Fabrication and Design of Structures of T-1 Steel

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• T–1* CONSTRUCTIONAL alloy steel (*1*) was discussed in 1957 and was at that time a material relatively new to highway and bridge engineers, although its use in other fields had been well established. This paper discusses some of the methods of shop fabrication of this material as well as some of the design considerations affecting its use.

The mechanical properties, chemical composition, and principal characteristics of T–1 steel will be briefly reviewed. It is an alloy high yield strength steel which obtains its properties by heat treatment; specifically, by quenching in water and subsequently tempering.

CHEMICAL COMPOSITION

A typical chemical composition is shown in Table 1; T–1 steel is basically a low-carbon alloy steel to which small percentages of several alloying elements have been added to increase hardenability, improve notch toughness, and resist softening at elevated temperatures.

TABLE 1
CHEMICAL COMPOSITION RANGE OF T-1 STEEL

Chemical	Percent
Carbon	0.10/0.20
Manganese	0.60/1.00
Phosphorus	0.04 max
Sulphur	0.05 max
Silicon	0.15/0.35
Nickel	0.70/1.00
Chromium	0.40/0.80
Molybdenum	0.40/0.60
Vanadium	0.03/0.10
Copper	0.15/0.50
Boron	0.002/0.006

Inasmuch as the principal production of this steel is in plates, the methods of treatment and the properties obtained in plates should be considered. A plate of T–1 steel starts with an open hearth melting practice which conforms to industry and plant standards for quality alloy steel. It is rolled on standard slabbing and plate mills; however, unlike practices for rolling structural carbon steel, controlled cooling, in plates over 1 in. thick, is used to prevent thermal ruptures. After cooling from the rolling operation, the plate is reheated to a temperature of about 1650 F, and quenched in water. This may be done under a press with a water spray to insure flatness and uniformity, or may, in heavier gages, be done by immersion in a tank of agitated water. After quenching, the plate is tempered at a temperature of approximately 1200 F to relieve quenching stresses and to produce the desired mechanical properties. Only in the tempered condition does T–1 steel have the high yield strength, good ductility, and excellent toughness which are necessary in service and which help in shop fabricating operation. Following the heat treating operation, the plates are sheared or gas-cut to size.

MECHANICAL PROPERTIES

Depending upon the user's requirements, T–1 steel plates may be furnished in regular, flange, firebox or aircraft quality. A comparison of T–1 steel with materials now in widespread use in bridge construction is given in Table 2. Because of improvements in mill control of heat treatment, the minimum tensile and yield strengths for T–1 steel are

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TABLE 2

TENSILE PROPERTIES OF T-1 STEEL COMPARED WITH ASTM A7 AND A242 STEELS

Plate Thickness, in.	Min. Yield Strength, psi	Tensile Strength, psi	Min. Elong. in 2 In., %	Min. Red. of Area, %
		(*a*) T-1 Steel		
3/16 to 2½	100,000	115/135,000	18	50[1]
Over 2½ to 4	90,000	105/135,000	17	50
Over 4 to 6	90,000	105/135,000	16	45
		(*b*) ASTM A7 Steel		
¾	33,000	60/72,000	21 (in 8 in.)	—
6	33,000	60/75,000	21.5	—
		(*c*) ASTM A242 Steel		
¾	50,000	70,000 min	18 (in 8 in.)	—
4	42,000	63,000 min	23.5	—

[1] 40 percent for ¾-in. thickness and less.

somewhat higher today than those previously (*1*) obtainable.

Table 2 shows that T–1 steel in plate thicknesses up to 2½ in. has three times the yield strength of A7 structural carbon steel, and two times the yield strength of A242 steel. The ductility of T–1 steel is measured by its percentage of elongation in a 2-in. gage length rather than the conventional 8-in. gage length, because of the common characteristic of alloy steels that elongation takes place over a shorter length. In thicknesses over 2½ in., the properties of T–1 steel are slightly reduced because of the lesser hardening effect of the quench on heavier sections.

FABRICATION

The first consideration in fabrication is cutting plates to specified dimension or shape. T–1 steel can be sheared on any good shop shear, and the capacity of this shear for T–1 steel will generally be about two-thirds of its capacity for structural carbon steel. In other words, if the shear is rated at 1-in. maximum thickness for carbon steel, it may be used for shearing T–1 steel in thicknesses up to about ⅝ in. The shear knives should be sharp and set with clearance of about 0.010 to 0.015 in. The smoothest cut will result when the shear is set at a high angle, but this may result in some shear bow being produced in the plate. If shear bow cannot be tolerated, a flatter shear angle must be used.

For plate thicknesses beyond the limits of available shears, cutting to size or shape may be readily done with conventional gas cutting equipment. No preheating or post-heating is necessary to gas cut T–1 steel. It may be cut using the same gas pressure, nozzle size and travel speed as are used for other structural steels. The flame cut edge, although approaching a hardness of about 400 Brinell, is nevertheless tough and may generally be formed without difficulty. If it is necessary to machine this edge, it is best to take a deep milling or planer cut which will get below the hardened zone into the softer parent material and thus cut the hardened area away. Alternatively, the edge may be flame softened locally by the application of heat which must not exceed 1100 F. Tandem cuts or the use of a follower post-heating torch will also soften the edge. Distortion of long straight cuts may be minimized by cutting several pieces at one time with parallel torches.

Bending may be performed on conventional brake or forming presses, using a bending radius of twice the thickness for plates up to 1 in. thick, or three times the thickness for plates up to 2 in. thick. To insure these radii, the span of the lower die should be 16 times the thickness, as compared with 8 times the thickness for structural carbon steel. Furthermore, air bends rather than closed die bends are preferable, and over-forming allowance about three times that for carbon steel will be necessary because of the greater springback. Quite often it will be found necessary to make a bend so close to a plate edge that a lower die span of 16 times the thickness cannot be used. In this case, heating of the area to be bent will be helpful, but the temperature of this area should never exceed 1100 F. If a slot furnace is used for heating, the operator should realize that the bottom surface of the plate may be 300 F to 400 F hotter than the top surface, and be guided accordingly. Temperature indicating crayons, and not guesswork, should control heating operations.

