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MILITARY HANDBOOK

STRUCTURAL ENGINEERING

STEEL STRUCTURES

NO DELIVERABLE DATA REQUIRED BY THIS DOCUMENT

AREA FACR

ABSTRACT

Basic criteria for the design of structural elements and systems fabricated of structural steel or cold-formed light gage steel are presented for use by experienced engineers. Design standards are established for Class A (Bridge), Class B (Building), and Class C (Special) structures. Guidance for the design of special structures includes crane runways, towers, stacks, storage tanks, and bins for storage of bulk solids. Problems of corrosion, abrasion, design of expansion joints, and exposure to extreme temperatures are discussed. Design cautions based on previous experience are presented in an appendix. A discussion of design practices that promote economy in the cost of materials, fabrication, and erection is also included in an appendix.

FOREWORD

This military handbook has been developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command (NAVFACENGCOM); other Government agencies, and the private sector. This handbook uses, to the maximum extent feasible, national professional society, association, and institute standards. Deviations from this criteria, in the planning, engineering, design, and construction of Naval shore facilities, cannot be made without prior approval of NAVFACENGCOM HQ Code 04.

Design cannot remain static anymore than can the functions it serves or the technologies it uses. Accordingly, recommendations for improvements are encouraged and should be furnished to Naval Facilities Engineering Command, Northern Division, Code 04AB, Building 77 Low, U.S. Naval Base, Philadelphia, PA 19112; telephone (215) 897-6090.

THIS HANDBOOK SHALL NOT BE USED AS A REFERENCE DOCUMENT FOR PROCUREMENT OF FACILITIES CONSTRUCTION. IT IS TO BE USED IN THE PURCHASE OF FACILITIES ENGINEERING STUDIES AND DESIGN (FINAL PLANS, SPECIFICATIONS, AND COST ESTIMATES). DO NOT REFERENCE IT IN MILITARY OR FEDERAL SPECIFICATIONS OR OTHER PROCUREMENT DOCUMENTS.

STRUCTURAL ENGINEERING CRITERIA MANUALS

Criteria Manual	Title	PA
DM-2.01	General Requirements	HDQTRS
DM-2.02	Loads	HDQTRS
MIL-HDBK-1002/3	Steel Structures	NORTHDIV
DM-2.04	Concrete Structures	LANTDIV
MIL-HDBK-1002/5	Timber Structures	NORTHDIV
MIL-HDBK-1002/6	Aluminum Structures, Masonry Structures, Composite Structures, Other Structural Materials	NORTHDIV
MIL-HDBK-1002/7	Seismic Site Response Spectra	HDQTRS
MIL-HDBK-1002/8	Blast Resistant Design	NORTHDIV
DM-2.09	Masonry, Structural Design for Buildings (Tri-Service)	ARMY

Note: Design manuals, when revised, will be converted to military handbooks and listed in the military handbook section of NAVFAC P-34.

STEEL STRUCTURES

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Section 1: INTRODUCTION

1.1 Scope. This military handbook prescribes the structural design criteria for structures fabricated of structural steel and related materials. Recommendations in this handbook pertain to structures constructed from hot rolled steel plates and shapes, high strength alloy steels, cold-formed light gage steel components and decking, and prefabricated steel components such as joists, prefabricated steel buildings, steel wire strand and rope, and stainless steel sheet, plate, and shapes. They also pertain to composite concrete and steel beams, girders, and metal decking.

1.2 Cancellations. This handbook cancels and supersedes NAVFAC DM-2.03, May 1980.

1.3 References. A number enclosed with ( ) refers to references at the end of this handbook.

1.4 Abbreviations for Standards Organizations

AASHTO	-	American Association of State Highway and Transportation Officials
AISC	-	American Institute of Steel Construction
AISE	-	Association of Iron and Steel Engineers
AISI	-	American Iron and Steel Institute
API	-	American Petroleum Institute
AREA	-	American Railway Engineering Association
ASCE	-	American Society of Civil Engineers
ASM	-	American Society for Metals
ASME	-	American Society of Mechanical Engineers
ASTM	-	American Society for Testing and Materials
AWWA	-	American Water Works Association
AWS	-	American Welding Society
CISC	-	Canadian Institute of Steel Construction
EIA	-	Electronic Industries Association
FM	-	Factory Mutual
FHWA	-	Federal Highway Administration
IASS	-	International Association for Shell and Spatial Structures
ISO	-	International Organization for Standardization
MBMA	-	Metal Building Manufacturers Association
NAVFACENGCOM	-	Naval Facilities Engineering Command
NBS	-	National Bureau of Standards
SDI	-	Steel Deck Institute
SJI	-	Steel Joint Institute
SSPC	-	Steel Structures Painting Council
SSRC	-	Structural Stability Research Council
UL	-	Underwriters Laboratories
WSTI	-	Welded Steel Tube Institute

Section 2: STANDARD DESIGN CRITERIA - CLASS A STRUCTURES

Class A structures are those to which standard specifications for bridge type structures are applicable. Included are bridges, trestles, viaducts, and their components. The basis for classification as a Class A structure is the type of loading applied. This type of loading consists of groups or trains of wheels moving on the structure with impact effect. In addition the wheels and tires are presumed to be within a size range and range of inflation pressure (generally less than 100 psig) corresponding to those of wheels and tires for passenger car and truck usage. Class A includes structures carrying automobile and truck traffic, railroad traffic, certain types or materials-handling equipment such as forklift truck (other than those having solid tires), and straddle carries. Class A does not include supports for overhead traveling cranes (Class B), mobile cranes or types of heavy-lift cranes generally used for waterfront work (Class C), equipment or other equipment operating on tracks or oversize tires or forklift trucks having solid tire (Class C). In general, consider supports for machinery under Class B, with due consideration for impact and resonant response.

2.1 Steel Highway Bridges. Design in accordance with AASHTO, Standard Specifications for Highway Bridges (1.1). Section 10 of Division 1 gives design and detailing requirements for steel bridges.

2.1.1 Load Factor Design. When load factor design is used, the overall load factor for the combined loads (dead load plus live load) shall be at least the specified amount, but not less than 1.5 for loads without wind load, and not less than 1.25 for loads that include wind.

This provision is required because for structures designed primarily for dead load or for earth load, the load factor of 1.3 specified for these load categories results in an overall load factor for combined design loads that is reduced below a level that can be justified by previous experience or available statistical data.

2.1.2 Composite Design. Criteria for the design of steel beams, girders, and box girders with composite concrete flanges are given in Division 1, Section 10, of reference (1.1).

2.1.3 Orthotropic Steel Plate Decks. Design criteria and guidance are given in AISC, Design Manual for Orthotropic Steel Plate Deck Bridges (2.11).

2.1.4 Fatigue. Design guidance to avoid brittle fracture due to fatigue is given in AISC, Bridge Fatigue Guide Design and Details (2.14) and in AISC Journal, 1st Quarter, 1977, Rolfe, Fracture and Fatigue Control in Steel Structures, (2.55).

2.1.5 Details. Guidance for practical details is given in AISC Journal, Jan. 1969, Lally and Milek, Bridge Construction Details (2.41).

2.2 Steel Railway Bridges. Design in accordance with AREA, Manual for Railway Engineering (Fixed Properties) (6.1). Chapter 15, "Steel Structures" gives design and material requirements for steel bridges.

2.3 Steel Culverts and Drainage Structures. Design in accordance with reference (1.1). Section 12 of Division 1 presents design and detailing requirements for corrugated steel culverts and drainage structures. Section 16 of Division 1 presents design requirements for steel tunnel liner plates. Additional guidance is provided in AISI, Handbook of Steel Drainage and Highway Construction Projects (3.6).

2.4 Other. Unless special considerations exist, design in accordance with reference (1.1). Specifically, the AASHTO Standard may be used for the design of waterfront structures supporting mobile cranes, and cranes moving on tracks. For methodology in distributing concentrated loads on tracks to supporting steel members, refer to Chapter 15 of reference (6.1).

Section 3: STANDARD DESIGN CRITERIA - CLASS B STRUCTURES

Class B structures are those to which standard specifications for building-type structures are applicable. Portions of waterfront structures (piers and wharves) which are designed for uniform live load are included in Class B.

3.1 Structural Steel Buildings. Design in accordance with AISC, Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, with Commentary (2.2), or Load and Resistance Factor Design Specification for Structural Steel Buildings (2.6). Related standards as published in the AISC, Manual of Steel Construction, 8th Edition, and the Load and Resistance Factor Design Manual of Steel Construction, First Edition (2.1), shall also apply.

3.1.1 Additional Design Requirements. Design and detailing shall also conform to the following requirements that are not included in the standard reference above.

3.1.1.1 Magnified Moment in Girders of Unbraced Frames. Design the beams or girders of unbraced rigid frames and their connections using an appropriate "magnified" moment that is consistent with the magnified column moments specified in the above-referenced standard to account for non-linear frame sway deflections. See AISC Journal, 2nd Quarter, 1977, LeMessurier, W., A Practical Method of Second Order Analysis, Part 2, "Unbraced Frames," (2.42), and AISC Journal, April 1971, Yura, J., The Effective Length of Unbraced Columns (2.60), for a more detailed discussion of this requirement.

3.1.1.2 Stability of Frames with Both Rigid and Non-Rigid Beam-to-Column Connections. When a structural frame contains both rigid and non-rigid connections at different beam-to-column joints in a particular story, provide the additional stiffness needed in the rigid jointed frames to adequately brace all the columns in the entire story. Methods for determining required rigid frame stiffness are described in reference (2.42) and reference (2.60), Structural Stability Research Council (SSRC), Fritz Engineering Lab No. 13, Lehigh University, Bethlehem, PA 18105, Guide to Stability Design Criteria for Metal Structures (edited by Johnston, B.), 3rd Ed., 1976 (34.1)

3.1.1.3 Web Stiffening in Beams Over Columns. When beams frame over the tops of columns, provide beam web stiffener plates extending between beam flanges and welded to the beam web in the same plane as the column web or flanges, depending on relative orientation of beam and column. This is to ensure that adequate strength is provided to transfer concentrated loads through this type of connection and that the top of the column and the bottom flange of the beam are laterally braced.

3.1.1.4 Stability of Frames with Semi-Rigid Connections. When semi-rigid (Type 3) connections or Type 2 Construction with Wind Moment Connections are used, increase the effective length of columns to allow for the reduced girder stiffness resulting from these connections for column design.

See AISC, Type 2 Construction with Wind Moment Connections, A Return to Simplicity (2.16); AISC Journal, April 1966 (Errata July, 1966), DeFalco and Marino, Column Stability in Type 2 Construction (2.29); and AISC Journal, 4th Quarter, 1981, Driscoll, Effective Length of Columns with Semi-Rigid Connections (2.32), for design procedures.

3.1.1.5 Minimum Bolting. Provide a minimum of two bolts in all bolted connections, unless a specially designed single pinned joint is required for adequate structural performance.

3.1.1.6 Lateral Support of Slender Purlins and Girts. Provide properly spaced sag rods or other lateral bracing, where required, to resist loads in the weak direction of roof purlins or wall girts with webs inclined to the vertical direction. Sag rods or bracing in pitched roofs must be supported by members and connections with sufficient strength to resist both unsymmetrical and symmetrical combinations of design loads on one or both sides of the pitched roof.

3.1.2 Welded Connections. Design and detailing of welded connections shall conform to reference (2.2) or reference (2.6), and to AWS Structural Welding Code-Steel, D1.1-86 (13.1). See AISC Journal, 4th Quarter, 1980, Blodgett, Detailing to Achieve Practical Welded Fabrication (2.21), for design guidance. See AWS Welding Handbook (13.2) and Lincoln Electric Co., The Procedure Handbook of Arc Welding (28.2) for technical information. See also the discussion of lamellar tearing with restrained welded connections in Appendix A, Paragraph A.3.

3.1.3 Bolted Connections. Design and detailing of connections using high-strength bolts shall conform to reference (2.2) or reference (2.6) and to Specification for Structural Joints Using ASTM A325 or A490 Bolts, approved by Research Council on Riveted and Bolted Structural Joints of Engineering Foundation, Endorsed by American Institute of Steel Construction and by Industrial Fastener Institute (26). See also the following references in AISC Journal: 4th Quarter 1978, Birkemoe and Gilmore, Behavior of Bearing Critical Double Angle Beam Connections (2.20); 1st Quarter 1983, Brockenbrough, R., Considerations in the Design of Bolted Joints for Weathering Steel (2.24); 1st Quarter, 1982, Driscoll and Beedle, Suggestions for Avoiding Beam-to-Column Web Connection Failure (2.33); 2nd Quarter, 1985, Thornton, Prying Action - A General Treatment (2.58); and Fisher and Struik, Design Criteria for Bolted and Riveted Joints, Wiley & Sons, 1974 (18). Additional technical information about standard practice for mechanical connectors is available from Industrial Fastener Institute (26).

3.1.4 Steel to Concrete Connections. For design guidance, see the following references in AISC Journals: 1st Quarter 1981, Hawkins and Roeder Connections Between Steel Frames and Concrete Walls (2.38); 1st Quarter 1980, Hawkins, Mitchell and Roeder, Moment Resisting Connections for Mixed Construction (2.39); 2nd Quarter, 1983, Shipp and Haninger, Design of Headed Anchor Bolts (2.56).

3.1.5 Load and Resistance Factor Design. Design of steel structures is usually based on the working stress method in conformance with the provisions given in (2.2). An acceptable alternate procedure based on the strength design method is given in reference (2.6).

If this method is used, the overall load factor for the combined loads (dead load plus live load plus snow load, etc.) shall be at least the specified amount, but not less than 1.4 for loads without wind load and 1.2 for loads that include wind. Additional design guidance for load and resistance factor design (plastic design) may be found in AISC, Plastic Design of Braced Multistory Steel Frames (2.5) and ASCE, Plastic Design in Steel (7.2).

3.1.6 Design and Detailing Aids. Useful aids for design and detailing of structural steel are found in the following AISC reference publications:

a) Engineering for Steel Construction (2.9)

b) Detailing for Steel Construction (2.7)

c) Torsional Analysis of Steel Members (2.8)

d) AISC Journal, 2nd Quarter, 1982, Johnston, Design of W-Shapes for Combined Bending and Torsion (2.40).

e) Web openings. AISC Journals: Oct. 1971, Bower, Recommended Design Procedures for Beams with Web Openings (2.23); Oct. 1971, Redwood, Simplified Plastic Analysis for Reinforced Web Holes (2.50); and Jan. 1972, Redwood, Tables for Plastic Design of Beams with Rectangular Holes (2.51).

f) Bearing plates. AISC Journal, April 1970, Fling, Design of Steel Bearing Plates (2.37).

g) Domes and space frames. AISC Journals: Oct. 1963, Stevens and Odom, The Steel Framed Dome (2.57); Oct. 1965, Buchert, Buckling of Framed Domes (2.25), Oct. 1968, Buchert, Space Frame Buckling (2.26); ASCE Struct. Div. Journal, Feb. 1965, Wright, Membrane Forces and Buckling in Reticulated Shells (7.7).

h) Single story rigid frames. MBMA, 1981 Lee, Ketter, and Hsu, Design of Single Story Rigid Frames (29.2), provides design aids for uniform and tapered member rigid frames.

3.1.7 Criteria for Tubing. Additional criteria and technical data for design of components fabricated from steel tubing and pipe are given in:

a) AISI, Tentative Criteria for the Structural Application of Steel Tubing and Pipe (3.7).

b) Welded Steel Tube Institute, Manual of Cold Formed Welded Structural Steel Tubing (40.1).

