44

Structural Steel

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44.1 Properties and Processes

Types of Steel

The term *structural steel* is generally taken to include a wide variety of elements or components used in the construction of buildings and many other structures. These include beams, girders, columns, trusses, floor plates, purlins and girts. Many of these elements are made from standard hot rolled structural shapes, cold formed shapes or made up from plates using welding. They are joined at connections made using plate, structural shapes, welding, and fasteners. Fasteners include bolts, nuts, washers and stud shear connectors. For a more complete listing of *structural steel* elements see the American Institute of Steel Construction (AISC) Code of Standard Practice for Steel Buildings and Bridges, Section 2.1.

A variety of steel types are used to produce these structural shapes, plates and other components depending on the intended use and other factors such as the importance of cost, the weight of the structure and corrosion resistance. The properties of steel products result from a combination of the chemical composition, the manufacturing processes and the heat treatment. The properties most commonly used as a basis for specification and design are the specified minimum yield stress (yield point or yield strength) and the specified minimum tensile strength (or ultimate strength), both obtained from tensile tests on small representative specimens of the steel (see below). Other properties also required are ductility, weldability, and fracture toughness (or notch ductility) although these requirements are often not explicitly stated. The steels most commonly used are:

- carbon steels
- high-strength low-alloy steels
- corrosion resistant, high-strength low-alloy steels
- quenched and tempered alloy steels

The elastic modulus is virtually the same for each of these types of steel, but their yield stress and ultimate strength vary widely. The range of strengths for each of these types of steel is shown along with the typical products and uses in Table 44.1.

The production of structural steel structures and buildings is a complex process involving making the steel, processing it into useful products, fabricating these products into useful assemblies or structures,

TABLE 44.1 Types and Usage of Steel Used In Construction

Steel Type	ASTM Specification	Grade	Products	Usage
Carbon	A36	36	Shapes, plates and bars	Riveted, bolted or welded bridges, buildings and general structural purposes
	A529	50 55	Shapes, plates and bars Shapes, plates and bars	Riveted, bolted or welded buildings and general structural purposes
High-strength, low-alloy	A572	42 50 55	Shapes, plates, sheet piling and bars	Riveted, bolted or welded structures
		60 65		Riveted or bolted bridges, or riveted, bolted or welded construction in other applications
	A992	50	W shapes	Building framing
Corrosion-resistant, high strength low-alloy	A242	50	Shapes, plates and bars	Riveted, bolted or welded construction for weight saving or added durability
	A588	50	Shapes, plates and bars	Riveted, bolted or welded construction, primarily for welded bridges or buildings for weight saving, added durability or good notch toughness
Quenched and tempered alloy steels	A852	70	Plates	Riveted, bolted or welded construction, primarily for welded bridges or buildings for weight saving or added durability
	A514	90 100	Plates to 6 in thickness	Welded bridges and other structures
	A913	50 60 70	Shapes	Riveted, bolted or welded bridges, buildings and other structures

and erecting and assembling these components, assemblies, and structures into buildings, bridges, cranes, and the myriad of other structures and pieces of machinery and equipment constructed using structural steel. It requires a team effort on the part of specialists who understand in detail the processes and pitfalls that are associated with each step in the overall process. Such is the complexity of each of these areas that specialists are required in each area and each in turn must contribute appropriately for the final product to be successful.

The first step in the production process, steel making and rolling to form finished products suitable for use in buildings and structures, are themselves very complex processes requiring extensive knowledge of the metallurgy and mechanical forming of steel. Designers should be aware of the importance of certain parameters and processes because they can have a major effect on the performance of a steel structure, but they normally do not specify or need details of precisely how the steel is produced, rolled or formed. Indeed structural steels are usually specified in terms of a very limited number of parameters that describe their essential attributes.

Structural steels are usually produced by rolling steel cast from the steelmaking process after reheating it to the austenizing range (above 850°C). Rolling consists of passing the steel through a series of rolls that form the cast steel into the shape and/or thickness required. A very wide range of shapes and sizes are currently rolled or available in the U.S. (Table 44.2). The shapes are designated as follows:

- W shapes are parallel flanged I or **H** sections used as beams and columns
- S shapes are I sections with inside flange surfaces that slope used as beams
- HP shapes are parallel flanged **H** sections used as beams with flanges and webs of similar thickness
- M shapes are I sections not qualifying as W, S or HP shapes

TABLE 44.2 Range of Structural Shapes Available in the U.S.

Shape Size Groupings for Tensile Property Classification\ (as given in ASTM A6/A6M – 1998)				
Group 1	Group 2	Group 3	Group 4	Group 5
	W40 × 149 to 268	W40 × 277 to 328	W40 × 362 to 655	
	W36 × 135 to 210	W36 × 230 to 300	W36 × 328 to 798	W36 × 920
	W33 × 118 to 152	W33 × 201 to 291	W33 × 318 to 619	
	W30 × 90 to 211	W30 × 235 to 261	W30 × 292 to 581	
	W27 × 84 to 178	W27 × 191 to 258	W27 × 281 to 539	
W24 × 55 & 62	W24 × 68 to 162	W24 × 176 to 229	W24 × 250 to 492	
W21 × 44 to 57	W21 × 62 to 147	W21 × 166 to 223	W21 × 248 to 402	
W18 × 35 to 71	W18 × 76 to 143	W18 × 158 to 192	W18 × 211 to 311	
W16 × 26 to 57	W16 × 67 to 100			
W14 × 22 to 53	W14 × 61 to 132	W14 × 145 to 211	W14 × 233 to 550	W14 × 605 to 873
W12 × 14 to 58	W12 × 65 to 106	W12 × 120 to 190	W12 × 210 to 336	
W10 × 12 to 45	W10 × 49 to 112			
W8 × 10 to 48	W8 × 58 & 67			
W6 × 9 to 25				
W5 × 16 & 19				
W4 × 13				
M shapes to 18.9 lb/ft				
S shapes to 35 lb/ft				
	HP shapes to 102 lb/ft	HP shapes over 102 lb/ft		
C shapes to 20.7 lb/ft	C shapes over 20.7 lb/ft			
MC shapes to 28.5 lb/ft	MC shapes over 28.5 lb/ft			
L shapes to ½ in	L shapes over ½ in to ¾ in	L shapes over ¾ in		
Tees cut from W, M and S shapes remain within the group of the shape from which they are cut.				

