal, although a small weld may be used at the centre of a compression diagonal if there is concern about buckling.

7.3.12 Failure to Consider Not-in-Service Loads

What happens to a structural member before it is put into use can affect its design. The designer should be aware that all members are subjected to handling, loading, shipping, unloading, and erection operations. As an example, the deep plate girder, shown in @1[Detail53.pcx], was shipped to the site tied down to a flat-bed railroad car. An inspection of the girder after shipment uncovered cracks in the web near the lower end of the transverse stiffener. It was generally agreed that the cracking was the result of fatigue.

Repeated displacement of the top flange relative to the bottom resulted during shipment. These lateral displacements resulted from the many thousands of jolts encountered during transit. This action demanded high flexibility from the one-inch unstiffened portion of the web between the flange and the stiffener. Unfortunately, the cyclical bending stresses were too much for such a small section of the web and fatigue cracks developed. It was later recommended that the stiffener should stop back at least four to six times the web thickness from the tension flange to spread the flexing over a larger area.

7.3.13 Failure to Understand Matching Electrode Conditions

Although the following example refers to American codes, the Canadian codes are similar in nature and so the principle concept remains valid.

The designer or fabricator should be familiar with prevailing codes, and especially any recent changes that simplify design or fabrication tasks. A shop was fabricating a girder from A572 grade 65 steel. Table 4.1.1 of the AWS D1.1 **Structural Welding Code** indicated that the matching weld metal would be from an E80 electrode wire for either gas metal arc or flux-cored arc welding meeting this strength level. But an E80 electrode was not really required. Because the girder was to be fillet welded, the very common and abundant E70 weld metal was all that was required. Not only did E70 metal fully meet codes, but it would also reduce the chance for cracking and other problems. The company had gone wrong by assuming AWS Table 4.1.1 **demanded** that matching weld metal be used with the A572 steel. The engineers should have taken a look at

Table 8.4.1 for buildings or Table 9.3.1 for bridges. If they had, they would have found that matching weld metal is required **only** for complete-penetration groove welds with tension applied normal to the effective throat. This means that butt welds in the flanges of tension girders would require matching weld metal - but there were only fillet welds in the subject girder. According to AWS Table 8.4.1, weld metal with a strength level equal to **or less than** matching weld metal could be used. An E70 electrode would meet code specifications.

8 Determine Weld Size

Overwelding is one of the major factors determining welding cost. Specifying the correct weld size is the first step toward obtaining low-cost welding. Therefore, simple methods are necessary to determine the appropriate volume of weld needed to provide adequate strength for all types of connections.

The general design provisions for welds in structural steel work are defined in the CSA standard - W59-M1989. There are "welding electrode - base metal" matching conditions laid out in this standard which may or may not be mandatory depending on the type of weld and the type and direction of loading. These matching conditions are intended to ensure structural continuity by keeping the variation in mechanical properties across the weld zone within tolerable limits.

In the presence of demanding service conditions, the complete joint penetration groove weld is preferred. Since the groove weld, properly made, has equal or better strength than the thinner plate joined, there is no need for calculating the resistance of the weld. On the other hand, depending on the direction and type of load, the size and resistance of a partial joint penetration groove weld will usually have to be determined. The CSA standard, W59 - M1989, lists many prequalified complete and partial joint penetration groove welds in order to simplify the design and fabrication procedure.

Many standardized connections utilize the fillet weld. The determination of correct fillet weld size is essential to maintaining structural integrity while minimizing distortion.

Plug and slot welds are quite limited in their usefulness in welded structures. They are generally only used when the use of a groove or fillet weld has been deemed impractical.

8.1 Welding Electrode - Base Metal Matching Conditions

The basic philosophy behind the matching principle is to maintain an ultimate strength of weld metal (Xu) that is greater than the ultimate tensile strength of the base metal (Fu). However, to maintain a modicum of material property continuity across the heat affected zone, the matching electrode is generally stipulated such that Xu is just greater than Fu. As an example, for a base metal minimum tensile strength of between 400 and 480 MPa, the matching electrode ultimate strength is 480 Mpa. And for Fu between 480 MPa and 550, Xu is 550 MPa.

It should be noted that the yield strength of neither the weld metal nor the base metal has been considered here. In general, however, the ratio of yield to ultimate tensile strength for the weld metal will be higher than that of the base metal thereby resulting in a weld yield higher than the base yield.

Tables 11-1 and 12-1 of W59-M1989 for, respectively, statically and dynamically loaded structures contain tabulated "matching requirements" for many grades of steel.

8.2 Design of Complete Joint Penetration Groove Welds

Whether designing for allowable stresses or factored resistances, the capacity of the complete joint penetration groove (CJPG) weld is considered to be at least as good as the base metal, assuming matching conditions apply.

The W59 standard defines the effective weld area (ie. That section through the weld assumed effective in transferring stress) as the effective weld length multiplied by the effective throat thickness. The effective weld length of any groove, square or skewed to the direction of stress, shall be the width of the part joined. The effective throat thickness is the thickness of the thinnest part joined.

Transition of thickness or width:

The following transition rules are governed by @1[W59_11.4.8 Transition of Thickness or Width] for statically loaded structures and by @1[W59_12.4.8 Transition of Thickness or Width] for dynamically loaded structures. Under static loading the following condition applies.

The transition zone of butt joints composed of plates of unequal width or thickness shall not incorporate slopes steeper than 4 in 10. Figures @1[W59_1101.pcx], @1[W59_1102.pcx], and @1[W59_1103.pcx] illustrate three methods of achieving the 4 in 10 slope in the transition zone of joints with unmatched thickness. The detail in figure @1[W59_1104.pcx] shows the transition through unequal widths.

The W59 Clauses stated above should be viewed for further details and possible exemption from the above requirement.

8.3 Design of Partial Joint Penetration Groove Welds

Partial joint penetration groove (PJPG) welds are used in welded design when the full capacity of the base metal need not be developed. Structural applications include: field splices of columns, built-up box sections for truss chords, web to flange joints in flexural members, etc.

PJPG welds are permitted to transmit any combination of loads at allowable stresses or factored resistances as specified in clauses @1[W59_11.3 Design Provisions] and @1[W59_12.3 Design Provisions] of W59-M1989 for, respectively, statically and dynamically loaded structures. An important point worthy of emphasis is; single partial joint penetration groove welds are not permitted to be subjected to bending loads about the longitudinal axis of the weld if such load produces tension at the root of the weld.

Effective weld area, length, and throat thickness:

As with CJPG welds the effective area of a groove weld is the effective length multiplied by the effective thickness. The effective weld length, squared or skewed to the direction of stress, is the width of the part joined. The effective throat thickness shall be as indicated in Table_5 below.

Table_5

*Effective Throat Thickness (mm) for Tension and Shear

ß, Angle at the Root of the Groove (same as groove angle for V and Bevel grooves)	Partial Joint Penetration Groves	Partial Joint Penetration Grooves Reinforced with Fillet Welds
60 > ß >= 45	S - 3	t - 3
ß > 60	S	t

* For partial joint penetration grooves in compression, add to the listed effective throat the thickness of the joint fitted in contact bearing.

Note1: S is the depth of chamfer and t is the shortest distance between the root of the groove and the surface of the fillet. See @1[W59_A01.pcx], @1[W59_A02.pcx], and @1[W59_A03.pcx]. These Figures and the above table were taken from Appendix A of CSA standard W59_M89.

Note2: The effective throat thicknesses stipulated in Table A-1 above are based on the ease at which full root penetration may be achieved and they assume a root gap of zero. It should be stated that greater penetration would be feasible with root openings greater than zero however the author found no accountability of this in the standard.

Minimum groove depth partial joint penetration groove welds:

Just as fillet welds have a minimum size for thick plates because of fast cooling and greater restraint, so partial penetration groove welds have a minimum groove depth which should be used. Refer to @1[W59_4.3.4.1 Partial Joint Penetration Groove Welds] to view Table 4-3 Minimum Groove Depth for Partial Joint Penetration Groove Welds (Not Combined with Fillet Welds).

Partial joint penetration groove welds in tubular construction:

Partial joint penetration groove welds used in tubular construction may be considered as having equivalent strength to complete joint penetration groove welds if certain conditions are met. Appendix L of W59_M89 contains more information on this subject.

Flare groove welds:

As stipulated in W59_M89, all flare groove welds are classified as partial joint penetration groove welds.

The determination of the effective throat thickness of flare-V and flare-bevel groove welds on hollow structural sections is the responsibility of the contractor. Unless otherwise approved by the engineer, the effective throat thickness shall be established by means of trial welds for each set of procedural conditions. The trial welds shall then be sectioned and measured to establish welding techniques that will ensure that the design throat thickness is achieved in production. The effective throats for flare grooves on solid bars are defined in section @1[W59_4.3.1.6.2 Solid Bars] of W59-M89. In most cases of flare groove welds on solid bars no joint preparation is necessary because of the inherent shape of the bars.

8.4 Design of Fillet Welds

Fillet welds may be continuous or intermittent, although certain restrictions apply with the use of intermittent fillet welds in dynamically loaded structures. Fillet welds may be used to transmit any combination of loads at allowable stresses or factored resistances as specified in Clause

@1[W59_11.3 Design Provisions] for statically loaded structures or Clause @1[W59_12.3 Design Provisions] for dynamically loaded structures. Single fillet welds may not transfer bending about the longitudinal axis of the weld if it produces tension at the root of the weld. Fillet welds may be used for connecting parts whose respective fusion faces form an angle between 60 and 120 degrees. When the angle between the faces is less than 60 degrees the weld shall be considered a partial joint penetration groove weld. For angles greater than 120 degrees, the fillet weld may not be relied upon to transmit calculated load.

