Field Welding to Existing Steel Structures

DAVID T. RICKER

Field welding to existing members is becoming increasingly common. During the recycling of older structures, new performance requirements often require addition of reinforcing material to increase load-carrying capacity, to restore areas eroded by corrosion, to strengthen fire-weakened members or perhaps alter the appearance of a member by changing its shape for aesthetic reasons. One of the many advantages of a steel-framed structure is that it can be reworked more readily than structures of other materials.

Field conditions are often far from ideal and it is necessary to ascertain the effects of the field work on the existing structure, especially the common method of attachment—welding.

Over the years, various rules-of-thumb have been applied: no welds where calculated stress exceeds 50% of working stress; no transverse welds across tension members; no welds where calculated stress exceeds three ksi unless shored; make all welds parallel to stress lines, etc. Most of these made sense at the time and were the result of practical experience plus common sense.

Safety, economic considerations and the endless search for understanding and refinement require us to delve deeper into the subject of field welding to existing steel structures. These items merit attention:

1. Weldability of existing and new steel
2. Selection and design of the weld
3. Anatomy of the weld
4. Heat input
5. Position of the weld
6. Surface conditions
7. Weather conditions
8. Nature of the load
9. Nature of the reinforcing
10. Shoring and stress relieving
11. Reinforcing connections
12. Effect of field alterations on the entire structure
13. Fire hazards and precautions
14. Testing and inspection

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In pursuing these topics, it is necessary to address three parties in the discussions—the designer (engineer), the fabricator and the erector (welder). These remarks are often directed to all three simultaneously, but occasionally only to one or two. It is hoped the temporarily uninvolved party bears with us until his turn comes again. Some very basic information is included for the benefit of those less-experienced who desire to increase their understanding of the subject. Some information is repeated in certain chapters to benefit those who seek specific knowledge or who cannot, for some reason, read the entire article.

1. THE WELDABILITY OF STEEL

By 1900, steel had almost replaced completely wrought iron as a building component, although the latter continued to be used for certain applications mainly associated with its relative imperviousness to atmospheric corrosion. Cast iron, once a popular material for columns and the fittings used in heavy timer construction, was fast dying out. Steel became the dominant building metal. Prior to about 1910, there was little standardization in the industry. Each steel producer used his own recipe and rules. This resulted in a wide variety of chemical and mechanical properties. Some of this early steel is very weldable and some is not. But the fact that it may have been made after 1910 does not guarantee automatically its weldability.

The obvious first test for weldability is to examine the existing steel work to see if welding was used during the original fabrication and erection or if the structure has been successfully welded onto previously.

Any steel whose weldability cannot be confirmed by such positive evidence, or whose chemical and mechanical properties cannot be verified with mill certificates or other documentation, should be tested. This can be done by cutting or drilling samples from redundant parts of the existing steel and having them *mechanically* tested for ductility and *chemically* tested for carbon content and other ingredients. The test samples should be taken from the thickest

**This paper was developed as part of ASCE Committee on Steel Building Structures.*

parts of the members, since these areas usually exhibit the greatest property fluctuation (Fig. 1). If possible, test the specific members on which the welding is to be performed.

A simple on-site test can be made by welding a lug of weldable steel to the existing member and beating it with a hammer (Fig. 2). Refer to the Fillet Bend Test as described in AWS Spec. D1.1, paragraphs 5.38.3 and 5.39.3.[2] If the weld deforms without fracturing, the steel can be considered weldable. If the weld separates from the base metal at the junction of the weld and base metal it indicates the base metal is subject to hardening, often the result of high carbon content. The separation exhibits a gray granular surface and usually follows the profile of the heat affected zone, appearing as a shallow crater in the base metal. This is not a very sophisticated test (about as refined as the hammer test on welded shear studs), but it is better than nothing and is useful in emergencies or when a testing agency is not readily available. It is not recommended if other methods of determination are available. The use of low-hydrogen welding electrodes and preheating can improve the weldability of most base metal.

If it is suspected the existing material is wrought iron, welding should be avoided if possible. If welding is mandatory, the welds should be oriented so tension loads are not exerted on the member in a direction perpendicular to the

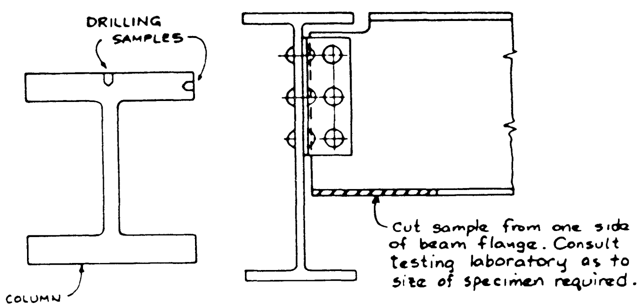

Figure 1

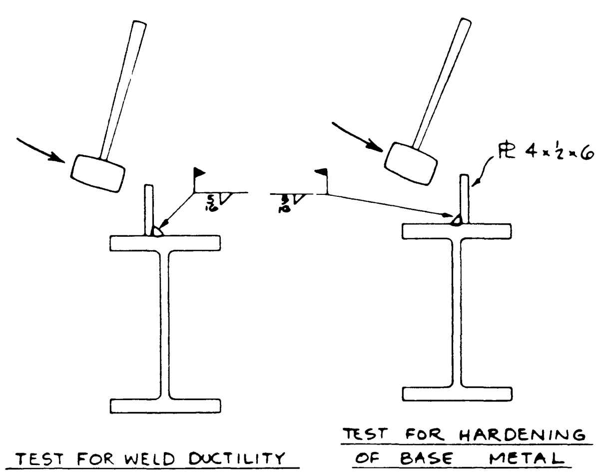

Figure 2

direction of rolling. Wrought iron tends to contain slag inclusions which form in layered laminations and does not possess good trans-lamination strength. It is recommended that a competent, experienced welding authority be consulted before attempting to weld on wrought iron. The same recommendation pertains to welding to cast iron.

Three of the ingredients of structural steel have considerable effect on its weldability: carbon—too much of which results in high hardenability and eventual loss of ductility, excess phosphorus which increases brittleness, and high sulfur which often results in porous welds.[3] The American Society for Testing and Materials (ASTM) lists bounds for these and other ingredients of steel by which we can determine its weldability.

If the chemical properties of the existing steel are known, the weldability can be determined by the *carbon equivalent*. There are quite a few carbon equivalent formulas in circulation today. A carbon equivalent of less than 0.48 in the following formula in Ref. 8 generally assures good weldability.

Carbon equivalent $= C + Mn/6 + Cr/5 + Mo/5 + V/5 + Ni/15 + Cu/15 + Si/6$

where C = carbon content (%)
Mn = Manganese content (%)
Cr = Chrominum content (%)
Mo = Molybdenum content (%)
V = Vanadium content (%)
Ni = nickel content (%)
Cu = copper content (%)
Si – silicon (%)

Another comprehensive carbon equivalent formula in Ref. 7 is:

Carbon equivalent $= C + Mn/6 + Ni/20 + Cr/10 + Cu/40 - Mo/50 - V/10$

If the results of this formula are below 0.40 the material is readily weldable. For values between 0.40 and 0.55, use of preheat or low-hydrogen electrodes is suggested. Values above 0.55 indicate an increased likelihood that cracks may develop unless special precautions are exercised—precautions which are not likely to be feasible under field conditions.

When a carbon equivalent formula is used, the laboratory analysis report should be required to list the quantities of each of the elements in the formula, even if the percentage reported is zero.

Much of the steel used in buildings in the first 60 years of the 20th century was ASTM-A7, a medium carbon steel. This is generally accepted as being a weldable steel, but it had a wider range of permitted carbon content than its ASTM-A36 eventual successor. The interim ASTM-A373 steel generally is considered to have good weldability. Although weldable with most electrodes in the E60, E70 classes, certain levels of carbon and other ingredients in ASTM-A7 steel may require low-hydrogen electrodes and preheating. The following carbon equivalent formula in Ref. 7 is useful in determining *preheat* requirements:

Carbon equivalent = $C + Mn/6 + Ni/15 + Cr/5 + Cu/13 + Mo/4$

The preheat temperatures suggested for several ranges of carbon equivalent are:

Carbon equivalent	*Suggested preheat temperatures**
0 to 0.45%	preheating optional
0.45 to 0.60%	200°-400°F.
over 0.60%	400°-700°F.**

If excessive carbon equivalent values are indicated, then further investigation of the existing steel should be considered, such as destructive testing. *Before welding to existing steel, its weldability must be established positively by one of the methods mentioned.* The AISC publication, *Iron & Steel Beams 1873 to 1952* is helpful in identifying and dating hot-rolled sections prior to 1952. It also lists some allowable stresses of that era.

