Let's Revitalize Welded Structural Steel

By Russell S. Hale*

The use of welded structural steel in buildings and bridges has been well accepted for many years—so well accepted, in fact, as to lull the fabricator and erector into a sense of complacency. It has been said that complacency is the greatest enemy of management—and certainly, until recent years, wa have been complacent about the use of steel in these structures, with the consequence that constructional concrete has made tremendous inroads as an alternative building material.

As late as 1967, 82% of the floor area of multi-storey buildings completed in Sydney has a steel frame—these are buildings over 14 stories in height erected in the Sydney area. In 1968, using the same criteria, steel frames accounted for only 27%, and in 1969, 0%. Concrete had supplanted steel as the choice of owners and designers as a constructional material. Happily for those in the steel business, work in progress at the present time is approximately divided evenly between the two materials.

And why the dramatic decline in the use of steel in buildings—and a decline paralled in bridge work—with the increased acceptance of concrete ? Why, when we analyse the advantages of steel, do so many builders, designers and engineers prefer to use concrete ? Steel frames offer numerous advantages to the designer and owner—the steel frame results in reduced dead weight of up to 50% for buildings between 15 and 30 stories, with concurrent reduction in foundation loads, on the average of 10%. Steel frames result in an increase of 1% to 2 % in usable floor area on a grid of 24′×24′ and a greater increase with increased spans. In addition, an average reduction in floor to floor height of 4″-8″ (based on the 24′×24′ grid) can be achieved with steel.

Of increased significance today is the reduced erection time for a building with a steel frame versus the erection time for a concrete building. With the "credit crunch" we are experiencing, this marked reduction in erection time means earlier occupancy, and less lengthy periods of capital invested with no return.

Notwithstanding these significant advantages there has been the rapid increase in the use of structural concrete. The reason—a simple case of economics. Despite the advantages of steel, concrete may offer overall economies to the owner—who in the final analysis, is the one most concerned with costs.

It can be debated whether first costs are the true cost and whether in the long run—including demolition of the building 50 years hence—steel is the lower cost material—but what must be accepted is the fact that the designer, engineer, architect or owner selects concrete because he thinks it is a lower cost building material.

To revitalise welded steel structures, costs must be reduced and success in restoring the use of welded steel in these structures will be directly proportional to the degree of success achieved in reducing costs.

But who is responsible for the costs of the welded steel structure ? Is it the fabricator and erector who supplies and erects the building frame or bridge structure ? Is he not the link in the chain from architect to engineer to fabricator to erector to inspector to owner that primarily determines the cost ?

Although no one will deny that the fabricator/erector can affect the cost of the welded structure, his role in so far as costs are concerned is far less than that

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of the designer, structural engineer, and the code group or person responsible for specifying weld requirements and inspection standards. In fact, the fabricator erector should be an object of sympathy, as he is caught in the vice of inflexible selling price and rising material and labour costs. The price of fabricated structural steel rose by less than 10% between 1965 and 1970 while wages rose 30% and raw materials by over 16%. If welded structures are to be revitalized—and such revitalization depends on reduction of overall structural steel costs—then we must look elsewhere for help of major significance.

The designer and structural engineer play the major role in the cost of the welded structure. They, along with code bodies, determine the steel to be used, the weld joint, its position and preparation, the welding process to be used, the weld deposit mechanical properties, and the inspection standards. Each of the above areas can contribute substantially to the cost of the welded structure.

Practical experience over the years has proved that all steels cannot be welded with the same degree of ease. For example, low carbon steels having less than .20% carbon can be readily welded, while the welding of high carbon (greater than .30% carbon) steels require extra caution. The designer today can select a wide variety of steels for his structure, but, in the main, has concerned himself with the physical properties, cost and availability of these steels, and to a minor degree with their weldability. In many instances, a choice is possible between two different steel specifications each with similar mechanical properties but with different weldabilities. Selection of the steel with poor weldability will require the use of special techniques, such as preheating, with an adverse affect on the cost of the structure.