3.1.8 Composite Design. Criteria and technical data for design of steel beams with composite concrete flanges are given in:

a) Reference (2.2).

b) Reference (2.6).

c) Reference (15.1).

d) See also the following references in the AISC Journal: July and Oct. 1970, Fisher, Design of Composite Beams with Formed Metal Deck (2.34); 1st Quarter, 1977, Fisher, Grant and Slutter, Composite Beams with Formed Steel Deck (2.36); and 2nd Quarter, 1983, Lorenz, Some Economic Considerations for Composite Floor Beams (2.44), for additional design guidance.

3.1.9 Floor Vibrations. Floors with large open spaces without partitions are particularly susceptible to objectionable vibrations. These include lobbies, retail stores, restaurants, ballrooms, laboratories, hospital operating rooms, and microelectronic facilities.

Design criteria and guidance for control of floor vibrations in structural steel framing are given in the following references in the AISC Journal: 2nd Quarter 1981, Murray, Acceptability Criterion for Occupant-Induced Floor Vibrations, (2.47); 3rd Quarter, 1975, Murray, Design to Prevent Floor Vibrations, (2.48); 3rd Quarter, 1977, Murray and Hedrick, Floor Vibrations and Cantilevered Construction, (2.49).

Human occupancy acceptability levels for continuous floor vibrations, as induced by machinery and equipment located inside or outside of a building, are given in the following references:

a) International Organization for Standardization (ISO), Standard ISO 2631-1978, Guide for the Evaluation of Human Exposure to Whole-Body Vibration (25.1).

b) Harris, C.M., Handbook of Noise Control, Chapter 18 (21).

c) Harris, C.M. and Crede, C.E., Shock and Vibration Handbook, Chapter 44 (22).

d) Richart, F.E., Foundation Vibrations, Trans. ASCE, 1962, Vol. 127, Part 1, pp. 863-898 (7.5).

Methods for isolating vibrations produced by machinery and equipment are given in references (b) and (c) above and in:

e) Jones, R.S., Noise and Vibration Control in Buildings (27).

The operation of sensitive equipment is affected by environmental vibration sources that may be grouped into three categories: external to the building, internal activities, and service machinery. External sources include ambient vibrations of the ground at the site, such as road and rail traffic, nearby construction activities, and machinery operating in nearby buildings. Internal activities include human motion (foot falls), maintenance and repair, in-plant vehicles, and production machinery and equipment. Service machinery includes the air conditioning system, furnaces, pumps and compressors, elevators and mechanically activated doors, and loading platforms. Limits on permissible environmental vibrations for sensitive equipment can best be obtained from the manufacturers of such equipment. Permissible limits for equipment used in the microelectronics industry are given in the following references from the Proceedings of a 1985 ASCE Symposium on Noise and Vibration Measurements - Predication and Mitigation.

f) Ungar, E. E., and Gordon, C. G., Cost-Effective Design of Practically Vibration-Free High Technology Facilities (7.6).

g) Nelson, J. T., Blazier, W. E., and Saurenman, H. J., Site Selection and Building Design for Minimizing Vibration (7.4).

3.1.10 Ponding Due to Deflection of Roof Structure. Requirements for minimum roof slopes to minimize ponding problems are given in NAVFAC DM-1.05, Roofing and Waterproofing (Proposed) (38.1). Design criteria and guidance for control of ponding on roofs are given in reference (2.2). Additional guidance is given in the following references in the AISC Journal: 1st Quarter, 1973, Burgett, Fast Check for Ponding, (2.27); April 1963, Chinn, Failure of Simply-Supported Flat Roofs by Ponding of Rain, (2.28); July 1966, Marino, Ponding of Two-Way Roof Systems, (2.45).

3.1.11 Snow Drift Loads for Large Multi-Level Roofs. When the upper level of a multi-level roof has a large dimension perpendicular to the line of separation between roof levels, snow drift loads may be significantly larger than the loads given in ANSI A58.1, Minimum Design Loads for Buildings and Other Structures (4.1). In such buildings, use the larger of the snow drift loads given in reference (4.1) or in the Metal Building Manufacturers Association (MBMA) Design Practices Section, Metal Building Systems Manual (29.3).

3.1.12 Mill and Other Industrial Buildings. Additional criteria and technical data for the design of steel mill and other industrial buildings are given in:

a) Association of Iron and Steel Engineers (AISE), Specifications for the Design and Construction of Mill Buildings, AISE Technical Report No. 13 (14.1).

b) MBMA Crane Manual for Metal Building Systems, 1982 (29.1).

c) See also Fisher and Buettner, Light and Heavy Industrial Buildings, 1979, AISC (2.15), and AISC Journal, 3rd Quarter 1981, Fisher, Structural Details in Industrial Buildings (2.35); AISC Journal, 4th Quarter 1977, Bakota, Mill Building Design Procedures (2.19).

3.1.13 Steel Shapes Prior to 1953. Strength and dimensional properties of steel and iron structural shapes in use prior to 1953 are given in AISC, Iron and Steel Beams 1873-1952 (2.10).

3.2 Steel Joists. Design in accordance with SJI, Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders - K-Series, LH-Series, DLH-Series, Joist Girders (32.1).

3.2.1 Joists With Unsymmetrical Members. If any elements of a joist are unsymmetrically arranged, the manufacturer of the joist should demonstrate, both by design calculations and by tests, that the joist is not overstressed as a result of the eccentric transfer of forces through unsymmetrically located members and joints, such as single angle compression webs.

3.2.2 Existing Joist Structures. Information in SJI Technical Digest No. 7: Fifty-Year Steel Joist Digest - A Compilation of Specifications and Load Tables 1928-1978 (32.6), may be useful for determining the design strength of existing joist structures.

3.2.3 Floor Vibrations. Joist floors shall be designed to avoid objectionable vibrations with certain types of human occupancy. This is more likely to be of concern in floors of large open spaces without partitions. See SJI Technical Digest No. 5, Vibration of Steel Joist-Concrete Slab Floors (32.4), Lenzen, Vibration of Steel Joist Concrete Slab Floors, (2.43), and references (2.47) and (2.48) for guidance.

3.2.4 Ponding. Joist roofs shall be designed to avoid excessive ponding of water that overloads the roof. See (38.1) for minimum roof slopes. See SJI Technical Digest No. 3, Structural Design of Steel Joist Roofs to Resist Ponding Loads (32.3).

3.2.5 Snow Drift Loads for Large Multi-Level Roofs. See design load requirements in Paragraph 3.1.11 of this manual.

3.2.6 Uplift. Uplift forces due to wind action sometimes act on steel joist roof structures that frequently have dead loads that are insufficient to counterbalance the wind uplift forces. See SJI Technical Digest No. 6: Structural Design of Steel Joist Roofs to Resist Uplift Loads (32.5).

3.2.7 Bridging. Requirements and considerations for bridging are given in SJI Technical Digest No. 2, Spacing of Bridging for Open Web Steel Joists (32.2).

3.2.8 Welding. Requirements and limitations for welding of connections of and to steel joists are given in SJI Technical Digest No. 8, Welding of Open Web Steel Joists (32.7).

3.3 Light Gage Cold-Formed Steel. Design in accordance with AISI, Specification for the Design of Cold-Formed Steel Structural Members (3.2).

3.3.1 Technical Information. AISI reference (3.2) contains the design specification, a commentary, supplementary information, illustrative examples, and design aids in the form of charts and tables.

3.3.2 Exterior Masonry Curtain Walls with Steel Studs. For brick masonry-steel stud curtain wall systems, design the backup wall light gage steel studs to support the full design wind load within the allowable stresses given in the above reference standard (3.2) and with the maximum deflection of studs acting without considering stiffening from the masonry no greater than the span length (unsupported height) divided by 600. If windows are supported on the curtain wall system, provide sufficient strength and stiffness in the stud adjacent to windows to support the lateral forces transferred from the window wall. Normally, doubled or special, extra-strength studs are required adjacent to windows.

3.4 Steel Decking. Design in accordance with reference (3.2). Also refer to Steel Deck Institute (SDI) Design Manual for Composite Decks, Form Decks and Roof Decks, Publ. 25-85 (31.1), for guidance.

3.4.1 Commercially Available Decking. For technical data on commercially available decking, refer to SDI Inc. File (31.3).

3.4.2 Composite Deck. Design composite concrete and steel deck slabs in accordance with ASCE, Specifications for the Design and Construction of Composite Slabs, with Commentary (7.3). Follow additional recommendations in reference (31.1).

3.4.3 Diaphragm Design. Design criteria for roof deck diaphragms are given in NAVFAC P-355, Seismic Design for Buildings (38.17) and the SDI Diaphragm Design Manual, Publ. DDM01-82 (31.2).

3.4.4 Roof Deck. Design roof deck to meet criteria for roofing and insulation given in ASTM E 936-83, Standard Practice for Roof System Assemblies Employing Steel Deck, Preformed Roof Insulation, and Bituminous Built-up Roofing (10.18). Also follow design recommendations in Factory Mutual System Loss Prevention Data 1-28, Insulated Steel Deck (17.1).

3.5 Prefabricated Steel Buildings. Design in accordance with reference (2.2) or reference (2.6) and reference (3.2). Include any applicable additional requirements given in Paragraphs 3.1 to 3.4 of this manual.

3.5.1 Supplementary Design Guidance. Refer to reference (29.3) for additional design guidance and fabrication and erection practices.

3.5.2 Design Loads. Use the design loadings and their method of application that are specified in NAVFAC DM-2.02, Structural Engineering-Loads (38.3). Use the design load combinations given in (29.3). For low roof areas adjacent to high roofs, use the greater of the snow drift loads given in (4.1) or in reference (29.3). The latter criteria accounts for the size of the upper roof, which has been found to be very significant when the upper roof has a large dimension perpendicular to the roof separation.

Loads may be reduced for temporary or minor structures as specified in Paragraphs 8 and 11 of NAVFAC DM-2.01, Structural Engineering-General Requirements (38.2).

3.5.3 NAVFAC Guide Specifications. Requirements in NFGS-13121, Preengineered Metal Buildings (38.21) take precedence over provisions in the MBMA Design Manual.

3.6 Crane Support Systems. Design information for cranes and crane support systems is given in:

(a) Reference (29.1)

(b) Reference (14.1).

(c) AISC Journal, Jan. 1965, Mueller, Lessons from Crane Runways (2.46); 4th Quarter 1982, Ricker, Tips for Avoiding Crane Runway Problems (2.54). See also Paragraph 3.1.12.

The MBMA Crane Manual is applicable to structures supporting cranes used in a range of service from standby and infrequent service to heavy duty service, while AISE Technical Report 13 is primarily concerned with structures supporting cranes used in severe duty and steel mill service. Criteria are given in these references for lateral loads from changes in acceleration of moving cranes, for impact, for forces on runway stops, and for limits to vertical and lateral deflection of runway beams. These vary with the type and expected use of the cranes. These publications also give references that provide further technical data about cranes and valuable recommendations for good design and detailing practices for crane support structures.

3.6.1 Deflection Limits for Crane Runway Girders. Limit maximum vertical deflection of runway girders for live load without impact on overhead cranes to span length/1000. Limit maximum horizontal deflection to span length/500.

3.6.2 Runway Crane Stops. Design runway crane stops to develop the maximum force applied to it by the crane striking the stop at the velocity specified by the manufacturer. Guidance for very heavy duty service cranes is given in AISE Standard No. 6, Specification for Electric Overhead Traveling Cranes for Steel Mill Service (14.2). The magnitude of the force applied on the stop is dependent on the stroke and the properties of the energy-absorbing device provided on the crane. The magnitude and point of application of this force shall be provided by the crane manufacturer. Design the crane runway for the force developed by the runway stop.

3.7 Stainless Steel.

3.7.1 General Guidelines. AISI, Design Guidelines for the Selection and Use of Stainless Steel (3.3). See also CSI, Spec. Data - Stainless Steel (3.1).

3.7.2 Light-Gage Cold-Formed Stainless Steel. Design in conformance with Specification for the Design of Cold-Formed Stainless Steel Structural Members as contained in AISI Stainless Steel Cold-Formed Structural Design Manual (3.8).

3.7.3 Fasteners. For guidance in selecting fasteners refer to AISI Stainless Steel Fasteners-A Systematic Approach to Their Selection (3.9). Also, for guidance in anchoring stone cladding, refer to AISI, Stainless Steel Stone Anchors (3.10).

3.7.4 Technical Data for Materials Selection. See ASM, Source Book on Stainless Steels (11.1). See also ASM, Source Book on Industrial Alloy and Engineering Data (11.2).

Section 4: STANDARD DESIGN CRITERIA - CLASS C STRUCTURES*

Class C covers special structures not readily classified in either of the above two categories. These include storage tanks, cable guyed structures, floating structures, structures supporting heavy-lift cranes and heavy earth-moving equipment, airport runways, catapults, and aircraft operating adjuncts, and other designed as special structures for which criteria are not specifically provided. Consider special codes or other information available in technical literature and manufacturers' publications in establishing standards for design.

4.1 General. The provisions of the Standard Design Criteria for Class B structures shall apply, except as described in reference (38.2) or elsewhere herein.

4.2 Wire Strand and Rope. Technical information about wire strand, rope, and fittings is available from various wire and fittings manufacturers. Working loads for various types of wire rope, including guys, but not including running ropes such as in cranes or derricks, and not including wire rope used in other types of equipment or machinery, shall be as follows:

a) Prestretched Zinc-Coated Steel Wire Rope and Strand (ASTM A603, Standard Specification for Zinc-coated Steel Structural Wire Rope, Specifications for (10.16), A586-81, Standard Specification for Zinc-coated Parallel and Helical Steel Wire Structural Strand (10.14); and A475, Standard Specification for Zinc-coated Steel Wire Strand (10.11)): For guyed towers, the provisions of EIA, RS-222-C, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures (16.1) (see Towers) shall apply. For other types of structures, consult NAVFACENGCOM. In general the factor of safety, based on breaking strength, shall not be less than 2.0 and shall be increased for cases where occupied areas would be threatened by failure of the rope or strand.

b) Other Types of Wire Rope and Strand and Non-Prestretched Wire Rope and Wire Strand: Consult NAVFACENGCOM.

c) Fasteners: For speltered fasteners, follow recommendations in the ASTM Standard applicable to the type of rope (or strand) being used. For threaded fasteners in guyed towers, consult NAVFACENGCOM regarding the desirability of increasing the factor of safety.

4.3 Towers.

4.3.1 Basic Design Standards. Use reference (2.2) or reference (2.6), reference (13.1), and other related standards given previously for Class B structures, except:

a) Reference (16.1), for loads and other criteria for towers 300 feet or less in height.

b) Consult with NAVFACENGCOM for criteria for each project involving towers over 300 feet in height.

c) Do not use an increased allowable stress for wind effects.

4.3.2 Free Standing Towers.

4.3.2.1 Geometry. Taper free standing towers inward toward the top. For high towers, the tapering can consist of two or more slopes. The upper part of the tower can be uniformly shaped. Use a partial bottom tier only where functionally required (for access, to bring in equipment, or to straddle an obstruction). Otherwise, use diagonals in the bottom tier and connect the bottom struts to the tower legs close to the foundation.

4.3.2.2 Foundations. The foundation for each leg shall have a factor of safety against uplift, overturning, and sliding, including the weight of earth cover, that conforms to the requirements of reference (38.2), Section 7. It shall also have a factor of safety of at least 1.0 against uplift, overturning, and sliding when the weight of the earth cover is neglected.

