



GOVERNMENT OF INDIA

AERB SAFETY STANDARD

**DESIGN, FABRICATION AND ERECTION
OF STEEL STRUCTURES
IMPORTANT TO SAFETY OF
NUCLEAR FACILITIES**



ATOMIC ENERGY REGULATORY BOARD

AERB SAFETY STANDARD NO. AERB/SS/CSE-2

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**Atomic Energy Regulatory Board
Mumbai 400 094**

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- I. Nuclear Facilities - Steel Safety-Related Structures for Design, Fabrication and Erection, American National Standard No. ANSI/AISC N690, 1984 Edition:**
Sections/sub-sections: Q1.8.2, Q1.8.3, Q1.9.1.1, Q1.9.2.1, Q1.10.4, Q1.10.8, Q1.10.9, Q1.13.1, Q1.13.2, Q1.14.1 to Q1.14.5, Q1.15.5.4, Q1.15.6 to 1.15.10, Q1.16.2, Q1.16.6, Q1.18.1, Q1.19, Q1.20, Q2.3, Q2.10(1), (2);
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Equations: Q1.15-1 to Q1.15-3, Q1.11-2 to Q1.117.

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- II. Manual of Steel Construction, Allowable Stress Design - Volume II, Ninth Edition:**
Finishing requirements for column base plates on page 3-111;
Figure 1 and Figure 2 on page 3-106.

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- III. Manual of Steel Construction, Allowable Stress Design - Volume III, Ninth Edition:**
Sections/sub-sections: B5.1, I-5, J1-4, K1-8;
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Equations: E2-1, E2-2, F1-1 to F1-8, F2-1, F2-3, F3-1, F3-3, F4-2, G1-1, G1-2, G2-1, G3-1, G4-2, G4-3, G5-1, J3-4, J8-1, J8-2, K1-1 to K1-9, A-B5-1 to A-B5-8.

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- IV. Rules for Construction of Nuclear Power Plant Components, Section III, Division 2, Code for Concrete Reactor Vessels and Containments, ASME Boiler and Pressure Vessel Code, 1989 Edition:**
Figures: CC-4542.2-1 (a to d), CC-4542.2-2 (a to f), CC-4542.2-3 (a to d), CC-4542.2-5 (a & b), CC-4542.2-6 (a to d), CC-4642.1-1 (a to h) and CC-4545-1.

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FOREWORD

Safety of public, occupational workers and the protection of environment should be assured while activities for economic and social progress are pursued. These activities include the establishment and utilisation of nuclear facilities and use of radioactive sources and have to be carried out in accordance with relevant provisions in the Atomic Energy Act, 1962.

Assuring high safety standards has been of prime importance since inception of the nuclear power programme in the country. Recognising this aspect, the Government of India constituted the Atomic Energy Regulatory Board (AERB) in November 1983, vide statutory order No. 4772 notified in the Gazette of India dated 13.12.1983. The Board has been entrusted with the responsibility of laying down safety standards and framing rules and regulations in respect of regulatory and safety functions envisaged under the Atomic Energy Act of 1962. Under its programme of developing safety codes and guides, AERB had issued four codes of practice in the area of nuclear safety covering the following topics:

Safety in Nuclear Power Plant Siting

Safety in Nuclear Power Plant Design

Safety in Nuclear Power Plant Operation

Quality Assurance for Safety in Nuclear Power Plants

Civil engineering structures in nuclear installations form an important feature having implications to safety performance of these installations. The objective and minimum requirements for the design of civil engineering buildings/structures to be fulfilled to provide adequate assurance for safety of nuclear installations in India (such as pressurised heavy water reactor and related systems) are specified in the Safety Standard for Civil Engineering Structures Important to Safety of Nuclear Facilities (AERB/SS/CSE). This standard is written by AERB to specify guidelines for implementation of the above civil engineering safety standard in the design, fabrication and erection of steel structures important to safety.

This standard may be revised as and when necessary in the light of experience as well as developments in the field. The appendices included in the document are an integral part of the document, whereas the annexure, footnotes, and bibliography are to provide information that might be helpful to the user.

Emphasis in the codes, standards, guides and manuals is on protection of site personnel and public from undue radiological hazard. However, for aspects not covered in these documents, applicable and acceptable national and international codes and standards shall be followed. In particular, industrial safety shall be assured through good engineering practices and by complying with the Factories Act, 1948 as amended in 1987 and Atomic Energy (Factories) Rules, 1996.

This safety standard on civil and structural engineering (CSE) has been prepared by the professionals from AERB, BARC, NPC, DCL, TCE and BIS. In its preparation, the relevant national and international documents (mentioned in the “Bibliography” section of this standard) have been extensively used. It has been reviewed by experts and amended by Advisory Committees before issue. AERB wishes to thank all individuals and organisations who have contributed in the preparation, review and amendment of the safety standard. The list of persons who have participated in committee meetings, along with their affiliation, is included for information.



(Suhas P. Sukhatme)
Chairman, AERB

DEFINITIONS

Acceptable Limits

Limits acceptable to the Regulatory Body for accident condition or potential exposure.

Accident Conditions

Substantial deviations from Operational States which could lead to release of unacceptable quantities of radioactive materials. They are more severe than anticipated operational occurrences and include Design Basis Accidents and severe accidents.

Anchorage (Prestressing)

In post-tensioning, a device used to anchor tendon to concrete member, in pre-tensioning, a device used to anchor tendon during hardening of concrete, a means by which force is transferred to the concrete.

Anchor Head

A nut, washer, plate, stud, or bolt head or other steel component used to transmit anchor loads to the concrete by bearing.

Approval

A type of regulatory consent issued by the Regulatory Body to a proposal.

Atomic Energy Regulatory Board (AERB)

A national authority designated by the Government of India having the legal authority for issuing regulatory consent for various activities related to the nuclear facility and to perform safety and regulatory functions including enforcement for the protection of the public and operating personnel against radiation.

Attachment

An attachment is an element in contact with or connected to the inside or outside of a component. It may have either a pressure retaining or a non-pressure retaining function.

Audit

A documented activity performed to determine by investigation, examination and evaluation of objective evidence the adequacy of, and adherence to, applicable Codes, Standards, Specifications, established procedures, instructions, administrative or operational programmes and other applicable documents, and the effectiveness of their implementation.

Base Temperature/Stress-free Temperature

Temperature at which it is assumed that the material is free of thermal stresses.

Commissioning

The process during which structures, systems and components of a nuclear and radiation facility, having been constructed, are made functional and verified to be in accordance with design specifications and to have met the performance criteria.

Competent Authority

Any official or authority appointed, approved or recognised by the Government of India for the purpose of the Rules promulgated under the Atomic Energy Act 1962.

Construction

The process of manufacturing, testing and assembling the components of a nuclear or radiation facility, the erection of civil works and structures, the installation of components and equipment and the performance of associated tests.

Decommissioning

The process by which a nuclear or radiation facility is finally taken out of operation, in a manner that provides adequate protection to the health and safety of the workers, the public and of the environment.

Defects

Any deviation from the pre-defined acceptable limits or any non-conformance with the stated requirements.

Design

The process and the results of developing the concept, detailed plans, supporting calculations and specifications for a nuclear or radiation facility.

Design Basis Accident (DBA)

Design Basis Accidents are a set of postulated accidents which are analysed to arrive at conservative limits on pressure, temperature and other parameters which are then used to set specifications that must be met by plant structures, systems and components, and fission product barriers.

Design Inputs

Those criteria, parameters, bases or other requirements upon which detailed final design is based.

Design Outputs

Documents such as design, reports, drawings, specifications that define technical requirements necessary for manufacture, installation and operation of structures, systems and components.

Disposition

An act to determine how a departure from a specified requirement is to be handled or settled.

Documentation

Recorded or pictorial information describing, defining, specifying, reporting or certifying activities, requirements, procedures or results.

Earthquake

Vibration of earth caused by the passage of seismic waves radiating from the source of elastic energy.

Embedded Part (EP)

Any structural member, plates, angle, channel, pipe sleeve or other section anchored to a concrete structure through direct bond or other anchors.

Embedment

The Embedment is that portion of the component in contact with the concrete or grout used to transmit applied loads to the concrete structure through direct bond or other anchor. The embedment may be fabricated lugs, bolts, plates, reinforcing bars, shear connectors, expansion anchors, inserts or any combination thereof.

Examination

An element of inspection consisting of investigation of materials, components, supplies, or services to determine conformance with those specified requirements which can be determined by such investigation.

Inspection

Quality control actions which, by means of examination, observation or measurement, determine the conformance of materials, parts, components, systems, structures as well as processes and procedures with predetermined quality requirements.

Item

A general term covering structures, systems, components, parts or materials.

Items Important to Safety

The items which comprise:

- (1) those structures, systems, equipment and components whose malfunction or failure could lead to undue radiological consequences at Plant site or off-site.
- (2) those structures, systems and components which prevent Anticipated Operational Occurrences from leading to Accident Conditions;
- (3) those features which are provided to mitigate the consequences of malfunction or failure of structures, systems or components.

Main Structural Members

The structural members which are primarily responsible to withstand, carry and distribute the applied load.

Maintenance

Organised activities covering all preventive and remedial measures, both administrative and technical, necessary to ensure that all structures, systems and components are capable of performing as intended for safe operation of plant.

Non-conformance

A deficiency in characteristics, documentation or procedure which renders the quality of an item unacceptable or indeterminate.

Normal Operation

Operation of a plant or equipment within specified operational limits and conditions. In the case of nuclear power plant these include start-up, power operation, shutting down, shut-down state, maintenance, testing and refuelling.

Nuclear Power Plant (NPP)

A nuclear reactor or a group of reactors together with all the associated structures, systems, components and equipment necessary for safe generation of electricity.

Nuclear Facility

All nuclear fuel cycle and associated installations encompassing the activities covering from the front end to the back end of nuclear fuel cycle processes and also the associated industrial facilities such as heavy water plants, beryllium extraction plants, zirconium plant, etc.

Objective Evidence

Term used in context of Quality Assurance, qualitative or quantitative information, record or statement of fact, pertaining to quality of an item or service which is based on observation, measurement or test and which can be verified.

Operating Basis Earthquake (OBE)

The "Operating Basis Earthquake" (OBE) is that earthquake which, considering the regional and local geology and seismology and specific characteristics of local sub-surface material, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake which produces the vibratory ground motion for which the features of Nuclear Power Plant (NPP) necessary for continued safe operation are designed to remain functional.

Operation

All activities following commissioning and before decommissioning performed to achieve, in a safe manner, the purpose for which a nuclear/radiation facility was constructed, including maintenance.

Postulated Initiating Events (PIEs)

Identified events that lead to Anticipated Operational Occurrence or Accident Conditions, and their consequential failure effects.

Primary Stress

Primary stress is any normal stress or shear stress developed by an imposed loading which is necessary to satisfy the laws of equilibrium of external and internal forces and moments. The basic characteristic of a primary stress is that it is not self-limiting.

Prescribed Limits

A limit established or accepted by the Regulatory Body.

Qualified Person

A person, who having complied with specific requirement and having met certain conditions, has been approved by the Regulatory Body, where necessary to discharge specified duties and responsibilities.

Quality

The totality of features and characteristics of a product or service that bear on its ability to satisfy a defined requirement.

Quality Assurance

Planned and systematic actions necessary to provide adequate confidence that an item or a facility will perform satisfactorily in service as per design specifications.

Quality Control

Quality assurance actions which provide a means to control and measure the characteristics of an item, process or facility in accordance with established requirements.

Records

Documents which furnish objective evidence of the quality of items and activities affecting quality. It also includes logging of events and other measurements.

Reliability

The probability that a device, system or facility will perform its intended function satisfactorily under stated operating conditions.

Repair

The process of restoring a non-conforming item to a condition such that the capability of this item to function reliably and safely is unimpaired, even though that item still may not conform to the prior specification.

Responsible Organisation (RO)

The organisation having overall responsibility for siting, design, construction, commissioning, operation and decommissioning of a facility.

Rework

The process by which a non-conforming item is made to conform to a prior specified requirement by completion, remachining, reassembling or other corrective means.

Safe Shutdown Earthquake (SSE)

The "Safe Shutdown Earthquake" is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local sub-surface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems and components are designed to remain functional. These structures, systems, and components are those which are necessary to assure;

1. The integrity of the reactor coolant pressure boundary, or
2. The capability to shutdown the reactor and maintain it in a safe shutdown condition, or
3. The capability to prevent the accident or to mitigate the consequences of accidents which could result in potential off-site exposures higher than the limits specified by the Regulatory Body, or
4. The capacity to remove residual heat.

Safety

Protection of all persons from undue radiological hazards.

Safety Limits

Limits upon process variables within which the operation of the facility has been shown to be safe.

Safety Critical System (Safety Systems)

Systems important to safety, provided to assure, under anticipated operational occurrences and accident conditions, the safe shut down of the reactor (Shutdown System) and the heat removal from the core (Emergency Core Cooling System), and containment of any radioactivity (Containment Isolation System).

Secondary Stress

Secondary stress is a normal stress or shear stress developed by the constraint of adjacent material or by self-constraint of the structure. The basic characteristic of a secondary stress is that it is self-limiting.