From the foregoing, it should be evident the higher the carbon content the less adaptable is the steel to welding and for unweldable situations other means of fastening should be considered. Other means usually refers to bolting.

The past service record of the structure may be very helpful. Were members subjected to fatigue, impact, vibration, stress reversal, cyclical loading, a corrosive atmosphere or extremes of temperatures? A record of this past history, together with the intended future use, helps to determine the anticipated extended life of the rejuvenated structure and the means to accomplish it.

2. SELECTION AND DESIGN OF THE WELD[4,5]

The following suggestions may aid in deciding on weld selection and placement:

1. Use fillet welds in preference to groove welds where the design permits. Groove welds are more costly and often result in overkill. Fillet welds are made easily and are dependable.
2. Arrange the welds if possible so they are made in the flat or horizontal position (Fig. 3).

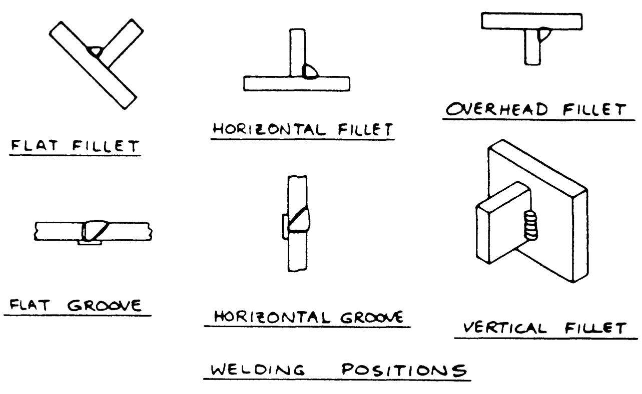

Figure 3

3. Try to avoid cutting across stress lines with the weld, especially if the member is carrying load. If unavoidable, then consider shoring or reducing the load to a safe level. If undercut is a potential hazard, design the weld so that any undercut will occur in the new material where it can be accounted for (Fig. 4). If an undercut occurs, especially in a member loaded in tension or subject to fatigue, it should be ground out, filled with sound weld metal, and ground smooth. If possible keep the grind marks parallel to the stress lines or reduce the grind mark profile by finishing up with light pressure.
5. Avoid biaxial and triaxial stresses at welds or in the immediate vicinity by altering the geometry of the joint (Fig. 5).
6. Avoid overwelding, both excessive size or quantity, since this may accentuate distorsion and shrinkage.
7. Avoid abrupt geometric discontinuities which are located at welds. This is a major source of weld failures, especially in tension-loaded members (Fig. 7).

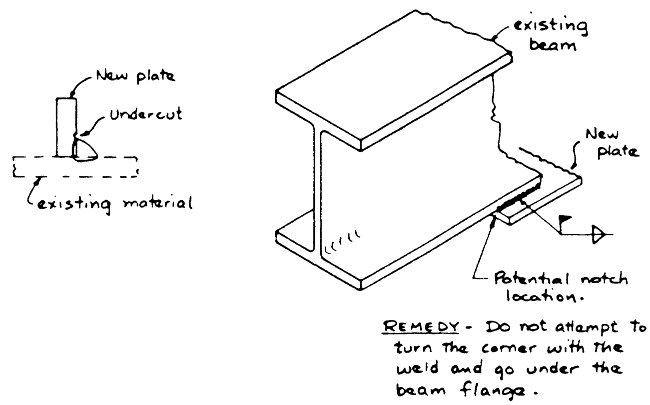

Figure 4

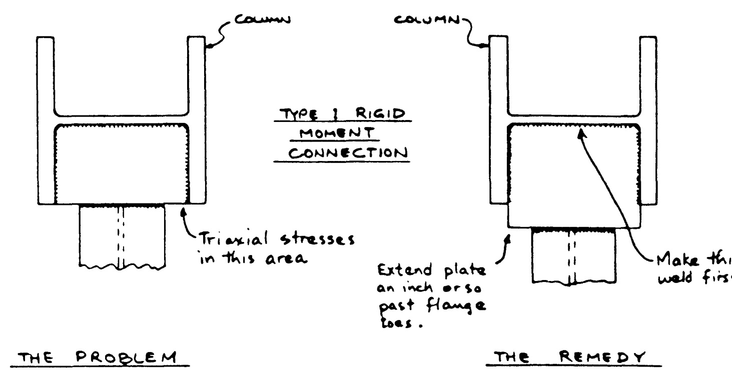

Figure 5

*If AWS specifications prevail it is recommended that AWS Spec. D1.1—latest edition (Table 4.2) be followed.

**Some codes recommend that maximum preheat not exceed 450°F.

Minimum Preheat and Interpass Temperatures for Common Building Steels			
Steel Specification	Welding Process	Material Thickness	Min. Preheat Temp. – F°
ASTM A7, A9, A140, A373, A36, A53, A500, A501	SMAW-other than low hydrogen electrodes	Up to 3/4" Incl.	None
		Over 3/4" thru 1-1/2"	150°
		Over 1-1/2" thru 2-1/2"	225°
		Over 2-1/2"	300°
ASTM A7, A373, A36, A53, A242, A441, A500, A501, A572 Gr.50, A588	SMAW-with low hydrogen electrodes, SAW, GMAW, FCAW	Up to 3/4" Incl.	None
		Over 3/4" thru 1-1/2"	50°
		Over 1-1/2" thru 2-1/2"	150°
		Over 2-1/2"	225°

Figure 6

Matching Filler Metal Requirements for Common Building Steels					
Steel Specification	Min. Yield Point (ksi)	Min. Tensile Strength (ksi)	Matching Electrode Specification	Min. Yield Point (ksi)	Min. Tensile Strength (ksi)
ASTM A7	25-33	50-65	SMAW		
A9	25-33	50-65	AWS A5.1 or A5.5		
A140	33	60-72	E60xx or	50	62 Min.
A36	36	58	E70xx	60	72 Min.
A53 Gr.B	35	60	SAW		
A500	41	58	AWS A5.17 or A5.23		
A501	36	58	F6x-Exxx or	50	62-80
			F7x-Exxx	60	70-90
			GMAW		
			AWS A5.18		
			ER 705-x	60	72 Min.
			FCAW		
			AWS A5.20		
			E6xT-x	50	62 Min.
			E7xT-x	60	72 Min.
ASTM A242	42	63	SMAW		
A441	42	63	AWS A5.1 or A5.5		
A572	42	60	E7015, E7016	60	72 Min.
A588	50	65	E7018, E7028	60	72 Min.
			SAW		
			AWS A5.17 or A5.23		
			F7x-Exxx	60	70-90
			GMAW		
			AWS A5.18		
			ER705-x	60	72 Min.
			FCAW		
			AWS A5.20		
			E7xT-x	60	72 Min.

Figure 6a

8. Consider a joint design that uses the least volume of weld. Least weld usually means least heat and least distortion (Fig. 8).
9. If appropriate, use a partial-penetration weld in place of a full-penetration groove weld (for column splices, for example, and other *static compression* welds).
10. For static loads, consider intermittent fillet welds in lieu of continuous welds. Do *not* use intermittent welds for cyclical loads which may result in metal fatigue since the allowable stress for this type is heavily penalized. See AISC 8th Edition Manual Appendix "B".[6]
11. After the desired weld has been determined, make sure the proper weld symbol is used on the drawings.
12. Orient welds so contraction strains are imposed on the base metal in a longitudinal direction, to diminish the possibility of lamellar over stress.
13. Follow AISC Spec. 1.5.3 and AWS Spec. D1.1–Table 4.1.1 with regard to matching weld metal. Complete penetration groove welds perpendicular to tension stress lines *must* have weld metal which matches the base metal. Most other welds can be made using weld metal with a strength level equal to or less than matching weld metal. Weld metal one strength level stronger than matching weld metal will be permitted for most welds. But it is not recommended, because higher strength welds have less ductility and the potential for cracking increases. (Fig. 6).
14. Avoid arc strikes and splatter in highly stressed areas. These cause hard spots which can initiate cracks in the base metal, especially if the carbon content is high.
15. Do not attempt to weld to cast iron members. Special procedures are required which may be impossible or at best difficult to accomplish in the field.

3. THE ANATOMY OF A WELD[3]

Most field welding in building construction is done by the arc-welding process. This, in effect, creates a small electric furnace at the adjacent bounds of the material to be joined. It takes two electrodes to make an arc—one is the steel building and the other the welding electrode (rod). The welding rod, the smaller of the two by some degree, is designed to melt and be consumed into the molten pool in combination with the part of the structure which also melts.