- C shapes are channels with inside flange surfaces that slope similarly to S shapes
- MC shapes are channels that do not qualify as C shapes
- L shapes are equal and unequal legged angles

The groups in Table 41.2 are convenient groups based on the thickness of the shape at the point where standard tension test (see below) specimens are taken and are used in the ASTM materials specifications and in design specifications such as the American Institute of Steel Construction Load and Resistance Factor Design Specification for Structural Steel Buildings. They are useful because in some material specifications the specified minimum tensile properties differ for different thicknesses of steel and care must be exercised in design to use appropriate properties and fabrication and welding procedures for the shapes used.

The properties of steel largely result from the influence of microstructure and grain size though other factors such as non-metallic inclusions are also important. The grain size is strongly influenced by the cooling rate, to a lesser extent by other aspects of heat treatment and by the presence of small quantities of elements such as niobium, vanadium and aluminium. The chemical composition of steel is largely determined when the steel is liquid but for a given chemical content the structure is largely determined by the rate at which it is cooled and may be altered by subsequent reheating and cooling under controlled conditions. The microstructure of steel is classified in terms of several crystal structures called ferrite, austenite, pearlite, and cementite. Phase diagrams are used to map the steel structure in relation to temperature and carbon content and transformation diagrams are used to map the structure in terms of temperature and time. If steel is allowed to cool slowly, the crystal lattice structure and microstructure reaches equilibrium dependent on the steel temperature and the carbon concentration (represented by the phase diagram). If it is cooled faster, the structure is dependent on the cooling rate and the chemical content of the steel (represented by transformation diagrams).

There are several terms associated with cooling and heat treatment that will now be briefly introduced. Normalized steel is allowed to cool in air from temperatures above 850°C, developing a reasonably fine grain structure, moderate strength and good toughness. Controlled rolling involves rolling the steel to the desired form within a comparatively narrow temperature range and then allowing it to cool naturally in air. This results in higher strength than achieved through conventional rolling. Quenching is rapid cooling by immersing the steel in water or oil to produce a higher strength but less ductile and tough product. Tempering is a subsequent process in which the temperature is again raised, but not to the austenitizing range, and cooled more slowly resulting in a reduction in strength but increased ductility and toughness. All of these processes are used in producing the steel types listed in Table 44.1. In addition some structural steel shapes are produced by a patented process of quenching and self-tempering (QST) which produces shapes with mechanical properties equivalent to those that would normally be obtained using reheating after rolling.

Carbon steels are largely composed of iron with up to 1.7% carbon, but the addition of relatively small quantities of other elements greatly influences its behavior and properties. For structural purposes it is desirable that steel be ductile and weldable, and consequently most structural carbon steels are "mild steel" with carbon in the range 0.15 to 0.29% and may include small quantities of manganese, silicon and copper.

High-strength low alloy steels include alloying elements such as chromium, copper, nickel, silicon and vanadium. In addition to their higher strength these steels may also possess greater corrosion resistance than carbon structural steels.

Corrosion resistant, high-strength low-alloy steels, due to the addition of greater quantities of copper, have corrosion resistance properties substantially greater than carbon and low-alloy structural steels. Under suitable environmental conditions, including several cycles of wetting and drying, a stable layer of corrosion products is developed on the surface of these steels and further corrosion is inhibited.

Quenched and tempered alloy steels contain greater concentrations of alloying elements than carbon and low-alloy structural steels and specific heat treatments are incorporated in the manufacturing process primarily to increase their strength. Generally this increased strength is obtained at the cost of reduced ductility and great care must be exercised in their use.

Table 44.1 summarizes the range of structural steels available in the U.S. according to their American Society for Testing and Materials (ASTM) specification, the grades available and the products and uses for which they are intended. It should be understood that many shapes, plates and bars are not readily available in all of these grades and that their availability should be checked with suppliers or fabricators before they are specified in designs. The more readily available types and grades are:

- W shapes — A992
- M, S, HP, C and L shapes — A36 (and increasingly A572 Grade 50)
- HSS — A500 Grade B for round and A501 Grade B for square and rectangular (and increasingly Grade C for both)
- Plate — A36 and A572 Grade 50

Structural Properties

As mentioned above structural steels are primarily designated in terms of their strength as measured in tensile tests. In these tests normally conducted in accordance with ASTM 370 (Test Methods and Definitions) specimens of the steel cut from the shape or plate are tested in tension. Figure 44.1 shows a typical stress-strain diagram obtained from such a test for a carbon steel. It shows a well defined yield point and yield plateau, with subsequent strain hardening and substantial elongation before rupture. Figure 44.2 shows the low initial (low strain) portion of the stress-strain diagram shown in Fig. 44.1 in more detail.

The behavior shown in Figs. 44.1 and 44.2 with an extended yield plateau and substantial strain hardening results in structures that have a great capacity for deformation and redistribution of stresses before failure.

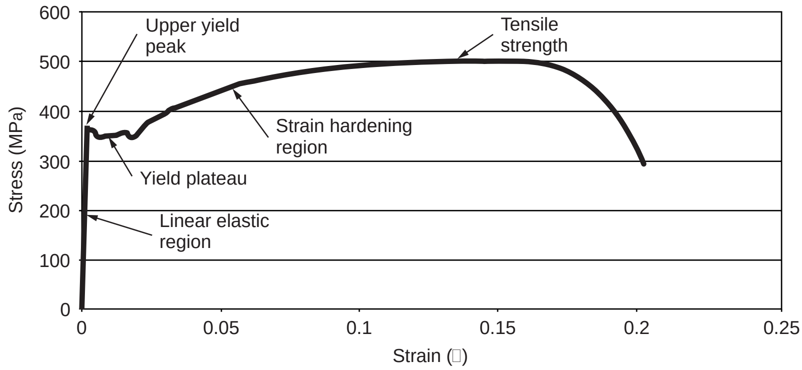

FIGURE 44.1 Typical stress-strain curve for carbon steel.