4.3.3 Guyed Towers.

4.3.3.1 Design Guidance. The IASS Recommendations for Guyed Masts (23.1), developed by IASS, Working Group 4, provides loading criteria, materials information, design guidance and procedures, and fabrication and erection requirements for guyed masts and towers. It also contains an extensive list of reference papers. Although this document reflects European design practice, it contains valuable guidance for all designers of guyed towers.

4.3.3.2 Special Design Requirements.

a) Consider the effect of temperature variations on guy tension and tower design stresses.

b) With no wind and with air temperature at the design value, initial tension in guy cables shall be no more than one-tenth of the cable breaking strength.

c) Working loads for guys shall be not greater than as given in Paragraph 4.2.1.

d) Working loads for insulators and eye bolts of fail-safe insulators shall not exceed 30 percent and 20 percent, respectively, of the manufacturer's guaranteed minimum breaking strength.

e) Design for a condition with any one guy broken, subject to one-quarter of the design load plus the dead load. Under this condition, the allowable stress may be increased by 33 percent.

f) Design towers that must have eccentrically located dead loads so as to minimize tower dead load deflections. The design should consider cambering the tower so that introduction of the eccentric dead load will result in a plumb condition.

4.3.3.3 Guys. Guys shall be prestretched. Follow recommendations in ASTM Standards applicable to the type of rope (or strand) being used.

4.4 Steel Stacks.

4.4.1 Basic Design Standard. Use reference (2.2) or reference (2.6).

4.4.2 Additional Design Criteria.

4.4.2.1 Local Buckling. (Due to axial compression and bending)

a) The allowable compressive strength of unstiffened, cylindrical stacks shall be reduced for local buckling when D/t is greater than $3300/F_y$, but D/t shall not exceed $13{,}000/F_y$.

The reduced compressive strength, F_{yr} is (3.2):

EQUATION:

$$F_r = \frac{660}{D/t} + 0.4\,F_y \qquad (1)$$

where

D = average diameter, inches
t = wall thickness, inches
F_y = yield strength, ksi

See reference (3.2) for further guidance.

b) Longitudinal or circumferential stiffeners may be used to increase the allowable compressive strength of a cylindrical stack shell whose strength is reduced by local buckling. Design procedures are described in references given in Paragraph 4.4.3. Special stiffeners may also be required to facilitate the transfer of forces and support against buckling in portions of the stack adjacent to openings for breeching ducts.

4.4.2.2 Compact Section. Cylindrical stacks may be designed with the increased allowable stresses allowed for compact sections when D/t is less than or equal to $1300/F_y$.

4.4.2.3 Beam Shear. An effective shear area of one-half of the gross cross-sectional area shall be used when calculating beam shear in the cylinder.

4.4.2.4 Deflection. The maximum beam deflection at the top of the stack shall not be more than 1/100th of the stack height.

4.4.2.5 Wind Induced Vibration. Low velocity winds may induce resonant vibrations in light steel stacks. This phenomenon may be more severe when multiple stacks are in series. Criteria for evaluating wind-induced vibrations of stacks and additional references are given in:

a) ASME Report 63-WA-248, Dynamic Response of Tall Stacks to Wind Excitation (9.1).

b) Gaylord, E.H., and Gaylord, C.N., Section 26, "Chimneys", in Structural Engineering Handbook (20).

c) Troitsky, M.S., Tubular Steel Structures, Chapter 5, "Self Supporting Stacks", and Chapter 6, "Multilevel Guyed Stacks" in (28.1).

4.4.2.6 Foundations. Design stack foundations using the uplift resistance criteria given in Paragraph 4.3.2.2 of this manual.

4.4.3 Design Guidance. Design procedures and aids for steel stacks are provided in reference (28.1). Basic procedures for calculating structural behavior of tubular structures are covered in Chapter 1, "Introduction", Chapter 2, "Local and Overall Buckling of Cylindrical Shells", Chapter 3, "Edge Effect at Tubular Structures", and Chapter 4, "Thermal Stresses in Tubular Structures". Design procedures for self-supporting stacks are covered in Chapter 5. Design procedures for multi-level guyed stacks are covered in Chapter 6.

4.4.4 Stainless Steel Stacks. Design criteria for stainless steel structural components, including cylindrical tubular structures, are given in reference (3.8). Information for the selection of stainless steel is given in publications referenced in Paragraph 3.7. See Paragraph 4.4.3 for design guidance.

4.4.5 Steel Chimney Liners. Criteria for the design and construction of steel chimney liners are given in ASCE, Design and Construction of Steel Chimney Liners (7.1).

4.5 Steel Tanks for Liquid and Gas Storage.

4.5.1 Vertical Tanks.

4.5.1.1 Water. Design in accordance with AWWA D100-84, Welded Steel Tanks for Water Storage (12.1), with the following exceptions:

a) Loads. Use design loads given in reference (38.3) in place of loads given in Section 3.1 of D100.

b) Corrosion Allowance. Modify Section 3.9 of reference (12.1) to require that corrosion allowances be added to flanges of beams and channels as well as to their webs.

c) Horizontal Girders Used as Balcony Floors. Provide the following minimum girder widths:

Tank capacity (gallons)	Minimum girder widths (inches)
75,000 or less	24
Over 75,000 to 100,000	27
Over 100,000 to 200,000	30
Over 200,000	36

d) Proportion thickness of hemispherical bottoms of elevated tanks in accordance with Section 4.8 of D100, but not less than the thickness of the lowest shell plate in the cylindrical part of the tank.

e) Painting. Use AWWA D102-78, Painting Steel Water-Storage Tanks (12.3).

f) Inspection and repair. Refer to AWWA D101-53, Standard for Inspecting and Repairing Steel Water Tanks, Standpipes, Reservoirs and Elevated Tanks for Water Storage (12.2).

g) Bolted tanks. Refer to AWWA D103-84, Standard for Factory-Coated Bolted Steel Tanks for Water Storage (12.4).

h) Refer to NAVFAC NFGS-13411, Water Storage Tanks (38.22). Follow Guide Specifications in case of conflict with above standards.

4.5.1.2 Petroleum Fuels. Design in accordance with the requirements in NAVFAC DM-22, Petroleum Fuel Facilities (38.11).

4.5.1.3 Other Liquids, Including Tanks with Gas Pressure. Use API Standard 650, Welded Steel Tanks for Oil Storage (5.2) with appropriate consideration of the compatibility of the selected steel materials with the stored liquid. Use API Standard 620-82, Recommended Rules for Revision of Design and Construction of Large, Welded, Low Pressure Storage Tanks (5.1), for large storage tanks that are subject to gas pressure. Use appropriate appendixes of this Standard for cryogenic liquid storage tanks.

4.5.1.4 Stainless Steel Tanks. Guidelines and rules for the design of stainless steel tanks at atmospheric pressure are given in Part IV of the AISI publication, Steel Tanks for Liquid Storage (3.11). Rules for stainless steel tanks for storage at low pressures of liquified hydrocarbon gases, particularly liquified ethane, ethylene, and methane are given in Appendix Q of reference (5.1).

4.5.1.5 Design Guidance.

a) Use all-welded, cylindrical construction, unless special conditions require other types of construction.

b) High strength and alloy steels permitted in reference standards for water storage tanks, petroleum fuels, storage tanks, and other types of tanks may be used for the respective types of tanks.

c) If two or more steels of different strengths are used in the same tank, plates must be permanently marked and also differentiated by a different thickness or by a different plate width. These differences must be shown on the shop drawings. These requirements are needed to minimize the potential for mislocating plates during tank assembly.

d) Take into account principal stresses, combining primary ring stresses and vertical compression, with secondary bending due to restraints offered by top and bottom plates for design of tank shells.

e) For local buckling resistance of riser pipe, see criteria in Paragraph 4.4.2.1.

f) Design tank foundations using the uplift resistance criteria given in Paragraph 4.3.2.2.

g) Design steel tanks in seismic areas based on criteria given in reference (38.17) and reference (12.1), for seismic design.

h) Provide ring beams for the foundations of all tanks greater than 10,000 bbl, and for tanks of all sizes in seismic Zones 3 and 4. Refer to reference (38.11) for more detail.

4.5.1.6 Design Aids. Reference (3.11) provides guidance on design procedures and materials selection and gives design aids and useful technical information about materials and fabrication and erection practices for large field fabricated vertical steel tanks.

4.5.2 Horizontal Tanks. Design of horizontal tanks is not covered in the previously described standards for vertical tanks since these standards are primarily concerned with design and construction of large field-erected tanks. However, many provisions in these standards relating to design, materials, fabrication, erection, and quality assurance are equally applicable to horizonal tanks. Refer to reference (38.11) for requirements for all petroleum storage tanks.

4.5.2.1 Underground Petroleum Storage Tanks. Design and fabricate in accordance with UL Standard 58, Standard for Steel Underground Tanks for Flammable and Combustible Liquids (36.2). Diameters of tanks without internal bracing are limited to 12 feet, and lengths are limited to six times the tank diameter.

Comply with local environmental requirements for the design of underground tanks that contain petroleum products or other hazardous fluids or that contain any products that have a potential to contaminate the surrounding soil and groundwater, if leaks develop.

4.5.2.2 Small Above-Ground Petroleum Storage Tanks. Tanks up to 12 feet in diameter and 40,000 gallon capacity that are shop fabricated shall be of welded steel construction, designed and fabricated in accordance with UL Standard 142-81, Standard for Steel Aboveground Tanks for Flammable and Combustible Liquids (36.1).

4.5.2.3 Design Guidance. Design procedures and practices for horizontal tanks and pressure vessels are described in Chapter 8, Horizontal Storage Tanks of reference (28.1). Additional technical information is found in Parts III, VI, and IX, Volume 2 in reference (3.12), in the series Steel Plate Engineering Data. Some of the information presented in reference (3.11) is applicable to horizontal tanks.

4.6 Pressure Vessels. Vessels designed for internal or external pressures greater than 15 psi shall be designed and fabricated in accordance with rules and requirements given in ASME, Boiler and Pressure Vessels Code, Section VIII (9.2). Frequently, design rules and guidance given in this code are also useful for horizontal vessels designed for less than 15 psi internal or external pressure. For additional guidance see ASME, Pressure Vessels and Piping: Design and Analysis, Vol. 1, Analysis, and Vol. 2, Components and Structural Dynamics (a compilation of technical papers) (9.3).

4.7 Hyperbaric Facilities. Design in accordance with NAVFAC DM-39, Hyperbaric Facilities (38.13). Consult with NAVFACENGCOM, Code 04B, for guidance.

4.8 Steel Bins for Storage of Bulk Solids.

4.8.1 Basic Design Standards. Use reference (2.2), (2.6), or (13.1), and other related standards given previously for Class B structures. Reference (5.2) should be used for allowable plate stresses, joint efficiency factors, and other relevant design criteria.

4.8.2 Design Guidance. Design criteria for bins and other design aids, including descriptions of typical bins, filling and emptying equipment, steel materials, properties of bulk solids, functional design of bins, loads from bulk solids, and design procedures for roofs, walls, hoppers, and foundations, and example designs are given in Gaylord, E.H., and Gaylord, C.N., Design of Steel Bins for Storage of Bulk Solids (19). Similar technical information, design procedures, and example designs are given in Chapter 7, Bins and Bunkers, of reference (28.1).

4.8.3 Clad Steels for Bins. Clad steel is formed by mill-rolling under pressure a sheet of cladding material and a sheet of base material until they bond integrally over their entire surface. Typically, ASTM A-36-81A Specification for Standard Structural Steel (10.1) or ASTM A283, Standard Specification Low and Intermediate Tensile Strength Carbon Steel Plates, Shapes, and Bars, (10.6) or ASTM A514 High-Yield Strength, Quenched, and Tempered Alloy Steel Plate, Suitable for Welding (10.12), alloy steels are used as base metal, while stainless steel, nickel or nickel alloys, and copper or copper alloys are used as cladding materials.

Typical clad steels are:

ASTM A-263-84, Standard Specification for Corrosion-Resisting Chromium Steel-Clad Plate Sheet and Strip (10.3)

ASTM A-264-84, Standard Specification for Stainless Chromium-Nickel Steel-Clad Plate or Sheet and Strip (10.4)

ASTM A-265-84, Standard Specification for Nickel and Nickel-Base Alloy-Clad Steel Plate (10.5)

Cladding thickness varies from 5 to 10 percent of the total plate thickness, depending on the application. Cladding thickness of 10 to 20 percent of the total plate thickness is most common when clad steels are used. See reference (19) for design information.

Section 5: SPECIAL CONSIDERATIONS

5.1 Expansion Joints.

5.1.1 Class A Structures. Design practice varies widely among the Departments of Transportation of the various states relative to the type and spacing of expansion joints in bridge decks and superstructures. Follow the bridge design practice of the locality where a structure is to be built. In general, the current trend is toward the use of longer lengths between joints, requiring fewer joints with greater potential movement at each joint.

5.1.1.1 Use of Expansion Joints and Provisions for Movement. The FHWA Technical Advisory T5140.13, Integral, No-Joint Structures and Required Provisions for Movement, January 28, 1980 (39.1) recommends the elimination of expansion joints in steel bridges whose lengths are less than 300 feet (for typical environments in the U.S.). They also recommend integral abutments unless the abutments are restrained against lateral movement, together with a suitable approach apron on the pavement tied to the abutment. This reference suggests that provisions for movement follow recommendations in the AASHTO Bridge Specification. These suggest an allowance for movement of 1 1/4 inches per 100 feet of bridge length for structures in cold climates and 1 inch per 100 feet of bridge length in moderate climates. See (39.1) for more details.

5.1.1.2 Design of Expansion Joints. See the following FHWA Technical Advisories for guidance:

a) Expansion Devices for Bridges, FHWA T5140.15, March 26, 1980 (39.2).

b) Bridge Deck Joint Rehabilitation (Retrofit), FHWA T5140.16, March 26, 1980 (39.3).

5.1.2 Class B Structures. Provide expansion joints in accordance with the following general rules:

a) Where structures are more than 300 to 500 feet in length, unless special conditions of climate or exposure exist, and except for structural frames exposed to outdoor environments, such as open parking structures. Follow rules for Class A structures for outdoor structures.

b) At junctures of T-, L-, U-shaped, and other irregularly shaped buildings.

c) Where there is such a change in the foundation soils or type of construction that differential settlements are expected to occur.

d) Guidance for expansion joint design. See Building Research Advisory Board, BRAB Technical Report No. 65, Expansion Joints in Buildings, National Academy of Sciences, 1974 (30.1).

5.2 Corrosion Control.

5.2.1 Allowance for Corrosion Loss (ASTM A-36). For purposes of estimating service life, the following provisions may be used as "first approximation." Where serious corrosion problems are anticipated, the advice of corrosion engineers should be sought. See also Corrosion Handbook (8.1) of the National Association of Corrosion Engineers (NACE).

5.2.1.1 Atmospheric Corrosion. See Figure 1 for typical time-corrosion curves for industrial and marine atmospheres for various types of uncoated steels. The industrial atmosphere is Kearny, New Jersey, and the marine atmosphere is Kure Beach, North Carolina. Use Table 1 to modify values for relative corrosion effects of atmospheres at different locations throughout the world to approximate corrosion loss with uncoated steel.

5.2.1.2 Corrosion in Soils. See Figure 2 for typical corrosion loss for buried steel. These curves do not include allowance for stray current effects.