In machining, two considerations are important: (a) the machining rates which must be used with this material, and (b) the possibility of distortion when large amounts of stock are machined away. In general, machining rates will be reduced about 40 percent with T–1 steel as compared to A7 steel. When drilling, for example, conventional speeds may be maintained but feeds should be reduced to about 60 percent of the normal rate. For all machining operations carbide tools are recommended, although high-speed steel tools may be satisfactory. As a matter of fact, a shop which is accustomed to machining rail steel will find that there is little change to be made. T–1 steel exhibits little tendency to work harden, although tools should be kept sharp and feeds should be maintained at all times. Plenty of coolant should be used; the choice of coolant is at the shop's option.

The second problem in machining has nothing to do with machinability, but arises from the fact that the heat treating operation frequently produces residual stresses in the T–1 steel plate or other section. For example, a distinct out-of-flat condition may exist which is removed by the cold work of roller leveling or press flattening. When such a plate is machined on either or both surfaces, the release of surface stresses may cause the out-of-flat condition to be restored. If extensive surface machining operations are contemplated, it is recommended that the plate be ordered stress relieved after heat treatment and flattening, which will reduce the residual stresses to a level preventing excessive movement during machining.

T–1 steel plates may be punched in thicknesses up to ½ in., but heavier thicknesses are apt to create excessive wear or breakage of the punches and possible damage to the punch press. Although T–1 steel is not particularly sensitive to cold work which exists around the punched hole, it is good practice from the standpoint of eliminating stress raisers that holes be subpunched and reamed or drilled.

Semi-finished or forging bar stock is available, and it may be forged at a temperature about 50 F lower than that used with carbon steels. A good blast of compressed air or steam should be available at the forging hammer, as the scale on alloy steels generally tends to be more sticky than that of carbon steel, and such a blast will prevent the formation of scale pits in the forging. Any forging over about 16 sq in. in cross-section should be cooled in lime, dry sand, or other medium to prevent formation of internal ruptures. Any forged piece must be reheat-treated by the method described subsequently in order that the desired mechanical properties may be restored.

Occasionally, it is necessary to make severe bending formations, which are impossible to perform if the steel is either cold or heated to about 1100 F, yet which do not qualify as forging operations. In such cases, T–1 steel should be heated to a temperature of from 1500 to 1800 F (not higher) and the desired formation made. This may be followed by air cooling. In such cases, the steel must be immediately reheat-treated as described in the following paragraph.

These operations of forging or hot forming will leave T–1 steel in a condition which is known to be undesirable for best toughness and ductility. As a consequence, reheat-treatment must be performed. To do this, the parts should be heated to a temperature of from 1650 to 1750 F, held at that temperature for about 1 hr per inch of thickness, and then quenched in agitated water until cold. They should then be tempered at a temperature of approximately 1200 F; the steel manufacturers' suggestions may be solicited as to the proper tempering temperature and time. This steel is not subject to quench cracking, and some hours can elapse between quenching and tempering without risk.

T–1 steel which has been hot-formed or forged should never be flame cut or welded before it is heat treated in accordance with the foregoing practice. Fabricators who have attempted to flame cut or weld T–1 steel in the as-rolled or as-forged condition have found that cracks initiate in the heat-affected zone of the weld or gas-cut edge and propagate for a considerable distance into the parent metal. For the same reason, a rolled or fabricated shape should never be locally flame straightened, because such heating destroys the desirable properties of the material and allows no opportunity to restore them.

WELDING

Hydrogen must be kept out of the welding operation. There are on the market many low-hydrogen coated manual welding electrodes, which may be used to produce satisfactory welds; but these welds will be satisfactory only so long as the low-hydrogen condition is maintained. To insure this, it is desirable that, immediately upon opening a sealed package of electrodes, they be placed in an oven maintained at about 250 F, and that these heated electrodes be taken from the oven only as needed. If the electrodes are taken from a container which is not airtight, they should be baked (at around 800 F) in accordance with the electrode manufacturer's instructions. The welder must not be permitted to turn off the oven overnight, and then start up with the same load of electrodes in the morning. This precaution cannot be emphasized too strongly. There is evidence that the popularity of iron-powder electrodes of the EXXX18 Class has not taken into account the increased sensitivity of these electrodes to moisture absorption; therefore, the drying precautions must be strictly applied. If heating of the electrodes is not practical, no more than a 2-hr supply should be opened at one time.

Several good methods of automatic submerged arc and inert-gas shielded arc welding are available, and these must be subject to rigid moisture control. Fluxes and shielding gases must be kept dry.

In periods of cold weather, shop temperatures may drop to 35 F or lower. If such a temperature drop results in moisture condensation, provision must be made to keep T–1 steel warm during welding.

There is no problem involved in welding T–1 steel to other steels such as ASTM A7 or A242. Low-hydrogen welding methods must be used, and the electrode may be of a strength sufficient to match that of the softer steel.

As a natural consequence of their high strength, weld metals used with T–1 steel will show ductility somewhat lower than those normally used for welding structural carbon steel. This lesser ductility may occasionally result in weld cracking in areas where restraint is high. This gives the welding engineer the responsibility of properly sequencing his welding operations to avoid high restraint. If, despite his best efforts, his design results in areas of high restraint, there are several possible courses of action. The first is to improve weld metal ductility by the use of a controlled preheat and heat input. Studies (*2*) have determined the maximum welding heat input which can be safely tolerated in welding T–1 steel without loss of joint

TABLE 3

MAXIMUM WELDING HEAT INPUT FOR T-1 STEEL

Preheat and Interpass Temperature, F	Heat Input, joules per in.[a]							
	3/16-In. Plate	1/4-In. Plate	1/2-In. Plate	3/4-In. Plate	1-In. Plate	1 1/4-In. Plate	1 1/2-In. Plate	2-In. Plate
70	27,000	36,000	70,000	121,000	Any	Any	Any	Any
200	21,000	29,000	56,000	99,000	173,000	Any	Any	Any
300	17,000	24,000	47,000	82,000	126,000	175,000	Any	Any
350	15,000	21,500	43,500	73,500	109,500	151,000	Any	Any
400	13,000	19,000	40,000	65,000	93,000	127,000	165,000	Any

[a] Joules per inch $= \dfrac{\text{Amperes} \times \text{Volts} \times 60}{\text{Speed in inches per minute}}$

efficiency or toughness, and these have resulted in the limits given in Table 3.

These limits apply to manual metal-arc, submerged-arc and inert-gas shielded arc welding methods. They apply to either butt-welded or fillet-welded joints. To keep heat input low, the stringer bead technique should be used, with little or no weaving of the electrode.