Immediately adjacent to this molten pool is the heat-affected zone (HAZ), a region about ⅛ in. to 3⁄16 in. in width between the molten mixture of deposited weld rod plus base metal and the heated but unchanged base metal (Fig. 9). In the HAZ, changes occur in the molecular and grain structure of the heated steel, and ductility and hardenability are affected. But oddly enough, relatively few weld-associated problems start here. Cracks usually do not start in the HAZ. They may start in unsound weld metal near the HAZ and travel into the base metal or vice-versa.[3]

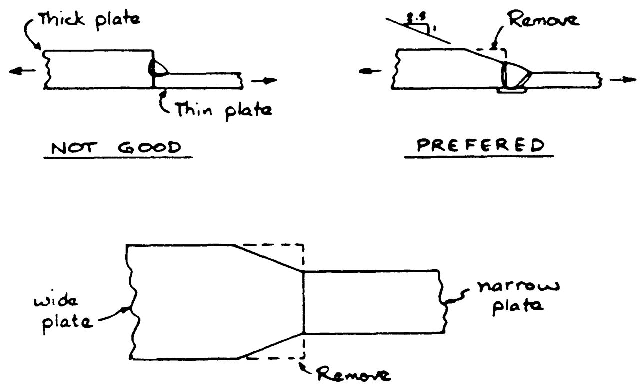

Figure 7

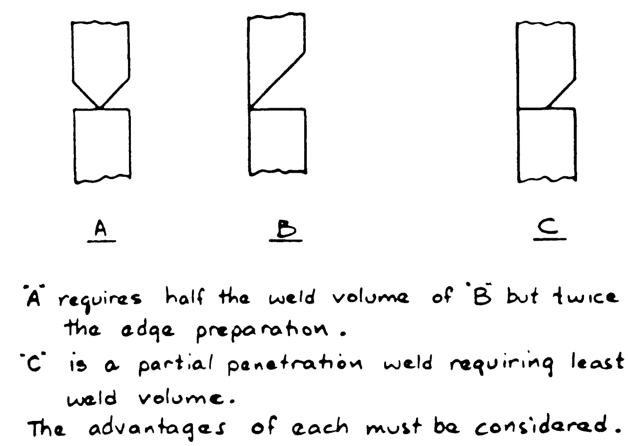

Figure 8

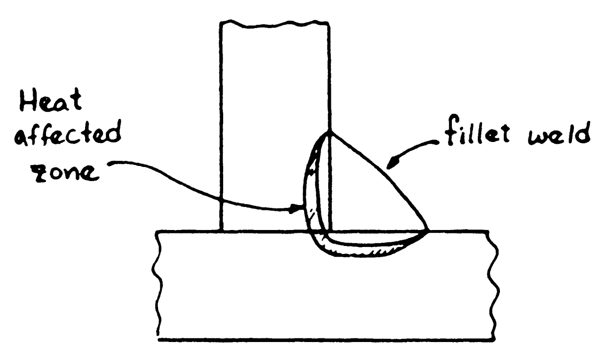

Figure 9

The temperature of the molten pool is near 3,000°F. (Steel melts at roughly 2,800°F. Fig. 10). The HAZ is in the 1,250 to 2,700°F. range. The base metal temperature ranges from about 1,250°F. on down to the ambient temperature. Some other significant temperatures are: 1,333°F., above which austenitic changes take place; 1,500°F., the yield stress is near zero and steel loses its magnetic properties. This is one reason why many codes limit the temperature of heat applications to about 1,200°F. maximum (for heat cambering or straightening for example). At 500-600°F., steel attains its greatest strength, but at the cost of some of its ductility.

The significance of all this to the subject at hand is that when we weld to an existing building member it is probably carrying a load. If so, we must not jeopardize or impair its ability to do so by melting away too much of the load-carrying area. That makes a lot of sense. More on this later.

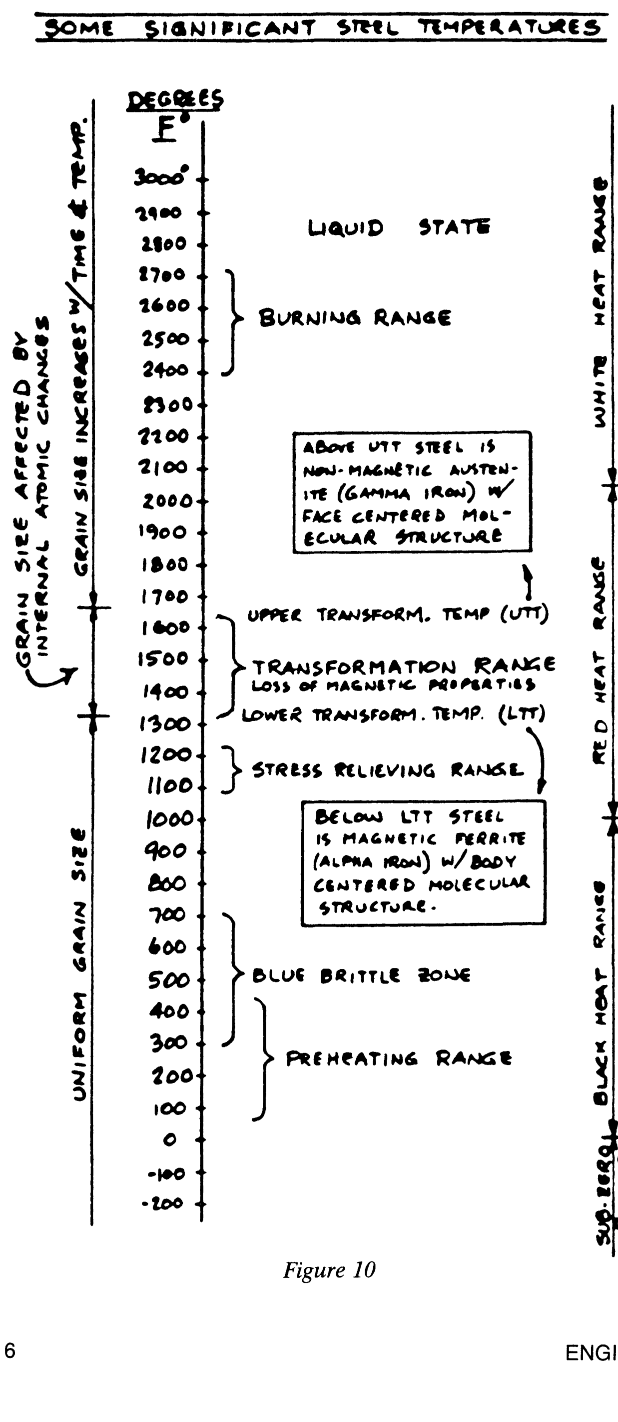

Figure 10

Weld metal is usually more pure than the metals it joins. A properly executed weld is almost always superior in mechanical properties to the base metals.

Much of the field welding today is done by the shielded metal arc process (SMAW), using manual stick electrodes. Another popular method is flux-cored arc welding (FCAW), a semi-automatically fed hollow-wire electrode with flux inside the hollow.

One does not have to be an engineer or metallurgist to lay a good weld, but a welder who develops an understanding of the basics of the subject can improve markedly the quality of his work.[3] Failures in service rarely occur in properly designed and executed welds. Most fractures start at a notch, groove, discontinuity or other type of stress-riser.

4. HEAT INPUT[3,3.1]

The welding process, by definition, produces heat. The members to be connected must be within a certain temperature range prior to the start of welding, the weld must be applied as rapidly as conditions will permit and the welded area must be allowed to cool properly. The process, routine under shop conditions, is often more complicated when applied to existing members under field conditions. But the same sets of rules apply. The AWS Spec. D1.1[2] has established easy-to-follow preheat rules. Table 4.2 in the same specification shows minimum preheat and interpass temperatures for various types and thicknesses of steel. Maximum preheat should generally not exceed 450°F. The preheat temperature must extend a minimum distance of 3 in., or the thickness of the material if over 3 in., in any direction from the weld and must be maintained adequately in advance of the weld. When two different strength steels are being welded together, the preheat for the higher strength steel governs. On multi-pass welds the proper interpass temperature must be maintained. The purpose of preheating is to slow down the *rate* of cooling. Welds cool principally by conduction. Thus, a thicker member will draw heat away from a weld faster than the same weld on a thinner member. This is one reason why thicker plate welds are more prone to cracking and the required preheat temperatures are higher than for thinner material (Fig. 6).

Welds also cool by convection—heat lost into the air. Thin plates are more susceptible to cooling by convection. *Rate* of cooling should not be confused with *time* required for cooling. Thick material may have a faster rate of cooling but the elapsed cooling time will be longer because many more BTU's must be dissipated than on a thin plate which has much less heat input. When welding is done transversely across existing members, it may be necessary to perform the welding in several stages to allow cooling time in between. This procedure results in residual stresses, but these are usually of little consequence.