5.2.1.3 Seawater Corrosion. For continuously submerged conditions, the rate of loss for carbon steel is approximately 0.004 inch per year for each surface exposed. In and above the splash zone, the rate of corrosion loss is greater. A general rate of 0.01 inch per year with random pits of 0.02 inch per year is suggested in AISI, Handbook of Corrosion Protection for Steel Pile Structures in Marine Environment (3.5).

5.2.1.4 Electrolytic Corrosion. Do not use dissimilar materials without separation by proper insulators, or cathodic protection, or both.

5.2.1.5 Corrosion in Tropical Climates. Except where specific values are presented in Table 1, assume that corrosion loss is usually increased in tropical climates (high humidity and temperatures). Unless local experience is available, a corrosion loss of twice the comparable exposure in temperate climates may be assumed.

5.2.2 Paint Coatings.

5.2.2.1 General. Paint coatings are the most frequent means of limiting corrosion of steel structures. Guidance for their selection and design is provided in NAVFAC MO-110, Painting and Protective Coatings (38.20). Additional guidance is given in SSPC, Steel Structures Painting Manual, Volume 2, Systems and Specifications (33.2); in SSPC, Steel Structures Painting Manual Volume 1, Good Painting Practice (33.1); and in AISC, A Guide to Shop Painting of Structural Steel (2.12). Recommendations for shop cleaning and painting and for painting various types of steel structures are found in these publications. Also, comparative cost data and life cycle cost information are presented for various paint systems.

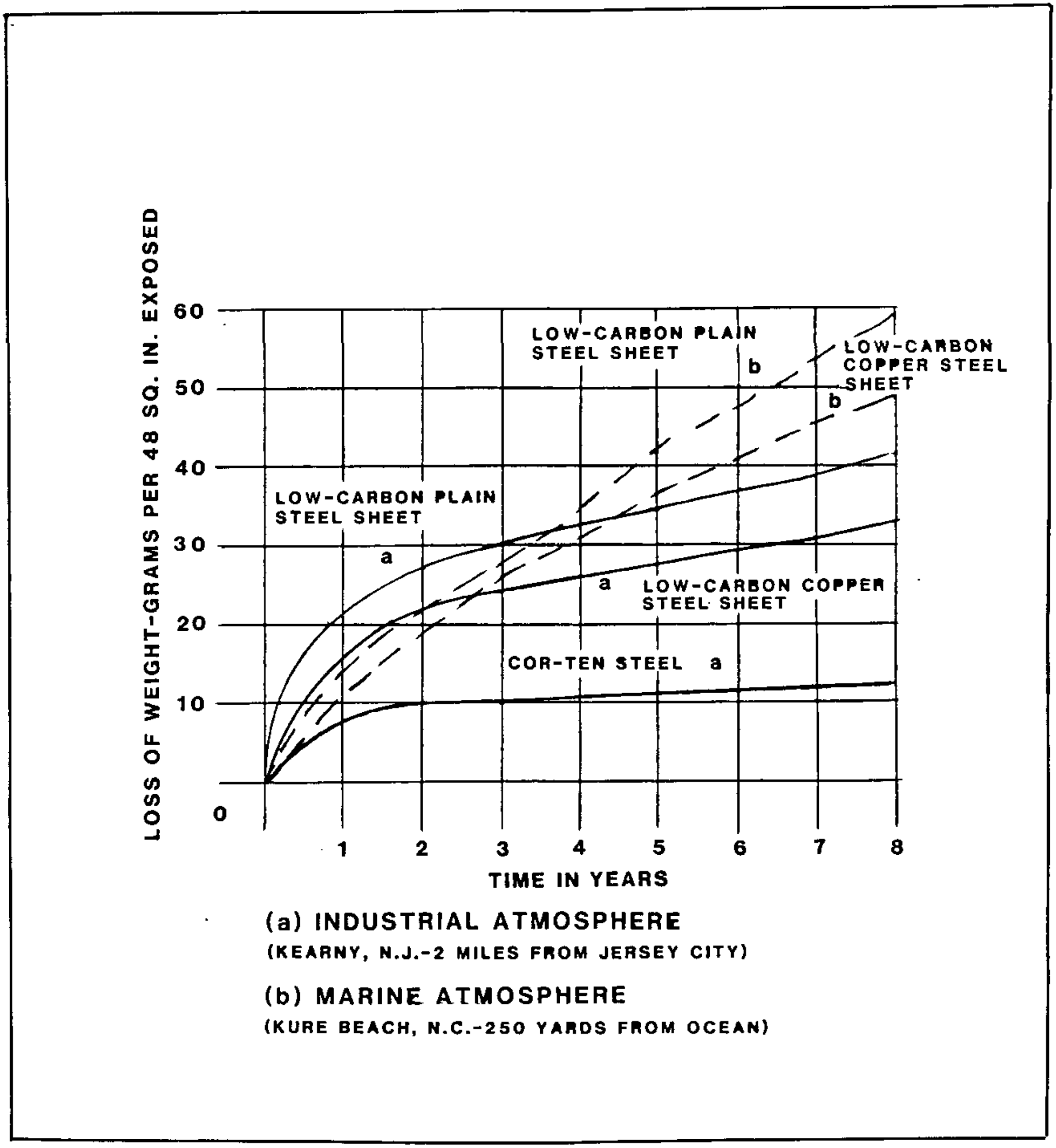
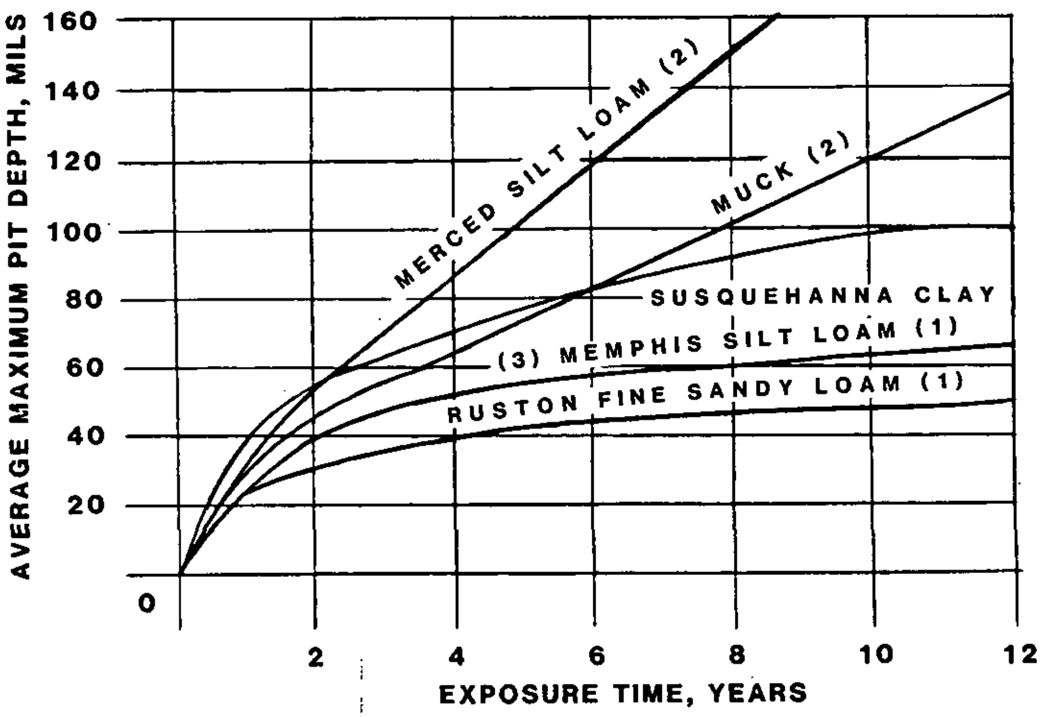


Figure 1
Time-Corrosion Curves for Industrial
and Marine Atmospheres



NOTE:

(1) WELL DRAINED SOILS

(2) POORLY DRAINED SOIL OF LOW RESISTIVITY

(3) MATERIALS USED IN TEST WERE CARBON STEEL, WROUGHT IRON AND CAST IRON

Figure 2
Time-Corrosion Curves in Soils

Table 1
Relative Corrodibility of Atmospheres at
20 Locations Throughout the World

Type of Location	Relative Atmosphere	Corrodibility
Khartoum, Sudan	Dry inland	1
Abisco, North Sweden	Unpolluted	3
Aro, Nigeria	Tropical inland	8
Singapore, Malaysia	Tropical marine	9
Basrah, Iran	Dry inland	9
Apapa, Nigeria	Tropical marine	15
State College, PA	Rural	25
South Bend, IN	Semi-rural	29
Berlin, Germany	Semi-industrial	32
Llanwrtyd Wells, U.K.	Semi-marine	35
Kure Beach, NC	Marine	38
Calshot, U.K.	Marine	41
Sandy Hook, NJ	Marine, semi-industrial	50
Congella, S. Africa	Marine	50
Kearny, NJ	Industrial-marine	52
Motherwell, U.K.	Industrial	55
Vandergrift, PA	Industrial	56
Pittsburgh, PA	Industrial	65
Sheffield, U.K.	Industrial	78
Frodingham, U.K.	Industrial	100

5.2.2.2 Water Tanks. Painting guidance for steel water tanks is given in reference (12.3).

5.2.2.3 Members Embedded in Concrete. Usually, painting is not required for steel members or surfaces that are to be embedded in or in contact with concrete unless chloride ions are present or likely to become present due to seawater, salt spray, or deicing salts. If chloride ion concentration exceeds or is expected to exceed a concentration of 1 pound per cubic yard, coat surfaces with epoxy. It is also not required for members in dry interior environments that are to be covered with membrane or sprayed-on fireproofing. In the latter case, however, it may be desirable to provide a minimum shop coat for protection during construction in typical outdoor environments where relative humidity may frequently exceed 70 percent or to consult the manufacturer or applicator of the sprayed-on fireproofing system.

5.2.2.4 Surface Preparation. Requirements for shop cleaning and preparation of surfaces and the shop paint coat shall be consistent with the painting system selected to meet the anticipated exposure requirements.

5.2.3 Metallic Coatings for Corrosion Control. Guidance on the use of hot dip galvanizing is given in Chapter 21, Hot Dip Galvanizing, in reference (33.1). A list of applicable ASTM standards for galvanizing is contained in this reference. Do not use galvanizing in high temperature environments. It has also been identified with brittle fracture of certain bolts.

5.2.4 Corrosion Control for Piles, Sheet Piles, and Structures in Similar Environments.

5.2.4.1 Marine Environments. Recommendations and criteria for corrosion protection of steel pile and sheet pile structures are given in DM-25.6, General Criteria for Waterfront Construction (38.12). Additional technical information is given in reference (3.5). Some of these recommendations are applicable to many other components of marine structures.

5.2.4.2 Other Environments. Follow recommendations in reference (38.12). Additional technical information about resistance of steel foundation piles is given in NBS Monograph 127, NBS Papers on Underground Corrosion of Steel Pilings - 1962-71 (37.1).

5.2.5 Corrosion Control Using Corrosion-Resistant Steel. These corrosion-resistant steels must conform to reference (10.1), reference (10.16), and A-588 Standard Specification for High Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum-Yield Point to 4 in. (100 mm) Thick (10.15).

5.2.5.1 Moisture Effects. Use only where all exposed surfaces of members are air-dried after contact with moisture, such as from intermittent rain. Do not use where moisture can remain in contact with the steel surface. Details used with exposed members must permit free drainage of moisture. No pockets that entrap water are permitted.

5.2.5.2 Paint. If paint coatings are used, steel is expected to have an increased life.

5.2.5.3 Staining. Do not use in locations where adjacent surfaces may be damaged by staining. Wind-driven moisture may extend the range of staining, depending on the elevation of the exposed steel.

5.2.5.4 Light-Gage Metal. Do not use for light-gage sheet steel architectural metal paneling. It is too difficult to be certain that all locations in panel joints will dry out after contact with atmospheric moisture.

5.2.5.5 Salt Water Exposure. No increase in corrosion protection over that provided by carbon steel is obtained when submerged in seawater. However, the type of steel described in reference (10.16) is sometimes used in marine environments because this steel has approximately two to three times greater resistance to seawater splash zone corrosion than ordinatry ASTM A36 carbon steel (10.1), but only where boldly exposed to the washing action of rain and the drying action of the wind or sun, or both. Follow recommendations in reference (38.12).

5.2.5.6 Buried Structures. Do not use in buried structures.

5.2.6 Corrosion Control Using Stainless Steel. Stainless steels are often used for their very good resistance to corrosion in many environments. See Section III: Corrosion Resistance and Protection in reference (11.1), for information about resistance in specific environments. The following is a limited summary of expected performance in certain common environments.

a) Rural environments without significant chemical pollution: Austenitic types (AISI Series 300) and AISI Types 410 and 430 give prolonged service without significant changes in appearance.

b) Industrial environments: Austenitic types and Type 430 provide long-term service essentially free of rust staining except when significant industrial chlorides are present.

c) Marine environments: AISI Type 316 is the most resistant to attack; Types 301, 302, and 304 may develop some staining, which is often easily removable. Types 410 and 430 will develop thin rust films in a relatively short time.

d) Fresh water: AISI 300 Series and Type 430 are almost completely resistant to corrosion at ambient temperatures, and Type 410 is much better than carbon steel, but is susceptible to attack in some applications.

e) Acid water: Series 300 steels have generally good resistance.

f) Salt water: AISI Types 316 and 317 are superior to other grades and provide the best resistance of the Series 300 steels in a wide variety of conditions. Even they are subject to pitting and crevice corrosion after significant exposure.

g) Soils: AISI Series 300 steels have generally excellent resistance to most soils except those containing chlorides. Types 410 and 430 are subject to attack. See NBS Circular 579, Underground Corrosion (37.2).

h) Other chemicals: See the above ASM reference (11.1).

5.2.7 Design Guidelines for Corrosion Control of Aboveground Structures.

5.2.7.1 Box-Shaped Members. Design box-shaped members so that all inside surfaces may be readily inspected, cleaned, and painted, or close them entirely.

5.2.7.2 Exterior Double Angle Members. In outdoor structures, provide a minimum of 3/8 inch of space between the flanges of two angle members.

5.2.7.3 Drainage. In outdoor structures, provide drain holes in pockets or depressions, or fill with concrete, mastic, or grout. Provide positive

drainage away from exposed steel. Terminate column bases on concrete curbs or piers above grade, and pitch tops of curbs or piers to drain. If tubes or box-shaped components are not sealed and can accumulate interior moisture, they must have drain holes to remove moisture. Freezing of moisture might produce bursting pressures.

5.2.7.4 Sheet Piling Ends. Ends of steel sheet piling may be capped with concrete or have a protective coating applied to eliminate rapid corrosion of exposed ends.

5.3 Wear.

5.3.1 Increase in Metal Thickness. Allow for wear by increasing the metal thickness of those portions of the design section subject to wear, beyond the stress requirements. The amount of such increase depends on the material to be handled and on the desired service life. Estimate wear requirements on the basis of previous experience and from observation of similar conditions at existing installations.