The direction of loading has a significant effect on the ultimate strength of a fillet weld. Transverse fillet welds have been found to be as high as 40 percent stronger than longitudinal fillet welds. However, this increased strength is not recognized by Canadian structural welding specifications. Instead, the strength is based on loading in the longitudinal direction irrespective of the true in service loading angle.

Effective fillet size:

W59_M89 has defined the effective throat area of a fillet weld to be equal to the effective length of the weld multiplied by the effective throat thickness. The effective length of a fillet weld incorporates the overall length plus end returns. In other words, if the weld section is full size throughout its length, no reduction in length is necessary for the start or termination of the weld. The effective throat thickness is defined as the shortest distance from the root of the diagrammatic weld to the face for all processes except SAW.

When submerged arc welding (SAW) is used, the effective throat thickness is considered to be the next larger size fillet, expressed in millimeters, providing the leg sizes are equal and sufficient penetration has been achieved to justify such an increase. In other words, the penetration must be deep enough to provide a diagrammetric weld size equal to or greater than the next larger fillet size. The design drawings will show the theoretical fillet size required and the actual size shall be not less than the minimum required in Table 4-4 of @1[W59_4.3.4 Minimum size of Fillet Welds].

Figure @1[wsize1.pcx] shows the effective cross-sectional area of an equal legged fillet weld in a right angled corner. The effective size and throat thickness are illustrated in this diagram. Similarly, @1[wsize2.pcx] illustrates the unequal legged fillet weld also fused into a right angled corner. Many times fillet welds are placed into joints where the fusion faces form an angle other than 90 degrees. The following paragraph describes briefly how the CSA standard W59-M89 deals with this scenario.

Although the strength of a fillet weld is determined from the shear capacity of the effective throat, the Canadian practice is to express weld dimensions in terms of an effective leg size. The effective size is the leg size of an equivalent fillet between fusion faces at 90 degrees having the same throat thickness as the measured fillet size under the skewed condition. Figure @1[W59_0403.pcx] illustrates the method used to determine the effective fillet weld size from the measured size and vice versa. This diagram shows the relationship between the effective size of the fillet weld (E), the measured size of the fillet weld (S), the gap (G), and the angle between the fusion faces (theta). The measured size (S) of each fillet shall be the smaller of the two leg sizes and gaps less than 1 mm may be ignored. In addition, the gap size may not exceed 5 mm on either side of the joint.

Minimum fillet weld size:

In addition to strength, the minimum size of fillet weld is governed by material thickness. The Canadian Standards Association recognizes that thick plates offer greater restraint, and produce a faster cooling rate for the welds. Too fast a cooling rate will produce undesirable weld properties and possibly lead to weld cracking. Table 4-4 of @1[W59_4.3.4b Minimum Length and Size of Fillet Welds] tabulates the minimum fillet sizes permitted in relation to the thickness of the thicker part joined. Table 4-4 is predicated on the theory that the required minimum weld size will provide sufficient welding heat input into the plate to give the desired slow rate of cooling. Note that the minimum weld size need not exceed the thickness of the thinner part joined unless required by calculated stress. With regard to this exception, sufficient heat input must be provided to ensure weld soundness. Also note that Table 4-4 need not apply for the welding of attachments to members that do not carry calculated stress.

Minimum effective length of fillet weld:

The minimum effective length of fillet weld is 40 mm or 4 times the size of the fillet, whichever is larger. If the weld is less than 40 mm in length than the effective fillet size shall be 1/4 of the effective length.

Maximum fillet weld size:

The maximum effective size of fillet weld should be such so as not to overstress the adjacent base material.

For fillet welds detailed along the material edge, the maximum effective leg size shall be:

(a) The thickness of the material, for material less than 6 mm thick; or

(b) The thickness of the material less 2 mm, for material greater than or equal to 6 mm thick.

If the detail drawing stipulates the weld to be built out to accommodate the full throat thickness than point (b) above need not apply.

For rolled edges, good welding practice suggests a maximum weld size of 3/4 the material thickness.

In T and corner joints, fillet welds on top of groove welds may be required by the Engineer. In such circumstances, the fillet size shall be not less than t/4, where t is the thickness of the enclosed groove welded member, and need not be more than 10 mm. This type of reinforcement is mandatory for T joints subjected to tension normal to the weld.

Fillet welds in lap joints:

Side or end fillet welds terminating at ends or sides, respectively, shall continue around the corner, where possible, for a distance at least equal to twice the nominal weld size.

In order to eliminate any excessive bending in stress carrying lap joints, a minimum length of overlap equal to 5 times the thickness of the thinner part joined is required.

Welds produced on opposite sides of a common plane are to be interrupted at the corners common to both welds, unless there is a requirement that an all around seal be provided.

Further design considerations regarding lap joints and other structural details may be found in @1[W59_11.4 Structural Details].

8.5 Design of Plug and Slot Welds

Because of the relative difficulty in producing sound, quality plug and slot welds, their use is limited to situations where the use of other weld types has been precluded. Plug and slot welds are only permitted to carry shear under static loading and the W59_M89 standard does not even provide for them under dynamic loads. Their most useful purpose seems to be in the strengthening or repair of existing structures under conditions where other methods might fail to maintain structural integrity.

For more information regarding plug and slot welds refer to:

@1[W59_4.4.2 Plug and Slot Weld Details]

@1[W59_B Plug and Slot Welds]

8.6 Determining Stresses on Welds

In many cases, the size of the weld my be determined by stress calculations. Precise calculation of weld stress, however, is hardly ever possible, in fact even stress analysis involving the simplest of shapes is difficult. The procedures outlined in the following examples, therefore, provide only rough estimates of nominal stress levels. The shapes and configurations of the examples are compared with the results from other simple loading systems and approximate stress levels are determined. Stress concentration effects which result from notches are temporarily ignored and weld sizes are adjusted until the "permissible" or design stress is not exceeded.

In view of the above, one may ask if such calculations are worth doing. The main justification of the nominal stress calculation is that it introduces a measure of uniformity into weld sizing procedures, and allows one to make use of a considerable body of previous experience which is based on a similar approach.

8.6.1 Allowable Stresses or Factored Resistances

A recommended permissible stress level, whether allowable or factored, characterizes successful design practice in a particular class of structure and loading. The Engineer should proceed cautiously with these structure classifications. Extrapolating recommended values from one class of structure to another involves the consideration of many variables (environment, workmanship, material properties, etc.) that are not always easy to predict.

Most code permitted design stresses are expressed as a proportion of the material yield or ultimate strength, except where fatigue loading is concerned. One suspected problem with the yield strength basis is that it encourages the use of high ratio of yield/ultimate strength steels that leave little room for reserve ductility.

The design/yield stress ratio can vary depending on the character of the stress. Statically determinate direct or membrane stresses, which could lead directly to collapse, will be held to lower levels than bending or shear stresses for which local yielding would be less of a problem. Normal and shear stresses acting at the same point are usually combined in terms of an 'equivalent' or Von Mises stress.

For welds experiencing compressive stress in regions of potential buckling, permissible stresses may be governed as a proportion of the theoretical buckling strength.

8.6.2 Stresses in Butt Welds

A complete joint penetration groove weld made in a butt joint may be considered an integral part of the loaded component. There is, therefore, no special analysis required on the weld. While it has been thought sensible to ignore weld overfill in a butt joint because it does not contribute to weld strength, the greater concern lies in the potential problems that could result from stress concentrations. A properly made butt weld will allow a smooth flow of stress from one element to the next.

The calculation of weld stress in a partial joint penetration groove weld should be based on the welded ligament only. Some standards discourage the use of these welds to carry primary tensile loads because they are prone to cracking in manufacture, and in other ways may increase the risk of fracture in service.

Because of code required matching conditions, the permissible stresses in butt welds are generally equal to or greater than those of the base metal.

8.6.3 Stresses in Fillet Welds

In contrast to groove welds, fillet welds are non-integral in character. Their shape and orientation relative to the loading almost disallows the application of simple stress analysis. Two simplifying assumptions are implied in most practical design methods for fillet welds:

1. The stress should be related to a cross-sectional area running the length of the weld. The throat plane is chosen as the representative area. For an equal legged fillet in a right angled corner the throat area is:

(area of throat) = 0.707 x (leg size) x (weld length)

2. The throat plane is parallel to or perpendicular to the direction of loading.

@1[wsize9.pcx] illustrates the above two simplifying assumptions. In the diagram, the welded joint is rearranged to resemble a butt weld in which the ligament area is determined by the throat area.

The inaccuracies of this approach are recognized through a reduction in the permissible stress.

8.6.4 Example: Load Parallel to Fillet Welds

Consider the lap style joint illustrated in part (a) of @1[WSIZE10.pcx]. If we assume the plates are rigid, part (b) of the figure, than the shear stress will be uniform throughout the length of the fillet welds. Then we can say .....

Tau(avg) = average shear stress = P / (2 x 0.707h x l)

On the other hand, if the plates strain significantly, more of the load will be taken at the end of the welds and the stress distribution will look more like that shown in part (c) of the figure. The maximum shear stress at the weld ends has been quantified (Mocanu and Buga, 1970) by the following formula:

Tau(max) = Tau(avg) + (Sigma x Lambda x l) / (12 x 0.707h)

where Sigma is the nominal stress in the plate and Lambda is the stiffness of a unit length of weld (determined experimentally) and therefore depends on the throat dimensions.

When the plates are thick, Sigma is low and the second term in the equation becomes less significant. Then the assumption of uniform stress distribution is reasonable. If we attempt to lower the average shear stress by increasing the weld length, an additional effect is a 'peakier' stress distribution resulting from the influence of the second term. It should also be noted that decreasing the weld lengths appreciably tends to generate elevated normal stresses in the plate near the edges.