A second course of action, applicable to fillet welds, is to use the lowest-strength welding electrode compatible with design requirements. Such electrodes will provide additional ductility to accept the high shrinkage stresses characteristic of fillet welds.

The contour of fillet welds in T–1 steel and other steels of high yield strength must be such that stress-raisers at the fillet toes are minimized. This demands better than average workmanship in securing good fusion and in avoidance of undercuts, craters and arc strikes. Peening between passes will be helpful.

Repairs in welds or base metal may be readily made. Either grinding or arc-air gouging should be used for removing metal; flame-gouging (scarfing) should be avoided because of the possibility of excessive local heat input.

The basic principles of welding T–1 steel have recently been described in non-technical language (*3*), and a slide-rule calculator has been provided for ready determination of welding heat input.

There is a considerable body of opinion, built up over years of experience with steels of high carbon content or of limited notch toughness, that stress relieving is necessary after welding steels of the strength level of T–1 steel. It is believed that this does not apply to the special type of micro-structure exhibited by T–1 steel. Even in the as-welded condition or after severe cold work, this material exhibits a high degree of notch toughness throughout the range of atmospheric temperatures encountered anywhere in the world. This opinion has been bolstered by laboratory and full-scale tests which have indicated no loss of toughness, even at −50 F, of welded, cold-worked or otherwise fabricated structures which have not been stress relieved. Thus, stress relieving for a bridge or other highway structure is believed unnecessary.

In considering all fabricating operations on steels of this high strength level, the importance of good and careful workmanship cannot be emphasized too strongly. The remainder of this paper deals in part with designs to withstand fatigue stresses. The ability of steel to withstand such stresses is greatly impaired by the presence of surface notches and irregularities. Typical of such stress raisers are rough bolt holes, under-cut or irregular weld beads, and misalignments which will create bending stresses where none were intended.

DESIGN CONSIDERATIONS

When subjected to loading conditions that produce fluctuating stresses, structural members can fail under stresses considerably less than those that would cause failure of the member when subjected to steady or static stresses. This phenomenon of decreased resistance of material to fluctuating stresses is called fatigue. Over the past 100 years, there has been increasing recognition of the importance of designing structures and machine parts to withstand fluctuating stresses. Although bridge engineers were among the first to recognize the importance of fatigue considerations in design, during most of this time the problem has been of particular interest to engineers concerned with the design of aircraft, automotive, and railroad equipment because fluctuating stresses are unavoidable and optimum design is a necessity. Within the past 25 years, with the advent and increased use of higher strength constructional materials, engineers concerned with practically all types of structures have been forced to recognize the effects of fluctuating stresses on design.

As in the case of procedures for the design of members subject to other types of stress, concise design procedures for members subject to fluctuating stresses cannot be formulated entirely from tests made in the laboratory on small, carefully prepared specimens. In the case of metals, in addition to chemical composition, mechanical and physical properties, method of fabrication and environment (all of which affect the behavior of metal members subject to steady or static loading), the behavior of metal members subject to fluctuating stresses is influenced by a large number of other factors. For example, grain size, surface roughness, coatings, corrosion, geometry, size, internal stresses and previous stress history influence the fatigue life of a metal structure. In spite of the valuable contributions that have been made to the understanding of the behavior of structural members when subjected to steady stresses, and notwithstanding the large amount of research that has been done during the past century on members subjected to fluctuating stresses, there still is not complete understanding of fatigue. Thus, in designing to resist failure by fatigue, engineers must rely almost entirely on limited test data and experience, supplemented by empirical rules and formulas.

It is not the purpose of this paper to discuss fatigue in general, but it does present results of fatigue tests made on samples of T-1 steel and compares them with the results of similar tests made on samples of steels having considerably less strength. This information should aid designers to have a better understanding of T-1 steel and its possible application in structures subjected to fluctuating loads.

Two types of fatigue tests—reversed-bending and rotating-beam— are normally made to obtain basic information on steel in a particular metallurgical condition. When conducting these tests, an attempt is made to eliminate all factors other than the metallurgical condition of the steel. Carefully machined and polished specimens of the steel are subjected to a periodic stress cycle, which varies from a maximum

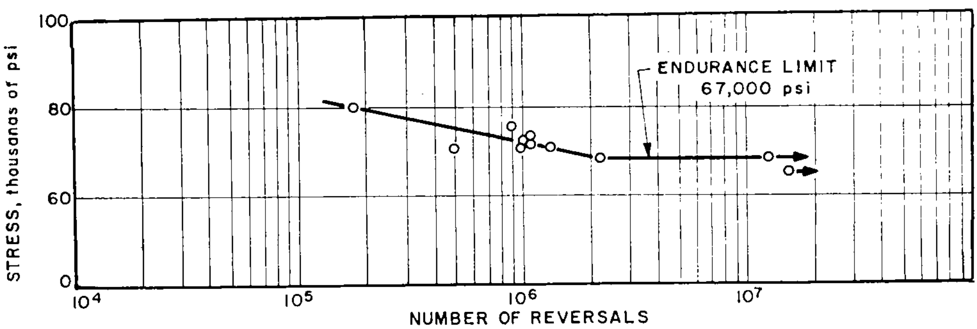

Figure 1. Results of polished rotating-beam fatigue tests on T-1 steel.

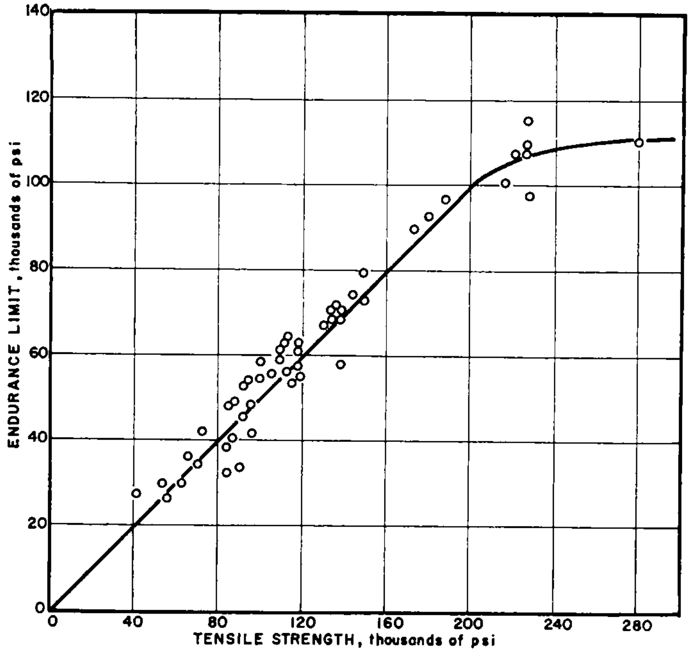

Figure 2. Correlation of endurance limits of steels with tensile strength, rotating-beam fatigue tests (from Ref. 4).