5.3.2 Wear Plates. Consider the use of replaceable wear plates where extremely severe wear conditions occur.

5.4 Climatic Requirements.

5.4.1 Class A Structures in Cold Regions. When these structures are to be exposed to extremely low temperatures, as will occur in Arctic and Antarctic zones and adjacent portions of temperate zones, special design provisions shall be implemented to reduce the probability of brittle fracture. These include the use of steels with improved fracture toughness, detailing to reduce stress raisers, and the control or elimination of welding to reduce stress raising defects and residual stresses from restraint of weld shrinkage. These design provisions are most important for structures subject to impact loading and repeated or cyclic loading (fatigue). Examples of these structure types are highway and railway bridges and crane girders. See reference (1.1), reference (2.14), and reference (2.55) for guidance.

5.4.2 Other Structures in Cold Regions. Consider using steels with improved fracture toughness for major load carrying components of structures in extreme cold environments. These include ASTM A-588 and A-572, Standard Specification for High-Strength Low-Allow Columbium-Vanadium Steels of Structural Quality, (10.13) for rolled sections and plates, and ASTM A-333, Standard Specification for Seamless and Welded Steel Pipe for Low Temperature Service (10.9) for pipe used for cylindrical piling. These and other low carbon steels that are "made to fine grain practice" will have improved toughness at low temperature compared to conventional ASTM A36 steel. When structural components in a low-temperature service are not subject to significant impact loads or fatigue conditions, it is generally more cost effective to specify a type of steel with inherently good fracture toughness, but to avoid a requirement for a specific Charpy impact strength at the reference temperature.

5.4.3 Tropic Zones. There are no adverse effects on the strength of steel members from the increased temperatures representative of tropic zones.

5.5 Elevated Temperatures.

5.5.1 Strength. The yield strength of steel is the strength at an ambient temperature of 80° F. The strength of steel decreases with increased temperature. These decreases are not significant until temperatures exceed 200° F. Strengths at about 1000° F. are generally about 0.6 to 0.7 of room temperature strengths. These relations may differ for heat-treated or other high-strength steels. The strength of such steels at elevated temperatures must be determined for each steel material over the range of service temperature environment.

5.5.2 Typical Strength Properties. Strength properties at elevated temperatures for ASTM A-36 mild carbon structural steel and ASTM A-441, Specifications for High-Strength, Low-Alloy, Structural-Manganese Vanadium Steel (10.10) are given in AISI, Fire-Resistant Steel Frame Construction (3.4).

5.6 Fire Resistance.

5.6.1 General. Steel structures are incombustible, but rapidly lose strength at elevated temperatures. Thus, they must be protected by an incombustible insulative covering in order to achieve a fire resistance rating.

5.6.2 Fire Resistance. Criteria for the fire resistance of steel structural systems are given in MIL-HDBK-1008, Fire Protection for Facilities Engineering, Design, and Construction (38.14). This document incorporates the requirements for fire resistance ratings required in the International Conference of Building Officials (ICBO), Uniform Building Code (24.1), for various types of construction and various occupancy types.

5.6.3 Fire Resistance Ratings. Fire resistance ratings for various assemblies of steel-framed structures, steel joist floors and roofs, light gage metal deck systems, and light-gage metal stud walls and framing systems are given in UL, Fire Resistance Directory with Hourly Ratings for Beams, Columns, Floors, Roofs, Walls, and Partitions (36.3). Ratings are given for various combinations of membrane (gypsum board), spray-on, concrete, and other protective non-structural materials with structural steel systems.

APPENDIX A

DESIGN CAUTIONS

A.1 Buckling Resistance. Steel framing systems involve the use of highly stressed members with relatively thin plate elements such as the flanges and webs of rolled sections. In view of this, their strength in compression involves the consideration of resistance to buckling in the inelastic or elastic stress range. This requires careful design of bracing to provide adequate stability of compression members and compression flanges of beams, as well as provision of sufficient width-to-thickness ratios of local plate elements for adequate resistance to local buckling. Whenever framed systems are a design requirement, local plate elements must have low enough ratios of width to thickness at each plastic hinge location to preclude inelastic buckling prior to sufficient hinge rotation to develop the required frame strength. Also, there must be lateral bracing at and near each plastic hinge location that has adequate strength, stiffness, and spacing to develop the required hinge rotation. See Paragraph 3.1.1 for additional design requirements for frame stability, lateral support, and local buckling resistance to cover common design problems that are not adequately defined in the AISC Specifications.

A.2 Brittle Fracture. The tensile strength of steel framing systems that are subject to repeated load applications or impact from dynamically applied loads requires consideration of resistance to brittle fracture. The dynamic application of load and the repeated application of loads that produce many cycles of loading, or significant stress reversal, may lead to fatigue and lowered tensile strength. These considerations are significant in the design of bridge beams, crane girders, and other structures subject to repeated applications of large stresses or repeated rapidly applied loads that cause impact. A low temperature service environment greatly increases the need to consider the resistance to brittle fracture.

Fracture toughness is increased by careful detailing to avoid local stress concentrations in regions subject to significant tension, as would be developed by notches, sharp changes in cross-sectional area or shape, defects in welds, intermittent welds, tack welds, and similar discontinuities. For applications involving impact, or fatigue where steel members are subject to low temperature service conditions, the designer should also select a steel material with improved fracture toughness. Some of these are described in Paragraphs 5.4.1 and 5.4.2. See reference (2.14) for guidance.

A.3 Lamellar Tearing. Steel plate and rolled shapes can develop fractures from excessive strain perpendicular to the plane of the plate or rolling direction of the shape as a result of impurities in the steel during rolling. These sometimes result from locked-in stresses produced by weld shrinkage at improperly detailed and highly restrained welded joints. See AISC, Engineering for Steel Construction (2.9) and AISC Journal, 3rd Quarter, 1973, Commentary on Highly Restrained Welded Connections (2.17).

A.4 Corrosion Protection. Use special paint or other coating systems that provide improved protection against corrosive attack from exterior moisture, chlorides, and other corrosive environments for steel structures that are exposed to such environments. Common examples include bridges and waterfront structures. Structures exposed to exterior environment where access for painting is expensive (such as towers, tanks, and bins) also should be protected with high quality protective coatings. The selection of coating system(s) should be based upon the following requirements: environment, facility use, aesthetic requirements, and life cycle costs. See Paragraph 5.2.

A.5 Floor Vibrations. Floors in places of public assembly, retail stores, restaurants, ballrooms, laboratories, hospital operating rooms, and other sensitive occupancies should be designed to avoid objectionable motion and vibration. Steel bar joist construction is particularly susceptible to perceptible motion from many different human occupancy loadings. See Paragraphs 3.1.9 and 3.2.3 for design guidance and references.

A.6 Rainwater Ponding on Flat Roofs. Roofs should be designed with adequate slope to drains and stiffness to avoid excessive ponding of water resulting from heavy rain, clogged roof drains, and melting snow. Bar joist roofs that are flat, or with small nominal slope, having long, clear spans are the most susceptible to excessive live loading from ponding on deflected roof structure. Design using high-strength steel and low live loads further exacerbates the problem. See Paragraphs 3.1.10 and 3.2.4 for guidance and references. See reference (38.1) for minimum roof slopes.

A.7 Snow Drifting on Large Multi-Level Roofs. Provisions in existing national and local building codes do not recognize the magnitude of snow drift loads that often develop on low roofs when the size of the adjacent upper roofs is very large, particularly in the direction perpendicular to the line of separation between the low and high roofs. This problem is most critical in low rise structures such as large warehouses, high-tech facilities, schools, and the like. Roof collapses have occurred in roofs designed for the drift loads given in common building regulations. Joists do not have reserve capacity to support large overloads. The Metal Building Manufacturers Association recommends designing for a drift load on the low roof of at least 25 percent of the design snow load on the adjacent upper roof, with the drift extending out on the low roof 4 times the drift height, unless the height of separation is not high enough for this much drift to occur-reference (29.3). See design guidance in Paragraphs 3.1.11, 3.2.5, and 3.5.1.

A.8 Common Design Errors. Some mistakes commonly found in design or detailing that have led to failure of steel structures include:

a) Absence of stiffeners adjacent to beam webs, when beams frame over columns. These stiffeners are particularly needed when large column loads are transmitted through a beam from an upper column to a lower column. They are also usually needed in roof framing systems that utilize "cantilevered beams" where the roof beams of alternate bays cantilever over a column to support shorter beams suspended between points of inflection. See Paragraph 3.1.1.3.

Stiffeners may also be needed (1) to transfer large compressive or tensile forces through a variety of beam column joints with various conditions of moment restraint or (2) to transfer large forces applied by bearing or hangers on flanges into a shear-resisting web.

b) Absence of adequate lateral bracing for the compression flanges of rigid frames and continuous beams in regions where the lower flange is in compression.

c) Inadequate local strength and stiffness at connections transferring tensile hanger forces.

d) Ignoring eccentricity in connections of members with axial load. Examples include truss and joist web members and x-brace systems that transfer wind loads.

e) Use of single angle (or otherwise eccentrically located) compression members in trussed components (joists) and wind bracing. The eccentric transfer of forces at member ends produces lateral deflection and a significant reduction in compressive buckling resistance as a result of the additional lateral moments produced from the deflected shape of the single angle compression strut. Note that a single angle member is by its nature an unsymmetrical shape that cannot be connected to other truss members without introducing eccentricities.

f) Designing the rigidly connected frames (beam-column systems) in an unbraced steel frame system having simple connections (AISC Type 2) at some beam-column joints in the frame without accounting for the reduction in overall frame stability caused by these non-rigid beam column connections. When the "effective length" (k-factor) method given in reference (2.2) is used in such mixed framing systems, the method of determining effective column lengths in the rigid frame portions of the system must be modified to obtain adequate stability of all columns in each story. This is explained in Paragraph 3.1.1.2 and references (2.42) and (2.60).

g) Inadequate strength of girders and girder end connections in unbraced rigid frames, if the magnified moments resulting from frame sway and required for column design are not also used for girder design. See Paragraph 3.1.1.1 and references (2.42) and (2.60).

h) Failure to reduce girder stiffness in unbraced frames designed as AISC Type 2 construction (simple beams) for vertical loads with moment connections designed for wind only, or in unbraced frames designed with AISC Type 3 (semi-rigid) connections. When only partially rigid connections are used in the design of an unbraced frame system that provides the entire stability for a structural framing system, the effective stiffness of girders must be reduced for determining effective column length because of the reduced connection rigidity. See Paragraph 3.1.1.4 and references (2.29) and (2.32).

i) Absence of ties or inadequately connected ties, at the base of one story long span rigid frames that develop significant horizontal thrusts. This problem sometimes results when the foundations are designed by a different organization than the superstructure, as may happen with pre-engineered metal building systems.

j) Inadequate provision for the effects of variations in live and snow load patterns in framing systems that utilize shop welded tapered girders with very high width-to-thickness ratios of flanges and webs. Such flanges and webs will not develop inelastic strains without local buckling. Thus, elastic moments that exceed design moments as a result of unanticipated live load distributions may cause local buckling of the thin flanges or webs prior to redistribution of moment by plastic hinge formation.

k) Inadequate provision for negative moments that produce excessive tensile stresses in floor slab concrete near column lines (causing cracking) when compositely designed beams are shored during concrete placement and concrete is not adequately reinforced to control crack widths produced by dead load of the slab. See Paragraph B.3.3 in Appendix B.

l) Use of excessively flexible light-gage steel back-up wall studs in brick veneer-steel stud back-up curtain walls. Also, failure to increase stud strength (such as adding studs) adjacent to window openings, where windows are supported on the back-up stud wall systems. See design guidance in Paragraph 3.3.2.

m) Use of excessively flexible metal decking for support of roofing, and having insufficient strength and stiffness for support of construction equipment used to apply roofing and gravel. See design guidance and references in Paragraph 3.4.4.

APPENDIX B

DESIGN FOR STRUCTURAL ECONOMY

B.1 General. Recommendations are given in this appendix for design practices that generally result in economical steel structures. These are given for general guidance leading to selection of cost effective steel structural systems. Each specific application and project requires a careful study of alternative design approaches (conducted in the preliminary design phase) to select the most appropriate and cost effective structural system for the conditions of that project.

B.2 Metal Decking for Roofs and Concrete Slab Floors on Steel Beams.

B.2.1 Roofs. Cold-formed steel roof decking is used in most roofs of steel framed buildings. It is usually used without a concrete slab, unless a better fire endurance rating than provided by bare deck systems is required. Most roof decking is 1 1/2-inch or 2-inch deep sections. Deeper sections are sometimes available, and their use may be economical with framing systems that employ rolled steel beams.

B.2.2 Floors. Cold-formed steel decking is nearly universally used as concrete slab forms with structural steel framing systems that have cast-in-place concrete slabs. In office buildings, the deck system may have electrical raceways incorporated to facilitate electrical flexibility with changing office layouts. Thus, both the electrical and structural functions must be considered in selecting the most economical deck system.

B.2.3 Depth, Shape, and Thickness. Floor decks are usually available in depths of 1 1/2 inches, 2 inches, and 3 inches. Corrugated metal floor decks of 1/2 inch or more depth are used with cast-in-place concrete slabs over closely spaced open web joists. See reference (31.3) for information about commercially available depths, flute configurations, and thicknesses.

B.2.4 Concrete Slabs Composite with Steel Deck. Additional economy can be achieved by using a composite type of steel deck to serve both as a form and as the bottom reinforcement in the concrete slab. These decks achieve shear connection by the use of special stampings in the sides of the deck flutes or by re-entrant type deck flutes that become more tightly gripped when the concrete shrinks. When the deck serves as the bottom slab reinforcing, the only other reinforcing in the slab often is a relatively light weld wire mesh. Minimum deck thickness should be 22 gage, and the deck should be galvanized.

Composite metal deck should not be used where the deck is exposed to chloride attack. This may occur from the top through cracks. In view of this, composite decks should not be used in parking structures in zones where highway salts are used, unless the owner is prepared to seal any cracks in the slab promptly and to maintain a protected top surface of the slab. Also, such parking decks should have free drainage and should be washed down at suitable intervals.

B.2.5 Use of Shoring. Composite metal deck is often selected such that deck shoring is not required during concrete placement. If shoring is used, a shallower or lighter deck may be adequate, and the weight and cost of additional concrete required because of deck deflection are reduced. It should be noted, however, that if the deck is shored during concrete placement, the negative moment produced by the weight of the deck (after removal of shoring) will cause significant flexural tension over the beams and may result in increased slab cracking. When the deck is shored, the slab should be reinforced for the negative moment.

B.2.6 Fireproofing. Metal deck and concrete slab assemblies, including composite deck systems, can attain significant fire ratings without additional protection from sprayed-on fireproofing or ceiling membranes. Also, the use of lightweight slab concrete permits a thinner slab and, generally, a more cost-effective structural system. Contact deck manufacturers for ratings of typical deck assemblies - see reference (31.3).

B.2.7 Use with Composite Concrete Slab and Steel Beams. Metal deck, including galvanized deck, may be used in composite slab-beam systems, if account is taken in the composite beam design of the effect of deck profile. The shear connectors usually employed are headed studs, resistance welded through the deck. The type of stud and its application system should be matched to the type of deck being used.

B.2.8 Edges. Screed angles at slab edges and openings often may be either rolled steel or cold-formed steel, connected by puddle welding. It is often more economical if they are cold-formed steel, furnished and installed by the metal deck contractor.

B.2.9 Fastening Method. Fastening methods available are screws and welds (and for joining side laps of adjacent slabs, "button-punching"). For decks that must resist large diaphragm forces, welding may be the only acceptable method, but generally the contractor should be permitted to use his option. See reference (31.1). Also, note that the use of powder-actuated fasteners in diaphragm decks is restricted to structures in Seismic Zones 0 or 1 and to locations with design wind velocity less than 100 miles per hour in Section 5-6 of reference (38.17).