Services

The performance by a supplier of activities such as design, fabrication, installation, inspection, non-destructive examination, repair and/or maintenance.

Site

The area containing the facility defined by a boundary and under effective control of the facility management.

Site Personnel

All persons working on the site, either permanently or temporarily.

Siting

The process of selecting a suitable site for a facility including appropriate assessment and definition of the related design bases.

Specification

A written statement of requirements to be satisfied by a product, a service, a material or a process, indicating the procedure by means of which it may be determined whether the specified requirements are satisfied.

Structure

The assembly of elements which supports/houses the plants, equipment and systems.

Supplier Evaluation

An appraisal to determine whether or not a management system is capable of producing a product or service of a stated quality, and generating evidence that supports decisions on acceptability.

Surveillance

All planned activities viz. monitoring, verifying, checking including in-service inspection, functional testing, calibration and performance testing performed to ensure compliance with specifications established in a facility.

Testing

The determination or verification of the capability of an item to meet specified requirements by subjecting the item to a set of physical, chemical, environmental or operational conditions.

Verification

The act of reviewing, inspecting, testing, checking, auditing, or otherwise determining and documenting whether items, processes, services or documents conform to specified requirements.

SYMBOLS

Unless specified otherwise, the following symbols apply to this standard and may not necessarily conform to the symbols adopted elsewhere for national and international use. Unless specified otherwise, SI units (millimeter for linear dimension and Newton for force) are adopted.

- A Cross-sectional area (A used with subscripts has been defined at appropriate places).
- A_b Nominal area of a fastener.
- A_0 Effective (net) area of concrete in composite design.
- A_{eff} Effective area of axially loaded tension members;
 $A_{\text{eff}} = C_t A_n$
- A_f Area of compression flange.
- A_g Gross area.
- A_n Net area of tension member.
- A_s Effective cross-sectional area of a member; Area of longitudinal reinforcing steel within the effective width of the concrete flange of composite beam.
- A_{st} Minimum combined cross-sectional area of stiffener or pair of stiffeners.
- A_w Effective cross-sectional area resisting shear for calculating the average shear stress or the maximum shear capacity of the members, generally the area of girder web.
- A_1 Maximum area of the portion of supporting surface that is geometrically similar to and concentric with the loaded area.
- A_2 Area of bearing steel.
- a Clear distance between transverse stiffeners.

b	Compression element width.
b_e	Effective width of stiffened or unstiffened compression elements.
b_f	Flange width.
b_0	Width of steel compression flange in encased member.
C_m	Coefficient applied to bending term in interaction formula and dependent upon column curvature caused by applied moments.
C_p	Stiffness factor for primary member in a flat roof.
C_s	Stiffness factor of secondary member in a flat roof.
C_t	Reduction coefficient in determining effective area of axially loaded tension member.
C_v	Coefficient relating allowable average shear stress in web to the yield stress of the web material.
D	Outside diameter of circular hollow section; overall depth of section.
DL	Dead load.
d	Depth of girder : to be taken as clear distance between flange angles or where there are no flange angles, the clear distance between flanges ignoring fillets.
d'	Diameter of expansion roller or rocker.
d_c	Web depth clear of fillets = $D - 2K$.
d_1	(i) For the web of a beam without horizontal stiffeners : the clear distance between flanges, neglecting fillets, or the clear distance between flange angles as appropriate. (ii) For the web of a beam with horizontal stiffeners : the distance between horizontal stiffener and tension flange, neglecting fillets, or between the horizontal stiffener and the inner toes of the tension flange angles as appropriate.

E	Modulus of elasticity for steel, taken as 2×10^5 MPa in this standard.
E_c	Modulus of elasticity of concrete.
E_0	Load effects due to the Operating Basis Earthquake, including responses of supported components, piping and equipment, hydrodynamic effects and dynamic effects of surrounding soil.
E_{ss}	Load effects due to Safe Shutdown Earthquake, including responses of supported components, piping and equipment, hydrodynamic effects and dynamic effects of surrounding soil.
F_h	Hydrostatic load due to internal flooding.
F_p	Allowable bearing force.
FF	Load effects due to design basis flood.
f_{cc}	Elastic critical stress in compression, also known as Euler critical stress.
f_{ck}	Specified or characteristic compressive strength (in MPa) of 150 mm concrete cube.
f_u	Ultimate stress.
f_y	Yield stress.
f_{yc}	Column yield stress.
f_{yf}	Minimum yield stress of flange material.
f_{yst}	Yield stress of stiffener.
f_{yr}	Yield stress of reinforcing steel providing composite action at section of negative moment.
f_{yw}	Minimum yield stress of the web material.
g	Gauge.

H	Lateral earth pressure.
H_s	Length of stud connector after welding.
h	Outstand of the stiffener.
h_c	Clear distance between flanges.
h_r	Nominal rib height.
I	Moment of inertia, subscript denoting the axis about which the moment of inertia is taken.
I_d	Moment of inertia of the steel deck supported on secondary members.
I_{eff}	Effective moment of inertia for deflection computations.
I_p	Moment of inertia of primary member.
I_s	Moment of inertia of secondary member.
I_{tr}	Moment of inertia of transformed composite section.
K	Distance from outer face of flange to web toe of fillet of member.
k	Effective length factor.
L	Span or length of member.
LL	Live load.
L_c	Length of channel shear connector.
L_p	Column spacing in direction of girder (length of primary members).
L_s	Column spacing perpendicular to the direction of girder (length of secondary members).
l_c	Unsupported length of compression members/columns.

M	Bending moment.
MA	Load and other effects of aircraft impact.
ME	Missiles due to external events other than those related to wind or tornado, explosion in transportation systems, disintegration of turbine and other components.
MI	Loading due to internal missiles.
Mt	Load and other impactive effects of turbine missile.
MT	Missiles, wind and overpressure generated from explosions in transportation systems on land, water or in air.
M_D	Section moment caused by loads applied prior to the time of M_t .
M_0	Lateral buckling strength in the absence of axial load.
M_p	Maximum plastic moment capacity of a section.
M_t	Section moment due to all causes subsequent to the time when the concrete will have reached 75% of its required strength.
M_1	Smaller moment at the ends of unbraced length of beam.
M_2	Larger moment at ends of unbraced length, taken about the strong axis of the member.
N_r	Number of connectors on each side of the section of maximum moment; Number of stud connectors in one rib at a beam intersection.
P	Axial force, compressive or tensile.
P_a	Design accident pressure.
P_{ac}	Calculated maximum load capacity of a strut.
P_{at}	Calculated maximum load capacity as a tension number.

P_{bf}	Computed force delivered by flange or moment connection plate multiplied by factor greater than 1.0.
P_e	Euler buckling load.
P_r	Beam reaction, divided by the number of bolts.
P_t	Test pressure.
P_v	Pressure loads resulting during normal operation condition.
P_y	Yield strength of axially loaded section.
	Density
Q_a	Reduction factor for slender stiffened compression elements.
Q_s	Reduction factor for slender unstiffened compression elements.
q	Allowable shear load per connector.
R	Reaction of a beam at the support.
R_a	Pipe and equipment reaction under thermal conditions generated by a postulated pipe break and including R_0 .
R_0	Pipe and equipment reactions during normal operation excluding dead load and earthquake reactions.
r	Radius of gyration of the section; subscript denotes the reference axis for the corresponding radius of gyration.
r_t	Radius of gyration of a section comprising compression flange plus 1/3 of the compression web area, taken about an axis in the plane of the web.
r_y	Radius of gyration about minor axis.
S	Transverse distance between centroids of welds; spacing of secondary members, allowable stress.

s	Longitudinal c/c spacing (pitch) of any two consecutive holes; staggered pitch.
s_i	Clear distance between adjacent beams; $i = 1,2$.
T	Mean thickness of compression flange; subscripted values have been defined at appropriate places.
T_a	Design accident temperature.
T_0	Thermal effects and loads during normal operation, solar radiation effects and effects during construction.
T_t	Thermal effects and loads during the test.
T_1	Thickness of flange of a section or plate in compression or the aggregate thickness of plates.
t	Thickness of slab; thickness of connected part; thickness of the critical part.
t_f	Flange thickness.
t_1	Flange thickness; thickness of circular section.
t_w	Web thickness.
V_h	Total horizontal shear to be resisted by connectors.
V_h'	Total horizontal shear to be resisted by connectors in providing partial composite action.
WC	Load effects due to the severe wind specified for the plant.
W_t	Loading effect due to wind-induced missiles generated by extreme wind and specific to the site.
w_r	Average width of concrete rib or haunch.
Y	Ratio of yield stress of web steel to yield stress of stiffener steel.

Y_j	Jet impingement load on a structure generated by a design basis accident.
Y_m	Missile impact load on a structure, such as pipe whip generated by design basis accident.
Y_r	Loads on the structure generated by the reaction of the broken high-energy pipe during design basis accident.
Z_{eff}	Effective section modulus.
Z_s	Section modulus of steel beam referred to flange where the stress is being computed.
Z_{sb}	Section modulus of steel beam referred to its bottom flange.
Z_{tr}	Section modulus of transformed composite section referred to its bottom flange, based on the permitted maximum effective width of the concrete flange.
Z_p	Plastic modulus of the section.
	Coefficient in the determination of minimum thickness of web; coefficient in the determination of fastener hole distance; coefficient of linear thermal expansion.
ϵ_{sc}	Allowable strain in compression.
ϵ_{st}	Allowable strain in tension.
$\epsilon_{u,y}$	Strain at ultimate and yield stress levels respectively.
σ_a	Computed axial stress.
σ_{ac}	Allowable axial compressive stress.
$\sigma_{ac,cal}$	Calculated axial compressive stress.
σ_{at}	Allowable axial tensile stress.

- $a_{t,cal}$ Calculated axial tensile stress.
- b Allowable bending stress.
- b_c Allowable bending compressive stress.
- b_{cx} Allowable bending compressive stress about x -axis.
- $b_{cx,cal}$ Calculated bending compressive stress about x -axis.
- b_{cy} Allowable bending compressive stress about y -axis.
- $b_{cy,cal}$ Calculated bending compressive stress about y -axis.
- b_s Allowable bending stress in slab base.
- b_t Allowable bending tensile stress.
- b_{tx} Allowable bending tensile stress in extreme fibre for bending about x -axis.
- $b_{tx,cal}$ Calculated bending tensile stress in extreme fibre for bending about x -axis.
- b_{ty} Allowable stress in extreme fibre for bending about y -axis.
- $b_{ty,cal}$ Calculated bending tensile stress in extreme fibre for bending about y -axis.
- c Allowable direct compressive stress in concrete.
- p Allowable bearing stress in a member.
- v Allowable shear stress.
- v_a Allowable average shear stress.
- $v_{,cal}$ Calculated average web shear stress.

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1. INTRODUCTION

1.1 General

- 1.1.1 The requirements for design/detailing of steel structures important to safety of Nuclear Power Plants (NPP) are different in some respects as compared to those of normal conventional structures. The Indian Standard (IS) Codes do not cover the special features related to safety demand of NPP structures. Keeping this in view, generally, the design standards for steel structures important to safety of NPP, used in other countries, e.g. USA, Canada, etc. were adopted along with IS codes and IAEA documents for the engineering of previous Indian NPP. One vital question that arose was how to assure the compatibility of the Indian material used in construction with the design specification of codes used by other countries. The second important point was the integration of Indian practice of design and construction with the codal requirements of other countries. In 1990, the AERB therefore decided to prepare its own document for the design, fabrication and erection of steel structures important to the safety of Indian NPPs.

AERB constituted the Code Committee for Civil and Structural Engineering (CCCSE) for this purpose. CCCSE was requested to prepare design standards, guides and manuals considering information on design specification of NPP structures which are acceptable to regulatory authorities of other countries. In the preparation of this document, codal provisions for the engineering of structures important to safety of NPP, used in other countries and acceptable to their regulatory authorities, were examined with respect to the need felt in India.

The present document may be viewed as the Indian design standard for steel structures important to safety of nuclear facilities. To prepare this standard as per the scope mentioned above, CCCSE derived assistance from the following codes/standards:

- (a) ANSI (1984), American National Standard : Nuclear Facilities - Steel Safety Related Structures for Design, Fabrication and Erection - ANSI/AISC N690-1984,

- (b) ASME (1992), Code for Concrete Reactor Vessels and Containments, Division 2, Rules for Construction of Nuclear Power Plant Components, Section III, (ACI Standard 359-92), The American Society of Mechanical Engineers, New York,
- (c) AISC (1989), Manual of Steel Construction, Allowable Stress Design, American Institute of Steel Construction Inc., Chicago, Illinois,
- (d) BIS (1984), Indian Standard Code of Practice for General Construction in Steel, Second Revision, IS 800, Indian Standards Institution (Bureau of Indian Standards), New Delhi.

1.1.2 The special functional and safety requirements of steel structures important to safety of NPP call for design criteria over and above those for conventional structures. The safety standard for Civil Engineering Structures Important to Safety of Nuclear Facilities (AERB/SS/CSE) describes the philosophy, safety design approach, and design requirements of civil engineering structures important to safety. This standard aims at describing the design criteria and methodology for the design, fabrication, and erection of steel structures and liners important to safety of nuclear facilities in line with the stipulations of AERB/SS/CSE.