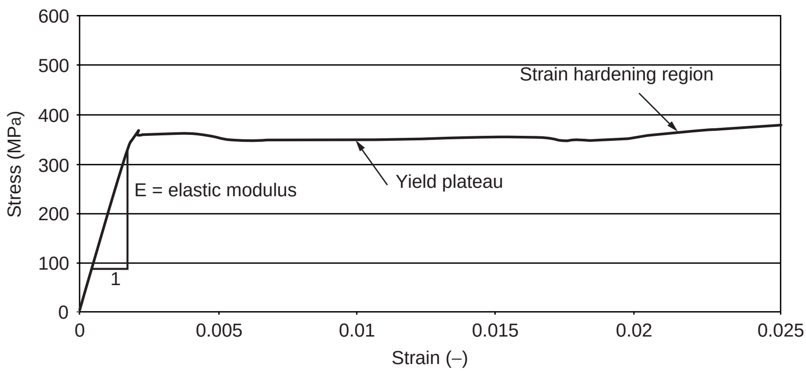

FIGURE 44.2 Detail of low strain region of stress-strain curve shown in Fig. 44.1.

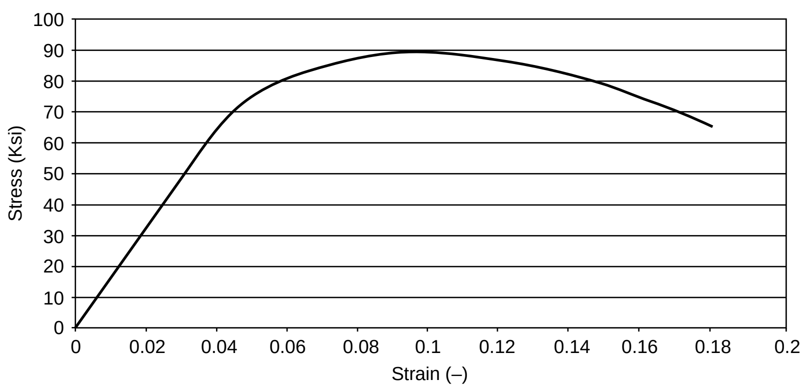

FIGURE 44.3 Form of stress-strain curve for high strength steels.

Higher strength structural steels (low alloy and quenched and tempered) do not show the well-defined yield plateau seen in Figures 44.1 and 44.2. The form of stress-strain curve obtained for such steels is shown in Fig. 44.3. The yield stress for these steels is determined from the yield strength, which is the stress at which a nominated strain is reached. The yield strength is determined by either the 0.2% offset method or the 0.5% extension-under-load method as shown in Fig. 44.4. Steels with this form of stress-strain curve do not exhibit the same elongation capability and stress redistribution capability as mild steels.

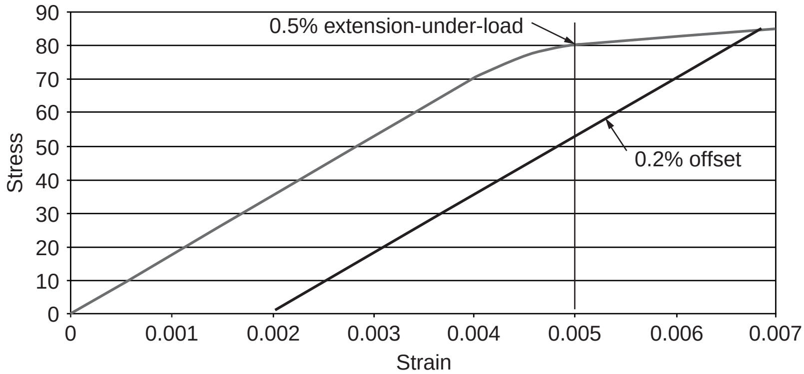

FIGURE 44.4 0.2% Offset and 0.5% elongation-under-load yield strength.

TABLE 44.3 Strength and Elongation Requirements for Structural Steels

Steel Type	ASTM Specification	Grade	Specified Minimum Yield Stress* Ksi (MPa)	Specified Minimum Tensile Strength* Ksi (MPa)	Elongation*
Carbon	A36	36	36 (250)	58 (400)	20%
	A529	50	50 (345)	70 (485)	18%
		55	55 (380)	70 (485)	17%
High-strength, low-alloy	A572	42	42 (290)	60 (415)	20
		50	50 (345)	65 (450)	18%
		55	55 (380)	70 (485)	17%
		60	60 (415)	75 (520)	16%
		65	65 (450)	80 (550)	15%
	A992	50	50 (345) max. 65 (450)	65 (450)	18%
Corrosion-resistant, high strength low-alloy	A242	50	50 (345)	70 (485)	18%
	A588	50	50 (345)	70 (485)	18%
Quenched and tempered alloy steels	A852	70	70 (485)	90 (620)	19%#
	A514	90			
		100	100 (690)	110 (760)	18%#
	A913	50	50 (345)	65 (450)	18%
		60	60 (415)	75 (520)	16%
		65	65 (450)	80 (550)	15%
		70	70 (485)	90 (620)	14%

* Note: 1. In some cases specified minimum elongation varies with thickness or product. Check specification for details.
2. # = elongation on 2 in. gauge length, elsewhere it is on an 8 in. gauge length

Typical specified minimum yield stress, ultimate stress and elongation for structural steels used in the USA are given in Table 44.3.

Heat Treatment

The production of steel and steel products involves heat and the effects of heating and cooling throughout. A brief description of some of the forms of heat treatment that may be used in producing structural steel plate and shapes was covered in Section 44.1. This section contains a brief description of forms of heat treatment that may be used in the subsequent processes of fabrication and erection of steel buildings and structures.

Many of the processes used in forming, fabricating and erecting structural steel can have an effect on the steel that is undesirable or that may affect the performance of the structure. These effects can include high residual stresses after forming or welding, excessive hardening (or softening) substantially altering the as-rolled properties of the steel, etc.

The processes that were described earlier can be used during or after the fabrication processes to reduce or repair some of these changes or effects.

Heat is sometimes used during fabrication to straighten or camber members and to bend or straighten plates. Welding always introduces residual stresses into the steel, but excessive welding or welding in restrained situations can be particularly concerning in that it can introduce residual stress fields that can significantly affect the performance of structural members or connections between members. It is possible to relieve (reduce) these residual stresses by uniformly heating the entire assembly or by heating a suitable portion of it.

In carrying out such heat treatment, it is important that full cognizance be given to the processes by which the components were made and possible changes in the structure or properties of the steel that can occur with incorrect or inappropriate heat treatment. This may involve limiting the maximum temperature to which the assembly is raised or limiting the duration of high temperatures.

