Chapter 3
Load and Resistance Factor Design

3-1. General

This chapter is intended to give a brief synopsis of LRFD methodology and to provide general guidance on LRFD for HSS. Appendixes B through I provide specific guidance and examples for different types of HSS. HSS designed by the LRFD method shall conform to guidance contained in AISC (1986), except as specified herein, and to the engineer manuals referenced in Appendixes B through I.

3-2. Design Basis

LRFD is a method of proportioning structures such that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The basic safety check in LRFD may be expressed mathematically as

$$\Sigma\gamma_i Q_{ni} \leq \alpha\phi R_n \qquad (2\text{-}1)$$

where

γ_i = load factors that account for variability in loads to which they are assigned

Q_{ni} = nominal (code-specified) load effects

α = reliability factor (see paragraph 3-4)

ϕ = resistance factor that reflects the uncertainty in the resistance for the particular limit state and, in a relative sense, the consequence of attaining the limit state.

R_n = nominal resistance

The expression $\Sigma\gamma_i Q_{ni}$ is the *required strength* and the product $\alpha\phi R_n$ is the *design strength.* Load factors and load combinations for specific structure types are listed in the appropriate appendix.

3-3. Strength Requirements

Strength limit states are related to safety and load-carrying capacity (i.e., the limit states of plastic moment and buckling). Formulas giving the load combinations for determining the required strength for buildings are given in American Society of Civil Engineers (ASCE) (1990) and AISC (1986). Similar load combinations pertaining to specific HSS are specified in Appendixes B through I. Structures shall have design strengths at all sections at least equal to the required strengths calculated for all combinations of factored loads and forces. The required strength of structural components shall be determined by structural analysis using appropriate factored load combinations. Each relevant limit state shall be considered. Elastic analysis is permitted unconditionally by this manual. Plastic analysis is permitted only with the approval of CECW-ED, and is subject to restrictions of paragraph A5.1 of AISC (1986).

3-4. Reliability Factors for HSS

For LRFD of HSS, resistance factors of AISC (1986) are multiplied by a reliability factor α. The reliability factor α shall be 0.9 except for the following structures where α shall be 0.85:

a. For those HSS where inspection and maintenance are difficult because the HSS is normally submerged and removal of the HSS causes disruption of a larger project. Examples of this type of HSS include tainter valves and leaves of vertical lift gates which are normally submerged.

b. For those HSS in brackish water or seawater.

3-5. Serviceability Requirements

Serviceability is a state of acceptable performance in which the function of an HSS, its maintainability, durability, and operability are preserved under service or operating conditions. Serviceability should be maintained for the expected life of the project (typically 50 years for navigation and local flood protection projects and 100 years for other projects). The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Limiting values of structural behavior (maximum deflections, vibrations, etc.) to ensure serviceability shall be chosen with due regard to the intended function of the structure. Serviceability may normally be checked using unfactored loads. The following limit states shall be considered in design for serviceability:

a. Deformation in the structural members and supports due to service loads shall not impair the operability or performance of the HSS.

b. Vibrations of the seals, equipment, or movable supports shall not impair the operability of the HSS.

c. Structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair serviceability or operability of the structure during its design life. Closure provisions shall be made as required to maintain the structure.

3-6. Fatigue and Fracture Control

a. Fatigue requirements. Fatigue design shall be in accordance with the provisions of Appendix K in AISC (1986) or AISC (1989) except as specified herein. The number and frequency of load cycles is a function of the HSS purpose and its environment. Determination of the total number of loading cycles shall consider known load fluctuations such as those due to operating cycles and fluctuations of hydraulic head. For certain HSS, vibration may result in unknown load magnitudes and number of cycles; therefore, a quantitative fatigue analysis is not possible. However, for HSS where vibration may produce significant cycles of stress, the choice of details shall be such to minimize susceptible fatigue damage (i.e., details with high fatigue resistance should be used where possible).

Welding processes induce significant residual stresses, and welded members may include high tensile residual stress in the welded region. Therefore, welded members which include any computed stress variation, whether it is tension or compression, shall be checked for fatigue. Deviation from this conservative assumption requires the approval of CECW-ED.

b. Fracture control requirements. For fracture-critical members (FCM) and/or components, the designer shall enforce controls on fabrication and inspection procedures to minimize initial defects and residual stresses, designate the appropriate temperature zone (see Table 3.1, Note 1), and specify the related minimum Charpy V-notch (CVN) fracture toughness. FCMs shall be defined as "members and their associated connections subjected to tensile stresses whose failure would cause the structure to be inoperable." Fracture critical members shall be identified by the designer (minimum requirements are given in Appendixes B through I). Minimum allowable CVN values shall be as given in Table 3.1. Tests to determine material CVN values shall be performed in accordance with the requirements of the American Association of State Highway and Transportation Officials (AASHTO) (1978). For construction of FCMs, fabricators, welding inspectors, and nondestructive examination personnel shall be certified according to AASHTO (1978). Designers are referred to American Welding Society (AWS) (1990) and AASHTO (1978) for guidance on developing adequate quality control and fabrication procedures that will minimize initial defects.