B.2.10 Deck Attachments. Ceiling hanger tabs and insulation clips should be designed and specified by the trade that will use them. Ceiling hanger tabs manufactured by cutting the deck and deforming short strips into integral tabs are suitable only to support very light loads.

B.3 Composite Concrete Slab - Steel Beam Construction.

B.3.1 Design. Composite concrete slab - steel beam framing systems are designed under the AISC Specifications in references (2.2) or (2.6). The depth and type of metal deck profile must be taken into account in the design. The AISC Specifications also permit the use of "partially" composite designs.

B.3.2 Economy. Composite beams are economical compared to non-composite beams for longer spans, wider bay spacings, and heavier loads. A general rule of thumb is that at least 6 pounds of total beam weight should be saved for each stud required for composite action. See A Guide to Economical Practices in Steel Design and Construction published by the Structural Steel Fabricators of New England (35). Partial composite design, 50 to 75 percent of full composite design, is often the most cost-effective design. Except for very heavy members, it is usually not economical to use coverplates on the bottom flange to reduce the rolled section weight with composite design. Also in bridge beams, the use of coverplates may promote brittle cracking at points where they terminate.

B.3.3 Shored Construction. Temporary shoring during concrete placement is usually not required for strength, although it is sometimes used for deflection control. It is sometimes used to allow the economical use of high strength steel and to avoid the need for cambering beams. However, if beams are shored during concrete placement, the dead load will cause large tensile stresses and probable cracking of the concrete at the ends of beams and girders. These stresses can be avoided by designing the composite system without the need for shoring. However, if shoring is required, potential slab cracking can be controlled by adequate slab reinforcing in the negative moment regions.

B.3.4 Camber and Deflection Control. Unshored composite beams deflect more than comparable non-composite beams because of their lighter weight. This may result in excessive deflection and ponding of wet concrete with long span beams. If the surfaces of slabs are held level, it also results in a significant increase in the weight and cost of the slab concrete as a result of the increased slab thickness near midspan. Beams may be cambered when dead load deflection control is needed. This may require a study of trade-offs between the cost of extra concrete, the cost of cambering beams, and the cost of shoring and extra negative reinforcing.

B.3.5 Floor Openings. Significant floor openings in the midspan vicinity of composite beams may reduce or eliminate the benefits of composite construction. In some types of structures, such as some industrial buildings, the possible introduction of future openings should be considered.

B.3.6 Stud Installation. Studs are normally installed in the field. Proper studs and accessories must be specified for each installation. Special ferrules are required for through-the-deck installations.

B.3.7 Partially Restrained End Connections. In braced frames and in some low unbraced frames, the use of partially restrained end connections may offer additional economy and advantages in composite structures with longer spans. The moment resisting end connections reduce deflections during concrete placement, as well as reducing the size of beams required to support construction loads without shoring, and total loads as composite members. Such design requires information about the moment-rotation behavior of the end connections and should be based on reference (2.6).

In zones of high seismicity, partially restrained connections that do not develop the full moment resistance of the steel member connected may not be permitted. Refer to reference (38.17) for requirements before considering partially restrained moment connections in lateral force resisting frames.

B.4 High Strength Steels.

B.4.1 Economy. In large projects, high strength steels such as ASTM A-572 may prove economical, in comparison with ASTM A-36, for heavily loaded columns and for girders and beams of moderate to long span where beam size is governed by strength and not stiffness for deflection or drift control. The availability of such steels may be limited to mill orders and mill extras for quantity, length, and shape and may be factors in the cost comparison.

B.4.2 Corrosion Resistance. Certain high strength steels are known as weathering steels (ASTM A-588 and certain alloys in ASTM A-242). These have improved resistance to weathering in some environments and sometimes may be left uncoated in atmospheres where all parts of the surface can dry out at least intermittently. It should be noted that structures constructed of weathering steel may experience severe crevice corrosion at bolted connections, resulting in premature or unexpected failure of the joints; therefore, close attention must be paid to the design and fabrication of these joints. See Paragraph 5.2.5.

B.4.3 Fracture Toughness. Certain high strength steels, particularly ASTM A-588, A-441, and A-572, have improved fracture toughness, making them more suitable than conventional ASTM A-36 steel for use in low-temperature service and for cyclic and impact loading. Note, however, that resistance to brittle fracture is also a function of detailing and welding practice and stress level. Fracture toughness of ASTM A-36 steel can also be improved by specifying steel that is kilned and made to fine grain practice. See Paragraph 5.4.1 for further guidance.

B.5 Cantilevered Framing.

B.5.1 Description and Use. In cantilever framing systems, roof beams are cantilevered over the tops of columns and extended approximately to the theoretical point in the adjacent spans that will produce equal positive and negative moments in the cantilevered beams. Simple beams are suspended between the cantilevered ends of beams from adjacent spans in alternate bays. Unbalances (patterns) in design roof snow or live loads should be considered when establishing the optimum length of cantilevers. This system produces significant weight savings in steel roof framing systems and permits simple fabrication and fast and safe erection. It is less suited to floor construction because it precludes the use of 2- and 3-tiered columns. Furthermore, economy is reduced with the need to consider full pattern live loads in floor structures. Finally, floor framing is typically designed to be simply supported when composite beams are used, unless unbraced frames are used for lateral force resistance.

B.5.2 Lateral Bracing. When cantilevered framing is used, stiffeners should usually be provided in the beam web over the column flanges to extend sufficient lateral bending stiffness to the bracing plane at roof level. Lateral bracing is sometimes provided at the top of the column (bottom of cantilever beam) by extending the bottom chords of an adjacent joist or other means. Additional lateral bracing of the lower compression flange of the cantilever beam away from the column line may or may not be needed, depending on beam depth and cantilever length.

B.6 Framing Systems for Lateral Load Resistance.

B.6.1 Typical Systems. Framing systems for lateral load resistance are either:

1) Braced frames with diagonals that provide vertical truss action.

2) Unbraced frames with fully rigid or partially rigid moment resisting connections.

3) "Tube" systems for tall buildings that involve the full perimeter structure of the building in lateral force resistance. In a tube system, the exterior wall framing resists the lateral forces using diagonal braces or closely spaced columns with deep spandrels. In a "tube-in-tube" system, a braced interior stair and elevator core structure provides additional lateral resistance.

4) Shear walls (reinforced concrete, reinforced masonry, or stiff steel plate) with pin-jointed frames, or used in concert with rigid jointed frames.

B.6.2 Economy. The choice of the most cost effective framing systems for lateral load resistance depends greatly on functional requirements, the height and base dimensions of the building, and the degree of lateral resistance required. In general, braced frame systems with AISC Type 2 (simple) connections are the most economical for all building heights, but their required diagonal members are sometimes incompatible with functional or aesthetic requirements. Shear wall systems with AISC Type 2 steel frames are theoretically efficient for moderate heights. If the shear wall is reinforced concrete or reinforced masonry, the requirement that different types of construction progress at the same rate may reduce the efficiency of steel erection. This can be mitigated by incorporating steel members in the shear walls that permit erection of the steel ahead of the walls, or by constructing the shear walls ahead of the steel frame erection using slip forming or jump forming techniques. Unbraced rigid jointed frames probably result in the highest cost steel frame of the above types, but allow the greatest functional and aesthetic flexibility to the architectural design. The most economical moment resisting frame for moderate height buildings probably is the system described in Paragraph B.6.4. However, this type of frame may not be permitted in zones of high seismicity. Refer to reference (38.17).

B.6.3 Drift Control and Frame Stability. Drift is the horizontal deflection of a tall building frame resulting from lateral loads - wind or earthquake. The maximum permissible drift is not specified in building codes or national design standards such as those referenced herein. However, reference (38.17) limits story drift produced by seismic design loads to 0.005 times story height. In the other standards, the drift limit is left up to the judgment of the design engineer. This limit should be a function of the probable effects of drift on frame stability, and the potential damage to non-structural components such as cladding, interior partitions and piping, and the stiffening expected from non-structural elements such as permanent interior walls.

Significant drift increases the forces that act on both braced frames and unbraced frames. Traditional limits on drift control may not provide adequate frame stability in some structures, particularly in unbraced frames where some joints between beams and columns are not moment resisting. These columns, termed "leaner" columns, depend on the frames with moment resisting joints for their stability. Current design standards do not provide guidance for design of frames with "leaner" columns, nor for the investigation of the effects of drifts in slender braced frames. Guidance for design or investigation of such systems may be found in references (2.42) and (2.60).

The increased moments produced by the interaction of drift and vertical load, commonly termed the P- effect, also cause increased moments in girders that frame into columns with moment resisting joints. Again, these are not covered in current design standards, but they should be considered in, design of girders in unbraced frames, as described in references (2.42) and (2.60).

Most columns in tall buildings are considered stressed in the inelastic range for design purposes. This results in a higher relative ratio of girder-to-column stiffness than indicated in the current AISC Specification. This increases the stability of columns in an unbraced frame over that determined using design aids in the AISC Specifications. The effect of inelastic behavior of columns can be taken into account using procedures given in Disque, R., Inelastic K-factor for Column Design (2.30).

B.6.4 Frames Designed Based on Type 2 (AISC) Connections for Vertical Loads and Moment Resisting Connections for Wind Load. These are frames designed with moment resisting connections sized only for moments caused by wind load, as described in reference (2.16). Beams are generally sized for effects of vertical loads acting as simply supported beams unless wind moments require larger beams. Many steel framed buildings with moment resisting connections have been designed using this simple assumption. These designs are usually an economical solution, if an unbraced frame is required for buildings of low to intermediate height.

In frames with partially rigid connections, the design unbraced lengths of columns should be increased because of the effects of partial joint restraint. A suggested procedure is given in reference (2.32). Also, if all connections in the framed system are not moment resisting, the "leaner" columns with pin-jointed beams must be braced by the other frames, requiring a further increase in the unbraced lengths of columns in the frames and a special review of frame stability. See references (2.42) and (2.60) for guidance.

Note that these types of partially rigid frames may not be permitted in zones of high seismicity. See reference (38.17).

B.7 Tubular Sections for Columns and Other Members.

B.7.1 Types and Sizes. Rectangular and circular tubes are readily available in sizes from 3 inches up to 12 inches in 36 ksi yield material. Larger rectangular sections are sometimes available up to 16 by 16 inches and 20 by 12 inches, as are tubes manufactured with 46 and 50 ksi yield material. Availability should be checked with local fabricators.

B.7.2 Economy. Tubular sections may show competitive costs with wide flange sections for low-rise building columns, while offering the following advantages: higher capacity for the same weight section due to more efficient shape as compression member, less painting and simpler fireproofing, easier to incorporate in metal stud walls, and more attractive appearance. Tubular sections show the best economy when connection requirements are simple. See White, R., Framing Connections for Square and Rectangular Structural Tubing (2.59). Tubular sections are also efficient for members subject to loads perpendicular to each axis, or loads that produce torsion. They also have been used in various large prefabricated trusses and three-dimensional space frame structures where the jointing system (usually proprietary) is the principal factor determining cost effectiveness of the usage. See Bouwkamp, J., Considerations in the Design of Large-Size Welded Tubular Truss Joints (2.22)

B.8 Beam Web Openings. The depth of a floor system can sometimes be minimized by designing beams with web openings for HVAC ducts. This is generally cost effective only if these openings do not require web reinforcement. The cost of web openings increases significantly where reinforcing is required. See reference (2.23), (2.50) and (2.51) for design guidance.

B.9 Fabrication Considerations. Economy can be promoted without loss of required performance by incorporating design requirements that promote efficient fabrication and erection and eliminate unnecessarily complex details. The following recommendations are a checklist of some commonly accepted means of reducing the cost of structural steel fabrication without a loss in required performance. See reference (35) for further discussion of economical practices in steel construction.

B.9.1 Size of Components. Design large components to be shop fabricated to the maximum extent possible. Components such as trusses are significantly more economical to assemble in the shop than in the field. This requires consideration of the envelope of maximum sizes of components that can be fabricated in the shop and shipped to the field as large assemblies. Typically, components up to 12 to 14 feet in width and 80 to 150 feet in length can be shipped with special procedures and scheduling.

B.9.2 Connections. To the extent practicable, allow the fabricator leeway to select fastener type, size, and details for connections. This enables him to take into account the equipment and other conditions that result in economical connections for fabrication in his shop. The fabricator's designs should be based on reactions and design criteria specified by the design engineer or on standard connections for the beam sizes shown on the design drawings, where permitted by the design engineer. See Reference (2.18) for predesigned bolted framing angle connections. The design engineer should provide complete details on the structural contract drawings for unusual or special connections. Other considerations that can reduce the cost of connections include:

a) Use fillet welds rather than welds requiring edge preparation.

b) Use longer single pass fillet welds (up to 5/16 inch) rather than equivalent shorter welds requiring multiple passes.

c) Use partial penetration welds where adequate, rather than a universal requirement for complete penetration welds.

d) Where intermittent fillet welds are appropriate, space welds so that their center-to-center distance is at least twice their length.

e) Minimize the number of different bolt types, sizes, and gages on a project.

f) Maintain the same bolt type, size, and gage on a steel piece.

g) Permit the use of single angle connections or shear plate connections where appropriate for light loads and where properly designed. See Disque, R. and Young, N., Design Aids for Single Plate Framing Connections (2.31); reference (2.33); Richard, R., Kriegh, J., and Hornby, D., Design of Single Plate Framing Connections with A-307 Bolts (2.52); and Richard, R., Gillet, P., Kreigh, J., and Lewis, B., The Analysis and Design of Single Plate Framing Connections (2.53) for guidance.

h) Use "bearing type" high-strength bolted connections for joints where slip is permissible (these include components not subject to vibrations or to reversing loads) because of the increased bolt capacity relative to "friction type" connections.

B.9.3 Stiffeners. Stiffeners should be designed in accordance with requirements in design standards. The use of oversize stiffeners results in unnecessary welding. Trimming of stiffeners should not be required unless necessary. Snipping to clear fillets in rolled shapes should be permitted. When partial depth beam and column stiffeners are adequate, they should be used instead of full depth fitted stiffeners. In some cases, it is cost effective to select a column with increased weight and increased flange thickness sufficient to eliminate the need for stiffeners in the column. A general rule is that the cost of 250 to 300 pounds of increased column shaft weight is equivalent to the cost of a pair of eliminated stiffeners. See reference (35).

B.9.4 Splices and Coverplates. Columns in multistory buildings are usually fabricated in 2- or 3-tier lengths. Although this usually results in some excess weight in the upper tiers, offsetting benefits are obtained from ease of erection, from eliminating splices, from reducing or eliminating stiffeners with a thicker flange, and from greater standardization of floor framing in details otherwise identical in each floor. A general rule is that the cost of each column splice is equivalent to the cost of about 400 pounds of column weight. See reference (35). Another option, seldom used, is to increase the weight of lower-tier columns by adding coverplates. The fabricator may be given the option to eliminate or modify splice locations shown on the structural drawings and to optimize the use of column coverplates.

B.9.5 Curved Members and Bent Plates. Steel rolled shapes may be curved about either axis, but there are limits to the length of members and radius of curvature that differ with locality. Also, cold bends are typically limited to a curvature that produces no more than 12-percent elongation at extreme fibers.

Plates may be bent to obtain cold-formed shapes, but plate thickness and configuration are limited by available fabrication equipment. Check limits with local fabricators.