Welding

Welding is perhaps the most important process used in the fabrication and erection of structural steelwork. It is used very extensively to join components to make up members and to join members into assemblies and structures. Welding used and done well helps in the production of very safe and efficient structures because welding consists of essentially joining steel component to steel component with steel that is intimately united to both. It can lead to very efficient paths for actions and stresses to be transferred from one member or component to another. Conversely, welding used or done badly or inappropriately can lead to potentially unsafe or ineffective structures – welds containing defects or inappropriate types or forms of joints can cause failure or collapse of members or structures with little or no warning. Thus care is required in the design of welds, in the design or specification of welding processes, in the actual process of welding components one to another, and in the inspection of welding to assure that it is as specified and fit for purpose.

As with the production of the structural steel components, specialist expertise is required for successful welding. This is built on a foundation of knowledge of the metallurgy of steel but also requires knowledge of the processes and materials involved in welding.

Welding of structural steel is usually the process of joining two pieces of similar (not necessarily identical) steel by casting a further quantity of steel between them and fused to each of them, but it may equally involve no filler material, simply the melting together of the two pieces to be joined. The process involves heating and melting the surfaces of the pieces to be joined and, when required, the steel to make the weld.

Currently the most common methods of welding may be classified as arc and resistance welding. Arc welding involves the process of striking an arc to pass a large electric current from one piece of metal to another. The arc itself is at a very high temperature and heats the materials nearby. The arc is normally struck between the metal that is to form the filler material and the workpiece, but in some processes is struck between a separate electrode and the workpiece. Resistance welding involves the very rapid discharge of a very high current between the two pieces to be joined resulting in the release of heat and the melting of the mating surfaces. Both processes involve rapid heating and cooling of the weld metal and the material adjacent to the weld and thus may alter the metallurgy and properties of these materials. Thus great care needs to be taken with the design, specification and execution of the welds to ensure that the final product is sound, has appropriate properties and does not lead to unintended effects such as excessive residual stresses.

There are several arc welding processes commonly used in structural steel fabrication and erection. Shielded metal arc welding (SMAW) is a manual process involving a consumable electrode (stick) which consists of a length of wire that is coated with a flux. The arc is struck between the electrode and the workpiece with the electrode coating forming a gas shield that protects the arc and molten metal from the air, stabilizing the arc and preventing the oxidation of the metal. After the weld is complete the solid residue of the coating (slag) solidifies over the weld metal and may be removed mechanically (chipping, etc). In gas metal arc welding (GMAW) the short stick of SMAW is replaced by a continuous uncoated wire electrode from a coil which is fed through a lightweight gun that also feeds a protective gas (often carbon dioxide) to protect the arc. GMAW has the advantage over SMAW of a continuous process and results in high deposition rates (of the filler metal).

A welding process often used in fabrication shops is submerged arc welding (SAW), which is usually performed automatically or semi-automatically. SAW usually uses a continuous wire electrode and the arc struck between the rod and the workpiece is protected by granular flux that covers the end of the electrode, arc and workpiece in the area near the arc. Thus the arc is submerged in flux and is not visible. Very high deposition rates can be achieved using SAW. Flux-core arc welding (FCAW) also uses a continuous electrode but in this case it is hollow and contains flux as a core. Often gas shielding in addition to the shielding provided by the flux material is provided to protect the work area.

Defects of various kinds can result from welding. These can be conveniently classified as planar and volume discontinuities (solid, such as slag, or gaseous inclusions) and shape (or profile) imperfections. In all cases these defects can result in the area of the weld being less than that intended and, often more importantly, in stress concentrations that increase residual and load induced stresses. The greatest stress concentrations arise from the planar discontinuities that in turn can be classified as arising from lack of penetration or fusion, hot cracking, cold cracking, lamellar tearing, and reheat cracking.

Lack of penetration or fusion (and slag inclusions) normally results from operator error or incorrect procedure specification. Hot or solidification cracking occurs during the solidification of the weld due to impurities such as sulfur and phosphorus deposited near the centerline of the weld. It is controlled by minimizing such impurities and by avoiding deep narrow welds. Cold or hydrogen-induced cracking results from the combination of a hardened microstructure and the effect of hydrogen in the steel lattice, and is avoided by keeping the hydrogen concentration to very low levels through control of electrode coatings and electrode storage. Lamellar tearing results from excessive non-metallic inclusions in rolled steel plates and shapes. Such inclusions result in planes of weakness within plates and the flanges and webs of shapes that split if subjected to high stresses produced by restraint of shrinkage as the weld metal cools. The problem is avoided by again keeping impurities low (in this case sulfides and silicates), and also by testing the tensile properties through the thickness (rather than along the length of the element as is usual) and requiring adequate ductility, and by avoiding welds where there is excessive restraint of welds in vulnerable locations. Reheat cracking occurs during stress relief treatment or prolonged high temperature exposure of steelwork, particularly of steels containing molybdenum or vanadium.

Steels are not uniformly suited to welding. Certain steel compositions make the resultant welds more likely to be hard and brittle, and thus more likely to be subject to cold cracking or brittle fracture. The weldability (that is suitability of a steel for welding using conventional practice) of steels is often assessed using the "carbon equivalent"(CE), particularly of carbon-manganese and low-alloy steels [ASTM A6]:

$$\mathrm{CE} = \mathrm{C} + \mathrm{Mn}/6 + \left(\mathrm{Cr} + \mathrm{Mo} + \mathrm{V}\right)/5 + \left(\mathrm{Ni} + \mathrm{Cu}\right)/15$$

This formula uses the concentrations of the various elements and effectively "scales" the propensity of each element to harden the steel compared with that of carbon.

44.2 Service Performance

Brittle Fracture

Structural steel members or elements may become liable to brittle fracture under some conditions, although this rarely occurs in practice.

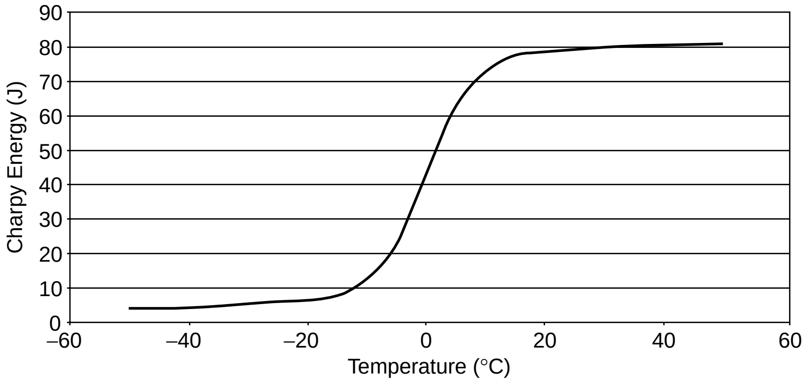

FIGURE 44.5 Illustrative Charpy energy transition curve.