stress in tension to the same maximum stress in compression until fatigue failure, usually by rupture, occurs. The results of a series of such tests made on similar specimens at different magnitudes of maximum stress are then plotted on an S-N diagram, in which the ordinate is the magnitude of the maximum stress in the stress cycle and the abscissa is the number of stress cycles that the specimen endured prior to failure. A diagram of this type is shown in Figure 1. Like most ferrous alloys, T–1 steel has a limiting value of stress; that is, a stress at which the S-N diagram becomes horizontal and below which the material can presumably endure an infinite number of stress cycles. This limiting stress is known as the fatigue limit or the endurance limit. Although there is no theoretical basis to assume that the endurance limit is related to the mechanical properties of steel obtained from static loading conditions, it has been found from tests made on a large number of different steels, that a close correlation exists between the endurance limit of a steel and the tensile strength of the same steel as obtained from a standard tension test. This correlation, based on results obtained from tests on polished, unnotched steel specimens subjected to completely reversed cycles of flexural stress, is shown in Figure 2 (*4*). For tensile strengths up to about 200,000 psi, the endurance limit of steel is about 50 percent of its tensile strength. Thus, for a steel having a tensile strength of 115,000 psi, the minimum tensile strength of T–1 steel in plate thicknesses up to 2½ in., an endurance limit of about 57,500 psi would be expected. The results of the tests shown in Figure 1, together with the results of other tests, confirm that the endurance limit for polished specimens of T–1 steel, when subjected to reversed bending, is about 50 percent of the tensile strength. Unfortunately, structural members are not polished, nor are they subjected to completely reversed stress cycles, so that, from the standpoint of design, the results in Figures 1 and 2 are interesting but not directly applicable in design. Nevertheless, it should be emphasized that, when only metallurgical conditions are considered, the behavior of T–1 steel in fatigue is similar to that of other steels.

Numerous investigations have been made in attempts to correlate fatigue conditions encountered in service with results obtained from laboratory fatigue tests on small-size specimens. Realizing the importance of conditions encountered in structural joints, the Welding Research Council and the Research Council on Riveted and Bolted Structural Joints have sponsored considerable research and testing to obtain information on the strength of welded, riveted, and bolted joints in different strength steels subjected to both steady and fluctuating loads. The work of the latter has demonstrated the superiority of bolted joints over riveted joints for resistance to fluctuating loads and has resulted in a specification (*5*) covering the use of bolts. The similar and even earlier work of the Welding Research Council has provided information to support and amplify design rules for repeated loading which were incorporated in the AWS specification (*6*) for welded carbon steel bridges first issued in 1936. Because the authors believe that these reports are the best source of information available on the fatigue strength of steel in structural members, a review of some of the results obtained in this work is presented.

Figure 3 shows a comparison of the results (*7–14*) obtained from axially loaded zero-to-tension stress cycle fatigue tests made on plate specimens of various constructional steels in thicknesses varying from ½ to ⅞ in. in the as-received condition and the tensile strengths of the same steels. The abscissa (fatigue strength) is the maximum stress in the stress cycle which the specimen of steel was able to endure prior to failure at the indicated number of cycles. Although somewhat scattered, a reasonable correlation is again exhibited between fatigue strength and tensile strength. The steels

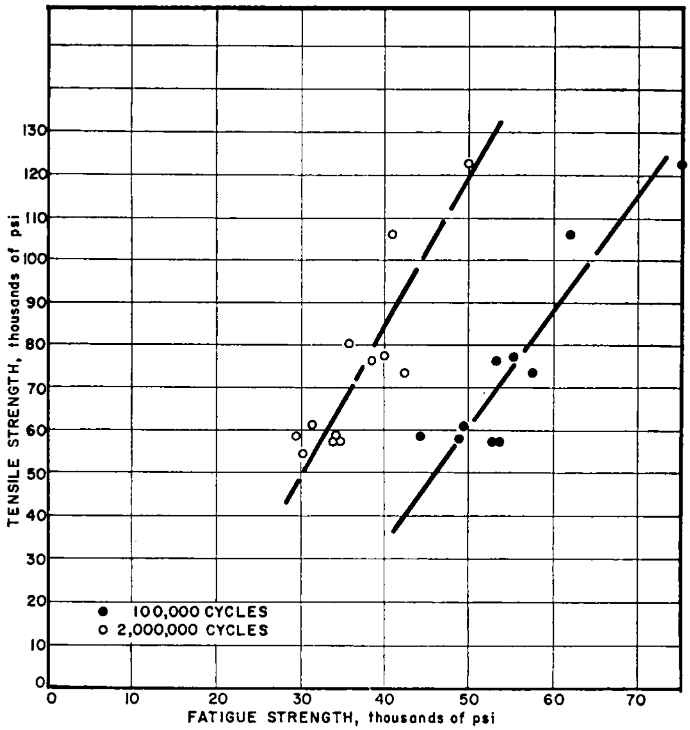

Figure 3. Zero-to-tension fatigue strengths of constructional steel plates, as received.