It is disturbing to note that the shear stress peaks should develop at the weld ends where the occurrence of weld defects is most probable. For this reason, it is considered good practice to continue the weld for a short distance around the corner, and along the end faces of the plates. Note that the small bending moment acting on the weld plane as a result of the eccentricity of the loads has been ignored.

8.6.5 Example: End Fillets in Lap Joint

@1[wsize11.pcx] illustrates this simple connection. In this brief analysis, the small bending moment will be ignored. If the plates are assumed rigid then the stress distribution will be uniform along the length of the welds. The average shear stress will then be given by:

Tau(avg) = P / (2 x 0.707h x l)

The assumption of uniform distribution across the width of the plate is reasonable, however, if the weld length is shorter then the plate width then the distribution will more probably be uniform in the center and peaked at the ends. This distribution would be similar to that of a stepped width plate.

8.6.6 Example: All Around Weld

@1[wsize12.pcx] illustrates an all around weld. The bending moment induced here through load eccentricity will be ignored. The average shear stress is given here simply by the following equation:

Tau(avg) = P / SUM(0.707h x l)

Fatigue tests clearly show that stress peaks occur at the plate corners. Although rounding of the corners may help the welder, it does not significantly reduce the stress peaks.

There is insufficient information to predict what percentage of the load is carried by the transverse fillet and what percentage is carried by the longitudinal fillets. However, it has been suggested that a heavier transverse weld might help to reduce the stress peaks seen in the longitudinal welds of @1[wsize10.pcx].

8.6.7 Example: Fillet Welds in Bending and Shear

The left side of @1[wsize13.pcx] illustrates fillet welds subjected to combined bending and shear. The weld plane can be seen to be the common plane between the elements being welded. The right side of the figure shows an idealization of the scenario whereby the throat plane has been rotated till it is perpendicular to the weld plane. On this idealized throat plane, the eccentric load P will produce both in- and out-of-plane shears with respect to the weld plane.

The maximum out-of-plane shear stress due to bending appears at the weld ends and is given by:

Tau(bend) = P d y / I = P d 12 / (2 x 0.707h(l)^3) l / 2

Note that the same initial equation would be used for normal stress if the throat plane were rotated into the weld plane instead of perpendicular to it.

The in-plane shear stress distribution, on the other hand, generates maximum shear stress at the weld center with zero shear stress at the ends. This distribution is commonly seen in bending problems. The magnitude of this maximum stress is given by:

Tau(max) = P A y(max) / b I

= {P x 2 x 0.707h x l/2 x l/4} / {2 x 0.707h x 2/12 x 0.707h(l)^3}

= 3 P / (4 x 0.707hl)

This treatment assumes that the elements extend sufficiently far out of the weld plane, and that the complementary shears implied by the above equation can be sustained in this direction.

Because both the in- and out-of-plane shear stresses are assumed to act on the same cross-sectional area, they may be added vectorially at any location to obtain the total stress.

Tau(total) = SQRT{Tau(bend)^2 + Tau(max)^2}

An alternate solution could have involved the bending stress treated as a normal stress instead of a shear stress and then Mohr's circle could have been used to sum the stresses. However, the numerical difference in the result is insignificant when considered relative to the gross inaccuracies in other respects.

8.7 Welds Subjected to Torsion

The following two separate idealizations have been made regarding weld groups subjected to torsion. It may be assumed that:

(1) A weld is capable of supporting shear stress due to torsion in any direction, or

(2) A weld is only capable of supporting shear stress along its length.

@1[wsize14.pcx] illustrates the above two idealizations. The primary assumption in the first idealization is that the components being joined are relatively rigid. @1[wsize15.pcx] depicts a scenario where both idealizations lead to the same result.

(1) shear stress in any direction:

As shown in @1[wsize16.pcx], the total stress on each element of weld is composed of a vertical shear stress and a torsional shear stress. The vertical stress is simply:

Tau(direct) = P / SUM(0.707h x l)

The torsional stress is determined by assuming the force on each element is proportional to the distance from the centroid of the weld group. This is not unreasonable because the strain is probably proportional to the distance, r. The maximum torsional shear stress may be determined from the following equation:

Tau(max. Tors.) = P d r(max) / J

where J is the polar second moment of area.

The vector sum of the above two calculated shear stresses will produce the largest magnitude shear stress in the weld. This value may then be compared to the allowable.

Instantaneous center calculation:

An extension to the idealization that the shear stress resulting from torsion may act in any direction involves the use of instantaneous centers. This method of analysis assumes the weld group to rotate about an instantaneous center of rotation in an effort to resist the applied shear and torsion. The Canadian steel code, S16.1, employs this analysis procedure to determine the capacity of framed beam shear connections. For more information on this topic, see @[09.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups].

(2) shear stress only along weld line:

The torsional shear stress under this assumption is determined by utilizing the weld group shear center. @1[wsize17.pcx] illustrates this principle. The torque is determined by multiplying the vertical force P by the distance to the shear center. The maximum torsional stress may be found from the following equation:

Tau(max) = 3 x (P e) x 0.707h(max) / SUM(l x (0.707h)^3)

In addition to the torsional shear stress, we also have direct shear in the vertical weld line. As seen in the diagram, this is given by:

Tau(direct) = P / 0.707(h1)(l1)

This direct stress is acting on the same line as the torsional stress and may therefore be subtracted directly from it.

8.8 Weld Sizing Through "Weld-Line" Analysis

One of the problems with sizing welds through stress analysis is that leg sizes must be known in advance. The general procedure involves: assuming a weld leg size, calculating subsequent weld stresses, and then checking for over or under stressing. If the result is too far off, then the weld leg size is readjusted and the procedure repeated. In addition, it can prove difficult to combine stresses from several loading types.

In contrast, the following is a simple method to determine the correct amount of welding required for adequate strength. In this method the weld is treated as a line, having no area, but a definite length and outline. This method has the following advantages:

1. By only considering the weld as a line, throat areas become irrelevant.

2. Properties of the welded connection may be found from a table without knowledge of the weld leg size.

3. Forces, not stresses, are considered on a unit length of weld, thus eliminating the knotty problem of combining stresses.

4. While it is true that the stress distribution within a fillet weld is complex, due to eccentricity of the applied force, shape of the fillet, notch effect of the root, etc.; these same conditions exist in the actual fillet welds tested and have been recorded as a unit force per unit length of weld.

Determining the force on the weld line:

Visualize the welded connection as a single line, having the same outline as the connection, but no cross-sectional area. Notice, [wsize18.pcx], that the area (A_W) of the welded connection now becomes just the total length of weld. Instead of trying to determine the stress on the weld, the problem now becomes a much simpler one of determining the force on the weld.

By inserting the property of the welded connection treated as a line into the standard design formula used for that particular type of load, [Wsize19.pcx], the force on the weld may be found in terms of kN (lb) per linear mm (in) of weld. Normally the use of these standard design formulae results in a unit stress, MPa (psi); however, when the weld is treated as a line, they result in a unit force.

In problems involving bending or twisting loads, Table 5 is used to determine properties of the weld treated as a line. It contains the section modulus (S_W), for bending, and polar moment of inertia (J_W), for twisting, of some 13 typical welded connections with the weld treated as a line.

Table 5

For any given connection, two dimensions are needed, width (b) and depth (d).

Section moduli (S_W) from these formulas are for maximum force at the top as well as the bottom portions of the welded connections. For the unsymmetrical connections shown in this table, maximum bending force is at the bottom.

If there is more than one force applied to the weld, these are found and combined. All forces which are combined (vectorially added) must occur at the same position in the welded joint.

Applying the System to Any Welded Connection:

1. Find the position on the welded connection where the combination of forces will be maximum. There may be more than one point to considered.

2. Determine the magnitude of each of the forces on the welded connection at one point and then:

(a) Use @1[wsize19.pcx] for the standard design formula to find the force on the weld.

(b) Use Table 5 to find the property of the weld treated as a line.

3. Combine (vectorially) all of the forces on the weld at this point.

4. Determine the required weld size by dividing this resultant value by the allowable stress or factored resistance as given in the appropriate standard.

Connecting for horizontal shear forces:

Outside of simply holding the flanges and web of a beam together, or to transmit any unusually high force between the flange and web at right angles to the member (for example, bearing supports, lifting lugs, etc.), the real purpose of the weld between the flange and web is to transmit the horizontal shear forces, and the size of the weld is determined by the value of these shear forces.

It will help in the analysis of a beam if it is recognized that the shear diagram is also a picture of the amount and location of the welding required between the flange and web. Transverse loading induces bending moments along the length of a beam. The variation in bending moment determines the magnitude of the horizontal shear which in turn determines the amount of weld required to transmit force between the web and the flange. When the shear force varies along the length of the beam it is usually greatest at the ends. Because the end fixity does not effect the shear diagram, the amount of welding between the flange and web will be the same regardless of the end conditions.

Consider the welded frame in @1[wsize20.pcx]. The moment diagram for this loaded frame is shown on the left-hand size. The bending moment is gradually changing throughout the vertical portion of the frame. The shear diagram shows that this results in a small amount of shear in the frame. Using the horizontal shear formula (f = Vay/In), this would require a small amount of welding between the flange and web. Intermittent welding would probably be sufficient. However, at the point where the crane bending moment is applied, the moment diagram shows a very fast rate of change. Since the shear value is equal to the rate of change in the bending moment, it is very high and more welding is required at this region.

In general, one should use continuous welding where loads or moments are applied to a member, even though intermittent welding may be used throughout the rest of the fabricated frame.

@1[wsize21.pcx] illustrates the concept of locating welds at points of minimum stress. Horizontal shear force is maximum along neutral axis. Welds in the top example must carry maximum shear force, however there is no shear on welds in bottom example.

@1[wsize22.pcx] illustrates examples of welds in horizontal shear. The leg size of the required fillet weld (continuous) is found by dividing the actual unit force (f) by the allowable for the type of weld metal used. If intermittent fillet welds are to be used divide this weld size (continuous) by the actual size used (intermittent). When expressed as a percentage, this will give the length of weld to be used per unit length.