B.9.6 Lengthwise Trimming. Trimming a rolled section, such as an angle, lengthwise is costly and may result in warping.

B.9.7 Detail Material. When high-strength steel is used for main members, the fabricator should be given the option of designing details such as connections and stiffeners using ASTM A-36 steel or of using the high-strength steel of the main members. Normally, ASTM A-36 steel is preferred for detail material. See reference (35).

B.9.8 Surface Treatment. Surface preparation and shop painting requirements should reflect the performance requirements for the expected exposure conditions. Do not require painting for steel to be embedded in concrete (except where exposure to chlorides is anticipated). Allow the fabricator's standard shop cleaning and paint coat for steel to be erected in normal exterior atmospheres for structures that will be enclosed without undue delay. Shop paint must be compatible with finish paint, if used, or with sprayed-on fireproofing or membrane fireproofing. Do not specify lead-based shop primers. Require surface preparation by blast cleaning and special paint systems only for severe exterior exposures such as in bridges, parking structures, water tanks, water pipe, and other structures exposed to aggressive environments. See references (2.12) and (33.1). Also follow applicable Navy guide specifications.

B.10 Erection and Field Assembly. Economy is promoted by designing for ease of erection and field assembly.

B.10.1 Shop Assembly vs. Field Assembly. Shop labor costs are generally lower than field labor costs. The availability of equipment for handling and for automated assembly also greatly increases the efficiency of shop assembled components. See Paragraph B.9.1.

B.10.2 Anchor Bolts. Anchor bolt layouts should be kept simple with uniform sizes and spacings throughout the project. The possibility of errors will be reduced when the fewest number of anchor bolt and base plate sizes are used. A careful check of the location and spacing of all anchor bolts should be required prior to the start of steel erection. Hole diameters for anchor bolts usually are specified 1/8- to 1/4-inch oversize for tolerance in setting of anchor bolts.

B.10.3 Leveling Devices. Preset 1/4-inch-thick steel leveling plates under column base plates can facilitate speed and safety in erection of light columns with shop-welded base plates. Leveling plates promote the safe erection of columns compared to the use of shims or wedges that may be knocked out. However, NAVFAC guide specifications require the grouting or drypacking of column base plates after the steel is plumbed and bolted. This requirement precludes the grouting or dry packing of leveling plates.

Larger columns with base plates that are grouted after the steel is plumbed and the bolted connections tightened require the use of leveling nuts and/or steel shims below the base plates. Leveling nuts are practical and cost effective for medium-sized base plates, but require at least 4 anchor bolts per column base, with proper spacing to develop adequate stabilizing moment restraint. They also require supplementary steel shims if the weight of steel and other structures to be erected prior to grouting exceeds the safe compressive capacity of the nuts, threads, or anchor bolts.

Larger column bases over about 36 inches in maximum plan dimension frequently are shipped separately and pre-set. Grout holes are sometimes provided along with special leveling devices.

B.10.4 Bolted Field Connections. Bolted field connections are usually preferred, except for connections that must transfer large direct forces (such as large moment connections or flange splices), where either bolted or welded connections may be more economical, depending on local conditions. Generally, high-strength bolts, see ASTM A-325-85, Standard Specification for High-Strength Bolts for Structural Steel Joints (10.8), should be used, rather than ASTM A-307 bolts, because their higher capacity permits fewer bolts. ASTM A-307-84, Standard Specification for Carbon Steel-Threaded Standard Fasteners, (10.7) may be economical for small structures. When ASTM A-325 bolts are used, bearing type connections are generally more economical because of higher allowable capacity, allowance of paint on faying surfaces, and reduced inspection requirements. Bearing type

connections should be specified unless loads are reversing (such as in wind and seismic frames in buildings) or cyclic (such as in highway and railway bridges) or unless oversized holes are used, in which case friction type bolts should be specified. The fabricator and the erector jointly should be given the option of using a torque-control (twist-off) type of bolt that may reduce installation labor
costs. Torque-control bolts or load-indicating washers may have a higher contractor cost, but improve reliability at reduced inspection costs where friction bolts are required.

Bolt installation specifications as contained in reference (2.4) now permit the installation of bearing-type, high-strength bolts by tightening to a snug-tight condition in connections where slip can be permitted and where loosening or fatigue due to vibration or load fluctuations are not design considerations. The snug-tight condition is defined as the tightness attained by a few impacts of an impact wrench or by the full effort of a man using an ordinary spud wrench. Bolts that need to be tightened only to the snug-tight condition must be clearly defined on the design and erection drawings.

Oversized (slotted) holes are sometimes used to facilitate tolerances in erection. Such use should be limited to situations where the extra tolerance is required, such as for reinforcement of, or additions to, existing structures and other special conditions. Standard holes (bolt diameter plus 1/16 inch) should usually be required for girder and spandrel connections to columns, in order to control more accurately the plumbness and dimensions between column centers. Blind (open) holes may be permitted when extra holes are detailed for the purpose of simplifying the detailing of identical members with opposite hand connections, or to facilitate the choice of alternate locations for bolting in areas of difficult access.

B.10.5 Field Welding. Avoid or minimize connection details that require overhead or vertical field welds, if other arrangements are available. Consider the use of partial penetration welds rather than full penetration groove welds in column splices, when permitted by the design standards. Minimize the use of field welding on small-to-medium sized projects where the quality control and assurance required for field welding increase the relative cost of welded connections compared to bolted connections.

B.11 Tolerances and Standard Practice. The AISC Code of Standard Practice (2.3) covers tolerances and standard fabrication and erection practice. AISC, Quality Criteria and Inspection Standards (2.13) provide additional guidance. Provisions in NAVFAC guide specifications shall take precedence in the event of conflicts with these references.

BIBLIOGRAPHY

Naval Facilities Engineering Command - Naval Facilities Guide Specifications (NFGS) - and Technical Specifications (TS) - copies can be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120

NFGS-02368	Rolled Steel Section Piles
NFGS-02369.1	Steel Sheet Piles
NFGS-05120	Structural Steel
NFGS-05210	Steel Joists
NFGS-05311	Steel Roof Decking
TS-05321	Steel Floor Decks with Amendment-1
NFGS-05400	Cold-Formed Metal Framing
NFGS-05500	Metal Fabrications
NFGS-09100	Metal Support Systems
NFGS-13411	Water Storage Tanks
NFGS-15176	Steel Tanks with Fixed Roofs

General Services Administration - Federal Specification copies can be obtained from General Services Administration, 7th & D Streets, SW, Washington, DC 20407.

RR-W-410D	Wire Rope and Strand

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO) specifications, 444 North Capitol St., NW., Washington, DC 20004.

   1.1 Standard Specifications for Highway Bridges, 13th Edition, 1983, with annual updates.

2. American Institute of Steel Construction (AISC) specifications, manuals, technical papers from Engineering Journal, and books, 400 North Michigan Avenue, Chicago, IL 60611-4185.

   2.1 Standard Specification for Manual of Steel Construction, 8th Edition, 1980, and Load and Resistance Factor Design Manual of Steel Construction, First Edition, 1986.
   2.2 Standard Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, with Commentary, 1978.
   2.3 Standard Specification for Code of Standard Practice for Steel Buildings and Bridges, 1976.
   2.4 Standard Specification for Structural Joints Using ASTM A325 or A490 Bolts, 1985 (approved by Research Council on Structural Connections and endorsed by AISC).
   2.5 Standard Specification for Plastic Design of Braced Multistory Steel Frames, 1968.
   2.6 Load and Resistance Factor Design Specification for Structural Steel Buildings, 1986.
   2.7 Standard Specification for Detailing for Steel Construction, 1983.
   2.8 Standard Specification for Torsional Analysis of Steel Members, 1983.
   2.9 Standard Specification for Engineering for Steel Construction, 1984.
   2.10 Standard Specification for Iron and Steel Beams 1873-1952, 1953.
   2.11 Standard Specification for Design Manual for Orthotropic Steel Plate Deck Bridges, 1963.
   2.12 Standard Specification for A Guide to the Shop Painting of Structural Steel, 1972.
   2.13 Standard Specification for Quality Criteria and Inspection Standards, 2nd Ed., 1980.
   2.14 Standard Specification for Bridge Fatigue Guide Design and Details, 1977.
   2.15 Fisher, J. and Buettner, D., Standard Specification for Light and Heavy Industrial Buildings (1979).
   2.16 Type 2 Construction with Wind Moment Connections, A Return to Simplicity. (No date.)

   AISC Engineering Journal - Technical Papers that Give Aids for Practical Design (see 20 Year Index 1964-1983)

2.17 AISC, Commentary on Highly Restrained Welded Connections, 3rd Quarter, 1973.

2.18 AISC, Predesigned Bolted Framing Angle Connections, 1st Quarter, 1982.

2.19 Bakota, J., Mill Building Design Procedures, 4th Quarter, 1977.

2.20 Birkemoe, P., and Gilmore, M., Behavior of Bearing Critical Double Angle Beam Connections, 4th Quarter, 1978.

2.21 Blodgett, D., Detailing to Achieve Practical Welded Fabrication, 4th Quarter, 1980.

2.22 Bouwkamp, J., Considerations in the Design of Large-Size Welded Tubular Truss Joints, July 1965.

2.23 Bower, J., Recommended Design Procedures for Beams with Web Openings, Oct. 1971.

2.24 Brockenbrough, R., Considerations in the Design of Bolted Joints for Weathering Steel, 1st Quarter, 1983.

2.25 Buchert, K., Buckling of Framed Domes, Oct. 1965.

2.26 Buchert, K., Space Frame Buckling, Oct. 1968.

2.27 Burgett, L., Fast Check for Ponding, 1st Quarter, 1973.

2.28 Chinn, J., Failure of Simply-Supported Flat Roofs by Ponding of Rain, April 1963.

2.29 DeFalco, F. and Marino, F., Column Stability in Type 2 Construction, April 1966; Errata, July 1966.

2.30 Disque, R., Inelastic K-factor for Column Design, 2nd Quarter, 1973.

2.31 Disque, R. and Young, N., Design Aids for Single Plate Framing Connections, 4th Quarter, 1981.

2.32 Driscoll, G., Effective Length of Columns with Semi-Rigid Connections, 4th Quarter, 1981.

2.33 Driscoll, G. and Beedle, L., Suggestions for Avoiding Beam-to-Column Web Connection Failure, 1st Quarter, 1982.

2.34 Fisher, J., Design of Composite Beams with Formed Metal Deck, July 1970; Discussion, Oct., 1970.

2.35 Fisher, J., Structural Details in Industrial Buildings, 3rd Quarter, 1981; Discussion, 2nd Quarter, 1982.

2.36 Fisher, J., Grant, J. and Slutter, R., Composite Beams with Formed Steel Deck, 1st Quarter, 1977.

2.37 Fling, R., Design of Steel Bearing Plates, April, 1970.

2.38 Hawkins, N. and Roeder, C., Connections Between Steel Frames and Concrete Walls, 1st Quarter, 1981.

2.39 Hawkins, N., Mitchell, D. and Roeder, C., Moment Resisting Connections for Mixed Construction, 1st Quarter, 1980.

2.40 Johnston, B., Design of W-Shapes for Combined Bending and Torsion, 2nd Quarter, 1982.

2.41 Lally, A. and Milek, W., Bridge Construction Details, Jan. 1969.

2.42 LeMessurier, W., A Practical Method of Second Order Analysis/Part 1 - Pin Jointed Systems, 4th Quarter, 1976; Part 2 - Unbraced Frames, 2nd Quarter, 1977.

2.43 Lenzen, K., Vibration of Steel Joist Concrete Slab Floors, July, 1966.

2.44 Lorenz, R., Some Economic Considerations for Composite Floor Beams, 2nd Quarter, 1983.
2.45 Marino, F., Ponding of Two-Way Roof Systems, July, 1966.
2.46 Mueller, J., Lessons from Crane Runways, Jan., 1965.
2.47 Murray, T., Acceptability Criterion for Occupant-Induced Floor Vibrations, 2nd Quarter, 1981.
2.48 Murray, T., Design to Prevent Floor Vibrations, 3rd Quarter, 1975.
2.49 Murray, T., and Hendrick, W., Floor Vibrations and Cantilevered Construction, 3rd Quarter, 1977.
2.50 Redwood, R., Simplified Plastic Analysis for Reinforced Web Holes, Oct. 1971.
2.51 Redwood, R., Tables for Plastic Design of Beams with Rectangular Holes, Jan. 1972.
2.52 Richard, R., Kriegh, J. and Hornby, D., Design of Single Plate Framing Connections with A-307 Bolts, 4th Quarter, 1982.
2.53 Richard, R., Gillet, P., Kriegh, J. and Lewis, B., The Analysis and Design of Single Plate Framing Connections, 2nd Quarter, 1980.
2.54 Ricker, D., Tips for Avoiding Crane Runway Problems, 4th Quarter, 1982.
2.55 Rolfe, S., Fracture and Fatigue Control in Steel Structures, 1st Quarter, 1977.
2.56 Shipp, J. and Haninger, E. Design of Headed Anchor Bolts, 2nd Quarter, 1983.
2.57 Stevens, D. and Odom, G., The Steel Framed Dome, Oct. 1963.
2.58 Thornton, W., Prying Action - A General Treatment, 2nd Quarter, 1985.
2.59 White, R., Framing Connections for Square and Rectangular Structural Tubing, July 1965.
2.60 Yura, J., The Effective Length of Columns in Unbraced Frames, April 1971; Discussion, Jan. 1972.

3. American Iron and Steel Institute, (AISI) manuals, handbooks and other design aids, 1000 16th Street, NW., Washington, DC 20036.

3.1 CSI Spec Data - Stainless Steel, 1976.
3.2 Standard Specification for Cold-Formed Steel Design Manual, 1983. Includes Specification for Design of Cold-Formed Steel Structural Members.
3.3 Design Guidelines for the Selection and Use of Stainless Steel, 1977.
3.4 Standard Specification for Fire-Resistant Steel Frame Construction, 2nd Ed., 1974.
3.5 Handbook of Corrosion Protection for Steel Pile Structures in Marine Environment, 1st Ed., 1981.
3.6 Handbook of Steel Drainage and Highway Construction Projects, 1983.
3.7 Sherman, D.R., Tentative Criteria for Structural Applications of Steel Tubing and Pipe, with Errata, 1977.
3.8 Stainless Steel Cold-Formed Structural Design Manual, 1974.

3.9 Stainless Steel Fasteners - A Systematic Approach to Their Selection, 1976.
3.10 Stainless Steel Stone Anchors, 1975.
3.11 Steel Tanks for Liquid Storage, Volume 1 in the series Steel Plate Engineering Data, 1982.
3.12 Useful Information on the Design of Plate Structures, Volume 2 in the series "Steel Plate Engineering Data, 1985.

4. American National Standards Institute, Inc., (ANSI) - 1430 Broadway, New York, NY 10018

4.1 ANSI A58.1-82. Minimum Design Loads for Buildings and Other Structures.

5. American Petroleum Institute, (API) - ANSI/API Standards - 1220 L Street, NW., Washington, DC 20005.

5.1 Standard 620-82 Recommended Rules for Revision Design and Construction of Large, Welded, Low-Pressure Storage Tanks.
5.2 Standard 650-80 Welded Steel Tanks for Oil Storage, 1984.

6. American Railway Engineering Association, (AREA) manual, Chicago, IL 60605.

6.1 Manual for Railway Engineering (Fixed Properties)

7. American Society of Civil Engineers, (ASCE) specifications, committee reports and technical articles, 345 East 47th St., New York, NY 10017-2398.