1.2 Safety Design Basis

- 1.2.1 The goal of nuclear safety, also important for safe design, is to protect the site personnel, the public and the environment by establishing and maintaining an effective safeguard against radiological hazard.
- 1.2.2 The structural system of steel structures important to safety shall be so designed that it serves the needs of safety function in two ways. It supports, houses and provides a controlled environment for safe operation of plants, systems and equipment such that no fault can occur due to the effects of site/plant-specific Postulated Initiating Events (PIEs) which might otherwise have caused release of activity. Secondly, given a condition, the activity beyond the structural boundary of the building is within prescribed limits under normal operating conditions and within acceptable limits during and following an accident condition.

- 1.2.3 Design bases of civil engineering steel structures important to safety shall specify the necessary capabilities of plant structures to cope with the specified range of operational states so as to maintain a prescribed limit. The bases shall also specify similar capabilities with respect to accident conditions so as to maintain an acceptable limit.
- 1.2.4 Steel structures important to safety shall be designed, fabricated and erected to perform the intended safety and service functional roles. (ref. section 2.3 of AERB/SS/CSE).
- 1.2.5 Design criteria of steel structures important to safety shall be derived from the above. These criteria are divided into the following categories:
- (a) structural strength;
 - (b) serviceability; and
 - (c) shielding requirement (if any).
- 1.2.6 Design bases of steel structures important to safety shall be developed considering their safety, seismic and design classifications as well as their quality assurance requirements.
- 1.2.7 All steel structures important to safety shall be classified depending on the safety functions required to be performed by the structures satisfying the provisions of sections 2.3 and 2.4 of AERB/SS/CSE. The design conditions and respective load combinations are specified on the basis of their classifications. The summary of classifications of civil engineering structures and the corresponding design conditions with load combinations are given in Table 1.1.

1.3 Scope

- 1.3.1 This standard describes methods for implementing the requirements of design, fabrication and erection of design class DC3 civil engineering steel structures and liners important to safety. Design class DC3 structures (ref. cl. 2.4.10 of AERB/SS/CSE) include internal structures of reactor buildings, auxiliary buildings and other structures important to safety.

- 1.3.2 This standard covers design class DC3 steel structures (carbon steel) which are parts of the nuclear safety-related systems or which support, house or protect nuclear safety-related systems or components, the failure of which would impair the safety-related functions of these systems or components.
- 1.3.3 This document covers carbon steel structural materials. If a material other than carbon steel is used or required to be used then para 1.7 should be referred to.
- 1.3.4 Besides steel structures and liners, this standard also covers composite structures of steel and concrete.
- 1.3.5 Reactor vessels or calandria vaults excluding liners, containment structures, equipment, components and systems, are outside the scope of this standard.
- 1.3.6 Safety assessment of other structures outside the plant area and site area whose performance influence the safety of the plant shall be carried out following the stipulations of this standard.
- 1.3.7 The data and relationship in this document are given only for guidance. The designer should generate data whenever required and it is his responsibility to validate the same.
- 1.3.8 This standard has been developed primarily for stationary thermal nuclear power plant. However, the provision of this standard may be found useful for various nuclear installations, failure of which may cause a radiological hazard in public domain.
- 1.3.9 For galvanised steel structures, provisions of this standard may be modified suitably to cater to special requirement of galvanising as per relevant Indian standard codes.

Use of the Standard

- 1.3.10 This standard shall be used in conjunction with "Safety Standard for Civil Engineering Structures Important to Safety of Nuclear Facilities" (AERB/SS/CSE) and other relevant AERB safety codes, standards and guides, and the Indian Standards published by the Bureau of Indian Standard (BIS) .

TABLE 1.1: SUMMARY OF CLASSIFICATIONS, DESIGN CONDITIONS AND LOAD COMBINATIONS FOR CIVIL ENGINEERING STRUCTURES

Sl. No.	Design Class	Safety Class	Seismic Category	Quality Requirement	Design ^(1,7) Conditions	Load ⁽²⁾ Combinations
1.	DC1	1	1	chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
					Abnormal	LC3, LC4, LC5, LC6
2.	DC2	2	1	chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
					Abnormal	LC3, LC4, LC5, LC6
3.	DC3	2,3	1	chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
					Abnormal	LC3, LC4, LC5, LC6 ⁽³⁾
		3 ⁽⁴⁾	2	chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
					Abnormal	LC4, LC5
		4 ⁽⁵⁾	2	chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
					Abnormal	LC4
4.	DC4	NNS ⁽⁶⁾	3			

Note:

- (1) Ref. cl. 3.2.2 of AERB/SS/CSE or cl. 2.1.1 of this standard,
- (2) Ref. cl. 3.5.5 of AERB/SS/CSE,
- (3) This load combination is applicable only for internal structures of reactor building,
- (4) Structures which do not perform the safety functions associated with supporting the core cooling systems and other systems related to safe shutdown of reactor or prevent/mitigate the consequences of accident which could result in potential off-site exposure comparable to relevant AERB guidelines. This class also includes structures of nuclear facilities with limited radioactive inventory whose functioning shall be maintained in the event of the design basis ground motion,
- (5) Structures of nuclear facilities with limited radioactive inventory whose loss of function may be permitted but should be designed against collapse in the event of the design basis ground motion. Lower safety level for load combinations LC2 and LC4 (smaller load factors for limit state design or plastic design and higher allowable stresses in allowable stress design method) is allowed for these structures,
- (6) Non-nuclear services not important to safety should meet the design requirements as per relevant national standard engineering practices,
- (7) Design requirements should be as per the relevant AERB standards/guides (ref. 3.5 of AERB/SS/CSE).

The provision contained in this standard will have precedence over the corresponding provision given in the relevant IS codes and standards.

1.4 Quality Assurance

1.4.1 Complete quality assurance programme for design and construction of steel structures important to safety shall be developed in line with chapter-9 of AERB/SS/CSE.

1.4.2 Design Documentation

- (a) The analysis and design of the structure shall be well documented and prepared, and shall include reasonableness and basis of all assumptions made in the analysis and design methods. The source of input data used in the calculations should be adequately referred so that all data are readily traceable. The design documents should include sketches, analysis and design assumptions, input data, output results and other items as may be deemed fit,
- (b) The design shall be independently checked and certified by competent persons as per safety importance of component/system,
- (c) Model tests, if performed, should be suitably documented,
- (d) Structural drawings, details, and specifications for all design class DC3 steel structures, shall be approved and signed by the authorised engineer and retained by the owner, as a permanent record. The design drawings, shop drawings and specifications shall be prepared complying with the provisions of section 1.5 of this standard.

1.4.3 Non-destructive Testing

- (a) Welds requiring non-destructive testing shall conform to the provisions of the AERB Safety Guide (AERB/SG/CSE-3) on Quality Assurance for Civil Engineering Construction Important to Safety,
- (b) When non-destructive testing is required, the categories of welds to be examined, the extent of examination of each category, and the method or methods of testing shall be specified,

- (c) Well laid-out procedures shall be prepared for non-destructive testing of welds. The procedure conforming to requirements stated in the specification shall also include acceptance criteria.

1.5 Plans (Design Drawings), Specifications, Drawings (Shop Drawings)

1.5.1 General

Plans, drawings and other documents as necessary shall be prepared according to latest revision/editions of the Indian Standards IS:10711 through IS:10715, IS:10720 and IS:962 or other procedures acceptable to AERB.

Plans and Specifications

- 1.5.2 The plans and typical details shall contain a complete design with sizes, sections and relative locations and orientations of various members.

The plans and specifications shall highlight the following:

- (a) Plans for structural elements or components shall indicate the type of material, type of construction (cl. 2.1.7), special fabrication and erection requirements, if any, location of working points for fabrication and offset dimensions,
- (b) All plans shall be supplemented by such data concerning the assumed loads, shears, moments and axial forces to be resisted by all members and their connections, as may be required for proper preparation of shop drawings or for preventing failure during construction,
- (c) For members designed from fatigue considerations, load information including the design value of number of cycles should be furnished appropriately,
- (d) Where joints are to be assembled with high-strength bolts and are required to resist shear between the connected parts, the plans shall indicate the type of connections to be provided, namely, friction or bearing,

- (e) The construction specification shall include applicable codes, material specifications, surface preparation and protective coating requirements, requirements of fabrication and/or erection, welding and bolting requirements, tests and inspection requirements, requirements for shop drawings, materials handling, shipping, storage and records.

1.5.3 Shop Drawings

Shop drawings, giving complete information necessary for fabrication of component parts of the structure, shall be prepared in advance. Shop drawings should be prepared in conformity with latest editions of relevant Indian standards and good engineering practices. Key diagram should be provided on each drawing.

1.5.4 Welding

Adequate information shall be provided on plans and on shop drawings for welding details. Welding symbols used on plans and shop drawings, shall be according to those given in Indian standard IS:813. For cases not covered by IS:813, other welding symbols may be used, provided a complete explanation thereof is shown on plans or drawings.

1.6 Construction

1.6.1 Fabrication

The fabrication details should conform to the AERB manual AERB/SM/CSE-3 or equivalent codes/guides/standards. Unless otherwise specified, tolerances for fabrication of steel structures shall conform to the provisions of the Indian Standard (IS: 7215). For general guidance on fabrication by welding, reference may be made to the Indian Standard (IS: 9595), except as indicated in AERB/SM/CSE-3.

1.6.2 Shop Painting

- (a) Painting shall conform to the AERB manual (AERB/SM/CSE-3) and appropriate provisions of IS:1477 (Parts 1 and 2),

- (b) Parts to be encased in concrete should not be painted to ensure proper bond between steel and concrete,
- (c) Contact surfaces within slip critical connection shall not be painted.

Erection

- 1.6.3 Tolerances for erection of steel structure should be in accordance with the specification approved by the engineer.
- 1.6.4 Whenever piles of material, erection equipment or other loads are supported during erection, adequate provisions shall be made to take care of stresses resulting from such loads.

1.7 Approval of Special Design and Construction Techniques

- 1.7.1 For novel or unproven methods of analysis, design and construction or for use of special construction materials (not covered in this standard), prior approval shall be obtained from AERB after ensuring comparable safety.

1.8 Structure of the Document

- 1.8.1 This standard comprises 8 chapters, 6 appendices and 1 annexure. Each chapter is divided into a number of sections and further sub-divided into paragraphs or clauses.
- 1.8.2 Chapter-2 : General design requirements
- Chapter-3 : Special design requirements
- Chapter-4 : Allowable stress design method
- Chapter-5 : Additional requirements for built-up members and column bases
- Chapter-6 : Composite construction
- Chapter-7 : Plastic design
- Chapter-8 : Steel liners

- 1.8.3 Appendix-A : Steel standards
- Appendix-B : Strength evaluation of existing structures
- Appendix-C : Requirements of welded construction for liner plates, penetrations and anchorage system
- Appendix-D : Material properties of structural steels
- Appendix-E : Limiting vertical deflection of crane runway girders
- Appendix-F : Loading conditions, material type and location, and maximum stress range for fatigue design
- 1.8.4 Annexure-I : Consideration of impulsive and impactive loadings.

2. GENERAL DESIGN REQUIREMENTS

2.1 General Requirements

2.1.1 Structures shall be designed for all loads (due to normal and abnormal conditions) likely to act on it throughout its life for both of the design conditions given below (ref. cl. 3.2.2 of AERB/SS/CSE):

- (a) Normal design condition which includes load combinations LC1 and LC2, i.e. normal and severe environmental load combinations respectively; and
- (b) Abnormal design condition which includes load combinations LC3, LC4, LC5, and LC6, i.e. extreme environmental, abnormal, abnormal severe environmental and abnormal extreme environmental load combinations respectively.

2.1.2 Two design methods, allowable stress design method (ASDM) and plastic design method (PDM) are described in this standard. These methods shall be used as follows:

- (a) Allowable stress design method shall be used for design of main structural elements of new plant,
- (b) Plastic design method is acceptable for design of secondary structures (which are not basically important to safety, and collapse or excessive deformation of which may jeopardise the function of structures, systems and components important to safety), and for purpose of retrofitting of main structural members provided the provisions of chapter-8 of AERB/SS/CSE are satisfied,
- (c) Members subject to heavy impact and fatigue, over 20,000 cycles, shall not be designed on the basis of plastic theory.