9 Flexible Connections

Flexible connections are an integral part of simple construction or simple design. They permit full rotational capacity while still maintaining the integrity of the connection. Although a small percentage of the fixed end moment may develop in the joint, it is considered negligible and is customarily ignored.

The following three modules provide further information relevant to this section:

@1[09.1 General Considerations for Flexible Connection Design]

@1[09.2 Standard Flexible Connections]

@1[09.3 Non-standard Flexible Connections]

9.1 General Considerations for Flexible Connection Design

In the design of welded connections the following must be given full consideration by the engineer:

1. Type and size of welds should be related to the type of service stress and load;
2. The number and location of welds should be governed by:

(a) good access;

(b) ease of execution;

(c) economy.

3. The volume of weld metal should be minimum so as to minimize residual stresses and distortion.

9.2 Standard Flexible Connections

Three types of flexible connections found their way into standard framing practice and are consequently covered in the CISC " Handbook of Steel Construction " in the limit state design format. They are:

@1[09.2.1 Framed Beam Connections], using header angles and relying on the flexibility of their outstanding legs to provide the required rotation capacity;

@1[09.2.2 Flexible Seated Beam Connections], in which the shear is transferred in direct bearing to the seat angle and the rotation capacity is provided by the flexibility of the clip angle connecting to the top flange of the beam and also to some degree by the deflection of the seat angle;

@1[09.2.3 Stiffened Seated Beam Connection], consisting primarily of a suitable "T" section supporting the beam and a clip angle attached to the top flange to provide the desired flexibility and at the same time some degree of lateral stability.

Note that the clip angle utilized in 2. and 3. above may be optionally connected to the top of the beam web if so desired.

9.2.1 Framed Beam Shear Connections

These types of connections are generally designed for strength requirements only, under the effects of factored loads, with the weld capacities based on factored resistances.

@1[09.2.1.1 General Design Considerations for Framed Beam Shear Connections] provide some basic considerations regarding the various systems.

The following five basic connections have become standardized over the years:

@1[09.2.1.2 Double Angle connection]

@1[09.2.1.3 Single Angle Connection]

@1[09.2.1.4 Tee Connection]

@1[09.2.1.5 Flexible End Plate Connection]

@1[09.2.1.6 Single Side Web Plate Connection]

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in the various connections.

9.2.1.1 General Design Considerations for Framed Beam Shear Connections

The selected weld sizes must be such as to:

(a) effect a "balanced design" with no overstress on the enclosed thickness of the web.

(b) conform to the Design Specification (CSA W59) and its rulings on mandatory "minimum size fillet to thickness of material" relationship.

(c) be within good welding practices in terms of recommended maximum sizes on sheared or rolled edges.

The minimum length of the header angles must be greater than 1/2 of the beam depth to maintain stability. The maximum length of the angle will be governed by the required access for horizontal fillets welded on the supported web connection, except when the beam is coped, in which case the acceptable edge distance related to fillet weld size will be the governing parameter.

There are a number of considerations bearing either on the design of the framing angles or on the execution of welding, which should receive due attention from the design engineer:

- Proper clearance and suitable access for welding must be provided.
- Ample edge distances for fillets must be recognized as a fundamental condition of sound workmanship in the execution of welding.
- Erection bolts should be placed in the lower half of the angles so as not to detract from the flexibility of the connection.
- Header angles should be placed close to the top flange with sufficient access for welding.

9.2.1.2 Double angle connection

The double angle connection is one of the most commonly utilized framing systems used to develop the required rotation capacity. Its symmetrical design combined with ease of fabrication provides for an efficient and economical flexible connection.

The WISDOM system will only be considering weld fasteners connecting the double angles to the supporting and supported members. Typically, these header angles are shop welded to one of the adjoining members, with the final welding for the connection being completed in the field. Coping of the lower beam flange may be required for erection purposes. @1[flex1.pcx] and @1[flex2.pcx] illustrate the typical double angle framed beam_column connection.

@1[09.2.1.2.1 Design Considerations for the Double Angle Connection] provide some basic considerations regarding this system.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

Alternatively, the double angle design spreadsheets may be accessed directly with one of the following options:

1. Beam to Column Flange .....@1[MAPLCALC.EXE b_to_cf]
2. Beam to Column Web .....@1[MAPLCALC.EXE b_to_cw]
3. Beam to Girder .....@1[MAPLCALC.EXE b_to_g]

9.2.1.2.1 Design Considerations for the Double Angle Connection

A look into the weld fasteners.

As a welded design problem, there are two basic weld groups which must be considered. These are the c_shaped weld group connecting the header angle to the web of the supported beam as well as the vertical weld connecting the outstanding leg of the header angle to the supporting member. A diagram of these double angle weld fasteners, @1[flex13.pcx] may be viewed for greater clarity. Note that the latter of the above two weld groups maintains a returned top weld (of length equal to twice the weld size) to guard against tearing in the tension zone. Traditionally, the design of these weld groups involved elastic analysis and vector sums versus allowable stresses at critical locations. The factor of safety determined using this method was found to be quite variable (approximate range: 3 to 7) and therefore unacceptable with todays limit state design standards. The state of the art today permits the use of instantaneous centers and ultimate load deformation responses to determine factored resistances allowing more uniform factors of safety. For more information on the latter refer to @1[9.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups].

Under the terms of the traditional elastic analysis, both weld groups were individually subjected to combined shear and torsion. The c_shaped welds on the web of the supported beam were quite straight forward, however, because of the twisting action of the angles under load, the vertical welds on the supporting member were analyzed with a special stress distribution. Separately published CISC allowable stress design tables have tabulated capacities calculated using the above analysis.

The modern approach using instantaneous centers and ultimate load deformation responses considers the c_shaped weld group on the supported web to be under the action of combined shear and torsion with the vertical weld on the supporting member sustaining combined shear and moment. The CISC handbook of steel construction uses this more modern approach to determine capacities for welded double angle beam connections. It should be noted, however, that the handbook ignored the effects of eccentricity when tabulating the weld capacities for connection angles over 310 mm in length.

Designing to maintain flexible connections:

In order to maintain flexibility, unequal legged triangles are typically used with the longer legs outstanding. The shorter legs on the web of the supported beam will then provide sufficient torsional resistance while minimizing the eccentricity.

The balance of leg length versus angle thickness plays a critical role in the design of this connection. The length of legs and the thickness of angles must be judiciously selected and the best compromise reached in satisfying the requirements; for suitable flexibility (favouring minimum thickness) and for sufficient shear capacity to maintain strength. Typical angle sizes are 65x75 and 75x90 with thicknesses ranging between 6 and 12 mm. In order to ensure flexible behavior, total width (ie. both outstanding legs) to thickness ratios for the outstanding legs must be within reasonable bounds. Past experience has suggested ratios in the range of 15 to 18 to maintain suitable flexibility. For example, 75 mm outstanding legs that are 10 mm thick give a ratio of 15. The design of fillet welds must encompass good welding practice which suggests weld sizes of 3/4 of rolled edge thicknesses.

Note that, to maintain continuous flexibility, a 10 to 12 mm gap has become the standard acceptable spacing between the back of the angles and the end of the beam. This will prevent the bottom of the supported beam from bearing against the supporting member.

9.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups

The analytical method which follows for predicting the ultimate load on eccentrically loaded fillet welded connections is similar to the approach used for determining the ultimate load on eccentrically loaded bolted connections. This approach uses the load-deformation response of the welds to predict the ultimate load of the connection.

Fillet welds, unlike bolts, are usually continuous fasteners. In addition, the maximum strength and deformation sustained by an element of the weld will depend upon the angle of the force applied on that element. In this approach, the continuous weld is divided into elemental lengths and the resisting force on each element of weld is assumed to act at its center. The continuous weld is then similar to a line of bolts in a bolt group, each element of weld being analogous to a bolt, except that in the case of the weld the maximum strength and deformation will depend upon the angle of the resisting force. It is assumed that the overall strength of the weld group will be the sum of the individual capacities. After predicting the ultimate load, a safe working load can then be determined by introducing an appropriate factor of safety.

The above approach applies wholly to in plane eccentricities where the weld group is free to deform. In the case of out of plane eccentricities, however, the direct bearing of the base metal in the compression zone prevents the deformation of the weld and must be considered in the analysis. Therefore, only the tension induced shear zone set up with an out of plane eccentricity will follow the above procedure. The axial and shear distributions acting in the compression zone may be determined with a few simple assumptions.

The prediction of the ultimate connection strength is based on the following assumptions:

1. The weld group, under an eccentric load, rotates about an instantaneous center of rotation.
2. The deformation which occurs at any point in the weld group varies linearly with the distance from the instantaneous center and acts in a direction perpendicular to a radius from that point.
3. The ultimate capacity of a connection is reached when the ultimate strength and deformation of some weld element is reached.
4. The ultimate strength of a fillet weld subjected to a tension induced shearing force is the same as for a similar weld loaded in compression induced shear.
5. The strength properties of fillet welds are assumed to be proportional to their leg size.
6. For out of plane eccentricities, the connecting plates in the compression zone of the connection are in direct bearing at the time that the ultimate load is reached.

9.2.1.3 Single Angle Connection

For some building applications, single angle connections provide satisfactory alternatives to double angle or end plate connections, and are particularly suitable for those cases where limited access prevents the erection of beams with double angle or end plate connections, and where speed of erection is a primary consideration.

Although the connection angle may be either bolted or welded to the supporting and supported members, usual practice involves shop fillet welding to the supporting member and field bolting to the web of the supported beam. @1[flex10.pcx] and @1[flex11.pcx] illustrate the typical connection.

