MANUAL
OF STEEL
CONSTRUCTION

LOAD & RESISTANCE FACTOR DESIGN

Volume 1

Structural Members,
Specifications,
& Codes

Second Edition

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FOREWORD

The American Institute of Steel Construction, founded in 1921, is the non-profit technical specifying and trade organization for the fabricated structural steel industry in the United States. Executive and engineering headquarters of AISC are maintained in Chicago, Illinois.

The Institute is supported by three classes of membership: Active Members totaling 400 companies engaged in the fabrication and erection of structural steel, Associate Members who are allied product manufacturers, and Professional Members who are individuals or firms engaged in the practice of architecture or engineering. Professional members also include architectural and engineering educators. The continuing financial support and active participation of Active Members in the engineering, research, and development activities of the Institute make possible the publishing of this Second Edition of the *Load and Resistance Factor Design Manual of Steel Construction.*

The Institute's objectives are to improve and advance the use of fabricated structural steel through research and engineering studies and to develop the most efficient and economical design of structures. It also conducts programs to improve product quality.

To accomplish these objectives the Institute publishes manuals, textbooks, specifications, and technical booklets. Best known and most widely used are the *Manuals of Steel Construction,* LRFD (Load and Resistance Factor Design) and ASD (Allowable Stress Design), which hold a highly respected position in engineering literature. Outstanding among AISC standards are the *Specifications for Structural Steel Buildings* and the *Code of Standard Practice for Steel Buildings and Bridges.*

The Institute also assists designers, contractors, educators, and others by publishing technical information and timely articles on structural applications through two publications, *Engineering Journal* and *Modern Steel Construction.* In addition, public appreciation of aesthetically designed steel structures is encouraged through its award programs: Prize Bridges, Architectural Awards of Excellence, Steel Bridge Building Competition for Students, and student scholarships.

Due to the expanded nature of the material, the Second Edition of the LRFD Manual has been divided into two complementary volumes. The present Volume I contains the LRFD Specification and Commentary, tables, and other design information for structural members, and a new Part 2: Essentials of LRFD. All of the information on connections is in an accompanying Volume II. Like the LRFD Specification upon which they are based, both volumes of this LRFD Manual apply to buildings, not bridges.

The Committee gratefully acknowledges the contributions of Roger L. Brockenbrough, Louis F. Geschwindner, Jr., and Cynthia J. Zahn to this Manual.

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REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

Part 6 (Volume I) of this LRFD Manual contains the full text of the following:

American Institute of Steel Construction, Inc. (AISC)
Load and Resistance Factor Design Specification for Structural Steel Buildings, December 1, 1993
Specification for Load and Resistance Factor Design of Single-Angle Members, December 1, 1993
Seismic Provisions for Structural Steel Buildings, June 15, 1992
Code of Standard Practice for Steel Buildings and Bridges, June 10, 1992

Research Council on Structural Connections (RCSC)
Load and Resistance Factor Design Specifications for Structural Joints Using ASTM A325 or A490 Bolts, June 8, 1988

Additionally, the following other documents are referenced in Volumes I and II of the LRFD Manual:

American Association of State Highway and Transportation Officials (AASHTO)
AASHTO/AWS D1.5–88

American Concrete Institute (ACI)
ACI 349–90

American Iron and Steel Institute (AISI)
Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members, 1991

American National Standards Institute (ANSI)
ANSI/ASME B1.1–82ANSI/ASME B18.2.2–86
ANSI/ASME B18.1–72ANSI/ASME B18.5–78
ANSI/ASME B18.2.1–81

American Society of Civil Engineers (ASCE)
ASCE 7-88

American Society for Testing and Materials (ASTM)

ASTM A6–91b	ASTM A490–91	ASTM A617–92
ASTM A27–87	ASTM A500–90a	ASTM A618–90a
ASTM A36–91	ASTM A501–89	ASTM A668–85a
ASTM A53–88	ASTM A502–91	ASTM A687–89
ASTM A148–84	ASTM A514–91	ASTM A709–91
ASTM A153–82	ASTM A529–89	ASTM A770–86
ASTM A193–91	ASTM A563–91c	ASTM A852–91
ASTM A194–91	ASTM A570–91	ASTM B695–91
ASTM A208(A239–89)	ASTM A572–91	ASTM C33–90
ASTM A242–91a	ASTM A588–91a	ASTM C330–89
ASTM A307–91	ASTM A606–91a	ASTM E119–88
ASTM A325–91c	ASTM A607–91	ASTM E380–91
ASTM A354–91	ASTM A615–92b	ASTM F436–91
ASTM A449–91a	ASTM A616–92	