The temperature of the molten weld pool is about 3,000°F. or greater. At these temperatures, the internal structure of the metal has changed. The extent of the change depends on the maximum temperature, the length of time this temperature is sustained, the composition of the metal and the rate of cooling. Steel maintained at a high temperature for an extended length of time produces large grain size and results in a cooled steel which is harder and less ductile. For this reason it is important the cooling process not be delayed.

As the molten area cools the motion of the atoms, as they skittle around seeking their rightful place in the grand scheme of things, becomes slower and slower until the freezing point of the metal is reached. At this stage the random motion of the atoms ceases and they have all joined hands and assume a fixed location with regard to each other, forming a solid metal. As the temperature further decreases, the atomic motion continues in the form of vibrations. Atoms arrange themselves in space lattices, each comprising a unit cell. A crystal (or grain) of metal may contain millions of unit cells. Rapid cooling usually results in the formation of many small grains because the unit cells do not have time to link hands with more than a few hundred thousand neighbors. A fine-grain texture results. Slower cooling permits more unit cells to join together into larger (but fewer) grains. The larger grains have greater spaces between them and the intergranular stresses are higher.

When steel is welded, a melting process that has been done previously is repeated. This is another of the many good qualities of steel. It can be re-melted and still regain its strength. The re-melting causes relief of the internal stresses, agitation of the atomic structure, but eventual recovery of the familiar properties we expect of steel. For these reasons, a new weld bead can be laid on top of a weld bead which may be many years old, and it will be effective and reliable.

A very important factor is the amount of heat applied—both from preheat and from the actual welding process. This should be kept to a minimum and the welding process completed as quickly as practical. To avoid excessive heat, use arc welding process, intermittent welds if conditions permit and skip around. And on multi-pass welds allow the weld to cool between passes.

Why dwell on this matter in discussing welding to existing members? When we create our little electric furnace during the welding process, we affect both the existing member and the new reinforcing piece. Although steel actually gains strength up to about 600°F., it then starts to taper off, and at 1,500°F. it has hardly any strength at all. When the member being welded is carrying a load it is important to know the extent to which we diminish its ability to maintain the load. (The same can be said of any load-carrying member whether we attack it with torch, drill or weld rod). When we create a molten weld pool we must know how much of the cross section is affected and to what extent. We have already mentioned that steel reaches its maximum strength at about 600°F., at which time it starts to trail off. At about 650°F., it has the same strength as at 75°F. (room temperature). The temperature of the steel dissipates rapidly as we move away from the weld so that in thick (over ⅜ in.) material 650°F. is attained about an inch away from the pool. In thinner material welded at slow weld speeds, this distance is about 1¾ in. The danger area seems easy to calculate. But remember, the welder is advancing his weld pool. How fast does the molten pool cool down to 650°F. behind him to the point where we know it will be strong enough to again carry its share of the load? This depends on the mass of the material and heat input, and can vary from a few minutes to over half an hour. If welding is done parallel to stress lines, this cooling time has less significance. But if the design requires a weld perpendicular to stress lines, the welder must be aware of the amount of cross section temporarily taken out-of-action at any given time. Temperature crayons or other suitable means should be used to monitor the temperature of the worked-on pieces.

Weld cooling is as important as heating up. When welds cool too rapidly or too slowly, crystalline changes occur. Internal stresses can increase to a troublesome point, sometimes causing local failures (otherwise known as cracks).

For low-carbon (0.10% to 0.30%) steel, cooling rates are relatively insignificant. However, a slower cooling rate results in more toughness and ductility, and rapid cooling gives harder, more brittle steel. For members subject to tensile stresses this is important. For compression members ductility is less important.

Another method used to delay the cooling rate is post heating. Post heating prolongs the cooling cycle and allows the hydrogen and other harmful gas bubbles to dissipate from the molten weld. Post heating is seldom required, except under extenuating circumstances, such as if preheating were not performed. Post heating requires heats to about 1,100°F., far greater than the 450°F. maximum for preheating. Post heating in the field often presents logistical problems. Preheating, much more effective than post-heating and generally less expensive, should be given priority.

Any discussion of the heat of welding must include the root pass. The root pass, especially when welding on thick material, is the most critical part of the weld. It is subject to the greatest rate of cooling (by conduction) and, since it is the thinnest part of the connection (at this stage), is subject to cracking (Fig. 11). If a root weld crack is detected it must be removed by grinding or air-arcing, since it is likely to spread into subsequent layers of the weld.

As a weld cools, stresses build up similar to the residual stresses in any hot-rolled shape. The established allowable stresses recognize the almost universal presence of these welding residual stresses. Residual stresses have little in-

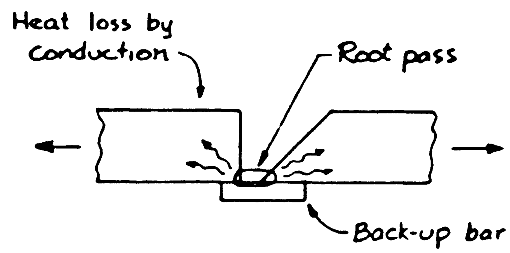

Figure 11

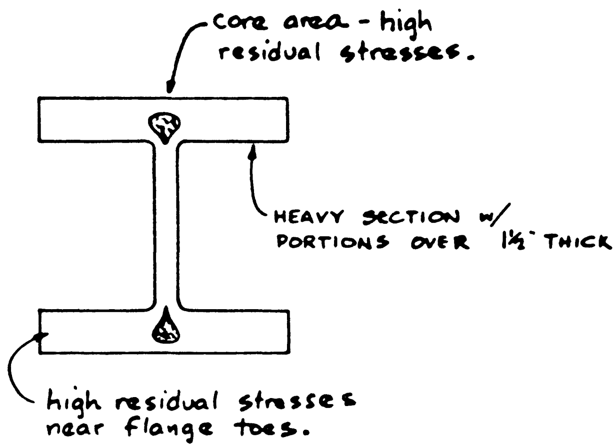

Figure 12

fluence in failures unless there is a flaw, notch, abrupt change in cross section or other stress riser present, which concentrates their effect. If the cooling weld is prone to crack it will usually have done so before the steel passes below 200°F. However, some cracks are known to have started several weeks after welds were made. All weld beads should be inspected carefully after cleaning.

There has been some concern in recent years as to the wisdom of welding to heavy wide-flange sections with parts exceeding one and one-half inches in thickness, especially when these members are used in tension applications (Fig. 12). The problem is that cross sections of these heavy members are sometimes non-homogenous, the result of slower cooling of the thickest parts which causes pockets of large-grain structure and high residual internal stresses. Segregation of the various ingredients of the steel may also occur.

If groove welding, rough grinding, flame cutting, undercuts, notching, arc strikes or other similar work is performed in the core area serious stress increases can occur and may spawn cracks. These cracks are sometimes subsurface and not easy to detect. Partial penetration welds in heavy tension members should be avoided because of the presence of stress concentrations in the root area.

However, heavy sections used in *compression* situations have a long history of reliable performance. The *surfaces* of heavy sections are not susceptible to the same deficiencies as the cores. Consequently there is no problem in welding to the surface of these members, provided the rules for weld size, preheating and post heating are observed.

5. POSITION OF THE WELD

There are four weld positions: flat, horizontal, vertical and overhead (Fig. 3). Flat and horizontal welds are the easiest and most economical, and usually the most reliable and best quality welds.

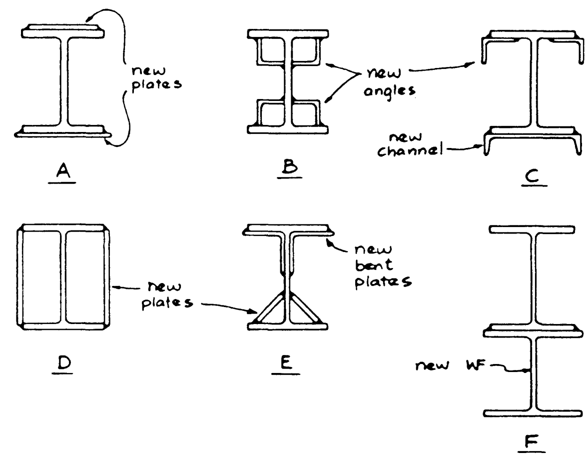

Figure 13

Unfortunately, in welding to an existing structure, the members are often at awkward or inaccessible locations. This presents a real challenge to the designer to devise a scheme and select reinforcing members most convenient to the welder and at the same time efficient to their performance (Fig. 13).

Fillet welds can be made in all positions with relatively little difficulty. Flat and horizontal *groove* welds are also fairly easy to accomplish. Vertical and overhead groove welds are more costly because of the greater volume of weld metal deposited and the relatively slow rate of deposition.