The wide choice of steels available also enables the designer to select the steel best suited to the stress levels required. Hybrid beams, with high strength flanges and lower strength webs can be fabricated with ease, and offer economies.

Although the cost of welding is only one of several items in the cost of even the simplest fabrication, an analysis of typical welded fabrications shows that welding can account for 20-30% of the total processing costs. Obviously, there are significant economies that can be achieved by judicious design of the weld metal required, and by planning for the positions in which the weld metal can be deposited.

The designer should be actuely aware of the high cost of weld metal—not the cost of purchased consumable products, but the cost of the weld metal which becomes a part of the welded structure. He must constantly strive to design structures with the minimum—the absolute minimum—of weld metal that is required by the stress requirements. Wherever possible, he should use standard sections, if such are available to him.

In today's computerized age, thorough stress analyses should be made and weld size specifications determined on the basis of calculated stress—not "seat of the pants" hypotheses. Recent changes in the AISC—AWS (American Institute of Steel Construction—American Welding Society) allowables for weld metal presage glad tidings for the Australian fabrication industry—if we choose to follow this thinking—thinking that permits weld metal to be used to the best of its ability. Of major significance are the following :

1. Fillet or partial penetration weld size may now be based on the strength level of the electrode being used, whether 60,000 or 110,000 p.s.i. ultimate tensile strength, with the proviso that the permissible unit stress, regardless of the electrode class used, must not exceed the allowable for the weld metal which matches the weaker base metal being joined.

2. Increased allowable shear stress in fillet welds of .3 x (min. specified tensile strength of electrode).

3. A credit for penetration obtained in submerged arc welding, by addition of 0.11″ to fillet welds greater than ⅜″ and for throat size equal to fillet size for fillets less than ⅜″.

What do these changes mean in practical terms ?

First, let us look at the allowable loads for fillet welds in the new AISC code compared to previous allowables :

	60 XX	70 XX	80 XX	90 XX	100 XX	110 XX
Now 1″ Fillet	12,730	14,850	16,970	19,090	12,210	23,330
Former 1″ Fillet	9,600	11,170	11,170	11,170	11,170	11,170

Weld metal costs vary directly with weld volume and thus weld area. Since the cross sectional area of the weld varies as the square of the leg size, the above allowables reduce leg size by 25% (from ½″ fillet to ⅜″ fillet), the weld area by 44% ; and costs of welding by a similar percentage.

If these codes are adopted by Australia, the knowledgable designer can, through the use of new stress allowables and submerged arc, obtain the same weld strength *with only 27% of the weld metal previously required.*

Example :

Old—$\frac{1}{2}=\frac{1}{2}$ x .707 x 8″ long x 15.8=44.6 KIPS
New—$\frac{1}{4}=\frac{1}{4}$ x $8\frac{1}{2}$″ long x 21=44.6 KIPS

Result :

73% reduction weld metal costs.

Since these allowables were determined after exhaustive tests and analysis, we should all work with fervor to have these or similar standards set for Australia, as they will certainly enhance our capability of competing. For the present, these examples illustrate the importance of designers and engineers accurately specifying weld sizes. It should also be noted that these codes permit higher tensile weld metal to be used in the welding of mild steel, provided strength calculations are based on the allowable for the lower strength materials, as discussed above.

New AISC specifications also expand the range of usage of partial penetration welds. They may be used anywhere **except** in splices in plate girders and beams. The AWS building code also prohibits their use in tension transverse to their axis if subjected to fatigue loading which by design criteria could produce fatigue failure. The cost advantages of the use of partial

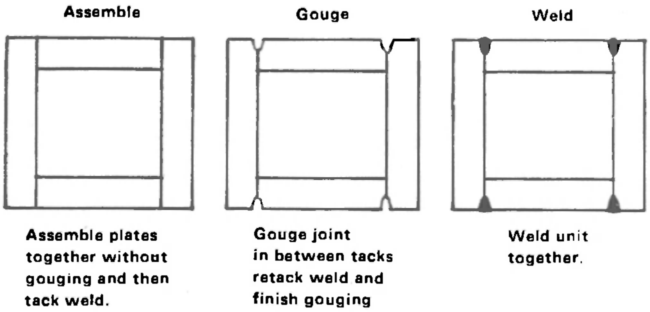

Fig. 2

penetration welds can be seen in the following design of a typical column (see Fig. 1).