7.1 Design and Construction of Steel Chimney Liners, 1975.
7.2 Plastic Design in Steel.
7.3 Specifications for the Design and Construction of Composite Slabs, with Commentary, 1985.
7.4 Nelson, J. T., Blazier, W. E., and Saurenman, H. J., Site Selection and Building Design for Minimizing Vibration, in Noise and Vibration Measurement: Prediction and Mitigation Proceedings of an ASCE Symposium, 1985.
7.5 Richart, F. E., Foundation Vibrations, Trans. ASCE, 1962, Vol. 127, Part 1, pp. 863-898.
7.6 Ungar, E. E., and Gordon, C. G., Cost-Effective Design of Practically Vibration-Free High Technology Facilities, in Noise and Vibration Measurements: Prediction and Mitigation, Proceedings of an ASCE Symposium, 1985.
7.7 Wright, D.T., Membrane Forces and Buckling in Reticulated Shells, ASCE Journal of the Structural Division, February 1965.

8. National Association of Corrosion Engineers, 1440 S. Creek Drive, Houston, TX 77084.

   8.1 Corrosion Handbook.

9. American Society of Mechanical Engineers, (ASME) reports and codes, 345 East 47th Street, New York, NY 10017-2398.

   9.1 ASME Report 63-WA-248. Dynamic Response of Tall Stacks to Wind Excitation.

   9.2 ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels Divisions I and II, 1983.

   9.3 Pressure Vessels and Piping: Design and Analysis, Vol. I, Analysis, Vol. II, Components and Structural Dynamics, a compilation of technical papers, 1972.

10. American Society for Testing and Materials, (ASTM) proceedings and standards, 1916 Race Street, Philadelphia, PA 19103.

10.1	A-36	Standard Specification for Structural Steel
10.2	A-242	Standard Specification for High-Strength Low-Alloy Structural Steel
10.3	A-263-84	Standard Specification for Corrosion-Resisting Chromium Steel-Clad Plate, Sheet, and Strip
10.4	A-264-84	Standard Specification for Stainless Chronium-Nickel Steel-Clad Plate, Sheet and Strip
10.5	A-265-84	Standard Specification for Nickel and Nickel Base Alloy - Clad Steel Plate
10.6	A-283	Standard Specification for Low and Intermediate Tensile Strength-Carbon Steel Plates, Shapes, and Bars
10.7	A-307-84	Standard Specification for Carbon-Steel Bolts and Studs 60,000 psi tensile
10.8	A-325-85	Standard Specification for High-Strength Threaded Standard Fasteners
10.9	A-333	Standard Specification for Seamless and Welded Steel Pipe for Low-Temperature Services
10.10	A-441	Standard Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel
10.11	A-475	Standard Specification for Zinc-Coated Steel Wire Strand
10.12	A-490	Specification for Heat Treated Steel Structural Bolts, 150 Ksi Minimum Tensile Strength
10.12	A-514	Standard Specification for High-Yield Strength, Quenched and Tempered Alloy-Steel Plate, Suitable for Welding
10.13	A-572	Standard Specification for High-Strength Low-Alloy Columbiun-Vanadium Steels of Structural Quality

10.14 A-586-81 Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand

10.15 A-588 Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick

10.16 A-603-70 Standard Specification for Zinc-Coated Steel Structural Wire Rope, Specification for

10.17 E-936-83 Standard Practice for Roof System Assemblies Employing Steel Deck, Preformed Roof Insulation, and Bituminous Built-Up Roofing

11. American Society for Metals (ASM), Metals Park, OH 44073.

11.1 Source Book on Stainless Steels, 1976.

11.2 Source Book on Industrial Alloy and Engineering Data, 1978.

12. American Water Works Association, AWWA standards and manuals, 6666 West Quincy Ave., Denver, CO 80235.

12.1 D100-79 AWWA Standard for Welded Steel Tanks for Water Storage, 1979, (Reaffirmed 1984).

12.2 D101-53 AWWA Standard for Inspecting and Repairing Steel Water Tanks, Standpipes, Reservoirs, and Elevated Tanks for Water Storage, 1953 (reaffirmed 1979).

12.3 D102-78 AWWA Standard for Painting Steel Water-Storage Tanks, 1978.

12.4 D103-84 AWWA Standard for Factory-Coated Bolted Steel Tanks for Water Storage, 1984.

13. American Welding Society (AWS) code and handbook, 2501 N.W. 7th Street, Miami, FL 33125.

13.1 ANSI/AWS D1.1-26 - Structural Welding Code-Steel, 1986.

13.2 Welding Handbook, 7th Edition, 1973.
Section 1 Fundamentals of Welding
Section 2 Welding Processes: Gas, Arc and Resistance
Section 3 Special Welding Processes and Cutting
Section 4 Metals and Their Weldability
Section 5 Applications of Welding

14. Association of Iron and Steel Engineers, (AISE) guide and specifications, Pittsburgh, PA.

14.1 Specifications for the Design and Construction of Mill Buildings, AISE Technical Report No. 13., 1979.

14.2 Specification for Electric Overhead Traveling Cranes for Steel Mill Service, Standard No. 6, 1969.

15. Canadian Institute of Steel Construction, (CISC) report 201 Consumers Rd., Suite 300, Willowdale, Ontario, Canada, M2J 4G8.

15.1 Chien and Ritchie, Design and Construction of Composite Floor Systems, 1984.

16. Electronic Industries Association (EIA) standard, 2001 Eye Street, NW., Washington, DC 20006-1807.

16.1 RS-222-C, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, March 1976.

17. Factory Mutual (FM) Systems Loss Prevention Data.

17.1 1-28 Insulated Steel Deck.

18. Fisher, John, and Struik, John, Design Criteria for Bolted and Riveted Joints, John Wiley & Sons, 1974.

19. Gaylord, E.H., & Gaylord, C.N., Design of Steel Bins for Storage of Bulk Solids, Prentice-Hall, 1984.

20. Gaylord, E.H. & Gaylord, C.N., Section 26, - "Chimneys", in Structural Engineering Handbook, McGraw-Hill, Book Company, New York, NY., 1968.

21. Harris, C. M., Handbook of Noise Control, Chapter 18, 2nd Edition, McGraw-Hill Book Company, New York, NY, 1979.

22. Harris, C. M. and Crede, C. E., Shock and Vibration Handbook, Chapter 44, 2nd Edition, McGraw-Hill Book Company, New York, NY, 1976.

23. IASS recommendation. International Association for Shell and Spatial Structures, Alfonso XII, 3, Madrid-7, Spain.

23.1 Working Group 4, Recommendations for Guyed Masts, 1981.

24. International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA 90601.

24.1 Uniform Building Code

25. ISO, International Organization for Standardization, ISO Central Secretariat, Case postale 56, CH-1211 Geneve 20, Switzerland.

25.1 Standard ISO 2631-1978, Guide for the Evaluation of Human Exposure to Whole-Body Vibration.

26. Industrial Fastener Institute, 1505 East Ohio Building, Cleveland, OH 44114.

28.1 Tubular Steel Structures - Troitsky, M., 1982.
28.2 The Procedure Handbook of Arc Welding, 12th Edition, 1973.

27. Jones, R.S., Noise and Vibration Control in Buildings, McGraw-Hill Book Company, New York, NY 1984.

28. James F. Lincoln Arc Welding Foundation - technical books.

27.1 Tubular Steel Structures, Troitsky, M., 1982.
27.2 The Procedure Handbook of Arc Welding, 12th Edition, 1973.

29. Metal Building Manufacturers Association, MBMA specifications, manuals and technical books. 1230 Keith Building, Cleveland, OH 44115-2180.

29.1 Crane Manual for Metal Building Systems, 1982.
29.2 Lee, G., Ketter, R., and Hsu, T., Design of Single Story Rigid Frames, 1981.
29.3 Low Rise Building Systems Manual, 1986.

30. National Science Foundation, Building Research Advisory Board (BRAB), Washington, DC.

30.1 BRAB Technical Report No. 65, Expansion Joints in Buildings, 1974.

31. Steel Deck Institute (SDI) manuals, P.O. Box 3812, St. Louis, MO 63122.

31.1 Steel Deck Institute Design Manual for Composite Decks, Form Decks, and Roof Decks, 1985 (Publ 25-85).
31.2 Diaphragm Design Manual, 1st Edition, 1981 (Publ DDM01-82).
31.3 SDI, Inc. File, 1986.

32. Steel Joist Institute (SJI) specifications and technical digests, 1205 48th Avenue N., SUite A, Myrtle Beach, SC 29577.

32.1 Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders - K-Series, LH-Series, DLH-Series, Joist Girders.
32.2 Standard Specification for Technical Digest #2, Spacing of Bridging for Open Web Steel Joists, 1970.
32.3 Standard Specification for Technical Digest #3, Structural Design of Steel Joist Roofs to Resist Ponding Loads, 1971.
32.4 Technical Digest #5, Vibration of Steel Joist - Concrete Slab Floors. (No date.)
32.5 Technical Digest #6, Structural Design of Steel Joist Roofs to Resist Uplift Loads. (No date.)

32.6 Technical Digest #7, Fifty-Year Digest - A Compilation of SJI Specifications and Load Tables, 1928 - 1978.
32.7 Technical Digest #8, Welding of Open Web Steel Joists, 1983.

33. Steel Structures Painting Council (SSPC) specifications and guides, 4400 Fifth Avenue, Pittsburgh, PA 15213.

33.1 Steel Structures Painting Manual, Good Painting Practice, - Volume 1, Second Edition, 1982.
33.2 Steel Structures Painting Manual, Systems and Specifications - Volume 2, Third Edition, 1982.

34. Structural Stability Research Council (SSRC), Fritz Engineering Lab No. 13, Lehigh University, Bethlehem, PA 18105.

34.1 Guide to Stability Design Criteria for Metal Structures (edited by Johnston, B.), 3rd Ed., 1976.

35. The Structural Steel Fabricators of New England, A Guide to Economical Practices in Steel Design and Construction, 1983.

36. Underwriters Laboratories Inc. (UL), 333 Pfingsten Road, Northbrook, IL 60062.

36.1 UL 142-81, Standard for Steel Aboveground Tanks for Flammable and Combustible Liquids.
36.2 UL 58-86, Standard for Steel Underground Tanks for Flammable and Combustible Liquids.
36.3 Fire Resistance Directory, with Hourly Ratings for Beams, Columns, Floors, Roofs, Walls, and Partitions.

37. U.S. Department of Commerce, National Bureau of Standards. Copies can be obtained from the National Technical Information Service, 5285 Port Royal, Springfield, VA 22161

37.1 NBS Monograph 127, NBS Papers on Underground Corrosion of Steel Pilings - 1962-71.
37.2 NBS Circular 579 by M. Romanoff, Underground Corrosion, 1957.

38. U.S. Department of Defense. NAVFAC Design Manuals (DM), Guide Specifications (NFGS), Maintenance Operations (MO) and Military Handbooks (MIL-HDBK). Copies can be optained from the U.S. Naval Publications and Forms Center, Philadelphia, PA 19120. Telephone number: AUTOVON-442-3321; commercial 215-697-3321.

38.1 Standard Specification for NAVFAC DM-1.05, Roofing and Waterproofing (Proposed)
38.2 Standard Specification for NAVFAC DM-2.01, Structural Engineering-General Requirements, 1980.

38.3 Standard Specification for NAVFAC DM-2.02, Structural Engineering-Loads, 1986.
38.4 Standard Specification for NAVFAC DM-2.04, Structural Engineering-Concrete Structures, 1986.
38.5 Standard Specification for MIL-HDBK-1002/5, Timber Structures, 1987.
38.6 Standard Specification for MIL-HDBK-1002/6, Aluminum Structures, Composite Structures, Structural Plastics, and Fiber Reinforced Composite, 1987.
38.7 Standard Specification for NAVFAC DM-2.09, Masonry Structural Design for Buildings (Tri-Service), with Changes Nos. 1 and 2, 1982.
38.8 Standard Specification for NAVFAC DM-7.01, Soil Mechanics, 1986.
38.9 Standard Specification for NAVFAC DM-7.02, Foundations and Earth Structures, 1986.
38.10 Standard Specification for NAVFAC DM-7.03, Soil Dynamics, Deep Stabilization and Special Geotechnical Construction, 1983.
38.11 Standard Specification for NAVFAC DM-22, Petroleum Fuel Facilities, 1982.
38.12 Standard Specification for NAVFAC DM-25.6, General Criteria for Waterfront Construction, 1981.
38.13 Standard Specification for NAVFAC DM-39, Hyperbaric Facilities, 1982.
38.14 Standard Specification for MIL-HDBK-1008, Fire Protection for Facilities - Engineering, Design and Construction, 1985.
38.15 Standard Specification for NAVFAC DM-50, NAVFAC Index to Engineering and Design Criteria, 1985.
38.16 Standard Specification for NAVFAC P-34, Engineering and Design Criteria for Navy Facilities.
38.17 Standard Specification for NAVFAC P-355, Seismic Design for Buildings (Tri-Service), 1982.
38.18 Standard Specification for NAVFAC P-397, Structures to Resist the Effects of Accidental Explosions (Tri-Service), March 1971.
38.20 Standard Specification for NAVFAC MO-110, Painting and Protective Coatings, 1981.
38.21 Standard Specification for NFGS-13121, Preengineered Metal Buildings (Rigid Frame)
38.22 Standard Specification for NFGS-13411, Water Storage Tanks

39. U.S. Department of Transportation, Federal Highway Administration, Office of Engineering, 800 Independence Avenue, SW, Washington, DC 20590, Washington, D.C.

39.1 FHWA Technical Advisory T5140.13, January 28, 1980, Integral, No-Joint Structures and Required Provisions for Movement.

39.2 FHWA Technical Advisory T5140.15, March 26, 1980, Expansion Devices for Bridges.

39.3 FHWA Technical Advisory T5140.16, March 26, 1980, Bridge Deck Joint Rehabilitation (Retrofit).

40. WSTI manual. Welded Steel Tube Institute, Inc., Structural Tube Division, 522 Westgate Tower, Cleveland, OH 44116.

39.1 Manual of Cold-Formed Welded Structural Steel Tubing, 1st Edition, 1974.

CUSTODIAN
NAVY-YD

PREPARING ACTIVITY
NAVY-YD

PROJECT NO.
FACR - 0188

STANDARDIZATION DOCUMENT IMPROVEMENT PROPOSAL

(See Instructions – Reverse Side)

1. DOCUMENT NUMBER
MIL-HDBK -1002/3

2. DOCUMENT TITLE
STRUCTURAL ENGINEERING STEEL STRUCTURES

3a. NAME OF SUBMITTING ORGANIZATION

4. TYPE OF ORGANIZATION *(Mark one)*

☐ VENDOR
☐ USER
☐ MANUFACTURER
☐ OTHER *(Specify)*: ______
☐ EFD/PWO
☐ AE
☐ CONTRACTOR
☐ OICC/ROICC

b. ADDRESS *(Street, City, State, ZIP Code)*

5. PROBLEM AREAS

a. Paragraph Number and Wording:

b. Recommended Wording:

c. Reason/Rationale for Recommendation:

6. REMARKS

7a. NAME OF SUBMITTER *(Last, First, MI)* – Optional

b. WORK TELEPHONE NUMBER *(Include Area Code)* – Optional

c. MAILING ADDRESS *(Street, City, State, ZIP Code)* – Optional

8. DATE OF SUBMISSION *(YYMMDD)*

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