@1[09.2.1.3.1 Design Considerations for the Single Angle Connection provide some basic considerations regarding this system.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.3.1 Design Considerations for the Single Angle Connection

Tests carried out at the University of British Columbia demonstrated that certain welded-bolted single angle connections possess adequate rotational capacity when the larger leg is bolted to the supported beam web. It was further noted in these experiments that ultimate failure occurred in the bolts when the weld pattern included welding along the heel and ends of the connection angle. The tests also demonstrated that the use of horizontal slotted holes in the connection angle reduced the moment at the bolts without affecting the ultimate capacity of the connection. The capacities tabulated in the CISC handbook are very specific as the research is rather limited. It should therefore be recognized that a general single angle connection design that is not covered by the tables will not be supported by any research or test data.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.4 Tee Connection

Standard tee-type beam connections combine some of the characteristics of single angle connections with the web-framing leg bolted in single shear, and of double-angle connections with the outstanding legs welded to the supporting member. Two diagrams of the typical tee_type beam_column connection illustrate the final product. @1[flex3.pcx] and @1[flex4.pcx]

Their main advantage is speed and ease of erection (framing from one side) as the connection can be completed at the time the beam is erected. They are also commonly used where hole making in the supporting member is undesirable, such as for connections to HSS columns, and to avoid having to cope the bottom flange of the beam for erection purposes.

@1[09.2.1.4.1 Design Considerations for the Tee-Type Connection] provide some basic considerations regarding this system.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.4.1 Design Considerations for the Tee-Type Connection

The flexibility of this connection can be assured if the thickness of the tee flange relative to its width is comparable to the equivalent double angle connection. In other words, the distortion of the tee flange under service loads must be nearly identical to the distortion that would be seen in the outstanding legs of an efficiently designed double angle connection under the same service loads. Consequently, here again a ratio of flange width to thickness in the range of 15 to 18 is desired. For example, a 150 mm wide flange that is 10 mm thick would provide suitable flexibility. The Canadian design approach suggests that the welds on the supporting member be designed for shear and bending as was the case for the double angle connection. The c_shaped weld (if used) on the web of the supported beam may be designed for shear and torsion. However, bolting of the beam web is more popular. The CISC Handbook only tabulates factored resistances for, welded Tee flange to supporting member and bolted Tee stem to supported web. An eccentricity of 65 mm was assumed in the tabulation of factored weld resistances. In the design module, this eccentricity will be an input value.

Of course, due consideration of the shear and bending strength of the T-stem and the shear strength of the beam web are required to ensure the structural integrity of this connection.

Fabrication considerations control the location of the beam end relative to the supporting member. The customary 10 to 12 mm gap does not apply in this connection. The beam end will generally be required to remain a distance k, plus some 3 mm for reserve, away from the back of the Tee-flange. (ie. the gap will be closer to 30 mm.)

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.5 Flexible End Plate Connection

End plate connections with the end plate welded to the supported beam web and bolted to the supporting member have combined good economy with ease of fabrication to yield satisfactory performance. Research conducted to date on simple beam end plate shear connections indicates that their strength and flexibility compare favorably with double angle shear connections for similar material thickness, depth of connection, and arrangement of bolts. @1[flex6.pcx] and @1[flex7.pcx] illustrate the final product.

Because it is most common to weld the end plate to the beam web and then bolt to the column later in the field, the design considerations for this connection will not be considered separately here. The design of the bolts will not be elaborated on because their design has nothing to do with welding. The weld fasteners joining the end plate to the supported beam web are assumed to be subjected to shear loading only. Therefore, the design of the weld will be somewhat trivial. It is suggested for practical reasons that the minimum thickness of end plates for simple beam connections be 6 mm, and that the maximum thickness be limited to 10 mm for adequate flexibility. The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.6 Single Side Web Plate Connection

The single plate framing connection incorporates a single plate fillet welded to the supporting member, either a column @1[flex16.pcx] or a beam @1[flex17.pcx], and then bolted to the supported beam web.

The single plate framing connection has always been considered by designers to be a flexible connection however it has only recently been given simple connection status. This distinction was the result of extensive research carried out in the U.S. in the late 1970's. A recommended design aid was published by the AISC Engineering Journal in the 4th quarter of 1981, "Design Aid for Single Plate Framing" by Young and Disque. This was a comprehensive design aid based on allowable stress design that was valid for A36 steel combined with A325 or A490 bolts. @1[09.2.1.6.1 Design Considerations for Single Side Plate Connections] provides some insight into connection performance and design.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.6.1 Design Considerations for Single Side Plate Connections

Other framed beam shear connections rely on flexural elements for their ductility. The single side plate beam connection, however, derives its ductility from the following:

1. bolt deformation in shear,
2. plate and/or beam web hole distortion,
3. out-of-plane bending of the plate and or beam web, and

4. bolt slippage if elongated holes are incorporated or if bolts are not in bearing at the time of loading.

The majority of the connection ductility and flexibility is provided by inelastic yielding and distortion of the holes in the beam web or connecting plate. The use of elongated holes, especially with the top and bottom bolts, will further enhance the connection flexibility.

Many tests have proved this connection quite capable of developing significant beam end moments. For two sided connections, rigid framing is assumed, however one-sided framing has shown that supporting element flexibility can significantly reduce the beam end moment.

Further research studies in the United States have indicated that, with the application of certain geometric and material parameters, the transfer of moment into the supporting member can be virtually eliminated. The "Design Aid for Single Plate Framing" by Young and Disque, AISC Engineering Journal, 4th Quarter of 1981, provides the conditions for such a claim. A few of the more significant conditions are summarized below.

a.) A maximum plate thickness is specified for a given bolt diameter to guard against bolt shear failure and maintain connection ductility. When elongated holes are used the maximum thickness criteria no longer applies and A307 bolts may be used in addition to the otherwise specified A325 or A390 bolts.

b.) A minimum plate thickness is specified for a given reaction, moment and plate size.

c.) Edge distances on both the plate and beam web are required to be at least as great as twice the bolt diameter.

d.) A fixed support is assumed along the weld line between the plate and the supporting member. Although this is not always true, it is conservative.

e.) The welds on the supporting member are designed for combined shear and bending.

f.) Certain limitations have been placed on the span to depth of beam ratio:

$l/d <= 36$ .... for noncomposite beams .... $Fy = 36$ ksi

$l/d <= 24$ .... for noncomposite beams .... $Fy = 50$ ksi

One important consideration regarding the fabrication of the connection is to avoid gaps between the plate and the supporting member when welding to ensure minimal misalignment after the weld cools and shrinks.

The @1[09.2.1.7 Framed Beam Shear Connection Design Module] permits the design of all weld fasteners used in this connection.

9.2.1.7 Framed Beam Shear Connection Design Module

This section accesses design modules that may be used in place of the tables in the "Framed Beam Shear Connections" section of the steel handbook. The design modules will only design for weld fasteners so any bolting will have to be designed elsewhere. Just as in the handbook, factored weld group capacities are determined using instantaneous centers and ultimate load deformation characteristics of the weld. The design modules are more versatile than the handbook tables as the user has more control over the input parameters.

1. DOUBLE ANGLE CONNECTIONS:

The following three connection design modules assume welded fasteners are used to connect the angles to both supporting and supported members. They are intended to provide the final design connection details and support code related restrictions.

a.) Beam to Column Flange .....@1[MAPLCALC.EXE b_to_cf]

b.) Beam to Column Web .....@1[MAPLCALC.EXE b_to_cw]

c.) Beam to Girder .....@1[MAPLCALC.EXE b_to_g]

If the connection is incorporating both welded and bolted fasteners than the following design modules may be accessed strictly for the determination of weld group capacity.

a.) Weld connecting angles to supporting member ... @1[MAPLCALC.EXE op_vert]

b.) Weld connecting angles to supported member .... @1[MAPLCALC.EXE c_open]

2. SINGLE ANGLE CONNECTIONS:

The steel handbook only provides tabulated capacities for one form of this connection, namely, welded to the supporting member and bolted to the supported web. The capacity of the weld on the supporting member may be determined in @1[MAPLCALC.EXE s_angle].

3. TEE CONNECTIONS:

The welded fasteners in this connection will be looked at separately because it is not common practice to field weld this type of connection and because the handbook contains no tabulated capacities of supported beam web welds.

a.) Shear and moment welds on supporting member ... @1[MAPLCALC.EXE op_vert] b.) Shear and torsion weld on supported beam web .. @1[MAPLCALC.EXE c_open]

4. END PLATE CONNECTIONS:

Because the weld design here involves only two parallel fillet welds subjected purely to shear, a design module will not be generated. The steel handbook contains the following tabulation "Factored Shear Resistance of Fillet Welds per Millimeter of Weld Length" that may be used to determine factored capacities quite easily.

5. SINGLE SIDE WEB PLATE CONNECTIONS:

The design module, @1[MAPLCALC.EXE op_vert], is setup to determine the factored capacity of two vertical parallel fillet welds subjected to combined shear and bending. Therefore the capacity of the welds joining the side plate to the supporting member may be determined but the capacity of the bolted connection joining the side plate to the beam web will have to determined elsewhere.

9.2.2 Flexible Seated Shear Beam Connections

All seated beam connections, both flexible and stiffened, are designed for simple shear only. The seated shear beam eccentricities are generally larger than for the framed beam shear connection, and as a result they may influence the design of the supporting members.

The flexible seated shear beam connection consists of a relatively heavy rolled angle (the seat angle) used for bearing with a generally lighter rolled angle (the clip angle) attached at the top of the beam for stability. Alternatively, the clip angle may be attached to the side of the beam web close to the top flange. @1[flex18.pcx] and @1[flex19.pcx] illustrate the details of this connection.

@1[09.2.2.1 Flexible Seated Shear Beam Connection Design Considerations] are provided to enhance the understanding of this connection.