It is also interesting to note that the above grooved joint preparation is intended to be made after assembly—a standard practice in many shops, but prohibited by inspectors in others. The new building codes specifically permit this technique, and so clarify the situation.

The influence architects, designers and engineers have on the cost of welding is a subject that could be amplified in much greater detail. The Australian Welding Institute has a prime responsibility to keep these important people fully informed on welding developments, the availability and suitability of welding consumables and, of greatest importance, the reliability and reproducability of the welding process. The mystique must be removed from welding and every assistance given to designers to enable maximum economies to be achieved.

Assistance must also be given to the structural and specifications engineer to escape from the maze of electrode and weld metal specifications confronting him. The practical engineer is continually faced with the problem of how to write his specification to provide an adequate margin of safety at a reasonable cost. Like quality control, weld metal specifications must be adequate to perform the job, for any less would be worthless. On the other hand, they should not be so unduly restrictive that they make the cost prohibitive to the customer.

Impact requirements, often needlessly specified for weld metal, are a prime example of specifications adding to the cost of welding.

For example, if a 1″ plate is butt welded with no specification as to low temperature impact values it can be welded one pass either side at a cost of 60c/ft. using submerged arc welding. If the same joint must be welded to meet 25 ft. lbs. at minus 40°F the cost will rise to $2.50/ft.

Certainly there is a large grey area with respect to the need for impact values in welded structures.

A useful approach is to first determine where low temperature notch toughness is important. If applications where this property is of no concern are eliminated attention can then be focussed on a much smaller "problem" area.

Notch toughness properties are important where all three of the following factors exist. These factors must be present before brittle fracture can occur.

1. The presence of high general stresses, either applied and/or residual.

2. The presence of a defect, notch or stress raiser.

3. Low notch toughness of the materials *at the service temperature to be encountered.*

On point 3 alone, impact values could be eliminated as a matter of concern in the vast bulk of welding applications in Australia.

Welding consumables with charpy impact values of 10-30 ft. lbs. at room temperature are widely used throughout Canada and other frigid areas with no problem, since these weld deposits minimise the likelihood of the weld defects needed to initiate brittle fracture at the low service temperatures involved (down to—50°F.)

There is a multiplicity of tests that purport to be a measure of a material's notch toughness. Because of the complex nature of the fracture process, however, no single test has so far been evolved that will provide a complete understanding of a material's notch toughness characteristics.

To date the charpy test has had widespread acceptance as a measure of the notch toughness of a material, but metallurgists have long been seeking a better test than this—a test that will correlate better with actual service conditions. One technique that has gained wide acceptance for fracture safe design of structural steels is the construction of a fracture analysis diagram based on the material's nil ductility transition temperature. This NDT temperature can be determined for a single drop weight test. It is a measure of the ability of a material to resist the propagation of a cleavage crack following initiation at a small flaw and growth and acceleration through the damaged zone adjacent to the flaw. Use of this NDT criterion will result in more realistic weld metal requirements with consesequent reduced fabricating costs.

Weld inspection standards specified for welded structures are other factors affecting costs. Not only can the engineers' specified level of tolerable weld defects and inspection requirements add significantly to the overall cost of welding, but the inspector's interpretation of what he believes the engineer wants can prove an expensive interpretation. Australian Standard CA 8, the code for welding in building construction, does not call for non-destructive testing. The service requirements of structures fabricated to this code require visual weld inspection to ensure satisfactory performance. The unnecessary addition of non-destructive testing for these structures will appreciably increase costs. For example, the X-ray inspection of the 1" but weld discussed above would increase the cost from 60c/ft. to $3.40/ft. One could easily inquire as to the extent of inspection accorded a similar concrete structure.