Brittle fracture normally only occurs when a critical combination of the following exist:

- a severe stress concentration due to a notch or severe structural discontinuity
- a significant tensile force occurs across the plane of the notch (or equivalent)
- low fracture toughness of the steel at the service temperature
- dynamic loading

The potential for brittle fracture is generally addressed by eliminating or minimizing the effect of each of these factors. Thus, using structural steels that have suitable notch ductility at the expected service temperatures, reduction in stress, particularly residual stresses due to welding or forming, and the use of details that do not give rise to severe notches or structural discontinuities are the preferred methods of reducing the risk of brittle fracture.

The fracture behavior of steel changes from brittle to ductile as the temperature increases. The Charpy V Notch impact test is used as a relatively low cost test that can be used to monitor the potential for brittle fracture of steel. For a given steel, the energy required to fracture the Charpy V Notch specimens typically is low at low temperatures, rises quickly with increase in temperature through a transition range and is relatively high at higher temperatures (Fig. 44.5). The change in energy required reflects a change in the mode of fracture: from brittle fracture at the lower temperatures to ductile or fibrous fracture at the higher temperatures, with a very variable mixture of the two through the transition temperature range.

The need for steel with high fracture toughness depends on the expected service temperature range. If the service temperature is expected to become low it is necessary to ensure that the transition from brittle to ductile fracture behavior under service conditions occurs at a temperature below the expected service temperature. The transition under service conditions normally takes place at temperatures substantially below those at which it takes place in the Charpy V notch test because the strain rate under service conditions is usually much lower than that which occurs during the test. Thus, specifications do not require the transition temperature to be lower than the service temperatures, for example, for bridges the steel specifications require the test to be performed at a temperature 38°C greater than the expected minimum service temperature. These specifications also provide different requirements for members depending on whether the members are fracture critical or not and more stringent requirements for weld metal compared with the steel in the members.

In general, designing structures so that they only incorporate details that provide good fatigue performance is a very effective way of reducing brittle fracture. Normally, structures such as bridges that have the potential to suffer both fatigue damage and brittle fracture, it is fatigue damage that is more likely to occur.

Fatigue

Fatigue of steel structures is damage caused by repeated fluctuations of stress leading to gradual cracking of a structural element. Most steel structures are not subjected to sufficiently great or sufficiently many fluctuations in load (and thus stress) that fatigue is a consideration in design. However, road and rail bridges, cranes and crane supporting structures, other mechanical equipment and machinery supporting structures are examples of steel structures that may be subject to fatigue.

In design for fatigue it is normal to design the structure for all of the other requirements (static strength, serviceability) first and then to assess the structure for fatigue. Fatigue design is normally undertaken with the actual (or estimated) loads that will apply to the structure rather than factored loads that are used in strength aspects of design. It is important in design for fatigue to ensure that all parts of the structure are considered and that structural details and connections are carefully detailed and specified as it is these details that will greatly influence the likelihood of fatigue damage occurring during the life of the structure.

The loads used in design should be the best estimates that can be made of those that will occur in practice and should take into account dynamic effects (for example, impact loads) and loads induced by oscillations of the structure or, for example, suspended loads. Moving or rolling loads may result in more than one cycle of load during their passage over a structure or part of the structure. Care should be taken to ensure that all of the load cycles on each element of the structure are considered in assessing the structure for fatigue.

In general in considering fatigue in steel structures it is not fatigue in the as rolled parent or base metal (that is, the steel from which structural elements are made away from connections or changes in shape or direction) that is important. Of far greater importance are points in the structure incorporating welds or bends or, to a lesser extent, bolts. At these points the general stress field in the elements is concentrated or amplified due to the presence of defects, notches, changes in section, etc or additional stresses are imposed on the steel due to residual stresses caused by the heating and cooling of welding or by the permanent deformation due to bending. These result in fatigue cracks generally initiating at such details.

The fatigue life of structures or structural elements can be considered to consist of two parts:

- the period until the commencement of cracking
- the period of crack growth from initiation until the crack grows catastrophically and the element fails

In design, consideration of these stages is not important as the basis for design is a classification of structural details (mainly weld types and configurations) in groups based on large numbers of tests of such details. For a given detail the number of cycles a structural element will endure before failure depends on the load on the element. The higher the load the lower the number of cycles. However, because there are many factors that influence the fatigue life of a particular element, there is much variability in the actual number of cycles before failure of a group of seemingly similar elements. The life can easily vary by a factor of ten around the mean for the whole group.

It has been found that the fatigue life (number of cycles to failure) for welded structures is best defined in terms of the stress range on the element or detail and the notch severity of the detail. The stress range is defined as the algebraic difference between the two extremes of stress that occur during a cycle of loading on the element or detail (Fig. 44.6).

The yield stress and tensile strength of the steel and the minimum, average or maximum stress in the components at the detail have little or no influence on the fatigue strength for a wide range of structural steel grades. Current design specifications base design for fatigue on full-scale testing of details carried out over many years and design largely consists of identifying an appropriate stress category for the detail under consideration, estimation of the stress range that results from the fluctuating loads and the number of cycles of load that the detail is likely to be subjected to during the intended life of the structure and comparing these with S-N curves provided in the relevant specification.

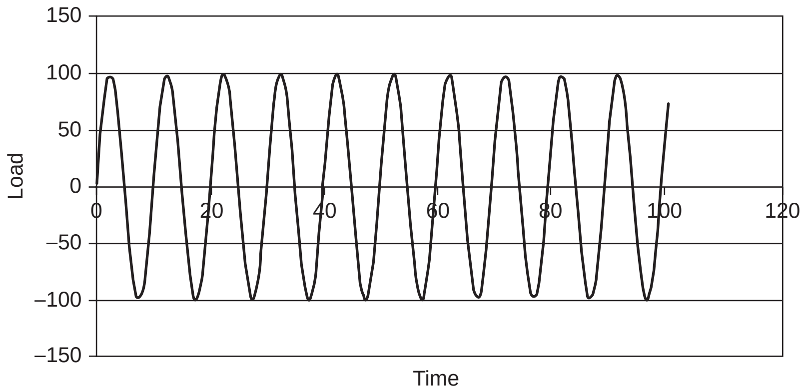

FIGURE 44.6 Uniform amplitude cycles of stress.