3-7. Commentary on Paragraph 3-2, Design Basis

Load factors and load combinations for structural steel design are based upon limit states of steel structures. Description of the methodology used in developing load factors and load combinations for buildings and other structures may be found in ASCE (1990), Ellingwood et al. (1982), Galambos et al. (1982), and McCormac (1990) and the commentary of AISC (1986). For HSS, the load and resistance factors are governed by items discussed in paragraph 3-8 (commentary of paragraph 3-4). The magnitude of a particular load factor is primarily a function of the characteristics (predictability and variability) of the load to which it is assigned and the conservatism with which the load is specified. A well known load with little variability or a conservatively specified load usually results in a relatively low load factor. Dead loads and static hydraulic loads are in this category. Transient loads are less known and, hence, they usually have a higher load factor.

3-8. Commentary on Paragraph 3-4, Reliability Factors for HSS

Reliability factors are applied to AISC (1986) resistance factors for HSS design. This is to reflect a higher level of uncertainty (compared to building design) due to more aggressive environments in which HSS are placed. Historically, HSS have been designed using a higher factor of safety than that used for building design to account for the unpredictable nature of various items. The variables which require additional consideration for HSS include: facility of inspection; maintenance and repair or replacement (may require dewatering or submerged work by divers); possibility of corrosion (water may be fresh, polluted, brackish, or saline); economic considerations (loss of benefits due to shutdown of a larger project if replacement becomes necessary); possibility of severe vibrations or repeated stress reversals (hydraulic flow may cause vibrations and operating procedures may cause stress reversals); relative importance (HSS may be critical in the project operation); and design life of the structure in severe environments (50 to 100 years). For these reasons, reliability factors are applied to the resistance factors specified by AISC (1986) to effectively increase the factor of safety.

3-9. Commentary on Paragraph 3-6, Fatigue and Fracture Control

Fatigue damage and brittle fractures in HSS are rare but as structure designs, fabrication, and construction become more complex, the probability of brittle fracture increases. Welded construction, with its emphasis on monolithic structural members, increases the need to add fracture criteria to strength and buckling criteria when designing a structure. Various HSS have failed due to fatigue and brittle fracture. Many of the cracking problems that have occurred in HSS originate from poor weld details or poor fabrication. For control of fatigue and fracture, consideration must be given to the following parameters: (a) stress range, detailing, and the number and frequency of load cycles to control fatigue and (b) geometry, toughness, and stress levels to control fracture.

a. Fatigue requirements.

(1) Fatigue is the process of formation and growth of a crack due to repeated fluctuating loads. The designer cannot control the number and frequency of load cycles since this is a function of the operational requirements of the HSS. However, design options include selection of larger members to control the stress range and choice of details with low stress concentrations which have a high fatigue life.

(2) Significant vibration may occur in certain HSS due to hydraulic flow, imperfect seals, movable supports and operating machinery, and impact of passing ice or debris which may occur during a single operating cycle. For these situations, the magnitude of load and the number of load cycles are unknown. Unless predictions for load magnitude and frequency may be made using probabilistic methods, a quantitative fatigue analysis is not possible. However, the possibility of fatigue damage can be controlled by considering the design options given in the previous paragraph.

(3) AISC (1986, 1989) do not require any fatigue check for members with a calculated repetitive stress variation from zero to compression, since crack propagation will not occur in the absence of tensile stress. However, whether a stress variation is tensile or compressive, paragraph 3-6a does require a fatigue check for welded members. This is due to the possible presence of large residual tensile stresses caused by welding processes. For example, if a residual tensile stress of 25 ksi exists, a calculated stress variation from zero to -10 ksi would actually be a variation from 25 ksi to 15 ksi, which could cause fatigue cracking. Tensile residual stresses for welded members are near the yield stress in most cases. The consideration of residual tensile stress is a conservative assumption for fatigue design. It is not currently a uniform practice in the United States; however, it is common in Europe. The assumption is currently favored by many welding specialists.

b. Fracture control requirements.

(1) Fracture is the sudden growth of a crack which may cause failure of a component. Fracture behavior is governed mainly by nominal stress level, material toughness, and geometry of the existing crack or flaw. The fracture control requirements specified herein are based on imposing material toughness requirements and limiting geometry of initial flaws for FCMs, the most critical structural components. Fracture toughness criteria are supplemented with welding and inspection requirements to form a complete fracture control plan. The toughness is controlled by imposing minimum CVN requirements per Table 3-1 and the geometry of initial flaws is controlled by imposing strict fabrication and inspection requirements. Project specifications should require qualification of fabricators and welding inspectors according to AASHTO (1978), to assure that FCMs and their components are in compliance with the requirements specified in paragraph 3-6.

(2) Table 3-1 values are the same as those required by AASHTO (1978) for steel bridges. The basic requirement used in the development of Table 3-1 was to ensure elastic-plastic behavior (i.e. prevent brittle fracture) under service loading at the minimum operating temperature. CVN tests were carried out under service load rates to determine the minimum CVN requirements to assure elastic-plastic behavior for various service temperatures (AASHTO 1978).