American Welding Society (AWS)

AWS A2.4–93
AWS A5.1–91
AWS A5.5–81
AWS A5.17–89
AWS A5.18–79
AWS A5.20–79
AWS A5.23–90
AWS A5.25–91
AWS A5.28–79
AWS A5.29–80
AWS B1.0–77
AWS D1.1–92
AWS D1.4–92

PART 1

DIMENSIONS AND PROPERTIES

OVERVIEW

To facilitate reference to Part 1, the locations of frequently used tables are listed below.

STRUCTURAL STEELS

Availability

Section A3.1 of the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* lists fifteen ASTM specifications for structural steel approved for use in building construction.

Five of these steels are available in hot-rolled structural shapes, plates, and bars. Two steels, ASTM A514 and A852, are available only in plates. Table 1-1 shows five groups of shapes and eleven ranges of thickness of plates and bars available in the various minimum yield stress* and tensile strength levels afforded by the seven steels. For complete information on each steel, reference should be made to the appropriate ASTM specification. A listing of shape sizes included in each of the five groups follows in Table 1-2, corresponding with the groupings given in Table A of ASTM Specification A6.

Seven additional grades of steel, other than those covering hot-rolled shapes, plates, and bars, are listed in Section A3.1a of the LRFD Specification. These steels cover pipe, cold- and hot-formed tubing, and cold- and hot-rolled sheet and strip.

The principal producers of shapes listed in Part 1 of this Manual are shown in Table 1-3. Availability and the principal producers of structural tubing are shown in Tables 1-4 through 1-6. For additional information on availability and classification of structural steel plates and bars, refer to the separate discussion beginning on page 1-133.

Space does not permit inclusion in Table 1-3, or in the listing of shapes and plates in Part 1 of this Manual, of all rolled shapes or plates of greater thickness that are occasionally used in construction. For such products, reference should be made to the various producers' catalogs.

To obtain an economical structure, it is often advantageous to minimize the number of different sections. Cost per square foot can often be reduced by designing this way.

Selection of the Appropriate Structural Steel

Steels with 50 ksi yield stress are now widely used in construction, replacing ASTM A36 steel in many applications. The 50 ksi steels listed in Section A3.1a of the LRFD Specification are ASTM A572 high-strength low-alloy structural steel, ASTM A242 and A588 atmospheric-corrosion-resistant high-strength low-alloy structural steels, and ASTM A529 high-strength carbon-manganese structural steel. Yield stresses above 50 ksi can be obtained from two grades of ASTM A572 steel as well as ASTM A514 and A852 quenched and tempered structural steel plate. These higher-strength steels have certain advantages over 50 ksi steels in certain applications. They may be economical choices where lighter members, resulting from use of higher design strengths, are not penalized because of instability, local buckling, deflection, or other similar reasons. They may be used in tension members, beams in continuous and composite construction where deflections can be minimized, and columns having low slenderness ratios. The reduction of dead load and associated savings in shipping costs can be significant factors. However, higher strength steels are not to be used indiscriminately. Effective use of all steels depends on thorough cost and engineering analysis. Normally, connection material is specified as ASTM A36. The connection tables in this Manual are for A36 steel.

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*As used in the AISC LRFD Specification, "yield stress" denotes either the specified minimum yield point (for those that have a yield point) or specified minimum yield strength (for those steels that do not have a yield point).

With appropriate procedures and precautions, all steels listed in the AISC Specification are suitable for welded fabrication. To provide for weldability of ASTM A529 steel, the specification of a maximum carbon equivalent is recommended.