2.1.3 (a) The design shall satisfy the strength and serviceability (such as limitations on deflection, settlement, amplitude, etc.) requirements specified for both the above design conditions as indicated in cl. 2.1.1 above irrespective of design method adopted as per cl. 2.1.2 above,

- (b) All internal forces and joint displacement shall be considered in the design of member section for strength under the action of seismic and other dynamic loads irrespective of the disposition of the member as beam, column, truss element, etc.
- 2.1.4 Any component important to safety, acting as a support or restraint, where rigidity considerations may be of primary importance, the design may be governed by stiffness requirements rather than strength considerations.
- 2.1.5 The DC3 structure may be of different safety classification. The same level of safety (load factor, strength factor, factor of safety, etc.) is provided for safety classes 2 and 3 steel structures in the design excepting the provision of cl.2.1.6 below. However variable level of safety in the design of DC3 structures depending on classification is acceptable provided safety equation of Annexure-I of AERB/SS/CSE is satisfied.
- 2.1.6 The following provisions are applicable to safety class 4 structures whose failure will release activity below the limit prescribed for buildings and structures associated with waste management facilities, D₂O upgrading plants, etc.:
- (a) These structures should be designed for load combinations LC1 and LC2,
 - (b) Permissible stresses (ref. Table 4.2) in allowable stress design method may be increased to 1.33S for all LC2 load combination without temperature effects. For plastic method the load factor could be taken as 1.3 for LC2 load combination Nos. 5&6 (ref. Table 7.1).

Types of Construction

- 2.1.7 Three basic types of construction and associated design procedures are permissible. Each type governs in a specific manner the size of members and the types and strength of their connections.
- (a) Type 1 : Commonly designated as “rigid frame” or continuous frame, this type assumes that beam-to-column connections have sufficient rigidity to hold virtually unchanged original angles between these connecting members,

- (b) Type 2 : Commonly designated as “simple frame” or unrestrained, pin-jointed frame, this type assumes that insofar as transverse loading is concerned, the ends of beams and girders are connected for shear transfer only, and these ends are free to rotate,
- (c) Type 3 : Commonly designated as “semi-rigid frame” or partially restrained frame, this type assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigid and flexible cases in types 1 and 2.

2.1.8 Design of all connections shall be consistent with assumptions as to the type of constructions.

2.1.9 Simple and Continuous Spans

The effective length of beams, girders and trusses designed on the basis of simple and continuous spans is equal to the distance between centres of supports.

2.1.10 End Connections

When designing the end connections the allowable stresses in cl.4.4 shall not be exceeded, except that some non-elastic but self-limiting deformation of a part of the connection may be permitted when this is essential to avoid overstressing of fasteners.

2.2 Materials

2.2.1 Unless otherwise specified all structural grade materials, used in construction, coming under the purview of this standard shall, conform to the relevant standards listed in Appendix-A.

2.2.2 In addition to satisfying the general requirements, the specification of the material of those structures or structural components, which may be subject to impact loads, (e.g. liners, jet shields, pipe restraint supports, pipe-whip impact barriers, etc.) shall be examined and supplemented by such requirements as product analysis, impact test, non-destructive examination, intergranular corrosion test, zero-ductility transition temperature test, etc. whenever felt necessary.

- 2.2.3 Laminations of material should be within acceptable limit and should meet QA requirements.
- 2.2.4 The dimensions, form, tolerances of all rolled shapes and other members or plate, wherever available, shall conform to appropriate standards.
- 2.2.5 For welded structures, only weldable quality structural steel shall be used.
- 2.2.6 For design values of the material properties of structural steels, suggested values are given in Appendix-D.

2.3 Deflections, Vibrations and Ponding

- 2.3.1 Beams and girders supporting floors and roofs shall be proportioned with due regard to deflection and/or vibration produced by design loads.

Limiting Vertical Deflection

- 2.3.2 The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads causing deflection. The maximum deflection for beam supporting brick wall should not exceed $1/325$ of the span. Maximum allowable deflection for other floors, trusses, girders, purlins, side girts, etc. shall be determined from service considerations.
- 2.3.3 For limits on maximum vertical deflection of crane runway girders, suggested values are given in Appendix-E.

Limiting Horizontal Deflection

- 2.3.4 The gantry girder, support brackets, etc. which support any other member without adequately designed anchorages, shall be designed such that the supported members will not get jammed or fall off their supports for any and all of the load combinations in cl.4.2.9.
- 2.3.5 Unless otherwise specified, the limiting horizontal deflection of crane girder due to transverse surge shall not be more than $L/2400$.

Ponding

2.3.6 Unless a roof surface is provided with sufficient slopes toward points of free drainage or adequate individual drains to prevent accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions, except as follows:

- (a) The roof system shall be considered stable and no further investigation will be needed if,

$$C_p + 0.9 C_s \leq 1.0, \quad (2.1)$$

and

$$I_d \geq 4.0 \times 10^{-11} S^4, \quad (2.2)$$

where

$$C_p = 2.0 \times 10^{-11} [(L_s L_p^4)/I_p], \quad (2.3)$$

$$C_s = 2.0 \times 10^{-11} [(S L_s^4)/I_s], \quad (2.4)$$

L_p = column spacing in direction of girder (length of primary members) (cm),

L_s = column spacing perpendicular to direction of girder (length of secondary members) (cm),

S = spacing of secondary members (cm),

I_p = moment of inertia of primary members (cm⁴),

I_s = moment of inertia of secondary members (cm⁴), and

I_d = moment of inertia of the steel deck supported on secondary members (cm⁴ per cm).

- (b) For trusses and steel open joists, the moment of inertia I_s shall be decreased by 15% when used in the above formulae. A steel deck shall be considered a secondary member when it is directly supported by primary members,

- (c) Total bending stress due to all loads, including ponding but excluding extreme environmental loading shall not exceed $0.80f_y$ for both primary and secondary members.

2.4 Stability of Overall Structures and Slender Compression Elements

Stability of Overall Structures

- 2.4.1 The stability of the overall structure shall satisfy the provisions of AERB safety guide on Geotechnical Aspects for Buildings and Structures Important to Safety of Nuclear Facilities (AERB/SG/CSE-2). In cases where dead load provides the restoring effect, only 0.9 times the dead load shall be considered. Restoring moments due to imposed loads shall be ignored. To ensure stability at all times, account shall be taken of probable variations in dead load during construction, repair or other temporary measures. The effect on load due to deflected or deformed shape of the structure or of individual elements of the lateral load-resisting systems shall be considered.

Stability of Braced Frames

- 2.4.2 In trusses and in those frames where lateral stability is provided by adequate attachment to diagonal bracing, to shear walls, to an adjacent structure having adequate lateral stability, or to floor slabs or roof decks secured horizontally, walls or bracing systems parallel to the plane of the frame, the effective length factor k for the compression members shall be taken as unity, unless analysis allows use of a smaller value.

Stability of Unbraced Frames

- 2.4.3 In frames where lateral stability is dependent upon bending stiffness of rigidly connected beams and columns, the effective length kl_c of compression members shall be determined by a rational method, and it shall not be less than the actual unbraced length.

Maximum Slenderness Ratios

- 2.4.4 All individual members of the structure, which have been designed for their dead and imposed loads, wind, earthquake or other loads, in conformity with

permissible stresses stipulated in this standard, shall be deemed to be adequately covered for elastic stability. Unless specified otherwise, in determining the slenderness ratio of an axially loaded compression member, the length shall be taken as its effective length kl_c , and r as the corresponding radius of gyration. For different members, the slenderness ratios (kl_c/r) shall be limited to:

(a)	a member carrying compressive loads resulting from dead and imposed loads primary member	-	180
(b)	a tension member in which a reversal of stress occurs due to loads other than extreme environmental loads	-	180
(c)	a member subjected to compression forces resulting from wind/earthquake forces, provided the deformation of such a member does not adversely affect the stress in any part of the structure	-	250
(d)	compression flange of a beam	-	300
(e)	a member normally acting as a tie in a roof truss or a bracing system but subject to possible reversal of stress resulting from the action of extreme environmental forces	-	350
(f)	tension members (other than pretensioned members)	-	400

Effective Length of Compression Members

2.4.5 The slenderness ratio of a compression member is the ratio of the effective length kl_c to the appropriate radius of gyration r , where l_c is the unsupported length, and k the effective length factor. In the absence of a detailed analysis, k can be obtained as stated below.

- | | | |
|-----|---|------------|
| (a) | compression member effectively held in position and restrained against rotation at both ends | $k = 0.65$ |
| (b) | compression member effectively held in position at both ends and restrained against rotation at one end | $k = 0.80$ |
| (c) | compression member effectively held in position and restrained at both ends, but not restrained against rotation | $k = 1.00$ |
| (d) | compression member effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position | $k = 1.20$ |
| (e) | compression member held in position and restrained against rotation at one end, and at the other end partially restrained against rotation but not held in position | $k = 1.50$ |
| (f) | compression member held in position at one end but not restrained against rotation and at the other end restrained against rotation but not held in position | $k = 2.00$ |
| (g) | Compression member held in position and restrained against rotation at one end, but not held in position nor restrained against rotation at the other end | $k = 2.00$ |

The sketches elaborating the above boundary conditions are given in Fig. 2.1.

Width Thickness Ratios

- 2.4.6 Most structural members and fabrications are composed of connected elements that, for purposes of analysis and design, may be treated as plates. When a plate is subjected to direct compression, bending or shear, the plate may buckle locally before the member as a whole and becomes unstable. This will have the effect of lowering the load-resisting capacity of the structural member. It will thus be necessary to prevent this premature buckling by either limiting the width or increasing the thickness of the connected elements or plates.

Unstiffened Elements Under Compression

- 2.4.7 Unstiffened (projecting) compression elements are those with one free edge parallel to the direction of compression stress. The width of an unstiffened element is the dimension measuring from the free edge to the first row of fasteners or welds. The width of legs of angles, channels, zee flanges, and stems of tees shall be taken as the full nominal dimension. The width of flanges of I- and H-shape members and tees is half the full nominal width. The thickness of sloping flange is measured at a section half-way between a free edge and the corresponding face of the web.

Requirements on Width-Thickness Ratios

- 2.4.8 Unstiffened elements subject to axial compression or compression due to bending shall be considered as fully effective when the ratio of width to thickness is not greater than those shown in Table 2.1 and applicable limiting proportions for channels and tees are shown in Table 2.2.
- 2.4.9 Steel sections are classified as compact, non-compact and slender element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios for its compression elements must not exceed the applicable limiting width-thickness ratios given in Table 2.1. Steel sections that do not qualify as compact are classified as non-compact if the width-thickness ratios of compression elements do not exceed the values shown for non-compact in Table 2.1. If the width-thickness ratios of any compression element exceed the limiting values of non-compact element, the section is classified as a slender element section.

2.4.10 For unstiffened elements which are supported along only one edge, parallel to the direction of the compression force, the width b shall be taken as follows:

- (a) For flanges of I-shaped members and tees, b is half the full nominal width,
- (b) For legs of angles and flanges of channels, b is the full nominal dimension,
- (c) For plates, b is the distance from the free edge to the first row of fasteners or line of welds,
- (d) For stems of tees, b is taken as the full nominal depth D .

2.4.11 For stiffened elements, i.e., supported along two edges parallel to the direction of the compression force, the width b shall be taken as follows:

- (a) For webs of rolled, built-up or formed sections, b is taken as the clear distance between flanges, h_c ,
- (b) For webs of rolled, built-up or formed sections, b is taken as the full nominal depth D ,
- (c) For flange or diaphragm plates in built-up sections, b is the distance between adjacent lines of fasteners or lines of welds,
- (d) For flanges of rectangular hollow structural sections, b is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the flat width may be taken as the total section width minus three times the thickness.

For tapered flanges of rolled sections, the thickness is the nominal value half-way between the free edge and the corresponding face of the web.