The X-Ray appears to have become the security blanket for the engineer—resulting in a comfortable feeling that all is well if he can put refer to an X-ray picture of the weld. If an X-ray is essential to the security of the specifier, the least he can do to control the added costs of such a requirement, is to have the X-rays interpreted by a competent inspector—*and to the proper X-ray standard* for the relevant code.

In short, if specifiers and engineers had greater confidence in welding—and welders, far less stringent inspection standards would be applied, with resulting cost reductions. In order to revitalize welded steel structures, this must be part of the goal.

And thus, the welded structure finds its way to the successful—though possibly not happy—bidder. The material to be fabricated has been specified, the type of welded joints (and sometimes their preparation) detailed, the weld deposit properties stipulated, the inspection standards stated—along with the required delivery. Within this narrow corridor of responsibility the fabricator/erector hopes that he can supply the structure and retain some measure of profit. For if there is no profit element in the price, the structure might just as well have been in concrete.

It must not be inferred that the fabricator/erector has no control over costs—that he is "locked in" with parameters of costs for which he has no responsibility. The weld metal requirements can be deposited with a variety of processes, all with favourable or unfavourable costs. Positioning equipment can be utilised to advantage, parts sub-assembled, welders trained and supervised competently and all other shop processes that affect welding costs, i.e. cutting, forming and cleaning operations controlled. In essence, the fabricator must analyse all shop practice, not solely welding.

The advantages and disadvantages of the various common welding processes have been analysed thoroughly and it is not proposed to restate them. It is important however from the cost viewpoint, to note the rapid acceptance of cored wire welding with or without a shielding gas for structural fabrication and erection.

Cored wire welding was first introduced in the U.S.A. in 1962, when approximately 1,000,000 lbs. were sold. In 1969, 80,000,000 lbs. were sold and it is estimated that double that figure will be marketed in 1974. The scope of this process can best be seen by comparing its usage in the U.S. with the total electrode production in Australia of 48,000,000 lbs.

Cored wire welding is done either with a wire which is self-shielding or one which requires an external shielding gas. Both have their advantages. These advantages would be.

Self-shielded

1. Unaffected by breeze or draughts and is therefore well suited to field erection work and shop work where curtaining is not easily available.
2. Equipment, particularly welding guns, is simple and no gas apparatus is required.
3. This process is characterised by low penetration, about on a par with stick electrode welding.
4. Because of 3 above, this process is capable of handling poor fitup.

Gas-shielded

1. Higher weld deposit quality.
2. Capable of welding more alloy steels.
3. Produces deeper penetration than stick electrode welding.

These processes offer substantial savings to fabricators and erectors. On a recent erection job completed in Sydney, using self-shielding cored wire, $18'' \times 30''$ beams were connected to the columns by 1″ thick moment plates on the top and bottom flanges. The 1″ moment plates were connected to the column with a single bevel butt weld from one side ; beam flanges were connected to the moment plate with $\frac{3}{8}''$ fillets. The time required to make this connection was reduced from 12 hours with manual electrode to $3\frac{1}{2}$ hours with cored wire. In another application of this process, $4' \times 3'$ box columns were field spliced, using a partial penetration butt joint $1\frac{1}{2}$ deep $\times$ 45° in 3″ plate. Time to weld this joint was reduced from 9 hours to $2\frac{1}{2}$ hours for each side. Weld metal required was approximately 60≠/ column splice. These are the economies available to fabricators and erectors if they are able, by design and planning, to use cored wire welding.

These are but a few of the factors that affect costs and thus the ability of welded steel structures to compete with alternative materials. Each however is an area in which changes can readily be made to reduce costs and thus ensure for the fabricating industry a major role in Australia's future growth.