ASTM A242 and A588 atmospheric-corrosion-resistant, high-strength, low-alloy steels can be used in the bare (uncoated) condition in most atmospheres. Where boldly exposed under such conditions, exposure to the normal atmosphere causes a tightly adherent oxide to form on the surface which protects the steel from further atmospheric corrosion. To achieve the benefits of the enhanced atmospheric corrosion resistance of these bare steels, it is necessary that design, detailing, fabrication, erection, and maintenance practices proper for such steels be observed. Designers should consult with the steel producers on the atmospheric-corrosion-resistant properties and limitations of these steels prior to use in the bare condition. When either A242 or A588 steel is used in the coated condition, the coating life is typically longer than with other steels. Although A242 and A588 steels are more expensive than other high-strength, low-alloy steels, the reduction in maintenance resulting from the use of these steels usually offsets their higher initial cost.

Brittle Fracture Considerations in Structural Design

As the temperature decreases, an increase is generally noted in the yield stress, tensile strength, modulus of elasticity, and fatigue strength of the structural steels. In contrast, the ductility of these steels, as measured by reduction in area or by elongation, and the toughness of these steels, as determined from a Charpy V-notch impact test, decrease with decreasing temperatures. Furthermore, there is a temperature below which a structural steel subjected to tensile stresses may fracture by cleavage,* with little or no plastic deformation, rather than by shear,* which is usually preceded by a considerable amount of plastic deformation or yielding.

Fracture that occurs by cleavage at a nominal tensile stress below the yield stress is commonly referred to as brittle fracture. Generally, a brittle fracture can occur in a structural steel when there is a sufficiently adverse combination of tensile stress, temperature, strain rate, and geometrical discontinuity (notch) present. Other design and fabrication factors may also have an important influence. Because of the interrelation of these effects, the exact combination of stress, temperature, notch, and other conditions that will cause brittle fracture in a given structure cannot be readily calculated. Consequently, designing against brittle fracture often consists mainly of (1) avoiding conditions that tend to cause brittle fracture and (2) selecting a steel appropriate for the application. A discussion of these factors is given in the following sections.

Conditions Causing Brittle Fracture

It has been established that plastic deformation can occur only in the presence of shear stresses. Shear stresses are always present in a uniaxial or biaxial state-of-stress. However, in a triaxial state-of-stress, the maximum shear stress approaches zero as the principal stresses approach a common value, and thus, under equal triaxial tensile stresses, failure occurs by cleavage rather than by shear. Consequently, triaxial tensile stresses tend to cause brittle fracture and should be avoided. A triaxial state-of-stress can result from a uniaxial loading when notches or geometrical discontinuities are present.

*Shear and cleavage are used in the metallurgical sense (macroscopically) to denote different fracture mechanisms.

Increased strain rates tend to increase the possibility of brittle behavior. Thus, structures that are loaded at fast rates are more susceptible to brittle fracture. However, a rapid strain rate or impact load is not a required condition for a brittle fracture.

Cold work and the strain aging that normally follows generally increase the likelihood of brittle fracture. This behavior is usually attributed to the previously mentioned reduction in ductility. The effect of cold work that occurs in cold forming operations can be minimized by selecting a generous forming radius and, thus, limiting the amount of strain. The amount of strain that can be tolerated depends on both the steel and the application.

The use of welding in construction increases the concerns relative to brittle fracture. In the as-welded condition, residual stresses will be present in any weldment. These stresses are considered to be at the yield point of the material. To avoid brittle fracture, it may be required to utilize steels with higher toughness than would be required for bolted construction. Welds may also introduce geometric conditions or discontinuities that are crack-like in nature. These stress risers will additionally increase the requirement for notch toughness in the weldment. Avoidance of the intersection of welds from multiple directions reduces the likelihood of triaxial stresses. Properly sized weld-access holes prohibit the interaction of these various stress fields. As steels being welded become thicker and more highly restrained, welding procedure issues such as preheat, interpass temperature, heat input, and cooling rates become increasingly important. The residual stresses present in a weldment may be reduced by the use of fewer weld passes and peening of intermittent weld layers. In most cases, weld metal notch toughness exceeds that of the base materials. However, for fracture-sensitive applications, notch-tough base and weld metal should be specified.