Different types of welding electrodes are available for welding in different positions. *Fast-fill* electrodes melt rapidly and are best suited for flat or horizontal welds where a high rate of deposition is called for. *Fast-freeze* electrodes solidify rapidly, hence are often used for vertical and overhead welds. *Fill-freeze* (also called fast-follow) electrodes are a compromise between the fast-fill and fast-freeze rods. A properly trained, experienced welder knows the correct type and diameter of electrode to use without advice from design drawings. The designer should indicate the proper classification of electrode to use. For example: low-hydrogen rods for high-strength steels (E7018, E7019, E7028); E8018-C1 or C3 electrodes when a good color match is required for weathering steels; E7018 on weathering steels where a color match is *not* important; low-hydrogen electrodes where adverse weather conditions are anticipated (such as high humidity or low temperatures). Most manufacturers of welding electrodes publish comprehensive guides on good welding practice, including electrode selection. Readers are urged to obtain these useful guides.

6. SURFACE CONDITIONS

Existing members are often in deplorable condition, corroded, covered by many layers of paint, oil, grease or other contaminants. These fouling elements should be removed prior to welding. Severely corroded steel must be cleaned to bare metal and any feathered edges removed by burning or grinding to present a proper weld surface. Most other surface contaminates can be removed with flame, brush, scraper or grinder. If the contaminates cannot be removed because of inaccessibility or extra tight adhesion, then the first surface passes should be made with E6010 or E6011 electrodes. A slow travel speed allows time for gas bubbles and contaminates to rise to the surface and boil out where they can be cleaned away prior to the next passes. E60XX and E70XX electrodes can be used at the same weld. If the existing members are galvanized, galvanizing should be ground off in the area to be welded. Good ventilation should be provided so the welder will not inhale any of the noxious gases produced by welding inadvertantly through a galvanized surface.

7. WEATHER CONDITIONS

When the welding must be performed in sections of the structure exposed to weather it is wise to heed this advice:

1. Do not weld if the temperature of the steel in the vicinity of the weld area is below 0°F. Steel, which possess great toughness (ductile strength) at normal working temperatures, becomes brittle at lower temperatures.
2. When the steel temperature is between 0° and 32°F., preheat and maintain the steel at 70°F. during the welding process, except where thickness or type of material requires greater preheat. Refer to AWS Spec. D1.1 Table 4.2 for preheat requirements.
3. It is just as important to observe preheating requirements in the summer because of the danger of electrode coatings absorbing moisture from the air, with subsequent absorption of hydrogen into the molten weld pool. The use of low-hydrogen electrodes helps solve this problems.
4. Do not weld on wet or frost-covered surfaces because of the danger of entrapping hydrogen from the moisture. The presence of hydrogen in steel increases its brittleness.
5. Frigid weather conditions may require post-heating in order to slow the cooling rate. This allows time for harmful gases to escape the melted area and helps assure a more ductile weld. The use of low-hydrogen weld electrodes lessens the amount of preheat and post-heat required.
6. Protection from wind should be provided while welding is done, so shielding gases will not be diluted and blown away, to retard the loss of preheat and interpass heat and also for the comfort of the welder in frigid weather. A steady hand is important to making good welds.
7. Do not artifically cool welded areas by water spray or forced air. This invites brittleness and loss of ductility as compared to the base metal.
8. Protect welding electrodes from adverse atmospheric conditions. Abide by exposure time and reheat recommendations provided by the manufacturer.
9. If the location of the completed joint is such that it may become wet, even by wind-blown spray, or dampened by condensation, the remainder of the joint not involved in the structural weld should be sealed with additional weld to prevent moisture entering between the joint plies. (Other sealants are also available for this purpose).

8. THE NATURE OF THE LOAD[4]

Four factors should be considered when welding to an existing member:

1. Type of load to be applied
2. Manner in which the load is applied
3. Duration of the load
4. Stresses in the existing member at the time welding is performed

The types of loads are: tension, compression, shear and combinations thereof. The loads may occur as static, cyclical (causing fatigue), vibration, reversible (extreme compression to extreme tension) and impact.

The history of the member, when known, aids the engineer in judging if and how to apply the weld. For instance, a transverse weld on a tension member which has been subject to fatigue loading over a long duration should be avoided because of the possibility of adverse granular changes. (Tests are available for determining the nature and extent of granular deformations.) Likewise, members subject to impact loads at extremely low temperatures should be welded with care, using low hydrogen electrodes. On the other hand, compression members statically loaded can be welded with less concern for future performance.

There are a great many other combinations of load conditions which affect the choice of welds. The following deserve special attention by the designer:

a. Welds tranverse to tension stresses
b. Welds on members with a long history of fatigue loading
c. Welds on members subject to extremes of stress reversal

When these conditions are encountered, the various rules and precautions should be observed as set out in codes and specifications. Proper allowable stress reductions should be applied for fatigue, impact and other penalty conditions. Refer to the AISC *Manual of Steel Construction*, 8th Edition.[6]

Some codes require that when a building is reinforced for one reason or another, the entire structure be upgraded to meet all current code requirements. For example: placing a new air-conditioning unit on an existing roof may require some minor reinforcing. But the current applicable building code may require a sizeable part of the roof be strengthened to meet increased snow-drift design requirements, even to requiring new foundation work. This can be a shock to an unsuspecting owner.

9. NATURE OF THE REINFORCEMENT

We have previously discussed various aspects of the weld itself. Let us examine the things being welded. Sometimes the reinforcement is relatively simple, such as an increase in the size or length of an existing weld. Sometimes we must add additional material, either to restore the member to its original strength or to increase its strength to support a greater load. The purpose of reinforcement may be to increase the area of the member cross section or to add to its stiffness so it will have an increased allowable stress. Occasionally, material is added for aesthetic reasons, such as plates to a wide-flange column to change its shape from an H to a square or rectangle. Sometimes entire sections of a member must be replaced.

After the member is reinforced, the end connections may require upgrading to carry greater loads (touched on later).

It is often necessary to add A36 material to high-strength steel members. If the addition is made for strength, differences in working stresses must be considered. If new material is added to increase stiffness or lessen deflection, then the size and arrangement of the material is of prime importance. ASTM-A36 material is appropriate to use with higher strength steels in either case.

If material is added to a column, such as channels or cover plates to improve bending strength, a tight fit bearing at base and cap plate is not necessarily required. But if the new material is added to increase the *bearing* area, then good bearing *is* required. This, however, does not call for a shrink fit. It is common practice to cut the material short by about ½ in. or so (depending on material thickness) and to fill the gap with weld metal to accomplish the proper bearing (Fig. 14).

It is recommended that the member be relieved of as much load as possible prior to reinforcing. In some cases this is impractical or impossible. However, the successful performance of multitudes of structures reinforced while under stress confirms the fact the procedure is safe and feasible in most cases.

When new material is to be added to an existing member *under stress*, some engineers require the area of the new piece to be computed using a stress equal to or less than the *allowable* stress of the base material minus the *actual* stress in the base material at the time the new steel is added. This is a conservative approach. When cover plates are added to a column, the anticipated allowable stress can be based on the properties of the built-up section.

Example

Original allowable column stress = 16 ksi

Anticipated allowable column stress = 18 ksi (a result of lesser L/r ratio after reinforcing)

Actual stress at time of reinforcement = 12 ksi

Stress to be used in calculating the area of the new steel is $18 - 12 = 6$ ksi.

To increase this usable stress in the new steel it is sometimes desirable to relieve some of the load on the original member prior to adding reinforcing. This can be done by removing live load or by shoring. In the example above, if the actual stress at time of reinforcement can be reduced from 12 to 8 ksi, then the usable stress in the new steel will increase from 6 ksi to 10 ksi.

The reverse is true if the anticipated alteration increases the column L/r value: for instance, if column flanges must be narrowed to clear an obstruction but are then reinforced with narrow plates to restore the necessary area. The following example illustrates:

Example

Original allowable column stress = 16 ksi

Anticipated allowable column stress (due to an in-

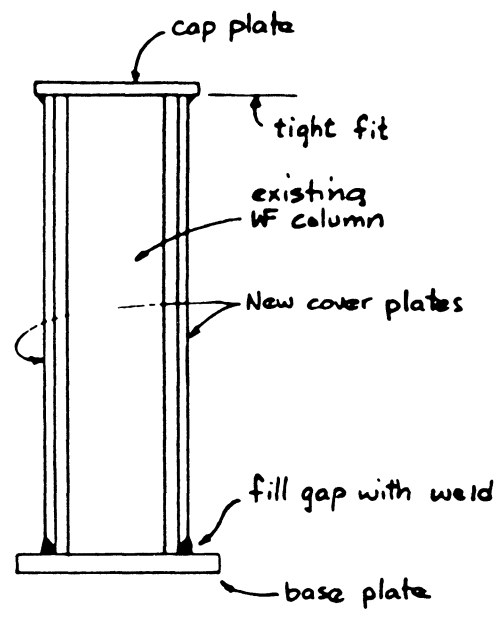

Figure 14

creased L/r ratio of the final reinforced section) = 15 ksi

Actual stress at time of reinforcement (the result of load relief by means of shoring) = 8 ksi

Stress to be used in calculating the area of the new reinforcing steel = 15 − 8 = 7 ksi

In this example, it is shown to be beneficial if the original section is left intact until the new reinforcement is in place and welded. However, this is not always possible because of accessibility requirements for welding.