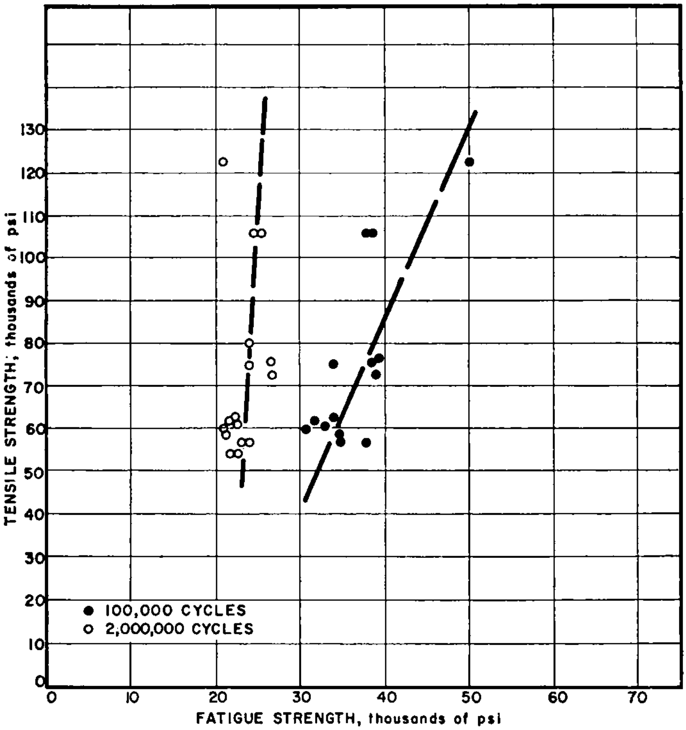

Figure 4. Zero-to-tension fatigue strengths of transverse butt-welded joints in constructional steel plates, as welded.

included in these tests were (a) structural carbon steel (ASTM A7 steel) having a 70,000-psi minimum tensile strength, (b) high-strength low-alloy steel (ASTM A242 steel) having a 70,000-psi minimum tensile strength, (c) structural silicon steel (ASTM A94 steel) having an 80,000-psi minimum tensile strength, and (d) T–1 steel having a 115,000-psi minimum tensile strength. Although the fatigue strength of the steel increases as the tensile strength of the steel increases for both the 100,000- and the 2,000,000-cycle lifetimes, the fatigue strength is not proportional to the tensile strength. Furthermore, the increase in fatigue strength is greater for 100,000 cycles than it is for 2,000,000 cycles.

Figure 4 shows a comparison of results (*7–14*) obtained from axially loaded zero-to-tension stress cycle fatigue tests made on specimens of transverse butt-welded joints in constructional steel plates varying in thickness from ½ to ⅞ in. and the tensile strengths of the same steels. These results are for joints in the as-welded condition; that is, with the weld reinforcement as-deposited, and without any post-weld heat treatment. With the exception of structural silicon steel, the grades of steel are the same as those previously described. Again, although scattered, there is a reasonable correlation between fatigue strength and tensile strength and, although the increase for 2,000,000 cycles is small, the fatigue strength increases as the tensile strength increases. Comparing Figure 4 with Figure 3, the presence of a transverse butt-weld in a plate reduces the fatigue strength of the plate considerably and this reduction of fatigue strength caused by a butt-weld increases as the tensile strength increases. Nevertheless, the fatigue behavior of T–1 steel is again similar to that of other steels.

In addition to the results of zero-to-tension stress cycle fatigue tests a limited number of tests has been made on transverse butt-welded steel plate

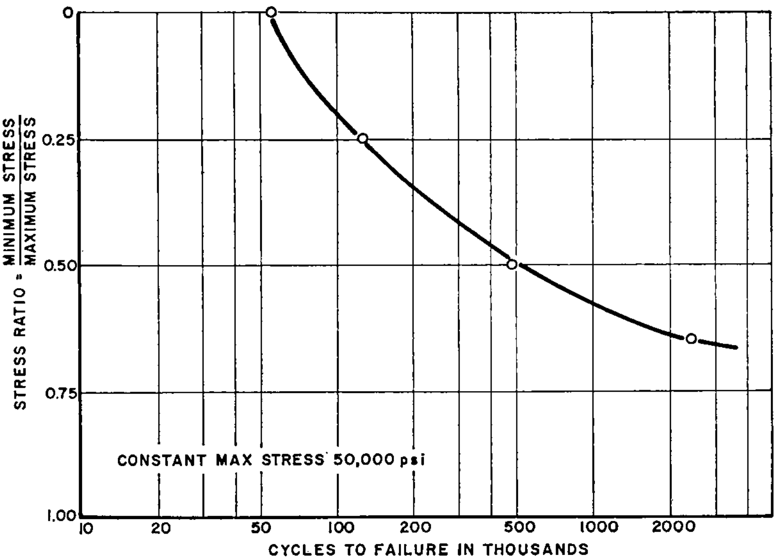

Figure 5. Results of fluctuating tension fatigue tests on transverse butt-welded joint of T-1 steel.

specimens subjected to different stress cycles. Figure 5 shows results obtained from axially loaded fluctuating tension stress cycle fatigue tests (*12*) made on specimens of transverse butt-welded joints in ½-in. thick plates of T–1 steel. The maximum stress in the stress cycle was the same for each test (50,000 psi), while the minimum stress in the cycle was different for each test. A stress ratio equal to 0.50 indicates that the stress in the stress cycle varied from 25,000 psi to 50,000 psi. For the same maximum stress, the number of stress cycles to produce failure increases as the stress ratio is increased. This behavior is typical for most steels and demonstrates the fact that fatigue failure is determined by three factors: (a) the maximum stress in the stress cycle, (b) the stress ratio

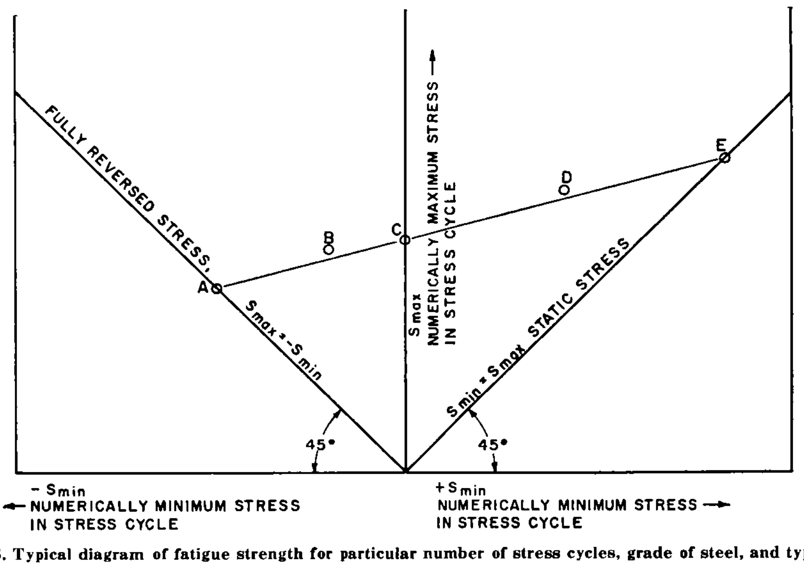

Figure 6. Typical diagram of fatigue strength for particular number of stress cycles, grade of steel, and type of joint.

or range of stress in the stress cycle, and (c) the number of applications of the stress cycle.