The residual stresses of welding can be greatly reduced through thermal stress relief. This reduces the driving force that causes brittle fracture, but if the toughness of the material is adversely affected by this thermal treatment, no increase in brittle fracture resistance will be experienced. Therefore, when weldments are to be stress relieved, investigation into the effects on the weld metal, heat-affected zone, and base material should be made.

Selecting a Steel To Avoid Brittle Fracture

The best guide in selecting a steel that is appropriate for a given application is experience with existing and past structures. A36 and Grade 50 (i.e., 50 ksi yield stress) steels have been used successfully in a great number of applications, such as buildings, transmission towers, transportation equipment, and bridges, even at the lowest atmospheric temperatures encountered in the U.S. Therefore, it appears that any of the structural steels, when designed and fabricated in an appropriate manner, could be used for similar applications with little likelihood of brittle fracture. Consequently, brittle fracture is not usually experienced in such structures unless unusual temperature, notch, and stress conditions are present. Nevertheless, it is always desirable to avoid or minimize the previously cited adverse conditions that increase the susceptibility of the steel to brittle fracture.

In applications where notch toughness is considered important, it usually is required that steels must absorb a certain amount of energy, 15 ft-lb or higher (Charpy V-notch test), at a given temperature. The test temperature may be higher than the lowest operating temperature depending on the rate of loading. See Rolfe and Barsom (1986) and Rolfe (1977).

Lamellar Tearing

The information on strength and ductility presented in the previous sections generally pertains to loadings applied in the planar direction (longitudinal or transverse orientation) of the steel plate or shape. It should be noted that elongation and area reduction values may well be significantly lower in the through-thickness direction than in the planar direction. This inherent directionality is of small consequence in many applications, but does become important in the design and fabrication of structures containing massive members with highly restrained welded joints.

With the increasing trend toward heavy welded-plate construction, there has been a broader recognition of the occurrence of lamellar tearing in some highly restrained joints of welded structures, especially those using thick plates and heavy structural shapes. The restraint induced by some joint designs in resisting weld deposit shrinkage can impose tensile strain sufficiently high to cause separation or tearing on planes parallel to the rolled surface of the structural member being joined. The incidence of this phenomenon can be reduced or eliminated through greater understanding by designers, detailers, and fabricators of (1) the inherent directionality of construction forms of steel, (2) the high restraint developed in certain types of connections, and (3) the need to adopt appropriate weld details and welding procedures with proper weld metal for through-thickness connections. Further, steels can be specified to be produced by special practices and/or processes to enhance through-thickness ductility and thus assist in reducing the incidence of lamellar tearing. Steels produced by such practices are available from several producers. However, unless precautions are taken in both design and fabrication, lamellar tearing may still occur in thick plates and heavy shapes of such steels at restrained through-thickness connections. Some guidelines in minimizing potential problems have been developed (AISC, 1973). See also Part 8 in Volume II of this LRFD Manual and ASTM A770, Standard Specification for Through-Thickness Tension Testing of Steel Plates for Special Applications.

Jumbo Shapes and Heavy Welded Built-up Sections

Although Group 4 and 5 W-shapes, commonly referred to as jumbo shapes, generally are contemplated as columns or compression members, their use in non-column applications has been increasing. These heavy shapes have been known to exhibit segregation and a coarse grain structure in the mid-thickness region of the flange and the web. Because these areas may have low toughness, cracking might occur as a result of thermal cutting or welding (Fisher and Pense, 1987). Similar problems may also occur in welded built-up sections. To minimize the potential of brittle failure, the current LRFD Specification includes provisions for material toughness requirements, methods of splicing, and fabrication methods for Group 4 and 5 hot-rolled shapes and welded built-up cross sections with an element of the cross section more than two inches in thickness intended for tension applications.

FIRE-RESISTANT CONSTRUCTION

Fire-resistant steel construction may be defined as structural members and assemblies which can maintain structural stability for the duration of building fire exposure and, in some cases, prevent the spread of fire to adjacent spaces. Fire resistance of a steel member is a function of its mass, its geometry, the load to which it is subjected, its structural support conditions, and the fire to which it is exposed.