In the examples just used, we assumed no part of the reinforced member will be stressed beyond the *allowable* working stresses as set forth in the AISC *Manual of Steel Construction*.

Before reinforcing steel is added to an existing member, a decision must be made as to whether to keep the working stresses within the normal allowable stress range or to allow some of the reinforced member to exceed the normal allowable stresses and approach the yield stress. The service conditions should be considered. Some designers are willing to permit certain members or parts of them to operate above normal working stress. The increased deformation from above-normal stresses should also be considered. When AISC established the ratio of working stress to yield at 0.6 it simultaneously set a fairly comfortable limit for deformation, a limit most construction materials can handle. The excessive deformation of one or more members may have an adverse effect on other parts of the structure and also on the connections. It may be wise to keep the reinforced member deformation from exceeding that intended for the original member. This is a matter of engineering judgment. For a distorted member it must be decided whether to freeze (lock) the piece into its damaged position or attempt to return it to its original shape. This depends heavily on the nature and location of damage and the effect on the rest of the structure. This accents another good feature of steel—heated steel regains its strength after cooling.

When welding a cover plate to a beam or column it is wise for the welder to start at one end and work in one direction toward the other end, or to start in the middle and work toward both ends. Do *not* start at the ends and work toward the middle. Since the cover plate and main member are rarely the same size, their heat absorption rates vary, resulting, if the ends are restrained, in a cover plate that looks like an inch worm.

There may be a temptation when cover plating a *compression* member to take advantage of this inch-worm effect to reduce compression stresses in the original member. There are many unpredictable variables, such as the amount of end restraint, exerted on heated members by the rest of the structure and the rates of heat absorption and cooling of member components. It seems unrealistic to rely on such nebulous factors to rationalize a stress reduction.

In a case where a column is reinforced with a pair of cover plates, it helps assure the final straightness of the column if cover plates are welded simultaneously and directly opposite each other. Testing at Lehigh University[9] of cover-plated WF columns produced these observations:

1. Heat generated during welding increased the length of the column and was followed by a shortening of the column when the member cooled. When the column length is held constant (or nearly so) there is a corresponding change in load. This load change is transmitted in turn to those other adjacent members of the structure involved in the restraining action.
2. Welding changed the residual stress distribution in the original WF column flange tips. In the sections tested (W8×31 with flange cover plates 7×3/8) the residual stress changes were beneficial. However, in heavier sections the change in residual stresses resulting from welding *may not* work to advantage. Further study is required on this subject.
3. Tests showed the ultimate stress of the reinforced members was not reduced by welding.
4. The influence of welding is confined to a very small area in the weld vicinity. Material properties were not affected enough to reduce the strength of the section.

After a cover plate is welded to another member it is assumed the weld deformation is so slight there is virtually no differential movement between the two elements. Visualize the loading progressions of a cover plate with zero initial stress welded to an existing WF member already carrying some load. If the allowable working stresses of *both* pieces are kept within elastic limits the mated elements will act in perfect unison, each load increment increasing the stresses and strains in direct proportion. However, if the load is increased so stresses in the WF members rise past the yield point into the plastic range, then, since the two elements are locked together by the welds, the plastic WF will tend to stretch the cover plate along with it. The cover plate, on the other hand, resists the effort to yield because it is still in the elastic range. At this stage, the behavior of the combined elements, when subjected to further load increment, depends greatly on the proportion of the areas of the two elements. If the area of the added plate is relatively small, then a small load increment will result in the plate also entering the plastic range. The resulting yield eventually will be arrested by strain hardening of the member, unless the welds fail or some other type of failure occurs first. If the area of the plate is relatively large, it takes a correspondingly larger load to get it into the plastic range. Early literature[10] suggests a safe condition exists as long as the maximum stress in the original member is not over one third greater than the average unit stress of the combined section. It was assumed the average unit stress would not exceed the allowable working stress.

The question to be asked is this: Is it wise to allow parts of the structural frame to approach the plastic stress range? In light of the difficulty in analyzing existing loadings, with the uncertainties of residual stresses and changes in residual stresses caused by welding, how much does an engineer dare to flirt with the yield point? If he keeps all his stresses within the normal working stresses (0.6 Fy) he should not have a problem either with strength or deformation. If he decides to exceed 0.6 Fy, the amount is up to his engineering judgment. If the reason to add the cover plate is because the normal allowable working stress would otherwise be exceeded, is it wise to go to the trouble of adding a cover plate and still have the original part of the member exceed the normal allowable working stress? One of the most important decisions facing the engineer involved in alteration work on existing steel structures is the selection of the working stress limit.

AWS Spec. 8.12[2] lists guide lines for welding together the components of built-up members with reference to stitch-weld spacing and unsupported width and length limits for plates. The integral parts of a built-up member are assumed to be connected so they act together, and that a given strain will produce a uniform stress change in all parts. Contrary to some belief, it is *not* necessary the new reinforcing material match the parent material.

Example:

Original member of A572 Gr.50 steel has an allowable bending stress = 0.6F = 30 ksi

Actual stress at time of reinforcement = 18 ksi

Usable stress in calculating the area of the new reinforcing = 30-18 = 12 ksi.

A36 reinforcing steel, which has an allowable stress greater than this, will do the job nicely.

If, on the other hand, to increase the effectiveness of the reinforcing, the decision is made to relieve the existing member of most of its load so the actual stress at time of reinforcing was reduced to, say 2 ksi, then the usable stress in calculating the area of the new reinforcing becomes 30-2 = 28 ksi. Since this is greater than the allowable stress for A36 steel, a higher strength steel must be used.

The load capacity of a non-composite beam supporting a concrete slab can be increased by adding shear connectors to make it composite. This is accomplished by core-drilling holes in the existing concrete slab over the beam at specified intervals, welding the studs and filling holes with non-shrink grout. If this method is contemplated, potential deflection should be investigated. For lack of a better method, welded shear connectors should be tested with the standard AWS "hammer test" (AWS Spec. 7.7), especially if the original beam material is unknown. Adding a bottom-flange cover plate will increase the beam capacity, especially when used in conjunction with the added studs (Fig. 15). Even without the studs, the bottom-flange cover plate can be an effective means of reinforcing. When the load capacity of any beam is increased, the existing *connections* should be investigated to determine if they, too, need upgrading.

When the area of a tension member is increased by adding angles or plates, reinforcing must be adequately connected to the gussett plates or whatever type of end connection exists. The resultant connection must be strong enough to transfer the new loads (Fig. 16).

If the new cover plates or angles are of a length which requires they be spliced in the field, the groove-welded splice should be made and allowed to cool before the new material is welded to the existing member, so groove weld shrinkage will not affect stresses in the connected pieces.

If welding is performed across a member, transverse to the stress lines, when the weld cools it will shrink and tension stresses appear. If the member was initially in *tension*, the weld will return to a tension condition and the resulting shrinkage stresses may be lesser or greater than original stresses. If the member was initially in *compression*,