Numerous mathematical formulas and graphical methods that take into account the interdependence of these factors have been proposed to present fatigue data. Perhaps the most useful of these is a semigraphical method or diagram recommended by Committee F of Welding Research Council (*15*) and previously adopted by the American Welding Society for presenting allowable stress requirements for joints in the first issue of AWS Specifications for Welded Bridges. Figure 6 is a typical diagram drawn according to these recommendations. The coordinates of each point on the diagram represent the value of the maximum stress and the value of the minimum stress in a particular stress cycle that produced failure at a particular number of repetitions of the stress cycle. Thus, for a particular number of stress cycles to produce failure, Point A represents an average of results obtained from "fully" reversed-stress-cycle fatigue tests (maximum and minimum stresses of the same magnitude but of opposite sign); Point C represents results of zero-to-tension-stress-cycle tests (zero minimum stress); and Point E is the static or ultimate strength of the particular steel or joint (maximum and minimum stresses of the same magnitude and sign, that is, a steady-state stress condition). Point B represents the average of results obtained from "partly" reversed-stress-cycle fatigue tests, and Point D, the average of results from fluctuating, but not reversing-stress-cycle fatigue. The results of numerous tests made on a particular type of specimen and plotted on such diagrams indicate that the curve-of-best fit through points representing conditions which produce failure in the same number of stress cycles can be approximated by a straight line. Thus, points on the line (Fig. 6) indicate the different stress cycles that would be expected to cause failure of a particular type of specimen after a particular number of repetitions of the stress cycle.

Figure 7 shows dependable values of fatigue strength of axially loaded transverse butt-welded joints in carbon steel plates for three different numbers of cycles (*6*, Fig. A-4). Because 100 percent joint efficiency is expected for properly welded joints in constructional steels, the point on the static strength line through which extensions of the lines-of-

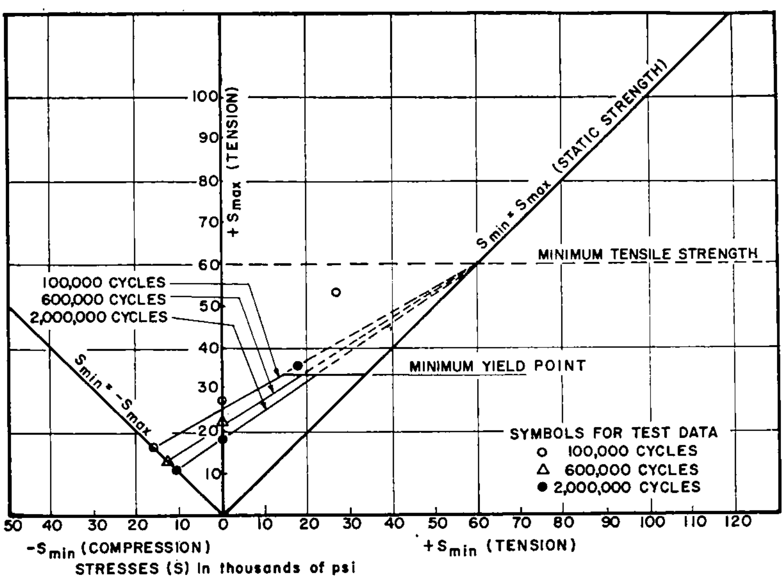

Figure 7. Fatigue strength of transverse butt-welded joints in carbon steel plates.

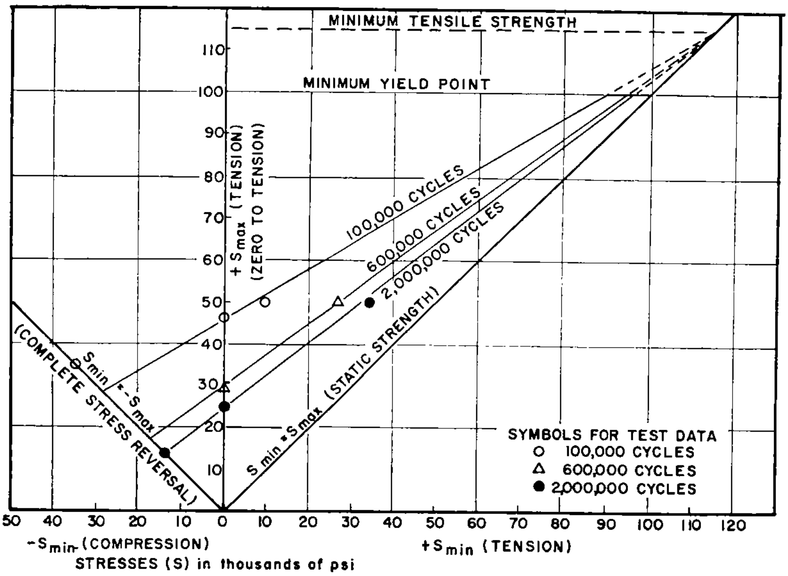

Figure 8. Fatigue strengths of transverse butt-welded joints in T-1 steel plates.

best fit pass is the minimum tensile strength for structural carbon steel. Because the stress requirements used in most structural designs are based on, and limited to, the yield strength of the materials, fatigue strengths for which the maximum stress in the stress cycle exceeds the yield point or yield strength of the steel cannot be used in place of the stress requirements based on yielding. Thus, the lines of constant lifetimes are limited to values of the maximum stress that are less than the minimum yield point or yield strength of the steel joined (Fig. 7).