The @1[09.2.2.2 Flexible Seated Shear Beam Connection Design Module] is provided as a designing tool for this connection.

9.2.2.1 Flexible Seated Shear Beam Connection Design Considerations

The following points should be duly considered when designing flexible seated connections:

1. The connection is generally satisfactory and efficient in carrying relatively light loads. Obviously the maximum for the loads applied will be set by the design capacity of the currently available largest sizes of rolled angles.

2. The length of the outstanding leg of the angle seat must be sufficient to accommodate the minimum length of bearing as established on the basis of rulings in the governing specifications plus a customary clearance of about 12 mm. As an added comment, it should be mentioned that there are diversified opinions amongst several authorities regarding the applicable level of bending stress on the critical section of the horizontal leg.

3. In finalizing the design of the seat angle the following aspects of its size must be satisfactorily resolved:

a.) the width (w) and the thickness (t) must be such as to keep the bending stresses in the critical section of the horizontal leg within allowable stresses or factored resistances;

b.) the width (w) of the angle should be such as to permit easy field welding along the toes of the beam flange with ample edge distance provided;

c.) the width (w) should also be such as to prevent the return welds (of length equal to twice the weld size) from infringing on the clearance provided for erection;

d.) the width (w) should also include ample edge distance for the fillets on the vertical leg and the supporting member;

e.) the thickness (t) and the length of the vertical leg (L) should be adjusted so as to keep the thickness at least 2 mm greater than the required fillet size.

4. The weld size must be within the "minimum size fillet/material thickness" rulings of the design standard CSA W59.

5. The recommended size of the clip angle on the top or optionally on the web of the beam is 100 x 100 x 6 or 75 x 75 x 5. Although its length will vary with the individual flange there are some efforts to standardize it at 100 and 150 mm.

6. The clip angle should be great enough to allow welding to the beam and column, but small enough not to inhibit rotation of the beam. When fillet welds are used to connect the clip angle to the beam or column, the fillet welds should be located along the toes of the angle.

7. This connection is more conducive to a supporting flange connection because welding access to a supporting web may be restricted.

9.2.2.2 Flexible Seated Shear Beam Connection Design Module

The design spreadsheet generated for this connection is meant to replace the welded portion of the "Seated Beam Shear Connections" Table in the Steel Handbook. This program will be much more versatile, with respect to welded design, than the Handbook tabulations because there are fewer restrictions on the input parameters. The program implements the following design procedure:

1.) Determine the end beam shear reaction, R

2.) Use the Web Crippling and Yielding formulae as laid out in Clause 15.9 of CAN3-S16.1-M89. Formula (i) of the end reactions section will provide a minimum length of bearing under the beam web.

3.) Specify a length of seat bearing that is greater than the minimum determined from 2.) above. See @1[flex21.pcx]. The size of the outstanding angle leg will be the chosen bearing plus a 20 mm gap between the beam end and the support face.

4.) Assuming a plastic moment develops in the angle seat at the critical location, determine the minimum length of seat given the thickness or visa versa. Seat angle outstanding leg capacities are developed using the plastic section modulus, Zx, and the full length, L, of the unstiffened seat. Although the gap between the end of the beam and the face of the supporting member is nominally 10 mm, calculated values will be based on a 20 mm gap to allow for fabrication and erection length tolerances.

The CISC Steel Handbook determines outstanding leg capacities by satisfying the following two conditions simultaneously:

1. crippling of the beam web and

2. minimum thickness of seat angle required.

The design spreadsheet utilizes the same principle.

Vertical leg weld capacities are calculated using the instantaneous shear center model with an out of plane eccentricity. For calculation purposes, the compression zone width for the angle is assumed to be 25 mm wide

To begin the design, link to @1[MAPLCALC.EXE un_stif].

9.2.3 Stiffened Seated Shear Beam Connections

The stiffened seated shear beam connection becomes necessary when the reaction is too large for the flexible seated connection to safely handle.

This connection consists primarily of a rolled Tee section, or Tee section formed from two plates, which is attached by shop welds to the supporting member in such a manner that the flange of the tee provides a seat for the end of the beam to be supported.

The behavior of the stiffened seated connection is essentially the same as that of the flexible seated connection. However, due to the greater stiffness of the horizontal element of the connection the lever arm for the reaction has been generally agreed to extend closer to the outer end of the seat.

For standardized cases of stiffened seated connections (as tabulated in Table 3-40 of the handbook) the eccentricity is taken as between 0.55*w and 0.60*w and the horizontal welds are taken as 0.2*L. See @1[flex22.pcx]

@1[09.2.3.1 Stiffened Seated Shear Beam Connection Design Considerations] are provided to enhance the understanding of this connection.

The @1[09.2.3.2 Stiffened Seated Shear Beam Connection Design Module] is provided as a design tool for this connection.

9.2.3.1 Stiffened Seated Shear Beam Connection Design Considerations

The following points should assist the designer in better understanding the essential requirements of this connection.

1. This connection is capable of supporting considerable loads. Architectural considerations rather than others may be responsible for setting an upper limit on the acceptable practical size of the seat.

2. The position of the reaction is undetermined, but for a general case it is assumed at the center of the bearing length "a" as established from crippling considerations laid down by governing specifications. This length for design purposes is measured from the end of the beam rather than from the outer end of the seat, which results in some discrepancy. In the standardized stiffened

seated connection the lever arm of the reaction has been arbitrarily set in the range 0.55 to 0.60 of the projected seat. For beams at right angles to the stems of the Tee the eccentricity is measured from the face of the supporting member to the center of the supported beam.

3. The welds connecting the T-section to the supporting member are assumed to be subject to combined shear and bending. The instantaneous center theory, @1[09.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups], is used to analyze the T-configuration of welds on the supporting member.

4. Because the vertical welds are very close it is the horizontal welds on the supporting beam that are relied on to provide some measure of torsional stiffness in the connection. These welds may vary individually from 0.2L to 0.5L although in the case of standardized connections they are set at 0.2L.

5. The fillet size must be within the "minimum size fillet/material thickness" rulings of the design specification (CSA W59).

6. If the T-section was welded and not rolled then that weld must have a greater capacity than the horizontal weld connecting the flange of the T-section to the supporting member.

7. The stem of the Tee should be kept on the shorter side so that buckling will give no cause for concern. The stem thickness should ordinarily be no thinner than the web of the beam, However, its actual thickness will be determined by one of the following:

- bearing stresses on the critical horizontal section,
- shearing stresses on the critical vertical section,
- the effective size of fillets, which for balanced design will require a comparatively strong stem.

Note that for the stem thickness and weld size to be in balance (assuming E480XX electrodes and 300 MPa steel) the stem thickness must be 1.7 times the fillet weld size.

Also, if the minimum yield stresses of the tee and the beam are unequal then the following equation must be satisfied ...

(ts * Fy)stiffener = (tw * Fy)beam

8. The recommended size of the clip angle is 75 x 75 x 6 mm or 100 x 100 x 6 mm. The only welds on these angles for both optional locations are the welds along the toes.

9. The width of the flange of the seat should be wide enough to accommodate the supported beam with ample edge distance provided for the side fillets along the flanges of the supported beam.

10. When stiffened seats are in line on opposite sides of a column web, the size of the vertical fillet welds (for E480XX electrodes) shall not exceed Fy/515 times the thickness of the column web, so as not to exceed the shear resistance of the column web. As an alternative to limiting the weld size a longer seat may be used to reduce the shear stresses in the column web.

9.2.3.2 Stiffened Seated Shear Beam Connection Design Module

Factored resistances, tabulated in the steel handbook, of stiffened seats for the T-shaped weld configuration are based on the use of E480XX electrodes and steel with a minimum tensile yield strength Fy = 300 MPa. These constants will become variables in the design spreadsheets. Factored resistances are computed based on instantaneous center theory for combined shear and moment modified for the T-shaped weld configuration used.

This design spreadsheet has not yet been created.

9.3 Non-Standard Connections

Many variations from standard framing practice exist and special attention is required to ensure the design and fabrication of efficient flexible connections. Two types of loads can be applied to various weld groups to produce either in plane eccentricities or out of plane eccentricities. Bearing pad connections (used mainly in the UK and Australia) will also be looked at as well as direct bearing connections.

The @1[09.3.1 Non-Standard - In Plane Eccentricity] module deals with both elastic and instantaneous center analyses.

The @1[09.3.2 Non-Standard - Out of Plane Eccentricity] module also deals with both elastic and instantaneous center analyses.

@1[09.3.3 Direct Beam Web Connections] are rare in fabrication because of their demanding tolerances.

@1[09.3.4 Bearing Pad Connections], although virtually nonexistent in North America to date, have been a part of both the UK and Australian framing practice for many years now.

9.3.1 Non-Standard - In Plane Eccentricity

This type of loading produces both shear and torsion on the weld group. Analysis of weld group capacity has traditionally involved linear elastic analysis however the handbook now uses the non-linear instantaneous center of rotation analogy to provide capacities of standard weld groups.

Elastic Analysis

The elastic analysis implemented here first involves the determination of the weld group centroid. The applied load is then transferred to the centroid with the resulting torsional moment. The stresses developed in the weld group due to the shear force and torsion moment at the centroid sum vectorially to produce a resultant stress which varies along the length of the weld. The most critical region(s) of the weld group will govern the design.

The @1[09.3.1.1 Design Module for In Plane Eccentricities] supports elastic design with both horizontal and vertical loading on several weld groups.

Instantaneous Center

Welded connections subjected to eccentric loading can be designed using limit states concepts. The method utilized is based upon the non-linear load-deformation response of the weld and the instantaneous center of rotation analogy. The ultimate load is reached when the ultimate strength and deformation of some critical weld element (usually the element farthest from the instantaneous center) is reached. The instantaneous center of rotation is located using the equations of statics. More information is in @1[09.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups].