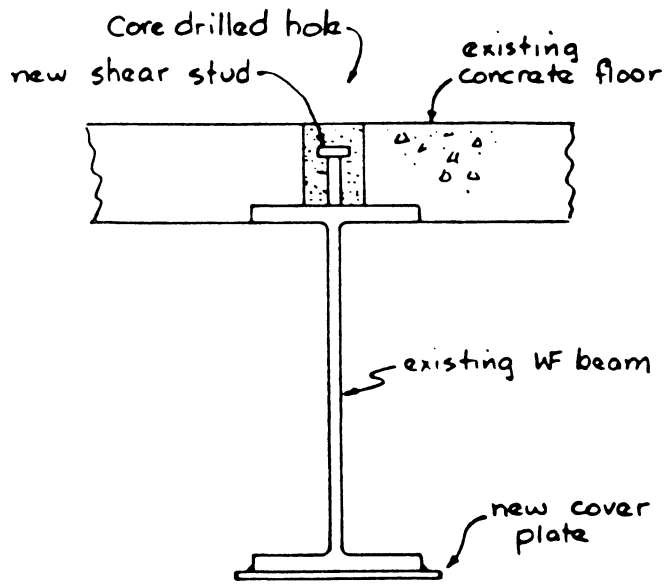

Figure 15

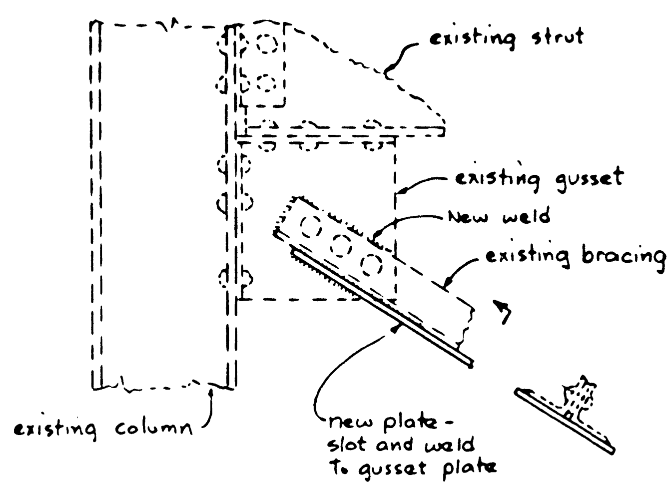

Figure 16

the welded area stresses become zero in the molten stage and then go into tension as the weld cools and shrinks. This puts part of the member out of action, or means the remaining unaffected cross section must take the load, or local yielding may take place which will return the entire cross section to compression. In such a case it is likely the stress distribution across the cross section will be highly variable. As previously stated, it is preferable to avoid transverse welding in highly stressed members, if at all possible.

Pre-tensioned reinforcement can be employed on occasion. Examples of pre-tensioned rods are fairly common. However, pre-loading of cover plates and similar reinforcing prior to welding is often very difficult under field conditions. And any cost advantage usually is cancelled by the added labor and equipment requirements.

10. SHORING AND STRESS RELIEVING

As just indicated, shoring before reinforcing can make reinforcing more efficient since it can increase the usable stress range in the added material. However, sometimes it is impractical or impossible to shore. The decision as to whether or not shoring is feasible must be made *prior* to the determination of the size and extent of reinforcing.

Shoring may be desired to reduce the deflection of an existing overloaded member. In this situation, some positive camber may be forced into the member prior to reinforcing. In a case where a plate is added to the bottom flange of a beam, the flange may go into compression when the camber is forced in, and this should be accounted for when sizing the cover plate.

Incidentally, the act of welding a cover plate to the bottom flange of an *unloaded beam* tends to make the beam arch upward because of the heating and cooling produced by welding. This is one reason to keep welding as symmetrical as possible.

Caution must be exercised when reinforcing a beam known to be composite is contemplated. It must be determined whether the original construction was shored or unshored before concrete work was done. If it were *shored* prior to the pour, so that both dead and live load were resisted by the composite action, there should be no problem jacking it up to its original no-load position. This no-load position necessarily may not be represented by a straight beam. In composite construction it is common to start with a cambered beam. If design drawings of the original building are available they indicate the type of composite construction and whether beams were cambered. (Engineers note - this information should be placed *on the design drawings* in addition to the job specifications. As years pass, the specification book is often lost, but drawings tend to be around a bit longer. Someone in the future may appreciate it.) If the original composite poured construction was *unshored*, then it is wise *not* to attempt to raise the beam. Some relief can be gained by removing as much of the *live* load as practical.

The AWS Spec.[2] recommends that if the dead load produces over 3 ksi of calculated stress, the member should be shored, prestressed or relieved of its excess load prior to welding. This is a very conservative, safe approach, but is not often followed in the real world, where welding may be required under trying or dangerous conditions in a condensed time frame.

The sound judgment of an experienced engineer in assessing the effects of cutting, welding and drilling on an existing structure may save his client needless expense and lost time by eliminating unnecessary shoring, cribbing, jacking, scaffolding and associated work.

11. REINFORCING CONNECTIONS

Whenever a member such as a beam or bracing is reinforced to increase its load-carrying capacity, end connections must be examined to see if they, too, must be strengthened. The original connection was devised with something specific in mind. It will probably fall into one of the three framing categories classified as Type I-rigid, Type 2 - simple or Type 3 - semi-rigid.

Unless the alteration design clearly states the type of connection is to be changed and the main members properly altered to accommodate, then it is wise to keep the connection as it was originally intended and any reinforcing work should be so tailored. Failure to do this may result in the introduction of unwelcome local stress concentrations.

There are many ways to reinforce or improve an existing connection. Here are a few:

1. Remove and completely replace the original connection with a new stronger connection.
2. Add weld length or increase weld size. There is no cause for concern when adding new weld to existing weld as long as the old weld is sound. The difference in strength between E60- and E70-class welding electrodes is small. And the difference is even less in the finished weld, which is a mixture of pure weld metal and base metal.
3. Remove old rivets or A307 bolts and replace with A325 or A490 bolts, or ream holes and use larger diameter bolts (worth mentioning even though it does not pertain to welding).
4. Extend the length of framing angles by welding on additional lengths (Fig. 17).
5. Add web framing angles to an original seated connection (Fig. 17).
6. Add weld to existing riveted or A307 bolted connections (Fig. 18). When this is done the capacity of the old bolts and rivets is normally discounted and the weld figured to take the entire load. This is a conservative approach. The AWS Spec. D1.1[2] states that where rivets or bolts are overstressed by the total load, they may be assigned the task of supporting the dead load, provided they can do so without over

stress. In such cases the reinforcing weld is added to support the live and impact loads. Despite this provision, consideration should be given to the following circumstance. Assume the original fasteners take the dead load and the new weld carries the subsequent superimposed loads. However, under the increased load, local yielding may take place in other parts of the connecting material. Since welds yield to a lesser extent than rivets or bolts which are attempting to plow their way through the connected material, the stiffer weld may end up taking more than its anticipated share of the final total load. If there is any doubt as to the final load sharing between the mechanical fasteners and the weld, it is safer to design the weld to take the entire load.

7. Add weld to existing high-strength bolted connections. High-strength bolts, if torqued properly, can be figured at their *friction* value when used in the *same connection plane* with added weld, the amount of added weld being sufficient to carry the difference between the total load and the load taken by the high-strength friction value bolts. If the original high-strength bolts were actually figured at bearing value with painted faying surface, it may be assumed the