Many steel structures have inherent fire resistance through a combination of the above factors and do not require additional insulation from the effects of fire. However, in many situations, building codes specify the use of fire-rated steel assemblies. In this case, ASTM Specification E119, Standard Methods of Fire Tests of Building Construction and Materials, outlines the procedures of fire testing of structural elements.

Structural fire resistance is a major consideration in the design of modern buildings. In general, building codes define the level of fire protection that is required in specific applications and structural fire protection is typically implemented in design through code compliance. In the United States, with a few notable exceptions, the majority of cities and states now enforce one of the following model codes:

- National Building Code, published by the Building Officials and Code Administrators International.
- Standard Building Code, published by the Southern Building Code Congress International.
- Uniform Building Code, published by the International Conference of Building Officials.

Building codes specify fire-resistance requirements as a function of building occupancy, height, area, and whether or not other fire protection systems (e.g., sprinklers) are provided.

Fire-resistance requirements are specified in terms of hourly ratings based upon tests conducted in accordance with ASTM E119. This test method specifies a "standard" fire for evaluating the relative fire-resistance of construction assemblies (i.e., floors, roofs, beams, girders, and columns). Specific end-point criteria for evaluating the ability of assemblies to prevent the spread of fire to adjacent spaces and/or to continue to sustain superimposed loads are included. In effect, ASTM E119 is used to evaluate the length of time that an assembly continues to perform these functions when exposed to the standard fire. Thus, code requirements and fire-resistance ratings are specified in terms of time (i.e., one hour, two hours, etc.). The design of fire-resistant buildings is typically accomplished in a very prescriptive fashion by selecting tested designs that satisfy specific building code requirements. Listings of fire-resistant designs are available from a number of sources including:

- Fire-Resistance Directory, Underwriters Laboratories.
- Fire-Resistance Ratings, American Insurance Services Group.
- Fire-Resistance Design Manual, Gypsum Association.

In general, due to the very prescriptive nature of fire-resistant design, changes in tested assemblies can be difficult to justify to the satisfaction of code officials and listing agencies. In the case of structural steel construction, however, the basic heat transfer and structural principles are well defined. As a result, relatively simple analytical techniques have been developed that enable designers to use a variety of different structural steel shapes in conjunction with tested assemblies. These analytical techniques are specifically recognized by North American building code authorities and are described in a series of booklets published by the American Iron and Steel Institute (AISI):

Designing Fire Protection for Steel Columns (1980)
Designing Fire Protection for Steel Beams (1984)
Designing Fire Protection for Steel Trusses (1981)

Since fire-resistant design is currently based on the use of tested assemblies, an important consideration is the degree to which a test assembly is "representative" of actual building construction. In reality, this consideration poses a number of technical difficulties due to the size of available testing facilities, most of which can only accommodate floor or roof specimens in the range of 15 ft by 18 ft in area. As a result, a test assembly represents a relatively small sample of a typical floor or roof structure. Most floor slabs and roof decks are physically, if not structurally, continuous over beams and girders. Beam and girder spans are often much larger than can be accommodated in available laboratory furnaces. A variety of connection details are used to frame beams, girders, and columns. In short, given the cost of testing, the complexity and variety of modern structural systems, and the size of available test facilities, it is unrealistic to assume that test assemblies accurately model real construction systems during fire exposure.

In recognition of the practical difficulties associated with laboratory scale testing, ASTM E119 includes two specific test conditions, "restrained" and "unrestrained." From a structural engineering standpoint, the choice of these two terms is unfortunate since the "restraint" that is contemplated in fire testing is restraint against the thermal expansion, not structural rotational restraint in the traditional sense. The "restrained" condition applies when the assembly is supported or surrounded by construction which is "capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures." Otherwise, the assembly should be considered free to rotate and expand at the supports and should be considered "unrestrained." Thus, a floor system that is simply supported from a structural standpoint will often be "restrained" from a fire-resistance standpoint. In order to provide guidance on the use of restrained and unrestrained ratings, ASTM E119 includes an explanatory Appendix. It should be emphasized that most common types of steel framing can be considered "restrained" from a fire-resistance standpoint.