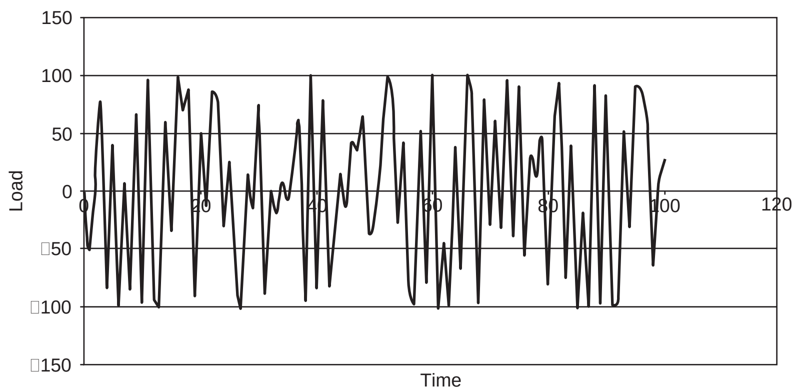

FIGURE 44.7 Varying amplitude stress cycles.

It is not necessary to consider fatigue at a detail if the stresses in the region are always compressive or if the stress range is always below a limiting value known as the threshold fatigue stress range for that detail. However, if any of the stress cycles that the detail is subjected to produces a stress range that is over the threshold value then all stress ranges including those below the threshold value must be considered in estimating the design life.

In practice, structures and structural elements are generally not subjected to uniform repeated applications of load such as those shown in Fig. 44.6. More usually the loads and stresses vary to some degree in the short term and through the life of the structure (Fig. 44.7). The design life is normally based on constant amplitude tests, so it is necessary to have a basis for determining the equivalence between the number of cycles to failure at a given constant stress range in tests and the number of cycles to failure when the stress range varies with each cycle either systematically or randomly.

This is usually done through the use of Miner's rule, which is the sum of the actual number of cycles at each stress range level divided by the number of cycles that are expected to cause failure, for each stress range level considered.

That is:

$$\sum_{i=1}^{m} \frac{n_i}{N_i}$$

where n_i = number of cycles for stress range group i
N_i = permissible number of cycles for stress range group i
m = number of stress range groups

When the stresses in a structural element vary during the life of a structure (as in Fig. 44.7) the "cycles" of stress are not obvious, and procedures have been developed for "identifying" and counting stress cycles using methods such as rainflow counting. Effectively histograms of the stress range are produced with stress ranges accumulated in groups so that the number of cycles that the structure or element will be exposed to at each stress range level during the life of the structure is determined and then used in Miner's rule (Fig. 44.8).

For fatigue design the structure is analysed using elastic analysis. This is appropriate because the peak stresses of all of the cycles must remain well below the yield strength of the steel. If the stresses induced are close to or exceed the yield strength the fatigue life (number of cycles to rupture) will be extremely limited. This case is usually not considered suitable for design purposes and is not covered in design codes.

The classification of details for fatigue is now fairly standard internationally and the relevant US specifications (AISC, AWS, AASHTO and AREMA) all have similar classifications. The S-N curves used for this classification are shown in Fig. 44.9. Comprehensive descriptions of the details are included in the specifications along with drawings of each detail. The parameters used to define each line in the S-N

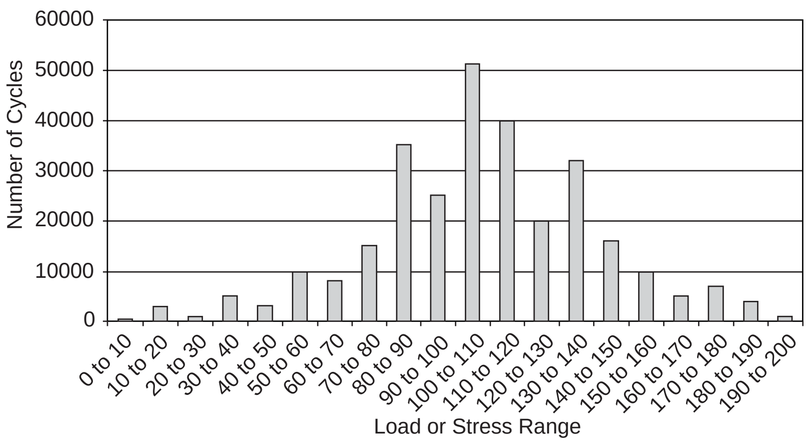

FIGURE 44.8 Histogram of stress cycles.

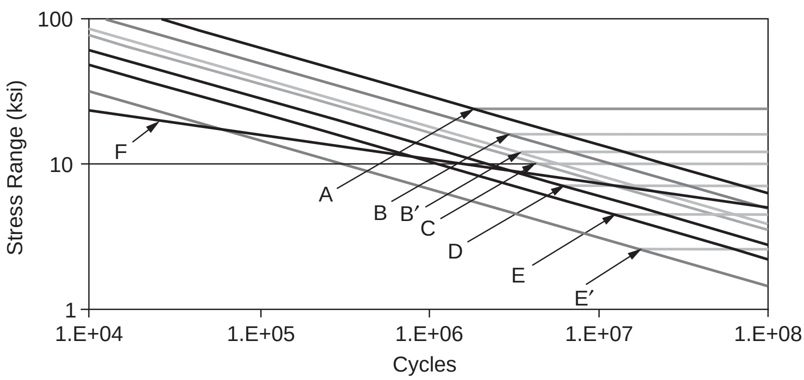

FIGURE 44.9 Design stress range S-N curves used in various USA specifications.