(3) Material toughness is affected by load rate, yield strength, service temperature, component thickness, and type of detail. Each of these effects was considered in the development of Table 3-1, and all but load rate are explicitly accounted for in Table 3-1. The following discussion is included to provide a brief explanation of toughness requirements for the various categories of Table 3-1. A more complete discussion is provided in AASHTO (1978) and Barsom and Rolfe (1987).

(a) Load rate. The effect of load rate was considered in the determination of required test temperatures. A consistent temperature shift exists between CVN values obtained for specimens subject to a given load rate (less than impact load rate) and those obtained for impact

Table 3-1
Fracture Toughness Requirements for Fracture Critical Members

Welded or Mechanically Fastened	Grade σ_{ys} (ksi)	Thickness (in.)	Zone 1 (ft-lb at °F)	Zone 2 (ft-lb at °F)	Zone 3 (ft-lb at °F)
Welded	36	t ≤ 1.5	25 at 70	25 at 40	25 at 10
		1.5 < t ≤ 4.0	25 at 70	25 at 40	25 at -10
Welded	50	t ≤ 1.5	25 at 70	25 at 40	25 at 10
		1.5 < t ≤ 2.0	25 at 70	25 at 40	25 at -10
		2.0 < t ≤ 4.0	30 at 70	30 at 40	30 at -10
Welded	70	t ≤ 1.5	30 at 20	30 at 20	30 at -10
		1.5 < t ≤ 2.5	30 at 20	30 at 20	30 at -30
		2.5 < t ≤ 4.0	35 at 20	35 at 20	35 at -30
Welded	100	t ≤ 2.5	35 at 0	35 at 0	35 at -30
		2.5 < t ≤ 4.0	45 at 0	45 at 0	Not allowed
Mechanically Fastened	36	t ≤ 1.5	25 at 70	25 at 40	25 at 10
		1.5 < t ≤ 4.0	25 at 70	25 at 40	25 at -10
Mechanically Fastened	50	t ≤ 1.5	25 at 70	25 at 40	25 at 10
		1.5 < t ≤ 4.0	25 at 70	25 at 40	25 at -10
Mechanically Fastened	70	t ≤ 1.5	30 at 20	30 at 20	30 at -10
		1.5 < t ≤ 4.0	30 at 20	30 at 20	30 at -30
Mechanically Fastened	100	t ≤ 4.0	35 at 0	35 at 0	35 at -30

NOTE:

1. Zone 1 minimum service temperature is 0°F and above; Zone 2 minimum service temperature is from -1°F to -30°F; and Zone 3 minimum service temperature is from -31° to -60°F.
2. Charpy impact tests are required on each end of each piece tested for Zone 3.

specimens. The CVN value for a specimen tested under a service load rate at service temperature is equivalent to the CVN impact value for a specimen tested at a temperature which is a constant magnitude greater (temperature shift) than the service temperature. For example (see Table 3-1), for welded 36-ksi components of thickness less than 1.5 in. which are subject to bridge service load rates and minimum service temperature, ductile behavior is assured if CVN impact values are at least 25 ft-lb for tests conducted at 70°F higher than the minimum service temperature. The temperature shift is dependent on service load rate. The temperature shift comparing static and impact load rates is maximum and as load rate increases, the temperature shift decreases. Adoption of bridge criteria for HSS is generally conservative since loading rates on bridges are likely higher than those which occur on most HSS.

(b) Yield strength. The more stringent requirements for steels of higher yield strengths are identified by higher CVN requirements and lower test temperatures. The higher CVN requirements for increased yield strengths are due to the fact that the design stress is generally higher which will result in more elastic stored energy. In order to attain the same degree of safety as in the lower yield steels, the CVN requirement is also increased. The reduced test temperatures are based primarily on the fact that the temperature shift between toughness under service load and impact load decreases with increasing yield strength; thus, lower CVN impact test temperatures are specified to reflect the decrease in temperature shift.

(c) Service temperature. The expected service temperature for a structure is a critical factor in determining toughness requirements since most steels exhibit a

transition from ductile to brittle behavior at a certain temperature. As temperature decreases, toughness and ductility decrease. Therefore, for lower minimum service temperatures, CVN specimens must be tested at lower temperatures to ensure that the steel has adequate toughness.

(d) Component thickness. For thick plates under tensile loading, through-thickness stresses at a crack tip are large due to the through-thickness constraint. This results in a triaxial stress state which reduces the apparent ductility of the steel by decreasing the shear stresses. Because yielding is restricted, the constraint ahead of the notch is increased resulting in reduced toughness. In order to assure ductile behavior, the CVN requirements of Table 3-1 are increased for increasing thickness.

(e) Detail. Welded details require more conservative CVN values than mechanically fastened details for certain thicknesses and service temperatures. The heat input due to welding can reduce toughness properties in the heat affected zone (HAZ). The HAZ is the area of unmelted parent material adjacent to the weld, which is sufficiently heated by the welding that its metallurgical properties are affected. This area may be of special importance in thick members since these usually have lower toughness and are subject to greater heat input during welding. Unfortunately, stress concentrations often overlap the HAZ of welds, thus combining the adverse effects of high stress and low toughness.