Figure 8 shows similar results obtained from fatigue tests made on transverse butt-welded joints in T-1 steel plates. With the exception of the results for fully reversed stress cycles, the results were obtained from axially loaded fluctuating-tension fatigue tests (*12*) and Figure 4. The results for fully reversed stress cycles were obtained from reversed-bending fatigue tests which, according to several investigators (*16*), are usually equal to, or greater than, the stresses required to produce failure in axially loaded fatigue tests. Again, the lines of constant lifetime intersect the static strength line at the minimum tensile strength for T-1 steel (115,000 psi) and are limited by the horizontal cutoff at S_{max} equal to the minimum yield strength for T-1 steel, 100,000 psi. Comparing the lifetime curves of Figure 7 for structural carbon steel with Figure 8 for T-1 steel, for lifetimes greater than 600,000 cycles and loading conditions that produce stress ratios between -1.0 and about 0.2 (stress ratios between $S_{min} = -S_{max}$ and $S_{min} = \frac{1}{5}S_{max}$), the fatigue strength of butt-welded joints in T-1 steel plates is only slightly larger than that of similar joints in structural carbon steel plates. Thus, for these lifetimes and stress ratios, the engineer may not be justified in specifying the higher strength, higher cost T-1 steel. However, for loading conditions that produce stress ratios greater than about 0.2, there is considerable advantage in using T-1 steel in place of structural carbon steel, and this advantage increases as the stress ratios increase above 0.2. Furthermore, for all stress ratios where the lifetime is less than 600,000 cycles, the advantage of using T-1 steel increases as the lifetime decreases.

Figure 9 shows the results obtained from a limited number of fatigue tests made on plate specimens of T-1 steel in

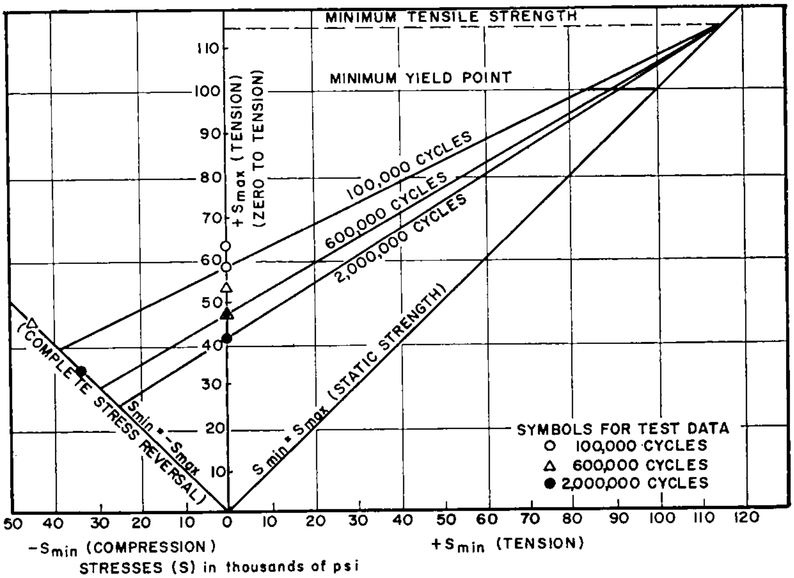

Figure 9. Fatigue strength of as-received T-1 steel plates.

the as-received condition varying in thicknesses from ½ to ¾ in. The lines-of-best fit are also shown. The results shown on the zero-to-tension line were obtained from fatigue tests made on axially loaded specimens, whereas the results shown on the complete stress reversal line were obtained from tests made on specimens subjected to reversed-bending fatigue. Again, the lines of constant lifetimes intersect the static strength line at the minimum tensile strength for T–1 steel and are further limited by the horizontal line at the minimum yield strength. Although these results are not compared here with similar results for plate specimens of structural carbon steel, a comparison of these results with Figure 8 will again indicate the deleterious effect a butt-welded joint has on the fatigue strength of a plate.

Based on information similar to that presented in Figure 7, the AWS Specifications for Welded Bridges provide formulas and rules for determining the cross-section of base metal and effective area of weld metal required to resist failure when subjected to fluctuating stresses. For transverse butt-welded joints, proportioned on the basis of fatigue considerations where the maximum stress in the cycle is tensile, the specification formulas seem to provide from about 33 to 42 percent more cross-sectional area for base metal and weld metal than the area of a joint that would fail if it were subjected to the same number of cycles. Thus, for butt-welds subject to fatigue, where the maximum stress in the stress cycle is tensile, the specification formulas imply a margin or factor-of-safety of from 1.33 to 1.42 against failure caused by fatigue. This margin-of-safety against fatigue failure is considerably less than that normally provided in specifications for similar structures subject to steady loading conditions. As stated in Appendix A of the previously mentioned specification, the justification for the use of the smaller margin-of-safety against failure for fluctuating stresses than is normally used for steady stresses is based on the consideration that "the margin against an event that will certainly happen once, or a few times, must be greater than for a succession of events which may never accumulate."

Based on the lines-of-best fit shown in Figures 8 and 9, formulas for permissible unit stress values or required cross-

TABLE 4
SUGGESTED UNIT STRESSES[a] FOR T-1 CONSTRUCTIONAL ALLOY STEEL PLATES AND WELDED JOINTS[b]
(Material Not Over 2½ Inches Thick)

Repetitions of Loading Producing Maximum Stress	Required Effective Cross-Sectional Area, sq in.		
	Base Metal Not Adjacent to Welds—Maximum Stress, Tension	Butt Welds—Maximum Stress, Tension	Fillet Welds
100,000	$A = \frac{\text{Max} - 0.50\ \text{Min}}{39,500}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.60\ \text{Min}}{31,000}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.60\ \text{Min}}{20,500}$ but $\geqq$ Max/37,000
600,000	$A = \frac{\text{Max} - 0.60\ \text{Min}}{33,000}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.75\ \text{Min}}{21,000}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.75\ \text{Min}}{14,000}$ but $\geqq$ Max/37,000
2,000,000	$A = \frac{\text{Max} - 0.65\ \text{Min}}{29,000}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.80\ \text{Min}}{16,500}$ but $\geqq$ Max/54,000	$A = \frac{\text{Max} - 0.80\ \text{Min}}{9,000}$ but $\geqq$ Max/37,000

[a] Max and Min refer to total stress; max is the numerically greater stress.
[b] For weld metal having strength equal to or greater than that of T-1 steel.

sectional area have been derived for T–1 steel plates and transverse butt-welded joints in such plates subject to fluctuating stresses. These formulas, which are similar to the AWS specification formulas and provide about the same margin-of-safety as is provided in the specification, are given in Table 4. The cross-sectional area of base metal and of welds is not to be less than the area obtained by dividing the maximum total stress by the suggested allowable basic tensile stress for T–1 steel, 54,000 psi. That is, the required cross-sectional area based on fatigue considerations must not be less than the cross-sectional area required from a consideration of yielding caused by the maximum load or maximum total stress in the stress cycle. The formulas for butt-welded joints are plotted as solid lines in Figure 10 together with the formulas for butt-welded joints in carbon steel plates given in the AWS specifications, plotted as dashed lines. Comparing the plots of the formulas for the two steels for similar lifetimes, the fatigue strength of T–1 steel is greater than that of carbon steel for all stress ratios and the difference between the fatigue strength of T–1 and carbon steel increases as the stress ratio increases and as the lifetime decreases. As mentioned previously, although the increased fatigue strength of T–1 steel over carbon steel is small for lifetimes greater than 600,000 cycles and stress ratios less than 0.2, its fatigue strength is substantially higher than that of carbon steel for stress ratios greater than 0.2, and also for all stress ratios when the lifetime is less than about 600,000 cycles.