The @1[09.3.1.1 Design Module for In Plane Eccentricities] supports instantaneous center design with vertical loading only on several weld groups.

9.3.1.1 Design Module for In Plane Eccentricities

This module accesses the following design programs:

1. *Elastic design - both vertical and horizontal loads*

   a.) Box shaped weld group ...@1[MAPLCALC.EXE boxweld]

   b.) C_shaped weld group ...@1[MAPLCALC.EXE cweld]

   c.) L_shaped weld group ...@1[MAPLCALC.EXE lweld]

   d.) Two parallel vertical weld lines ...@1[MAPLCALC.EXE vertweld] e.) Two parallel horizontal weld lines ...@1[MAPLCALC.EXE horweld]

The elastic design programs were written using spreadsheet format and they provide the user with the efficiency of the connection given all of the input parameters (factored shear loads, eccentricities, weld dimensions). Input parameters may be altered until the desired efficiency is achieved.

2. Instantaneous center design - vertical loads only

a.) Box shaped weld group .....@1[MAPLCALC.EXE box_all]

b.) C_shaped weld group - load on open side .....@1[MAPLCALC.EXE c_open]

c.) C_shaped weld group - load on closed side .....@1[MAPLCALC.EXE c_close]

d.) L_shaped weld group - load on open side .....@1[MAPLCALC.EXE l_open]

e.) L_shaped weld group - load on closed side .....@1[MAPLCALC.EXE l_close]

f.) Two parallel vertical weld lines .....@1[MAPLCALC.EXE par_ver]

g.) Two parallel horizontal weld lines .....@1[MAPLCALC.EXE par_hor]

The instantaneous center design programs were also written on spreadsheets however they require an external .EXE program to determine the coefficient C (in units of kN/mm^2) as seen in the handbook of steel construction. Once the input has been entered, macro execution provides for the coefficient. These programs were written to replace the design tables in the "Eccentric Loads on Weld Groups" section of the CISC handbook.

9.3.2 Non-Standard - Out of Plane Eccentricity

This type of loading produces both shear and bending on the weld group. The more traditional elastic analysis may be used or alternatively instantaneous center of rotation capacities may be determined.

Elastic Analysis

The traditional elastic method of design and analysis assumes that the shear force per unit length is uniform over the entire weld. The flexural force per unit length caused by the eccentricity of the load can be found with the usual bending equation by assuming the neutral axis to pass through the centroid of the weld group. The critical location occurs where the vector summation of the unit shear force and unit flexural force reaches a maximum.

The @1[09.3.2.1 Design Module for Out of Plane Eccentricities] supports elastic design, with vertical loading only, on several weld groups.

Instantaneous Center Analysis

As with the in plane eccentricities, this method is also based on the load deformation capacity of the weld group and the instantaneous center of rotation analogy. The one difference here is that the bearing area of the connecting plate in the compression zone becomes a factor in the analysis. The weld in the compression zone is assumed to carry shear only with the bearing stresses being carried entirely by the plate. The handbook only tabulates coefficients for parallel vertical welds, however with slight modifications in the analysis, many other weld configurations may be dealt with. More information on this topic may be found in the module @1[09.2.1.2.1.1 Instantaneous Centers and Eccentrically Loaded Weld Groups].

The @1[09.3.2.1 Design Module for Out of Plane Eccentricities] supports instantaneous center design with vertical loading for a few standard weld groups.

9.3.2.1 Design Module for Out of Plane Eccentricities

Options in this design module include the following:

1. *Elastic design - vertical loading*

    a.) Box shaped weld group

    b.) Single vertical weld line

    c.) Two parallel vertical weld lines

    d.) Two parallel horizontal weld lines

The elastic design programs were written on spreadsheets and they provide the user with the efficiency of the connection given all of the input parameters (factored shear loads, eccentricities, weld dimensions). Input parameters may be altered until the desired efficiency is achieved.

2. *Instantaneous center design - vertical loading*

    a.) Box shaped weld group

    b.) single vertical weld line

    c.) Two parallel vertical weld lines

    d.) Two parallel horizontal weld lines

The instantaneous center design programs were also written on spreadsheets however they must run an external .EXE program to determine the coefficient C (in units of kN/mm^2) as seen in the handbook of steel construction. Once the input has been entered, macro execution provides for the coefficient, C. These programs were written to replace the design tables in the "Eccentric Loads on Weld Groups" section of the CISC handbook.

9.3.3 Direct Beam Web Connections

Although this connection is quite rare in fabrication, it may be theoretically acceptable as a flexible connection with the proper detailing to produce low resistance to bending while still maintaining safe shear transfer.

Connection flexibility may be supplied by the supporting member as in the case of a supporting beam or girder. If the supporting member is essentially rigid than the rotation capacity must come from the weld detail which should be made short enough and stout enough to produce prior yielding in the supported beam web. The weld will be short enough if it is not greater than two thirds the depth of the beam and stout enough if its thickness is greater than four fifths the thickness of the supported beam web.

Practically speaking this connection has some serious drawbacks. There is no support for the beam inherent in this connection and therefore temporary supports are required before and during welding that must be removed later so as not to impair the flexibility of the connection. Also, the tight fit-up conditions necessary for producing effective fillet welds conflict with the erection clearances necessary for ease of construction. A comfortable compromise of say 3 mm would be difficult to achieve because of square cutting tolerances on the supported beam and rolling tolerances on the supporting column.

9.3.4 Bearing Pad Connections

One component in this connection consists of an end plate which is shop welded to either the upper end, the lower end, or the full depth of the beam. The end plate location determines the center of rotation for the beam end. Field bolting is accommodated with two holes in the bottom of the end plate. The end plate bears onto a bearing pad which is shop welded to the supporting member. A shimming plate may be used to control the erection gap.

The load is considered to be transferred by shearing and bearing. The end plate length and connecting welds are governed by shear. Beam web buckling and connection torsional stability should be checked under critical conditions.

As with the direct beam-web connection this connection also has some serious practical drawbacks. The mill tolerances on the column depths are very demanding in order to ensure the required minimum bearing length is achieved. Great care must be taken to ensure the accurate placement of both the end plate and the bearing pad.

10 Moment Connections

Welding is most efficient in structures designed for continuity. Rigid frame or continuous construction demands moment resisting beam-to-column connections. @1[moment2.pcx] illustrates a common rigid connection.

Frye and Morris demonstrated that all connections will develop a moment if the rotation becomes large enough. The diagram in @1[Moment1.pcx], which illustrates moment rotation curves for various connections, was developed by Frye and Morris. It can be seen from the diagram that varying degrees of end fixity produce varying degrees of connection rigidity. The most important difference between the rigid and semi-rigid connections is the magnitude of moment that the connection will develop at small rotations. The rigid connection will develop the full beam moment at relatively small rotations.

Under service conditions, it is assumed that rigid connections possess sufficient rigidity to maintain, virtually unchanged, the original angles between the beams and the columns with which they connect. In addition to adequate strength and stiffness, suitable rotation capacity (ie. ductile behavior) is required to permit inelastic deformations to occur. These inelastic deformations are necessary in developing the full strength of the connection.

The high degree of restraint associated with the rigid connection demands careful design considerations to properly handle possible stress concentrations. In elastically analyzed structures, stress concentrations are alleviated through small amounts of yielding. Plastically designed structures depend on connection ductility to enable the large amounts of yielding necessary for the redistribution of moments.

The beam end moments lead to both tensile and compressive flexural stresses that must be transferred into the column. This transfer may be accomplished directly or with the use of separate plate material. Similarly, shear transfer may be provided with a direct groove weld on the beam flange or with plated material such as header angles, one-sided plates, or seats. @1[moment3.pcx] illustrates both direct and plated flexural stress transfer. @1[moment4.pcx] shows the same only for shear.

In many instances, column stiffeners will be required to prevent local failures from destroying the integrity of the connection. Laminations existing in the column flange plate could also lead to failure, especially when welding under conditions of restraint. Refer to @1[06.4 lamelar tearing].

Built up column sections require additional considerations to ensure the web to flange weld develops the required capacity.

The Designer must provide the Fabricator with complete information regarding moments and shears to be developed by the connection. Because weld inspection plays such an important role in maintaining quality control, the Designer together with the Fabricator should determine the type and degree of inspection prior to fabrication to avoid unforeseen problems.

10.1 Rigid and Semi-Rigid Connections

Semi-rigid connections:

In semi-rigid connections, the beam is designed for the total elastic moment less the moment developed by the end connection. Unfortunately, it is no easy task to determine the moment developed by the end connection.

Semi-rigid connections are not often used for the following reasons:

1. The stiffness of the connection is difficult to define and, therefore, the structural analysis is more complicated. As such, the analysis is less well understood by design engineers.

2. The moment rotation curves used in the analysis must be determined from physical testing on full scale specimens.

3. Both simple and rigid connections have become more standardized over the years while semi-rigid connections still remain the hazy field in between.

Rigid connections:

Rigid connections develop the maximum beam moment while maintaining suitable rotation capacity. In order to maintain the integrity of this connection, the following considerations apply:

1. Local buckling of essential elements must be prevented.

2. The criterion for stiffening of the column web opposite the compression flange of the beam must be examined.

3. Similarly, the criterion governing the use of stiffeners opposite the tension flange of the beam must be investigated.

4. The column may need to be reinforced to handle combined axial and local stresses.

5. The connection must deliver rotation capacity demands.

Note that clause 21.3, Restrained Members, of S16.1-89 defines the requirements for the stiffeners in points 1 and 2 above.