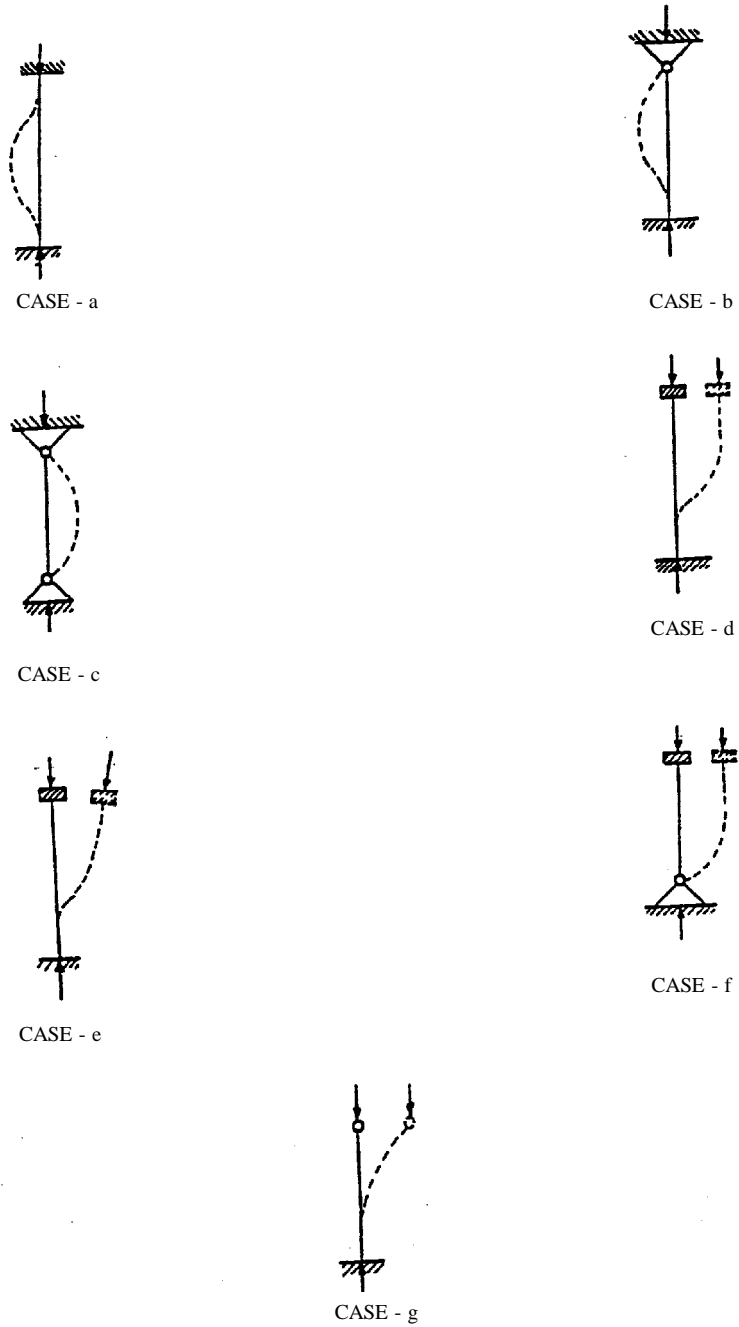


Fig. 2.1: ILLUSTRATION OF SUPPORT CONDITIONS FOR EFFECTIVE LENGTH FACTOR

Local Buckling - Slender Compression Elements

2.4.12 Unstiffened compression elements, whose width-thickness ratios exceed the applicable limits given in Table 2.1, shall be subject to a reduction factor Q_s in the allowable stresses. The value of Q_s shall be determined by equations (2.5) to (2.10), as applicable. When such elements comprise compression flange of a flexural member the maximum allowable bending stress shall neither exceed $0.6 f_y Q_s$ for the normal and severe load combinations, and $0.9 f_y Q_s$ for extreme and abnormal load combinations, nor the applicable values provided in chapter 4. The allowable stresses of axially loaded compression members shall be modified by appropriate reduction factor Q_s determined from the following expressions:

(1) For single angles;

when $200/(f_y)^{1/2} < (b/t_1) < 400/(f_y)^{1/2}$

$$Q_s = 1.340 - 1.7 \times 10^{-3} (b/t_1)(f_y)^{1/2}, \quad (2.5)$$

and when $b/t_1 \geq 400 / (f_y)^{1/2}$

$$Q_s = 1.07 \times 10^5 / [f_y (b/t_1)^2]. \quad (2.6)$$

(2) For angles or plates projecting from columns or other compression members, and for compression flanges of girders:

when $250/(f_y/k_c)^{1/2} < (b/t_1) < 500/(f_y/k_c)^{1/2}$

$$Q_s = 1.293 - 1.17 \times 10^{-3} (b/t_1) (f_y/k_c)^{1/2}, \quad (2.7)$$

and when $b/t_1 \geq 500 / (f_y/k_c)^{1/2}$

$$Q_s = 1.806 \times 10^5 k_c / [f_y (b/t_1)^2]. \quad (2.8)$$

For k_c , see note (e) of Table 2.1.

(3) For stems of tees: 22

**TABLE 2.1 : LIMITING WIDTH - THICKNESS RATIOS FOR
COMPRESSION ELEMENTS**

Sr. No.	Description of Elements	Width Thickness Ratio	Limiting Width-Thickness Ratios	
			Compact elements	Non-compact elements ^(c)
1.	Flanges of I-shapes rolled beams and channels in flexure. ^(a)	b/t_1	$170/(f_y)^{1/2}$	$250/(f_y)^{1/2}$
2.	Flanges of I-shaped hybrid or welded beams in flexure. ^(a)	b/t_1	$170/(f_y)^{1/2}$	$250/(f_y)^{1/2}$ if $h_c/t_1 \leq 70$ $250/(f_y/k_c)^{1/2(e)}$ if $h_c/t_1 > 70$
3.	Compression flanges of plate girders, angles or plates projecting from girders, built-up columns or other compression members.	b/t_1	NA	$250/(f_y)^{1/2}$ if $h_c/t_1 \leq 70$ $250/(f_y/k_c)^{1/2(e)}$ if $h_c/t_1 > 70$
4.	Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression; ^(d) flange cover plates and diaphragm plates between lines of fasteners or welds.	b/t_1	$500/(f_y)^{1/2}$	$625/(f_y)^{1/2}$

**TABLE 2.1 : LIMITING WIDTH - THICKNESS RATIOS FOR
COMPRESSION ELEMENTS (contd.)**

Sr. No.	Description of Elements	Width Thickness Ratio	Limiting Width-Thickness Ratios	
			Compact elements	Non-compact elements ^(c)
5.	Unsupported width of cover plates perforated with succession of access holes ^(b) .	b/t_1	NA	$833/(f_y)^{1/2}$
6.	Angles or plates projecting from girders, built-up columns or other compression members; compression flange of plate girders.	b/t_1	NA	$250/(f_y)^{1/2}$
7.	Legs of single angle struts; legs of double angle struts with separator; unstiffened elements supported along one edge.	b/t_1	NA	$200/(f_y)^{1/2}$
8.	Stems of tees	D/t_1	NA	$333/(f_y)^{1/2}$
9.	Other uniformly compressed stiffened elements i.e. elements supported along two edges.	b/t_1 or h_c/t_w	NA	$665/(f_y)^{1/2}$

TABLE 2.2 : LIMITING PROPORTIONS FOR CHANNELS AND TEES-UNSTIFFENED COMPRESSION ELEMENTS

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Web or Stem Thickness
Built-up or rolled channels	≤ 0.25	≤ 3.0
	≤ 0.50	≤ 2.0
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

$$Q_s = 1.908 \quad 2.72 \times 10^{-3} (b/t_1) (f_y)^{1/2}, \quad (2.9)$$

and when $b/t_1 \geq 460 / (f_y)^{1/2}$,

$$Q_s = 1.38 \times 10^5 / [f_y (b/t)^2]. \quad (2.10)$$

Stiffened Compression Elements

2.4.13 Stiffened compression elements are those having lateral support along both edges parallel to the direction of compression stress. The width of such elements shall be taken as the distance between nearest lines of fasteners or welds, or between roots of flanges in rolled sections.

Requirements on Width-Thickness Ratios

2.4.14 (1) Stiffened elements of non-circular tubular sections subject to axial compression or to uniform compression due to bending as in the case of the flange of a flexural member (except webs of flexural members) which are covered by the provisions of cl.5.3.4, shall be considered as fully effective when the width-to-thickness ratio is not greater than those shown in Table 2.1,

- (2) Except in the case of perforated cover plates, when the actual width-to-thickness ratio exceeds the values shown in Table 2.1, effective width, b_e shall be used in computing the flexural design properties of the section containing the element and the permissible axial stress, except that the ratio b/t_1 need not be taken as less than the applicable value permitted in Table 2.1.

Effective Width-Stiffened Compression Elements

- 2.4.15 (1) When the width-thickness ratio of a uniformly compressed stiffened element (except perforated cover plates) exceeds the non-compact limit given in cl. 2.4.14, effective width b_e shall be used in computing the design properties of the section containing the element. The ratio, b/t_1 , however, need not be taken as less than the applicable value permitted in cl. 2.4.14.

- (a) For flanges of square and rectangular sections of uniform thickness:

$$b_e = [665t_1/(\sigma_c)^{1/2}] [1 - 132/\{(b/t_1)(\sigma_c)^{1/2}\}] \leq b \tag{2.11}$$

- (b) For other uniformly compressed elements:

$$b_e = (665t_1/(\sigma_c)^{1/2}) [1 - 116/\{(b/t_1)(\sigma_c)^{1/2}\}] \geq b \tag{2.12}$$

where,

σ_c = computed compressive stress (axial plus bending) based on section properties in cl. 2.4.16 in MPa except as otherwise provided for extreme and abnormal loads,

b = actual width of a stiffened compression element as defined in cl.2.4.13,

t_1 = thickness.

- (2) When allowable stresses are increased for abnormal design conditions in accordance with Table 4.2, the effective width b_e shall be determined on the basis of 0.75 times the stresses caused by loading effects under abnormal design conditions.

Sectional Properties

2.4.16 The sectional properties shall be determined by conventional methods, using the full cross-section of the member except that in computing the moment of inertia and section modulus of flexural members, with respect to the axis of bending under consideration, the effective width of stiffened compression elements parallel to the axis of bending and having a width-thickness ratio in excess of the applicable limit given in cl. 2.4.14, rather than the actual width, shall be used, and the axis of bending shall be located accordingly, except that for sections otherwise symmetrical. The properties may conservatively and more easily be computed using a corresponding effective area on the tension side of the neutral axis as well. The portion of area that is neglected in arriving at the effective area shall be located at and symmetrically about the centre line of the stiffened element to which it applies. The stress due to axial loading and the radius of gyration r shall be computed on the basis of actual cross-sectional area. However, the allowable axial stress σ_{ac} shall be subjected to the form factor.

$$Q_a = (\text{effective area})/(\text{actual area}), \quad (2.13)$$

where,

$$(\text{effective area}) = (\text{actual area}) - [(b - b_e)t]. \quad (2.14)$$

2.5 Minimum Thickness of Members

2.5.1 Steelwork Directly Exposed to Weather

Where steel is directly exposed to weather and fully accessible for cleaning and repainting, the thickness shall be not less than 6 mm. Where the steel is directly exposed to weather and not accessible for cleaning and repainting, the thickness shall be not less than 8 mm. These provisions do not apply to webs of rolled sections.

2.5.2 Steelwork Not Directly Exposed to Weather

The thickness of steel in main members, not directly exposed to weather, shall be not less than 6 mm. The thickness of steel in secondary members, not directly exposed to weather, shall be not less than 4.5 mm.

2.5.3 Steelwork in Special Environments

The minimum thickness of light structural steel work or of sealed box section or of steel work in which special provision against corrosion, such as use of special coatings or paints has been made, or of steel work exposed to highly corrosive fumes or vapour or saline atmosphere, shall be decided considering all important factors. However, the minimum thickness shall not be less than those specified in cl. 2.5.1 or 2.5.2 except for light structural steel work.

2.5.4 Corrosion Allowance

Suitable corrosion allowance for specified structure or structural elements like tanks, chimneys, base plates, and exposed anchor bolts shall be provided in addition to designed thickness. The minimum value of such allowance shall be 2.0 mm.

2.6 Connections

2.6.1 Connections consist of connecting elements, e.g., stiffeners, gussets, angles, brackets, and fastening element, i.e. welds, bolts and rivets. These components shall be proportioned so that their design strength equals or exceeds the required strength determined by (a) structural analysis for load combinations in cl. 4.2.9; or (b) a specified proportion of the strength of connected members, whichever is appropriate. (see cl.2.1.7).

Minimum Requirements of Connections

2.6.2 Connections carrying calculated stresses, except in the case of lacing, sag bars, and girts, shall be designed to support not less than 50 Kilo-Newtons for normal design conditions.

Eccentric Connections

2.6.3 Axially stressed members meeting at a point shall have their gravity axes to intersect at a point. If not, provisions shall be made for bending stresses due to the eccentricity, both in connections and members which are eccentrically connected.

Placement of Bolts and Welds

- 2.6.4 Except as hereinafter provided, groups of bolts or welds at the end of any member transmitting axial stress into that member shall have their centres of gravity on the gravity axis of the member, unless provision is made for the effect of resulting eccentricity. Except in members subject to repeated variation in stress, (ref. section 3.3), disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single angle, double angle, and similar type members is not required. Eccentricity between the gravity axes of such members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but shall be considered in members subject to fatigue loading.

Unrestrained Members

- 2.6.5 Except as otherwise indicated by the engineer, connections of beams, girders or trusses shall be designed as flexible, and may ordinarily be proportioned for reaction shears only. Flexible beam connections shall accommodate the end rotations of such simple (unrestrained) beams. Inelastic action in the connection is permitted for such purposes.

Restrained Members

- 2.6.6 Flange to Flange Weld Connection

When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column-web stiffeners having a combined cross-sectional area, not less than that computed from formula (2.15) shall be provided whenever the calculated value of A_{st} is positive.

$$A_{st} = \{P_{bf} - f_{yc} t_w (t_f + 5K)\} / f_{yst} \quad (2.15)$$

where,

P_{bf} = computed force (in Newton) delivered by the flange or moment connection plate multiplied by 1.67 for normal design condition, and by 1.2 for abnormal design conditions,

f_{yc} = column yield stress,

t_w = column web thickness,

t_f = thickness of flange or moment connection plate, delivering concentrated force,

K = distance between outer face of column flange and web toe of its fillet, if the column is a rolled shape, or equivalent distance if the column is a welded shape,

f_{yst} = yield stress of stiffener.

Stiffeners

2.6.7 Notwithstanding the requirements of cl. 2.6.6, a stiffener or a pair of stiffeners shall be provided behind the compression flange when

$$d_c > \{10800 t^3 (f_{yc})^{0.5}\} / P_{bf}, \quad (2.16)$$

where,

d_c is the column web depth clear of fillets and t is the thickness of stiffeners.