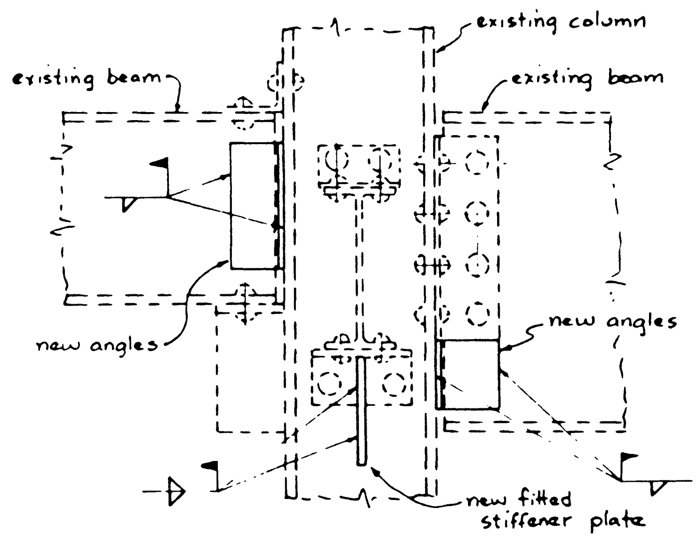

Figure 17

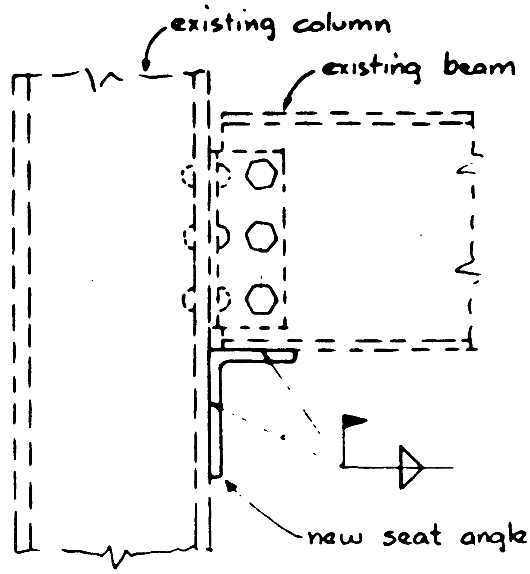

Figure 19

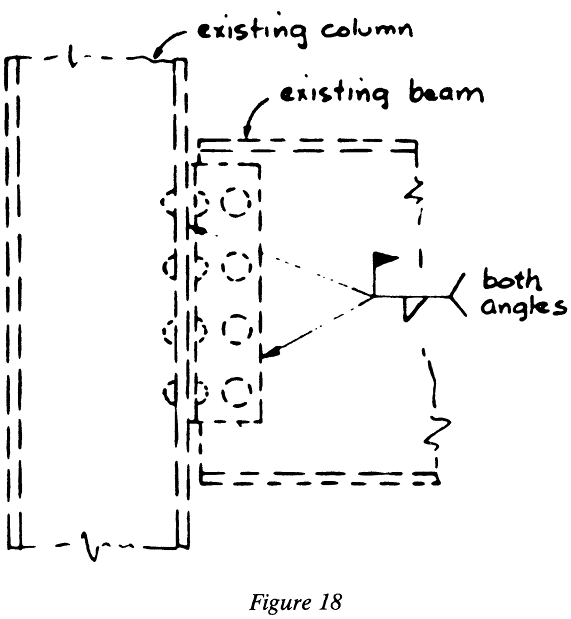

Figure 18

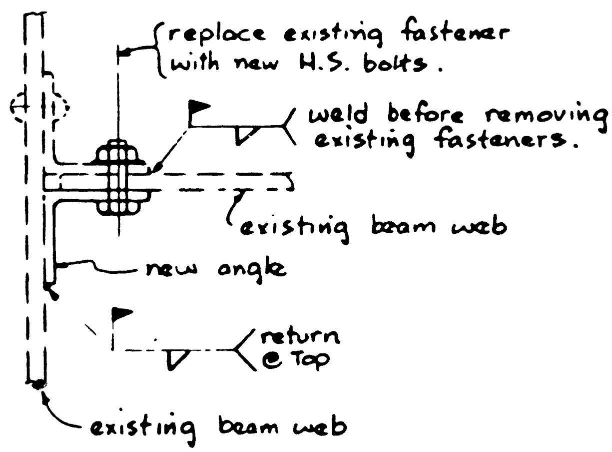

Figure 20

connection has already eased into bearing. However, with the recent relaxation of torquing requirements for bearing value high-strength bolts, it is suggested the bolts, if used in the same plane with welds, be torqued to friction-value requirements, painted faying surfaces or not.

8. Add a seat to a web framed connection (Fig. 19).
9. Add a second angle to a single angle web framed connection (Fig. 20).
10. It is common in renovation work to remove or relocate members, leaving open holes behind. AWS Specification D1.1 paragraph 3.7.7 pertains to the restoration of abandoned (unused) holes in steel members. Abandoned holes may be left open provided they are not required to be filled in for structural, aesthetic or weather-tightness reasons. If the holes must be restored by welding, rules in the previously mentioned specification must be followed. Members subject to dynamic tensile stress deserve special attention in this regard.

Looking into the innards of an ancient clock with all its wheels, gears, levers, ratchets and cogs, often kindles a feeling of admiration for the craftsman who devised those works. But a common structural joint is no less imposing. In fact, it is more so. In a clock, keen observation comprehends what is happening. In a joint, many things are happening which are not visible. Parts are being compressed, stretched, twisted, bent, often in two or three directions at once. To say that one completely understands a connection is unrealistic. Most connections have predictable patterns of performance. But it is beyond practicality to assign exact magnitudes of stress and strain to the various pieces or parts of the joint. A good joint must be designed so wide ranges of stress and strain do not cause its eventual failure. We know yielding takes place in parts of most connections. Another one of the many beauties of steel is its maintenance of strength after yielding—in fact, it increases in strength because of strain hardening.

A *field*-reinforced connection is no less impressive. Here we may have some bolting and/or riveting combined with welding. The same rules which apply to new connections must apply to reworked connections. There may be enough stress concentrations to send a purist into a cold sweat, yet the connection as a whole, if properly designed, performs well.

12. EFFECT OF FIELD ALTERATIONS ON THE ENTIRE STRUCTURE

We have talked about individual welds and the effects of field working individual members. It is often wise to step back for a brief overview of the effects of alteration work on the entire structure. If we stiffen a beam here and there has it affected the columns? We cut off a cantilever beam—what did we do to the back span? We remove X-bracing here and add moment connections. Can the columns absorb the added moments? Can the foundations take the change in loads? Often, in the name of expedience, alterations must be redesigned and reworked on short notice in the field by the tradesmen. Such work can be very ingenious and at times a life-saver. But it should be reviewed by the designer-of-record to assure its suitability to the overall structure. A designer will generally welcome suggestions from the field.

13. FIRE HAZARDS AND PRECAUTIONS

Existing state and local fire codes and regulations and safety rules must be observed. When welding is done, or cutting with oxyacetylene torches, grinding or air-arc gouging, red-hot metal droplets or sparks may fall on combustible materials.

Older buildings are often dirty and littered and contain a higher proportion of combustible materials than modern structures. If these conditions exist, protection should be provided in the form of fireproof blankets and readily available fire extinguishers, or other suitable means. If an explosive atmosphere is suspected, welding should not be performed until proper ventilation and all necessary safeguards are provided. It is often wise to maintain a fire watch *after* the work shift is ended and until the next shift starts work in case latent embers become active.

14. TESTING AND INSPECTION

The owner may wish to have his structure inspected either during or after work is performed. Inspection *during* the welding process is more effective. If a problem is discovered it can be corrected with less disruption of the welded area.

The inspector should inspect all welds visually for signs of poor quality or non-conformity. Visual inspection *after* welding detects only major surface imperfections, and does not detect below-surface defects, which can be the most serious. Non-destructive testing, such as ultrasonic or radiographic, is one means to examine the interior of a weld area. However the non-destructive proof of the existence of a weld flaw does not measure its serviceability. (Only a destructive test will give this information.) The non-destructive test is merely someone's opinion as to what is good or bad. The correlation between the flaw and serviceability is the important issue.

A flawed weld may have adequate quality to be acceptable. A perfect weld is hardly ever required in building work. Purity is a matter of interpretation. A flawed but acceptable weld does not mean the work is sloppy or careless. It means the degree of contamination, porosity, non-fusion, inclusions or other non-conformities is not great enough to prevent the weld from performing safely during its service life.

The engineer—and not the inspector—should be the judge as to the serviceability of the weld. These factors should be considered in judging weld quality:

1. Service conditions, stress level, nature of the stress, working temperature
2. Material properties
3. Risk of defects happening
4. Risk of defects not being found
5. Consequence of failure

If the weld quality is questionable, it is recommended the inspector consult a comprehensive text on the subject, such as Chap. 8 of the AWS *Welding Handbook*, Vol. 5, 7th Edition,[4] prior to passing his conclusions on to the engineer.

Various tests can be performed on the existing steel to determine certain characteristics. In addition to the aforementioned chemical analysis to establish weldability, and the tension test to confirm ductility, tests are available to determine fracture toughness, cyclic rupture, grain flow (by photo macrography) and susceptibility to hydrogen embrittlement. It is recommended a qualified testing laboratory be contacted to discuss the type of testing most effective and the size and quantity of samples required.

SUMMARY

1. Establish the weldability of existing and new steel members.
2. Determine the load condition of existing members to be field-worked.
3. Be aware of what welding can do to existing members and plan reinforcing and welding so there will be no harmful consequences.
4. Account for fatigue, impact and other factors that affect the design of the new reinforcing and welds.
5. Design the new reinforcing material based on the usable stress, the difference between established allowable stress in the existing member minus the actual stress at the time reinforcing is performed. Shore only if necessary.
6. Make sure connections are reinforced as required to satisfy new load requirements.
7. Account for effects of the alteration work on the balance of the structure, including foundations. Check to see that all applicable building code requirements are satisfied.
8. Make sure welds are properly executed.

If all these things can be accomplished successfully, the revitalized structure should provide many added years of service life.

REFERENCES

1. *James F. Lincoln Foundation* The Procedure Handbook of Arc Welding *Cleveland, O.*
2. *ANSI/AWS* Specification D1.1 Latest edition.
3. *Jefferson T.B., and Graham Woods* Metals and How to Weld Them *James F. Lincoln Arc Welding Foundation.*
4. *American Welding Society* Welding Handbook *Volumes 1 through 5, 7th Ed.*
5. *Blodgett, Omer* Design of Welded Structures *James F. Lincoln Arc Welding Foundation.*
6. *The American Institute of Steel Construction, Inc.* Manual of Steel Construction *8th Ed., 1982, Chicago, Ill.*
7. *George E. Linnert* Welding Metallurgy *Vol. 2, American Welding Society.*
8. *Bjorhovde, Reidar,* Unpublished notes *University of Arizona, Tucson, Ariz.*
9. *Nagaraja Ras, N.R. and Lambert Tall* Columns Reinforced Under Load *AWS Welding Journal, Vol. 42, April 1963.*
10. *Spraragen, W. and S. L. Grapnal* Reinforcing Structures Under Load *AWS Welding Journal, Vol. 23, Research Supplement 65-S, February 1944.*