10.2 Column Stiffeners

It is mostly the column which requires a thorough examination of its critical elements. Such investigation will include a check on requirements for:

1. Stiffening of the column web to prevent crippling under the compressive flexural flange load of the beam;

2. Stiffening of the column flanges for the tensile flexural flange load of the beam to prevent their deformation and ensure a reasonably uniform stress distribution in the tension joint of the beam;

3. Reinforcing of the column web to maintain shear stresses within those permissible;

4. Reinforcing of the column web-to-flange weld for build-up columns for all cases of excessive longitudinal shear stress in such welds.

The first three point have to be satisfied in light of the pertinent requirements of CSA Standard, S16.1-M89 Limit States Design of Steel Structures. In particular, clause 21.3, Restrained Members, gives requirements for stiffeners provided on the column web when beams are rigidly framed into the flange of an H-type column.

The resistance of the column section to local deformation is crucial for developing the full connection capacity. Column web yielding in tension or compression or column web buckling will seriously impair the moment carrying capacity of the connection. The use of stiffeners or doubler plates may be necessary for local strengthening of the column. Doubler plates are commonly used to increase the column web shear capacity in the panel zone. Inclined stiffeners may also be used to achieve the same result. If the plastic shear capacity, 0.55(phi)(w)(d)(Fy), of the panel zone is exceeded then stiffeners are required.

Weld failure in tension, usually from non-uniform stress distribution, is another possible mode of failure. A horizontal column stiffener adjacent to the beam tension flange will virtually eliminate the curling of the column flanges which would otherwise tend to overstress the central portion of the weld between the tension flange and the column. Horizontal column stiffeners are also used adjacent to the beam compression flange to prevent buckling of the column web. One sided stiffeners need only extend to one half of the column depth provided the welds connecting the stiffener to the column web develop ... P = A(st) x F(y).

On those occasions when beam flanges framing in from opposite sides of the column are at different heights, inclined stiffeners or two horizontal stiffeners may be used. According to Graham, Sherbourne, and Khabbaz, if the difference in flange height is less than 50 mm then only one stiffener need be used, however the stiffener thickness must be increased by a factor of 1.7. Three methods of framing for eccentric flange connections are shown in @1[moment9.pcx], @1[moment10.pcx], and @1[moment11.pcx].

A [test comparison of stiffener types] gives a brief summary on rigid beam-to-column connection testing done through the AISC in 1959.

10.2.1 S16.1-M89 21.3 Restrained Members

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load. When beams are rigidly framed to the flange of an H-type column, stiffeners shall be provided on the column web as follows:

(a) opposite the compression flange of the beam when

see [restrain.pcx]

except that for members with Class 3 or a webs,

see [restrain.pcx]

(b) opposite the tension flange of the beam when

Tr = 7 [phi] (tc)^2 Fyc < Mf / db

where

wc = thickness of column web

tb = thickness of beam flange

k = distance from outer face of column flange to web-toe of fillet, or to web-toe of flange-to-web weld in a welded column

Fyc = specified yield point of column

db = depth of beam

hc = clear depth of column web

tc = thickness of column flange

The stiffener or pair of stiffeners opposite either beam flange must develop a force equal to

Fst = {Mf / db} - Br

Stiffeners shall also be provided on the web of columns, beams, or girders if Vr calculated from Clause 13.4.2 is exceeded, in which case the stiffener or stiffeners must transfer a shear force equal to

Vst = Vf - 0.55 [phi] w d Fy

In all cases, the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of the column only, the stiffeners need not be longer than one-half of the depth of the column.

10.2.2 Test Comparison of Stiffener Types

1. Direct beam-to-column flange connection without the use of column stiffeners. Refer to @1[moment12.pcx]. In the actual connection tested, the beam was stiffer against rotation than the column. As would be expected, the beam tension flange normal stress distribution showed higher concentrations of stress at the center of the flange. Although the connecting groove weld also showed the same stress concentration, no weld failures occurred until after excessive rotation had taken place.

2. Horizontal stiffeners are added to the column adjacent to both beam flanges. Refer to @1[moment13.pcx]. The column is now as stiff against rotation as the beam. The normal stress distribution in the beam's compression flange was essentially uniform however the tension flange still exhibited higher magnitudes of stress in the center.

3. @1[moment14.pcx] illustrates the use of wide-flange Tee section stiffeners. Here again the columns are as stiff against rotation as the beam. From strain gage readings it was determined that each of the vertical plate stiffeners in the elastic range transmitted only about 3/16 of the forces coming from the beam flanges while the column web transmitted 5/8. Replacing the vertical stiffeners closer to the web may produce a more uniform distribution however it would defeat the primary purpose of this type of connection. The Tee stiffeners are used in a 4-way connection to provide easy access for joining and therefore the plates should be positioned flush with the column flange tips. At the working load, the stress distribution was uniform in both flanges. At 1.5 times the working load, however, large tensile stresses occurred at the mid-flange.

4. @1[moment15.pcx] illustrates 4-way framing without additional stiffeners. The connection shown in the figure was found to be stronger than its 2-way counterpart. This would indicate that the stiffening action provided by the beams framing into the column web outweighs the weakening by triaxial stress.

5. Tee stiffeners in a 4-way connection are considered here. @1[moment16.pcx] illustrates the connection. Under these conditions, the beams welded directly to the column flanges proved stiffer than those welded to the Tees. In the latter case, with the column flanges being relatively far away, the stems of the Tees are providing most of the resistance. Meanwhile the column web is ably assisted in preventing rotation at the connection by the flanges of the split-beam Tee stiffeners.

10.3 General Recommendations for Welded Rigid Connections

The following are the general recommendations which will affect the overall economy of welded rigid connections and which are offered for the fabricator's consideration:

1. Take full advantage of any possibility to create a high degree of repetition of details on shop drawings especially with respect to the fabrication of floor beams. These may not change in size for many floors and the slightly changing column sizes will allow maintenance of the same "centre-column to centre-hole in beam-web" distance;

2. Investigate the possibility of using standardized pre-assemblies welded in the shop for minimum welding and speedy erection in the field;

3. Use minimum number of separate pieces;

4. Confine holes to detail material or lighter sections to avoid handling of heavy structural sections for drilling or punching;

5. Use fillets rather than groove welds, where possible, to avoid preparation of edges and close fit-up requirements;

6. Design welds for actual stresses; do not overweld;

7. Keep the number of horizontal stiffeners in columns fitted at both ends to a minimum;

8. Use vertical stiffeners on toes of column flanges in cases of 4-way framing especially when the pair of column web connections is to be flexible; a plate or a T section may be conveniently used here but the selected alternative must be designed in size and thickness so as to be fully compatible with the expected behavior of the connection in question.

In the case of direct rigid connections:

9. Check the actual geometry of columns and beams prior to detailing, unless provisions are made to accommodate the allowable mill tolerances and the probable field misalignment in the fit-up, so that the actual joint configuration may be accepted from the point of view of execution of welding;

10. Make suitable provisions for the cumulative shrinkage effects caused by welding. For example, preset the columns and enlarge the holes in the beam webs to accommodate temporary bolts.

11. The use of fillet welded shear connections on the web will eliminate the refinement necessary in sequencing of welding operations when full direct butt welding is anticipated.

In the plated versus the direct rigid connections the following summary may be taken as representing the pertinent economical considerations applicable to the plated solutions:

A. Due to ample end clearances the mill tolerances lose their significance;

B. No specific fit-up or erection difficulties are normally encountered;

C. There are no cumulative effects of shrinkage due to welding. Butt welds are welded with no restraint present. Fillet welds are capable of absorbing shrinkage stresses when deposited under conditions of restraint;

D. Additional plate material required involves cutting and handling;

E. Double amount of welding is required.

The following are a number of observations with respect to moment connections using plates for the top and projecting elements (plates or legs of angles) for the bottom flanges.

In the case of the top plate the thickness of the plate must be chosen so as to allow ample edge distance for placing of fillet welds along the sides of the plates.

The amount of edge distance will be affected by:

1. The actual length of the plate;
2. The accuracy of fit-up at the column;
3. The out-of-squareness of the column flanges;
4. The mill tolerances in the width of beam flanges;
5. The actual amount of gap between the top of the beam and the sloping plate this gap will necessitate an appropriate increase in fillet size so as to maintain its required effective size.

10.4 Built Up Column Sections

In the case of built-up columns, where there is a large differential in beam moments causing high longitudinal shear stresses between the column web and flange, the weld capacities may have to be correspondingly increased within the depth of the connection. Increased fillets or partial grooves with superimposed fillets may be appropriately used here. The figure, @1[wsize20.pcx], serves well to illustrate the two basic requirements for the welds holding the component plates in a built-up column section together:

1. The entire length of the column must have sufficient welds to withstand any longitudinal shear resulting from moments applied to the column from wind or beam loads.
2. Within the region of beam connection to the column, this long longitudinal shear is much higher because of the abrupt change in the bending moment within the depth of the beam.

Also the tensile force from the beam flange will be transferred through a portion of this weld. These two conditions require a heavier weld in this region.

There are four common methods for joining the flange to the web and supplying additional weld strength at the critical panel zone location.

Method 1. This method is used when the required weld size is relatively small. The joint is first fillet welded along the entire length of the column and then additional passes are made in the beam connection region to bring that weld up to size. Refer to @1[moment5.pcx].

Method 2. The web plate is first prepared with a bevel on all 4 edges for the entire length of the column. Then, in the critical region around the beam connection, a fillet weld is made over top of the groove weld to achieve the necessary weld size. Refer to @1[moment6.pcx].

Method 3. This method starts off by only preparing the web for a groove weld in the region of the beam connection. The groove weld is then made flush with the surface in the connection region. Next, a fillet weld is produced along the entire length of the column. Refer to @1[moment7.pcx].

Method 4. First the column web is beveled to the necessary depth along the entire length of the column and then the connection region is further beveled to a significantly deeper depth. This region is groove welded first to build the plate edge up to the height of the first bevel. Then the entire length of column is groove welded to complete the joint. Refer to @1[moment8.pcx].

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