2.6.8 A pair of stiffeners shall be provided opposite the tension flange when

$$t_f < 0.4 (P_{bf}/f_{yc})^{1/2}. \quad (2.17)$$

2.6.9 Stiffeners required by provisions of cl. 2.6.6 to 2.6.8 shall comply with the following criteria :

- (a) the width of each stiffener, in the direction of the width of the flange, plus (1/2) the thickness of the column web shall be not less than (1/3)rd of the width of the flange or moment connection plate delivering the concentrated force;
- (b) the thickness of stiffeners shall not be less than $t_f/2$;
- (c) when the delivered concentrated force occurs only on one column flange, the stiffener length need not exceed half the column depth

unless the web stiffener for full depth of column is required from other considerations; and

- (d) the weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments, if any, on opposite sides of the column and axial load.

Connection with High Shear in Column Web

2.6.10 Connections having high shear in the column web shall be suitably designed.

Fasteners or Welds for End Connections

2.6.11 Fasteners or welds for end connections of beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connection.

Fillers for Bolted Connections

2.6.12 When bolts carrying computed stress pass through fillers thicker than 6 mm, except in friction-type connections assembled with high-strength bolts, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection.

Fillers for Welded Connections

2.6.13 In welded construction, fillers, 6 mm or thicker, shall extend beyond the edges of splice plate and welded to the part on which it is fitted with sufficient weld to transmit the splice plate stress, applied at the surface of the filler as an eccentric load. The welds joining the splice plate to the filler shall be sufficient to transmit the stresses in the splice plate, and shall have sufficient length to prevent overstressing of the filler along the toe of the weld. Any filler, less than 6 mm thick, shall have its edges made flush with the edges of the splice plate, and the weld size shall be the sum of the thickness of filler plate and the size necessary to carry the splice plate stress.

Connections of Tension and Compression Members in Trusses.

2.6.14 The connections at ends of tension or compression members in trusses shall develop the force due to design load, but not less than 50% of the effective strength of the member, based on the kind of stress that governs the selection of the member (see also cl. 2.6.2).

Compression Members with Bearing Joints

2.6.15 Where columns bear on bearing plates or when columns are finished to bear at splices, there shall be sufficient bolts or welds to hold all parts securely in place. Where other compression members are finished to bear, the splice material and its bolting or welding shall be arranged to hold all parts in line and proportioned for 50% of the computed stress. All the foregoing joints shall be proportioned to resist any tension that would be developed by specified lateral forces acting in conjunction with 75% of the calculated dead load stress and no live load stress.

Combination of Welds

2.6.16 If two or more types of welds (groove, fillet, plug, slot, spot) are combined in a single joint, in order to determine the allowable capacity of the combination, the effective capacity of each type of weld shall be separately computed with reference to the axis of the group.

Bolts in Combination with Welds

2.6.17 In new work high strength bolts used in bearing type connections shall not be considered as sharing the stress in combination with welds. Welds, if used, shall be provided to carry the entire stress in the connection. In friction-type connections, high strength bolts may be considered as sharing the stress with the welds.

2.6.18 In making alternations with welds, properly tightened high strength bolts may be utilised for carrying stresses resulting from existing dead loads, and the welding need be adequate only to carry all additional stresses.

Field Connections

2.6.19 High strength bolts or welds shall be used for all field connections.

2.7 Bolts

General Requirements

- 2.7.1 Bolts made from medium carbon heat-treated steel can be used either as friction-type (with minimum ultimate strength of 800 MPa but yield stress not exceeding 80% of the ultimate strength) or as bearing-type (with minimum ultimate strength of 400 MPa but yield stress not exceeding 60% of the ultimate strength).
- 2.7.2 Friction-type bolts shall be tightened to a minimum tension equal to the proof load (the lower boundary of the proportional or elastic limit of the material) of the bolts. This shall be done with an impact wrench. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings. Alternatively, all bolts shall be installed to meet the requirements of friction bolts.
- 2.7.3 Bearing bolts can be “snug” tightened with an impact or spud wrench. From the snug point, the nuts shall be tightened at least another (1/2) to (7/6) turns, depending on the lengths and diameters of the bolts. The corresponding nut rotations from the initial tightening position, which is the position at which not less than 10% of the specified proof load of the bolt is developed, are given in Table 2.3.
- 2.7.4 The tightness of friction bolts shall be verified with torque wrenches. The tightness shall be checked initially for 10% of the bolts in each connection, with a minimum checking of two bolts in each connection. Should any of the bolts fail to meet the test of minimum torque for tightening, all bolts in that connection shall be tested. Should inadequate torque be detected in one or more bolts per connection, then all bolts erected by the same gang of workers and or using the same wrench, shall be checked for the required torque for tightening.

2.7.5 Allowable stresses for friction and bearing bolts are given in cl. 4.4.21.

High-strength Bolts

2.7.6 Use of bolts, both friction and bearing type, shall conform to Indian standards IS: 3757, IS:1367 (Part III), and IS:4000, except as modified in section 2.7.

2.7.7 When required to be tightened to more than 50% of their minimum specified tensile strength, bolts in tension and bearing-type shear connections shall have a hardened washer installed under the bolt head. Nuts shall meet the requirements as specified in IS code IS:3757 or equivalent.

TABLE 2.3 : NUT ROTATION REQUIREMENTS FOR BEARING-TYPE BOLTS

Bolt Length (Nominal)	Nut Rotation (Minimum) (in degree)
$l_b \leq 2d$	180
$2d < l_b \leq 3d$	240
$3d < l_b \leq 4d$	300
$4d < l_b \leq 8d$	360
$l_b > 8d$	420

Note : d = thread size = outer diameter of bolt

l_b = nominal length of bolt

2.7.8 Bolts with ultimate strength higher than 1175 MPa shall not be used unless impact testing performed that the bolt is not subjected to stress corrosion cracking by virtue of the fact that (a) a corrosive environment is not present; and (b) no residual stresses or assembly stresses are present and frequent sustained service loads are not experienced.

Effective Bearing Area

2.7.9 The effective bearing area of bolts, threaded parts and rivets shall be the diameter multiplied by the length in bearing, except that for countersunk bolts half the depth of the countersink shall be deducted.

Size and Use of Holes

2.7.10 The maximum sizes of holes for bolts are given in Table 2.4 except that larger holes required for tolerance on location of anchor bolts in concrete foundations may be used in column base details.

2.7.11 Standard holes shall be provided in member-to-member connections. Finger shims up to 6 mm may be introduced into slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener.

TABLE 2.4 : NOMINAL HOLE DIMENSIONS (for bolts in shear)

Diameter of Bolt (Nominal) <i>d</i> (mm)	Hole Dimensions (mm)	
	Standard (Dia)	Oversize (Dia)
12.0	13.5	15.0
16.0	17.5	20.5
20.0	21.5	24.5
22.0	23.5	26.5
24.0	25.5	30.0
28.0	29.5	33.0
≥ 30.0	<i>d</i> + 1.5	<i>d</i> + 7.5

2.7.12 For friction grip bolts, the hole sizes shall be as follows: *d*+2 mm for bolts up to 24 mm dia, *d*+3 mm for bolts above 24 mm dia.

Minimum Spacing

2.7.13 The distance (in mm) between centres of standard or oversized fastener holes shall not be less than 8/3 times the nominal diameter (preferably 3d) of the fastener, nor less than that required as in the following.

(1) For Standard Holes,

$$[(P) / (f_u t)] + (d/2),$$

where

P = force transmitted by one fastener to the critical connected part, (Newtons);

f_u = specified minimum tensile strength of the critical connected part, (MPa);

t = thickness of the critical connected part, (mm);

d = nominal diameter of bolt, (mm); and

= 2.0.

(2) For Oversized Holes,

The distance required for standard holes in the preceding plus the applicable increment C_1 in Table 2.5. The clear distance between holes shall, however, be not less than one bolt diameter.

Minimum Edge Distance

2.7.14 The distance (in mm) from the centre of a standard hole to an edge of a connected part shall be not less than the applicable value given in Table 2.6 nor $3/2 d$ nor that obtained from

$$(P) / f_u t$$

where P , f_u , t and are as defined in cl.2.7.13.

2.7.15 The distance from the centre of an oversized hole to an edge of a connected part shall be not less than that required for a standard hole, plus the increment C_2 in Table 2.7.

TABLE 2.5 : VALUES OF SPACING INCREMENT C_1

Nominal Diameter of Fastener (mm)	C_1 (mm)
≤ 22.0	3.0
24.0	4.0
≥ 27.0	6.0

TABLE 2.6 : MINIMUM EDGE DISTANCE (CENTRE OF STANDARD HOLE⁽¹⁾ TO EDGE OF CONNECTED PART)

Nominal Bolt Diameter (mm)	At Sheared Edges (mm)	At Rolled Edges of Plates, Shapes or Bars-Machine flame cut or saw cut ⁽¹⁾ (mm)
12.0	22.0	19.0
16.0	29.0	22.0
20.0	32.0	26.0
22.0	38.0 ⁽²⁾	29.0
24.0	45.0 ⁽²⁾	32.0
28.0	51.0	38.0
32.0	57.0	42.0
over 32.0	1.75 x diameter	1.25 x diameter

Notes: (1) All edge distances in this column may be reduced by 3 mm when the hole is at a location where the design stress does not exceed 25% of the allowable stress in the element,

(2) These may be 32 mm at the ends of beam connection angles.

TABLE 2.7 : VALUES OF EDGE DISTANCE INCREMENT C_2

Nominal Diameter of Fastener (mm)	C_2 (mm)
≤ 22.0	1.5
24.0	3.0
≥ 28.0	3.0

2.7.16 At end connections of beams bolted to the web of another beam or column and designed for beam shear reaction only (without use of an analysis which accounts for the effects induced by fastener eccentricity), the distance from the centre of the nearest standard hole to the end of the beam web shall be not less than

$$(P_r) / (f_u t),$$

in which P_r is the beam reaction (Newtons) divided by the number of bolts, and f_u , t and are as defined in cl. 2.7.13.

Maximum Edge Distance

2.7.17 The maximum distance from the centre of bolt to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm.

Combined Tension and Shear

2.7.18 Anchor bolts shall be designed to provide resistance to all conditions of tension and shear at the base of columns, including the net tensile components of any bending moments which may result from fixation or partial fixation of columns. However, in case of high shear force acting at column base, adequacy of frictional resistance between the column base and foundation shall be checked. In case frictional resistance is not capable of transmitting the total shear force, shear lugs shall be provided to resist the same. Corrosion allowance for exposed anchor bolts shall be made as per cl. 2.5.4.

2.7.19 For bolts, studs and bars, the area of steel required for tension and shear shall be considered to be additive. The tensile stress area (in mm²) of a threaded anchor subject to direct tensile or tensile and shear stress shall be

$$\left(\frac{\pi}{4}\right)[D - (0.975/n)]^2,$$

where D is the major thread diameter in mm, and n the number of threads per mm. If the threads are excluded from the shearing plane, the gross area may be used in determining the shear stress.

2.8 Welds

General Requirements

2.8.1 The applicable provisions of Indian Standards IS:816 (Code of Practice for Use of Metal Arc Welding for General Construction in Mild Steel) and IS:9595 (Recommendations for Metal Arc Welding of Carbon and Carbon Manganese Steels) shall apply to the work performed under this standard.

Welding Material and Material Control

2.8.2 (1) Filler Metals

Electrodes, filler rods and wires shall conform to appropriate standards given in Appendix-A.

(2) Control of Welding Materials

Fabricators and erectors shall prepare and implement procedures to control the storage and handling of electrodes, flux and other welding materials to be used in the welding process.

Welding Procedures and Welders Qualifications

2.8.3 Welding procedures and welders shall be qualified as per relevant Codes.

Identification of Welders and Welding Operators.

2.8.4 Each qualified welder and welding operator shall be assigned an identifying number, letter or symbol by the fabricator or erector.

Corrosion-Resistant Weld Overlay Cladding

2.8.5 Welding procedures for cladding shall be qualified as per relevant Codes.

Welding Repair

2.8.6 Defects shall be removed or repaired by procedures prepared by the fabricator or erector and approved by the engineer.

Welding Symbols

2.8.7 Weld joints are classified as (a) Butt joint (b) T joint (c) Lap joint (d) Corner joint and (e) edge joint. Each has variations in the welding procedure, which is dictated among other factors by the thickness of the connecting elements. The scheme of symbols for indicating welds on drawings giving information regarding the type, size, position, finish, etc. of welds in welded joints, shall be as per IS:813, but complete details shall be given for any non-standard welds.

Effective Areas of Weld Metal for Groove Weld

2.8.8 The effective area of groove welds shall be determined as effective throat thickness of weld times the effective length of weld.

2.8.9 Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

Effective Throat Thickness of Complete Penetration Groove Weld

2.8.10 The effective throat thickness of a complete penetration groove weld shall be the thickness of the thinner part joined.