The standard fire test also includes other arbitrary assumptions. The specific fire exposure, for example, is based on furnace capabilities with continuous fuel supply and does not model real building fires with exhaustible fuel. Also, the test method assumes that assemblies are fully loaded when a fire occurs. In reality, fires are infrequent, random events and their design requirements should be probability based. Rarely will design structural loads occur simultaneously with fire. In addition, many structural elements are sized for serviceability (i.e., drift, deflection, or vibration) rather than strength, thereby providing an additional reserve strength during a fire. As a result of these and other considerations, more rational engineering design standards for structural fire protection are now being developed (*International Fire Engineering Design for Steel Structures: State-of-the-Art,* International Iron and Steel Institute). Although not yet standardized or recognized in North American building codes, similar design methods have been used in specific cases, based on code variances.

One such method has been developed by AISI for architecturally exposed structural steel elements on the exterior of buildings. In effect, ASTM E119 assumes that structural elements are located within a fire compartment and does not realistically characterize the fire exposure that will be seen by exterior structural elements. *Fire-Safe Structural Steel: A Design Guide* (American Iron and Steel Institute, 1979) defines a step-by-step analytical procedure for determining maximum steel temperatures, based on realistic fire exposures for exterior structural elements.

Occasionally, structural engineers will be called upon to evaluate fire-damaged steel structures. Although it is well known that the prolonged exposure to high temperatures can affect the physical and metallurgical properties of structural steel, in most cases steel members that can be straightened in place will be suitable for continued use (Dill, 1960). Special attention should be given to heat-treated or cold-formed steel elements and high-strength bolts and welds.

Effect of Shop Painting on Spray-Applied Fireproofing

Spray-applied fireproofing has excellent adhesion to unpainted structural steel. Mechanical anchorage devices, bonding agents, or bond tests are not required to meet Underwriters Laboratories, Inc. (UL) guidelines. In fact, moderate rusting enhances the adhesion of the fireproofing material, providing the uncoated steel is free of loose rust and mill scale. Customarily, any loose rust or mill scale as well as any other debris which has accumulated during the construction process is removed by the fireproofing application contractor. In many cases, this may be as simple as blowing it off with compressed air.

This ease of application is not realized when fireproofing is applied over painted steel. In order to meet UL requirements, bond tests in accordance with the ASTM E736 must be performed to determine if the fireproofing material has adequate adherence to the painted surface. Frequently, a bonding agent must be added to the fireproofing material and the bond test repeated to determine if the minimum bond strength can be met. Should the bond testing still not be satisfactory, mechanical anchorage devices are required to be applied to the steel before the fireproofing can be applied. The erected steel must still be cleaned free of any construction debris and scaling or peeling paint before the fireproofing may be applied.

Once it is determined that the bond tests are adequate, UL guidelines require that if fireproofing is spray-applied over painted steel, the steel must be wrapped with steel lath or mechanical anchorage devices must be applied to the steel if the structural shape exceeds the following dimensional criteria:

- For beam applications, the web depth cannot exceed 16 inches and the flange cannot exceed 12 inches.
- For column applications, neither the web depth nor the flange width can exceed 16 inches.

A significant number of structural shapes do not meet these restrictions.

The use of primers under spray-applied fireproofing significantly increases the cost of the steel and the preparation for and the application of the fireproofing material. In an enclosed structure, primer is insignificant in either the short- or long-term protection of the steel. LRFD Specification Section M3.1 states that structural steelwork need not be painted unless required by the contract. For many years, the AISC specifications have not required that steelwork be painted when it will be concealed by interior building finish or will be in contact with concrete. The use of primers under spray-applied fireproofing is strongly discouraged unless there is a compelling reason to paint the steel to protect against corrosion.

It is suggested that the designer refer to the UL Directory *Fire Resistance*—Volume 1, 1993, "Coating Materials," for more specific information on this topic.

EFFECT OF HEAT ON STRUCTURAL STEEL

Short-time elevated-temperature tensile tests on the structural steels permitted by the AISC Specification indicate that the ratios of the elevated-temperature yield and tensile strengths to their respective room-temperature values are reasonably similar in the 300° to 700°F range, except for variations due to strain aging. (The tensile strength ratio may