TABLE 44.4 Stress Category Details

Stress Category	C_f	I	F_{TH}
A	250×10^8	0.333	24
B	250×10^8	0.333	16
B′	250×10^8	0.333	12
C	250×10^8	0.333	10
D	250×10^8	0.333	7
E	250×10^8	0.333	4.5
E′	250×10^8	0.333	2.6
F	250×10^8	0.0.167	8

TABLE 44.5 General Description of Stress Category Details

General Description of Detail (See relevant specification for full description and diagram of details)	Stress Categories
Plain material away from welding	A, B, C
Connected material in mechanically fastened joints	B, D, E
Welded joints joining components of built-up members	B, B′, D, E, E′
Longitudinal fillet welded end connections	E, E’
Welded joints transverse to direction of stress	B, B′, C, C′, C″
Base metal at welded transverse member connections	B, C, D, E
Base metal at short attachments	C, D, E, E’
Miscellaneous details	C, E, E′, F

diagram are included in Table 44.4 along with the threshold fatigue stress range. The lines are defined by the equation:

$$F_{SR} = \left(\frac{C_f}{N}\right)^I$$

where F_{SR} = design stress range
C_f = fatigue constant
I = index
N = estimated number of stress range cycles in design life

Table 44.5 provides a very brief general description of each stress category detail.

Fatigue damage in structures occurs due to stresses whether they are primary stresses resulting directly from applied loads or secondary stresses resulting from distortion or relative displacements that are not normally considered in strength and serviceability design. In designing structures for fatigue it is generally best (where possible) to avoid the use of details that have low fatigue lives and to ensure that details that might result in unintended secondary stresses are also avoided.

Performance in Fire

The occurrence of fire in buildings and industrial installations is frightening and sometimes costly in lives, injuries, property damage and other consequences. Often the structures involved are not threatened, with most human losses occurring because of exposure of occupants to smoke and sometimes heat, or both. However, the structure is often incorporated in the fire safety system in the building as a means to separate the occupants from the effects of a fire so that they may leave the building safely and as a means to prevent fire spread and thus minimize property loss resulting from a fire.

Traditionally the requirements for structures have been incorporated in building codes (UBC, SBC, BCC, etc) in terms of the fire resistance rating required to be exhibited by building elements and

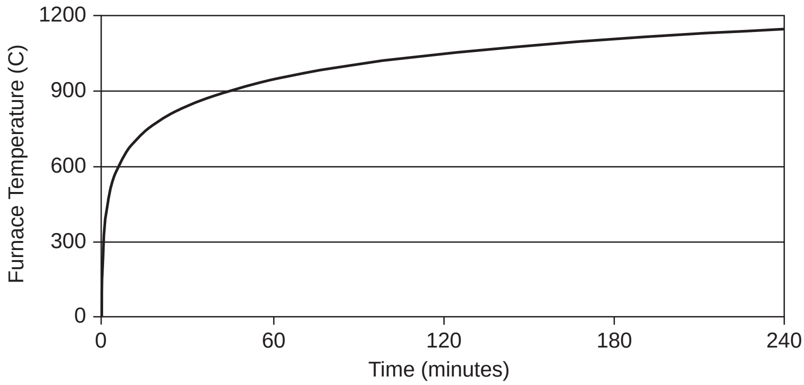

FIGURE 44.10 ASTM E119 furnace temperature — time relationship.

assemblies in standard fire tests such as the ASTM E119 test, which is similar to the ISO 834 test adopted in many countries. The time for which the element or system must survive without failure in the test is nominated as the required fire resistance in the building codes. In these tests the specified furnace temperatures are required to rise approximately as shown in Fig. 44.10.

In these tests the element or system to be tested, whether loaded or not loaded, is exposed to the standard temperature-time regime and the test continues with the element deemed not to have failed until specified failure criteria are reached. These failure criteria include limits on the deflection that may occur for a loaded element or on the temperature reached by the element or at some point on an assembly for unloaded tests. These tests are expensive and time consuming and consequently methods have been developed to allow estimation by calculation of the performance of elements in standard fire tests.

In the last few years "performance-based" design has become more common and estimation of the behaviour of structures in real fires is used in design more often than in the past. This has become more practical with the availability of faster computers that allow the complex calculations to be undertaken in reasonable time.

The performance of steel structures in fire (whether in standard fire tests or in unwanted fires in buildings) depends on:

- the severity of the fire
- the protection applied to the steel
- the loads applied to the steel
- the size and properties of the steel members

Each of these may contribute to variation in the performance of a real fire, but the severity of the fire and in the loads applied to the structure at the time of the fire generally contribute most to this variation, with variability in the steel behavior and properties generally being the least important contributor. Nevertheless an understanding of the behavior of steel under the elevated temperature conditions likely to be experienced during a fire is important in assessing the fire safety in buildings.

Steel when it is heated it expands, and when heated to a high enough temperature begins to lose strength and stiffness. Thus at high temperatures steel structures deform more than they would at normal ambient temperatures and their ultimate strength is lower. If the temperature of the steel increases sufficiently the structure will collapse.

The deformation of the structure that takes place can be thought of as the sum of several components:

- thermal (due to changes in temperature)
- stress related or "elastic" (dependent on temperature and load or stress)
- creep (dependent on temperature, load or stress, and time)

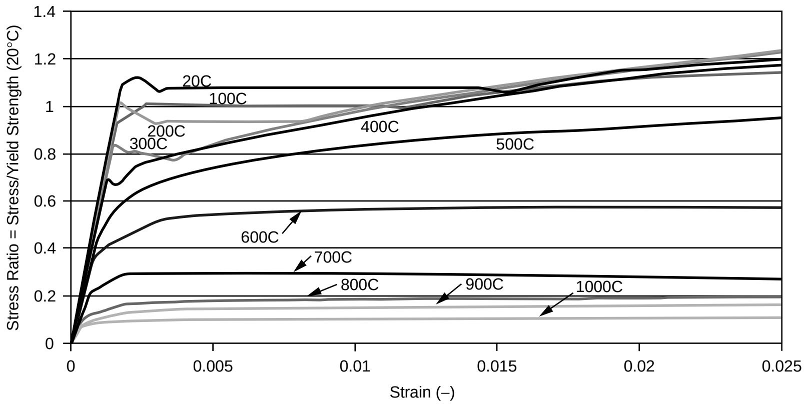

FIGURE 44.11 Effect of temperature on stress-strain relationship structural steel.

Testing of the properties of steel at high temperatures is more complex than at normal ambient temperatures because of the dependence on time of the creep component. This results in a much greater effect of the strain rate on the resulting stress-strain curves. Below about 400°C there is little effect on the stress-strain behavior of structural steels. However, above about 400°C the shape of the stress-strain curves is greatly influenced by the strain rate.

The shape of the stress-strain curve changes with temperature as shown in Fig. 44.11. In this figure the strain rate is fairly high, thus reducing the effect of creep. The stress-strain curves shown in Fig. 44.11 have been "normalized" by dividing the stress by the yield stress at 20°C. The relationships between high temperature properties and normal ambient temperature properties for many grades of structural steel are quite similar and curves of the form shown in this figure can, with reasonable accuracy, be applied to other grades of structural steel.