Effective Throat Thickness of Partial Penetration Groove Weld

2.8.11 The effective throat thickness of a partial penetration groove weld shall be as shown in Table 2.8. The minimum effective throat thickness of partial penetration groove weld is influenced by the thicker of the two parts joined except that the weld size need not exceed the thickness

of the thinnest part joined. The minimum required throat thickness of partial penetration groove welds is shown in Table 2.9.

TABLE 2.8 : EFFECTIVE THROAT THICKNESS OF PARTIAL PENETRATION GROOVE WELDS

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc	All	J or U joint	Depth of chamfer
Gas metal arc	All	Bevel or V joint $\geq 60^\circ$	
Flux-cored arc	All	Bevel or V joint $\leq 60^\circ$ but $\geq 45^\circ$	Depth of chamfer minus 3 mm

Effective Throat Thickness of Flare Groove Weld

- 2.8.12 (a) The effective throat thickness of a flare groove weld, when flush to the surface of the solid section of the bar, shall be as shown in Table 2.10,
- (b) Effective throat thickness, larger than those shown in Table 2.9, is permitted, provided the fabricator can establish, by qualification, that he can consistently provide such larger throat thickness. The qualification shall consist of sectioning the member, normal to its axis, at terminal ends and at mid-length of the weld. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

Effective Areas of Fillet Welds

- 2.8.13 The effective area of fillet welds shall be determined as effective throat thickness times the effective length of weld. The minimum size of fillet weld shall be as given in Table 2.11.

Effective Area of Fillet Weld

2.8.14 The effective fillet weld area shall be taken as the length times the effective throat thickness. The effective throat thickness of fillet weld shall be the shortest distance from the root to the face of the weld, except that, in fillet welds made by submerged arc process, the effective throat thickness shall be taken equal to leg size for 10 mm and smaller fillet welds, and equal to theoretical throat plus 2.8 mm for fillet welds over 10 mm.

TABLE 2.9 : MINIMUM EFFECTIVE THROAT THICKNESS OF PARTIAL PENETRATION GROOVE WELDS

Material Thickness of Thicker Part Joined (mm)	Minimum Effective Throat Thickness (mm)
Up to 6	3
over 6 to 12	5
over 12 to 18	6
over 18 to 37	8
over 37 to 55	10
over 55 to 150	12
over 150	15

TABLE 2.10 : EFFECTIVE THROAT THICKNESS OF FLARE GROOVE WELDS

Type of Weld	Radius (R') of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$(5/16)R$
Flare V Groove	All	$(1/2)R$

Note : For gas metal arc welding, except short-circuiting transfer process, effective throat thickness = $(3/8)R'$, when $R \geq 12$ mm

TABLE 2.11 : MINIMUM SIZE OF FILLET WELDS

Material Thickness of Thicker Part Joined (mm)	Minimum Size of Fillet Weld (leg dimension of fillet welds) (mm)
Up to 6	3
over 6 to 12	5
over 12 to 18	6
over 18	8

Effective Length of Fillet Weld

2.8.15 The effective length of fillet welds, except fillet welds in holes and slots, shall be the overall length of full-size fillet, excluding end returns. In the case of fillet welds in holes and slots, the effective length shall be the length of the centre line of the weld through the centre of the plane through the throat. However, in the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot in the plane of the faying surface. The minimum effective length of fillet welds shall not be less than 4 times the nominal size; or else the size of the weld shall not exceed (1/4) of its effective length.

Limitations

2.8.16 The minimum size of fillet welds shall be as shown in Table 2.11. Minimum weld size depends on the thicker of the two parts joined, except that weld size need not exceed the thickness of the thinner part. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld. Weld sizes larger than the thinner part joined are permitted if required by calculated strength. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld may be less than 1.5 mm provided the weld size is clearly verifiable.

2.8.17 The maximum size of fillet welds permitted along edges of connected parts shall be:

- (a) For material less than 6 mm thick :- not greater than the thickness of the material,
- (b) For material 6 mm or greater in thickness :- not greater than the thickness of the material minus 1.5 mm unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

2.8.18 The minimum effective length of fillet welds designed on the basis of strength shall be not less than 4 times the nominal size, or else the size of the weld shall be considered not to exceed $(1/4)$ of its effective length. If longitudinal fillet welds are used alone in end connections of flat bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. The transverse spacing of longitudinal fillet welds used in end connections of tension members shall not exceed 200 mm, unless otherwise specified.

2.8.19 Intermittent fillet welds are permitted to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than 4 times the weld size, with a minimum of 40 mm. However, intermittent welds are not permitted where the member is subjected to dynamic loading, reversal of stress and fatigue.

2.8.20 In lap joints, the minimum lap shall be 5 times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress shall be fillet-welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

2.8.21 Fillet welds in holes or slots are permitted to transmit shear in lap joints or to prevent buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap subject to the provisions of section 2.8. Fillet welds in holes or slots are not to be considered plug or slot welds.

2.8.22 Side or end-fillet welds terminating at ends or sides, respectively, of parts or members shall, wherever practicable, be returned continuously around the corners for a distance not less than twice the nominal weld size. This provision shall apply to side and top fillet welds connecting brackets, beam seats and similar connections, on the plane about which bending moments are computed. For framing angles and simple end-plate connections which depend on flexibility of the outstanding legs for connection flexibility, end returns shall not exceed four times the nominal size of the weld. Fillet welds occurring on opposite sides of a common plane shall be interrupted at the corner common to both welds. End returns shall be indicated on the shop drawings.

Effective Areas of Weld Metals for Plug and Slot Welds

2.8.23 The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

Limitations

2.8.24 Plug or slot welds are permitted to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members. The diameter of the hole for a plug weld shall be not less than the thickness of the part containing it plus 8 mm, rounded to the next larger or multiple of 1.5 mm, nor greater than the minimum diameter plus 3 mm or 2.25 times the weld thickness.

2.8.25 (a) The minimum centre-to-centre spacing of plug welds shall be four times the hole diameter,

(b) The minimum spacing of lines of slot welds in a direction transverse to their length shall be 4 times the width of the slot. The minimum centre-to-centre spacing in a longitudinal direction on any line shall be twice the slot length.

2.8.26 The length of slot for a slot weld shall not exceed 10 times the weld thickness. The slot width shall be not less than the thickness of the part containing it plus 8 mm, rounded to the next larger or multiple of 1.5 mm, nor shall be larger than 2.25 times the weld thickness. The ends of the slot shall be

semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the weld thickness shall be at least half the thickness of the material but not less than 16 mm.

2.9 Areas, Cambers, Expansions

Gross Area

- 2.9.1 The gross area of a member at any section shall be determined by summing the products of the thickness and the gross width of each element as measured normal to the axis of the member. The net area shall be determined by substituting for the gross width the net width computed in accordance with cl. 2.9.2 to 2.9.7.

Net Area and Effective Net Area

2.9.2 Chain of Holes

- (a) In the case of a series of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the series and adding, for each gauge space in the series, the quantity

$$s^2/4g$$

where s is the longitudinal centre-to-centre spacing (pitch) of any two consecutive holes (mm) and g is the transverse centre-to-centre spacing (gauges) of the same two holes (mm),

- (b) The net area A_n for design of the part is obtained from that series which gives the least net width,
- (c) In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

Axially Loaded Tension Members

2.9.3 The effective net area A_{eff} of axially loaded tension members where the load is transmitted by bolts through some of the cross-sectional elements of the member, shall be computed from the formula

$$A_{\text{eff}} = C_t A_n \quad (2.18)$$

where,

A_n = net area of the member,

C_t = a reduction coefficient,

= 0.75 for members whose connections have only two fasteners per line,

= 0.90 for structural shapes with flange width not less than (2/3)rd the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress,

= 0.85 for structural shapes not meeting the above conditions, structural tees cut from these shapes, and all other shapes, including built-up cross sections, provided the connection has no less than three fasteners per line in the direction of stress.

Bolted Splice and Gusset Plates

2.9.4 Bolted splice and gusset plates and other connection fittings, subject to tensile force, shall be designed in accordance with the provisions of cl. 4.4.21 to 4.4.24 where the effective net area shall be taken as the actual net area, except that, for design calculations, it shall not be taken as greater than 85% of gross area.

Angles

2.9.5 For angles, the gross width shall be the sum of widths of the legs less the thickness. The gauge for holes in opposite legs shall be the sum of gauges from the back of angles less the thickness.

Size of Holes

- 2.9.6 In computing net area, the width of a bolt hole is taken generally as 1.5 mm greater than nominal dimension of the hole normal to the direction of applied stress, given in Table 2.4 or cl. 2.7.12.

Pin-Connected Members

- 2.9.7 (a) Eyebars shall be of uniform thickness, without reinforcement at pin holes. They shall have “circular” heads in which the periphery of the head beyond pin hole is concentric with pin hole. The radius of transition between circular head and the body of the eyebar shall be equal to or greater than the head diameter,
- (b) The body width of the eyebar shall not exceed 8 times its thickness, and the thickness not less than 12 mm,
- (c) The net area of head through pin hole, transverse to the axis of eyebar, shall be not less than 1.33 or more than 1.5 times the cross-sectional area of the body of eyebar,
- (d) The pin hole diameter shall not be less than $(7/8)$ th of the body width of the eyebar. The pin hole diameter shall not be more than 0.8 mm greater than the pin diameter. For any steel having a yield stress greater than 480 MPa, the pin diameter hole shall not exceed 5 times the plate thickness,
- (e) In pin-connected plates other than eyebars, the tensile stress on the net area, transverse to the axis of the member, shall not exceed the permissible stress given in Table 4.2. The allowable bearing stress on the projected area of the pin shall be calculated in accordance with cl. 4.4.18. The minimum net area beyond pin hole, parallel to the axis of the member, shall be not less than $(2/3)$ rd the net area across pin hole,
- (f) The distance transverse to the axis of a pin-connected plate or any individual element of a built-up member, from the edge of the pin hole to the edge of the member or element, shall not exceed 4 times the pin hole thickness. The pin hole diameter shall be not less than 1.25 times the smaller of the distances from the edge of the pin hole

to the edge of a pin-connected plate or separated element of a built-up member at the pin hole. For pin-connected members in which the pin is expected to provide for relative movement between connected parts while under full load, the pin hole diameter shall not be more than 0.8 mm greater than the pin diameter,

- (g) The corners beyond pin hole may be cut at 45° to the member axis, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the member axis,
- (h) Thickness limitations on both eyebars and pin-connected plates may be waived whenever external nuts are provided to tighten pin plates and filler plates into snug contact. When the plates are thus contained, the allowable stress in bearing shall be no greater than that specified in cl. 4.4.18.

Camber

Trusses and Girders

- 2.9.8 Trusses of 25 metre or greater spans shall be cambered for approximately the dead load deflection. Crane girders 20 metres or greater spans shall be cambered for approximately the dead load deflection plus (1/2) the live load deflection.

Camber for other Structures

- 2.9.9 If any special camber requirements are necessary from other considerations, the requirements shall be specified on the plans and detailed drawings.

Erection

- 2.9.10 Beams or trusses, detailed without specified camber, shall be fabricated so that after erection any minor camber due to rolling or shop assembly shall be upward. If camber involves erection of any member under a straining force, this shall be noted on the erection diagram.

Cambering, Curving and Straightening

2.9.11 Local application of heat or mechanical means may be used to introduce or correct camber, curvature and straightness in structural elements. The temperature of heated areas, as measured by approved methods, shall not exceed 600°C to 650°C depending upon the grade of steel.

Expansion

2.9.12 Adequate provision shall be made for expansion and contraction as appropriate to design conditions of the structure.

3. SPECIAL DESIGN REQUIREMENTS

3.1 Fire-Resistant Design

- 3.1.1 Protection shall be provided against fire hazard by both direct measures and passive provisions.
- 3.1.2 Direct measures, i.e. detection and fire-fighting arrangement shall be developed in line with stipulations of AERB Safety Code of Practice on Design for Safety in Pressurised Heavy Water Based Nuclear Power Plants (AERB/SC/D).
- 3.1.3 Fire protection measures in steel structures shall be taken to prevent and delay structures affected by fire from collapse. The design criteria for fire resistance design shall be such that the structural components should sustain their integrity against a fire of a minimum specified period of duration and intensity. Structural components shall be provided to act as a fire barrier for the specified period of fire duration to restrict the fire locally within specific compartments.
- 3.1.4 The minimum fire rating of different structural components to be considered in the design shall comply with cl. 3.8.3 of AERB/SS/CSE.
- 3.1.5 The intensity of fire shall correspond to that which may develop from the contents of the plants. As a minimum, it shall correspond to that specified in relevant Indian standards for conventional buildings. If an unprotected steel section is raised to a temperature above that considered critical for given exposure conditions, or is found to be deficient in meeting the required rating, insulating material may be placed around the steel, only when approved by the engineer.
- 3.1.6 For fire conditions, design requirements on both strength and serviceability are to be satisfied.

3.2 Impulsive and Impactive Loadings

- 3.2.1 Steel structures important to safety shall be designed for impulsive and impactive loads (ref. Appendix-A of AERB/SS/CSE). These loads must be combined suitably with other loads in accordance with requirements in

determining the required resistance of structural elements. Information regarding design of steel structures subject to impulsive and impactive loadings is given in Annexure-I.