Another type of welded joint which must be considered is the fillet-weld subject to fluctuating stresses. Although considerable information obtained from tests is available on the fatigue strength of this type of joint in structural carbon steel plates, the authors are not aware of similar information on the fatigue strength of fillet welds on T–1 steel plates. Nevertheless, considerable information is available on the satisfactory service performance of T–1 plates joined by fillet welds in structures and structural members subject to fluctuating stresses. These applications include side, end and floor plates of railroad freight cars; dipper sticks for power shovels and dredges; bodies and supporting structural members of earth moving and mining equipment; and primary longitudinal supporting members of highway trailers. Comparing the AWS specification formulas for fillet or plug welds subject to

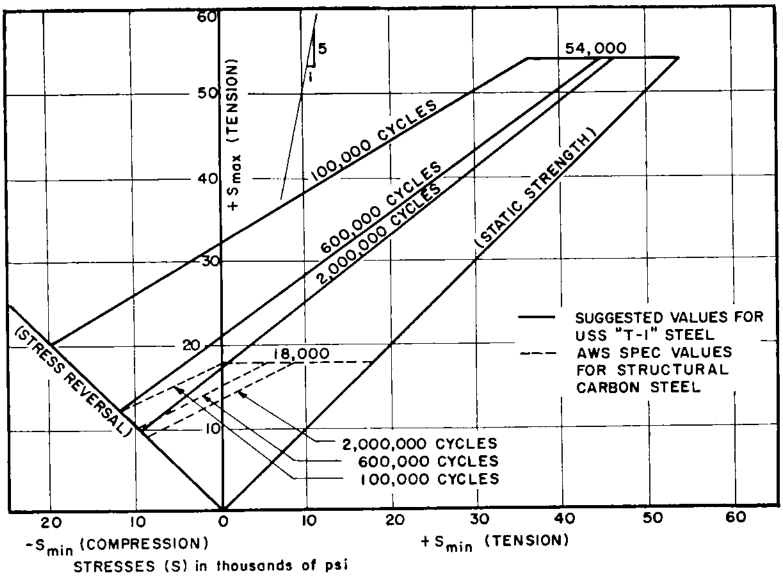

Figure 10. Permissible stress values for butt-welded joints in T-1 and carbon steel plates subject to fatigue.

fluctuating shearing stresses and the corresponding formulas for transverse butt welds in tension members subject to the same number of cycles of fluctuating longitudinal stresses, the corresponding formulas are directly proportional when the type of stress is not considered (shearing or longitudinal). Furthermore, when the type of stress is not considered and when the magnitudes and signs of the corresponding maximum and minimum stresses are the same, the required area for fillet welds subject to 100,000 and 600,000 cycles is 50 percent greater than that required for butt welds subject to the same numbers of cycles. Similarly, the required area for fillet welds subject to 2,000,000 cycles is approximately 88 percent greater than that required for butt welds subject to the same number of cycles.

Based on the satisfactory service performance of T–1 steel, the test results on plate and butt-welded joints which indicate a similarity in the fatigue behavior of T–1 steel and other steels, and the previously mentioned proportionality between the specification formulas for required areas for fillet and butt welds in carbon steel plates, formulas for the required area for fillet welds in T–1 steel plates subject to fluctuating stresses are presented in Table 4. The formulas for fillet welds are not based on test results, but are derived from the formulas for butt welds in T–1 steel plates so as to provide the same proportionality between formulas for fillet and butt welds as exists in the AWS specifications for carbon steel.

Information obtained from tests on the strength of butt welds in carbon steel plates subject to fluctuating shearing stresses and of carbon steel base metal adjacent to connections made by fillet welds in axially stressed tension members subject to fluctuating stresses is limited. Furthermore, there appears to be no direct correlation between formulas provided in the AWS specifications for these conditions and the formulas provided in the specification for other conditions. However, a comparison of the formulas indicates that those for butt welds in shear and for base metal adjacent to fillet welds are not much different from those for fillet welds. Therefore, it is suggested that the formulas for fillet welds (Table 4) be used for determining the required area of butt welds subject to fluctuating shearing stresses and also for the required area of base metal adjacent to fillet welds in axially stressed members subject to fluctuating stresses. It is also suggested that the areas obtained from the formulas for fillet welds not exceed (a) Max/37,000 when they are used for butt welds in shear and (b) Max/54,000 when they are used for base metal adjacent to fillet welds. The foregoing procedure should provide a satisfactory basis for proportioning butt welds in shear in T–1 steel and for T–1 steel base metal adjacent to fillet welds.

It is realized that the foregoing information on fatigue strength of T–1 steel is by no means complete. However, the limited information presented should be useful to engineers concerned with the design of structures made of this steel and should emphasize the importance of limitations imposed by fatigue considerations on the application of higher strength steels in structures.

CONCLUSION

The composition, treatment and fabrication practices for T–1 steel have been reviewed. This material may be shaped by any of the standard fabricating practices, if allowances are made for its high yield and tensile strengths. Guidance for welding with low-hydrogen methods is given, including recommendations for control of heat input. Flame-cutting or welding operations should only be performed after heat treatment. The importance of good workmanship is emphasized.

Examination of the fatigue characteristics of constructional steels shows that the conventional reversed-bending endurance limit as determined on a polished specimen will not predict performance in a gross structure containing stress-raisers such as weld beads, scratches and as-rolled surfaces. By use of a diagrammatic method of presentation of fatigue data recommended by the Welding Research Council, it is shown that the strength characteristics of T–1 steel may be used to advantage in applications subject to fluctuating stresses. Formulas are derived for suggested unit stress values or required cross-sectional area for butt and fillet welds.

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