Each of the tests represented in Fig. 44.11 was conducted at a constant temperature and at a constant strain rate. The curves would be different if the strain rate was changed, particularly the higher temperature curves.

Mathematical relationships of varying complexity have been developed to model the stress-strain-temperature-time relationship of structural steel for use in fire engineering calculations.

However, for many calculations simpler relationships such as those shown in Fig. 44.12 can be used. This is based on the curves in Fig. 44.11 and shows that the yield strength (or 0.2% proof stress at higher temperatures) decreases slowly up to about 500°C, then more rapidly until about 700°C, when about 73% of the ambient temperature yield strength has been lost. It then falls more slowly and falls to about 10% of the ambient value at 1000°C.

Creep and Relaxation

Creep is generally defined as time-dependent deformation that results from sustained loading and results in permanent deformation.

Steel at room temperature is not subject to creep or relaxation at normal ambient temperatures (say <50°C) unless subject to very high stresses. As pointed out above when subjected to high temperatures (say >400°C) steel can creep quite rapidly. At these temperatures and above steel creeps at a rate that is dependent on the applied stress. Creep is not required to be considered for most steel structures as they are not usually exposed to high temperatures.

In structures or structural elements that are subjected to high temperatures for long periods, it may be required to consider creep. This is normally covered in specialist codes and specifications appropriate to the usage.

In many cases this is simply done by specifying limits on long term stresses according to the temperature and the lifetime required. Thus the allowable stresses will be higher for a given grade of steel with lower

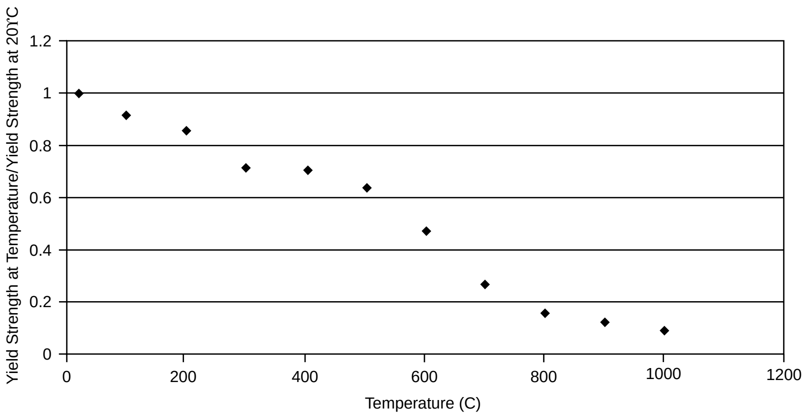

FIGURE 44.12 Effect of temperature on yield strength of structural steel.

temperatures or shorter exposure periods (design life). Allowable stresses will be correspondingly lower for higher temperatures or longer exposure periods.

Corrosion and Corrosion Protection

Many types of steel, including most common grades of structural steel will corrode if exposed to moisture and oxygen. If either or both of these are prevented from contacting the steel it will not corrode under normal circumstances.

Corrosion of steel takes place by a complex electro-chemical reaction between the steel and oxygen that is facilitated by the presence of moisture. In certain circumstances, corrosion can be exacerbated by pollutants or other contaminants in the water. Such materials can include common salt (sodium chloride) and many industrial chemicals. Where such materials are present special precautions should be taken to adequately protect exposed steelwork.

In the absence of such materials structural steel that is contained within the envelope of a building or structure and which is thus not subjected to periodic wetting by rain or other sources of moisture requires no corrosion protection.

Structural steelwork that is not protected in this way requires additional protection and the usual methods are paint systems and galvanizing. In considering a protection system it is necessary to consider the type of building or structure under consideration, its use, its expected life and the relative merits of initial cost against maintenance costs.

Paint systems vary between simple barrier systems that provide a protective film over the steel separating it from oxygen and moisture, and complex systems that include components that provide additional means of protection should, for example, the paint system be damaged by small scratches or holes.

Galvanizing consists of coating the steel with zinc (in some cases with significant quantities of other materials) to provide both barrier protection and cathodic (sacrificial) protection. Sacrificial protection means that the coating preferentially corrodes, leaving the steelwork intact. The zinc coating is metallurgically bonded to the steel providing a tough barrier. In addition, in most commonly occurring circumstances zinc is anodic to steel and thus provides cathodic protection should damage or minor discontinuities occur to the barrier.

An important attribute of all barrier systems is adhesion to the surface of the element to be protected. Thus careful preparation of the surface to be protected is required.

All corrosion systems have a limited life and the system used must be appropriate to the exposure and the lifetime required.

Additional information may be found through the American Institute of Steel Construction, the Society for Protective Coatings or the American Galvanizers Association.

Some grades of structural steel possess high enough levels of corrosion resistance that they can be used in certain applications uncoated. These are usually termed weathering grade steels and are most commonly used in highway bridges, but are also used in buildings and other structures. Although weathering grade steels are more expensive than equivalent normal grades of steel they can be cost effective through the elimination of painting. Successful use of weathering grade steels requires appropriate locations and careful detailing of the structure. Guidelines have been published by the Federal Highway Authority which provide advice on these matters and on appropriate maintenance practices.

Non-destructive Testing

Steel structures, particularly those that are at risk of fatigue damage or brittle fracture, may be required to be inspected using non-destructive testing methods such as dye-penetrant, magnetic particle, ultrasonic and radiographic examination.

The most common form of examination is visual inspection and for many steel structures this is sufficient to ensure a satisfactory level of workmanship in fabrication and erection. This form of inspection can only be used to detect defects in the fit-up of components, in the shape or form of welds and lack of fusion or cracks that are visible on the finished surface.

Methods that can be used to detect smaller surface or near surface defects include dye-penetrant and magnetic particle inspection.

Detection of subsurface defects is usually done using ultrasonic or radiographic techniques.

The choice of appropriate methods of examination is important as the cost of inspection varies widely depending on the method used. This requires consideration of the importance of the weld as well as consideration of the type and location of defects to be detected as the methods mentioned above have differing capabilities in relation to the size, orientation, depth and shape of defects that they can be used to detect.

Welding inspection is normally carried out to the requirements of AWS D1.1.

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