3.3 Members and Connections Subject to Repeated Variation of Stress (Fatigue)

General Requirement

- 3.3.1 Fatigue, as used in this standard, is defined as possible damage in fracture after a sufficient number of stress fluctuations. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, stress range shall be computed as the numerical sum of repeated maxima of the tensile and compressive stresses, or the sum of maximum shearing stresses of opposite directions at a given point, which may result from the differing arrangement of live loads or other cyclic and repetitive loadings.

Design for Fatigue

- 3.3.2 The design of members and connections subject to repeated variations of stress, shall consider the number of stress cycles, the expected range of stress, and the type and location of the member or detail.

For this purpose, loading conditions, their classification, and type and location of material are given in Appendix-F.

Maximum Stress

- 3.3.3 The maximum stresses shall not exceed permissible stresses provided in section 4.3 of this standard, and the maximum stress range shall not exceed that given in Appendix-F.
- 3.3.4 For bolt under fatigue loading the design strength is given in Table 3.1.

TABLE 3.1 : DESIGN STRENGTH OF BOLTS SUBJECT TO TENSION

Number of Cycles	Design Strength	
	Normal Design Conditions	Abnormal Design Conditions
$\leq 20,000$	0.28 $A_b f_u$ when threads are not excluded from shear plane.	0.45 $A_b f_u$ when threads are not excluded from shear plane.
	0.37 $A_b f_u$ when threads are excluded from shear planes.	0.60 $A_b f_u$ when threads are excluded from shear plane.
20,000 to 1,00,000	0.2 $A_b f_u$	0.32 $A_b f_u$
1,00,000 to 5,00,000	0.18 $A_b f_u$	0.29 $A_b f_u$
$> 5,00,000$	0.16 $A_b f_u$	0.25 $A_b f_u$

4. ALLOWABLE STRESS DESIGN METHOD

4.1 Design for Allowable Stress

- 4.1.1 Steel structures or parts, having design class DC3, shall be designed for allowable stress in accordance with the provision of this chapter unless specified otherwise. The design shall consider individual loads and their combinations as specified hereinafter. Effects such as vibration, impact, differential settlement, and any other form of transients shall also be considered.
- 4.1.2 Allowable stresses in various steel members and components are dependent, among various factors, on the material temperature. Allowable stresses, specified in this standard are valid for temperature up to 370°C, above which, appropriate changes in the values of the material properties, such as E , f_y and f_u , shall be considered in arriving at allowable stresses.

4.2 Loads for Allowable Stress Design

- 4.2.1 Structures and structural members should be so designed that the induced stress at all sections is not greater, and at the most equal to permissible value of stress for loads and forces in such combinations as stipulated for loads in cl. 4.2.3 through 4.2.8 and as combined in accordance with provisions specified in cl. 4.2.9.
- 4.2.2 (1) For all loads, consideration should be given to effects created by application of load and its interaction with the structure, or created by a particular response of the structure. Effects such as vibration, impact, differential settlements, shrinkage and any other form of transients should also be considered,
- (2) Loads as mentioned in this section should mean loads and/or related internal moments and forces due to effects of stipulated load described in AERB/SS/CSE. The effect of earthquake on structures shall be considered in the design in accordance with AERB/SS/CSE. For detailed description of individual loadings given in the following clauses, Appendix-A of AERB/SS/CSE shall be referred.

Individual Loads

4.2.3 Normal Loads

DL — dead load

H — lateral earth pressure

LL — live loads

P_v — pressure loads during normal operation condition

P_t — test pressure

T_t — thermal effects and loads during the test

R_0 — pipe and equipment reactions during normal operation excluding dead loads and earthquake reactions

T_0 — thermal effects and loads during normal operation, solar radiation effects and effects during construction.

4.2.4 The dynamic effects of live load should be considered in the analysis. Where a detailed dynamic analysis is performed for crane systems, elevators, or other moving machinery, the resulting load with dynamic amplification should be used. If such an analysis is not performed, the following increases over static effects should be used to account for dynamic effects.

supports of elevators	—	100 %
cab-operated travelling crane, support girders and their connections	—	25 %
pendant-operated travelling crane, support girders and their connections	—	10 %
supports of light machinery, shaft, or motor-driven equipment	—	not less than 20 %,

supports of reciprocating machinery or power-driven units.	—	not less than 50%
hangers supporting floors and balconies.	—	33 %

4.2.5 Crane Runway Horizontal Forces

- (1) In addition, the horizontal forces on the crane runway should be considered in the live load (*LL*) category. These lateral forces on the crane runways due to the effects of moving crane trolleys should, if not otherwise specified by designer or manufacturer, be 20% of the sum of the weights of the lifted load and that of the crane trolley, but exclusive of other parts of the crane,
- (2) Unless otherwise specified by crane vendor, the lateral force due to surge shall be considered acting on any one crane rail track, acting in either direction and equally distributed amongst all the wheels on one side of the rail track,
- (3) Longitudinal forces should, if not otherwise specified, be taken as 10% of the static wheel loads of the crane applied at the top of the rail. Gantry girders and their supports shall be designed assuming that either of the horizontal loads stated above may act at the same time as the vertical load.

4.2.6 Severe Environmental Loads

- E_0 — load effects generated by operating basis earthquake,
 WC — load effects generated by severe wind,
 FF — load effects generated by design basis flood.

4.2.7 Extreme Environmental Loads

- E_{ss} — load effects generated by safe shutdown earthquake,
 W_t — load effects and wind-induced missile due to extreme wind.

4.2.8 Accidental Loads

F_h — hydrostatic load due to internal flooding,

MA — load and other effects of aircraft impact,

ME — missiles due to external events other than those related to wind or tornado, explosions in transportation systems, disintegration of turbine and other components,

MI — loading due to internal missiles,

MT — missiles, wind and overpressure generated from explosions in transportation systems on land, water or in air,

Mt — load and other impactive effects of turbine missile,

P_a — design accident pressure,

R_a — pipe and equipment reactions under thermal conditions generated ~~by a postulated pipe break including~~ R_0 ,

T_a — design accident temperature,

Y_j — jet impingement load on a structure generated by DBA,

Y_m — missile impact load on a structure, such as pipe whip generated by DBA,

Y_r — load on structures generated by the reaction of the broken high energy pipe during DBA.

The magnitude of accidental loading shall include an appropriate dynamic load factor (DLF) when these loads are considered as equivalent static loads. Otherwise appropriate dynamic analysis is to be carried out to determine structural response.

Load Combinations

- 4.2.9 (a) Unless specified otherwise load combinations given in Table 4.1 shall be used,
- (b) Load combination number 12 of LC6 holds for internal structures of reactor building,
- (c) For safety class 3 and design class DC3 structures which do not perform safety functions associated with supporting emergency core-cooling systems and other systems related to safe shutdown of reactor or to prevent/mitigate the consequences of accident resulting in potential off-site exposure, comparable to relevant AERB Guidelines, the applicable load combinations are LC1, LC2, and LC5,
- (d) For safety class 4 and design class DC3 structures the applicable load combinations are LC1 and LC2,
- (e) Whenever applicable impact effect of moving load shall be included in the live load.
- 4.2.10 Steel structures, which by themselves are not classified as nuclear safety-related, but the failure of which may cause damage to nearby nuclear safety-related structures and components, should be evaluated for no gross failure for load combinations LC3 to LC6.

4.3 Permissible Design Stress

- 4.3.1 The permissible design stresses are given in Table 4.2. The allowable design stresses required for calculation of permissible design stresses shall be taken from section 4.4.
- 4.3.2 For DC3 safety class 4 structures increase in the permissible design stresses for LC2 load combination by 1.33 times is acceptable subject to limitations on their permissible values specified in notes (2) and (3) of Table 4.2.

TABLE 4.1 : LOAD COMBINATION FOR DESIGN OF STEEL STRUCTURES

Load Type	Combination	Load Comb. No.	Load Factors f_{ij}																		
			DL	(1) LL	(2) H	R_0	T_0	P_t	P_v	$E'_0 / WC / FF$	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA/ME $MI/MT/M_t$	F_h		
LC1	Normal Design Condition Normal Load Comb.	1	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
		2	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
		3	1.0	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-
LC2	Severe Env. Load Comb.	4	1.0	1.0	1.0	-	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-	-
		5	1.0	1.0	1.0	1.0	-	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-
		6	1.0	1.0	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-
LC3	Abnormal Design Condition Extreme Env. Load Comb.	7	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	1.0	-	-	-
		8	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	1.0	-	-
		9	1.0	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	1.0	-

TABLE 4.1 : LOAD COMBINATION FOR DESIGN OF STEEL STRUCTURES (contd.)

Load Type	Combination	Load Comb. No.	Load Factors f_{ij}																																		
			DL	(1) LL	(2) H	R_0	T_0	P_t	P_v	E'_0 / WC/ FF	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA/ME $MI/MT/M_t$	F_h																		
LC4	Abnormal Load Comb.	10	1.0	1.0	1.0	-	-	-	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			
LC5	Abnormal Severe Env. Load Comb.	11	1.0	1.0	1.0	-	-	-	-	-	-	-	1.0	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	
LC6	Abnormal Extreme Env. Load Comb.	12	1.0	1.0	1.0	-	-	-	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-

Note : (1) All load combinations above shall be checked for full and zero live load conditions,
(2) Effect of lateral earth pressure shall be considered in design when it is critical.

TABLE 4.2 : PERMISSIBLE DESIGN STRESSES FOR STEEL STRUCTURES

Load Combination	Permissible Design Stress	
	Other than shear stress	Shear stress
Normal Design Conditions		
LC1: Normal without temperature effect with temperature effect	S 1.5S	S 1.4S
LC2: Severe Environmental without temperature effect with temperature effect	S 1.5S	S 1.4S
Abnormal Design Conditions		
LC3: Extreme Environmental without temperature effect with temperature effect	1.5S 1.6S	1.4S 1.4S
LC4: Abnormal without temperature effect with temperature effect	1.5S 1.6S	1.4S 1.4S
LC5: Abnormal Severe Environmental without temperature effect with temperature effect	1.5S 1.6S	1.4S 1.4S
LC6: Abnormal Extreme Environmental without temperature effect with temperature effect	1.5S 1.7S	1.4S 1.4S

- Note : (1) Appropriate value of S should be taken from sec. 4.4,
- (2) The permissible tensile stress shall not be greater than the following in any load combination,
- Axial tension: $0.7 f_u$
- Axial and bending tension: $(Z_p/S_e) \times 0.7 f_u$
- where f_u = ultimate strength,
 Z_p = plastic section modulus,
 S_e = elastic section modulus,
- (3) Permissible stress under any load combinations shall not be more than $0.9 f_y$.

TABLE 4.3 : ALLOWABLE STRESSE(S) FOR STEEL STRUCTURES

Sl. No.	Stress	Allowable Values (S)
1.	Axial Tension (σ_{at}) On the gross area On the effective net area On the net area (at pin holes)	$0.6 f_y$ $0.5 f_u$ $0.45 f_y$
2.	Bending Tension ⁽⁴⁾ (σ_{bt})	$0.66 f_y$
3.	Axial Compression (σ_{ac}) General section Plate girder stiffeners Web crippling (rolled)	See Note (1) $0.6 f_y$ (on gross area) $0.75 f_y$
4.	Bending Compression ⁽⁴⁾ (σ_{bc}) Compression flange fully restrained Compression flange not restrained	$0.66 f_y$ See Note (2)
5.	Shear Stress General At the beam end connection, for combination of shear along a plane through fastener plus tension along a perpendicular plane.	$0.4 f_y$ For $h_c/t_w \leq 1000/(f_y)^{1/2}$ (See also cl.4.4.2) $0.3 f_u$
6.	Bending stress for slab base	$0.75 f_y$
7.	Bearing stress (in concrete)	$0.25 f_{ck}^{(3)}$

Note: (1) To be calculated on the basis of l_c/r ratio and section properties (ref. cl. 4.4.3 and 4.4.4),

(2) To be calculated on the basis of l_c/r ratio and section properties, (ref. cl. 4.4.5 to 4.4.17),

(3) Characteristic strength of concrete,

(4) Ref. cl. 4.4.5 to 4.4.17.

4.4 Allowable Stresses

4.4.1 Allowable stresses in design shall not exceed the value given in Table 4.3 unless otherwise specified in the following clauses of this section or by other provisions of this standard.

Shear Stress of Structural Steels

4.4.2 For $h_c/t_w > 1000/(f_y)^{1/2}$, the allowable shear stress on the clear distance between flanges times the web thickness is given by:

$$\tau_v = \frac{f_y}{2.89} (C_v) \leq 0.40 f_y, \quad (4.1)$$

where,

$$C_v = \frac{3.125 \times 10^5 K_v}{f_y (h_c/t_w)^2}, \text{ when } C_v \leq 0.8,$$

$$= \frac{500}{h_c/t_w} \left[\frac{K_v}{f_y} \right]^{1/2}, \text{ when } C_v > 0.8,$$

$$K_v = 4.00 + \frac{5.34}{(a/h_c)^2}, \text{ when } a/h_c \leq 1.0,$$

$$= 5.34 + \frac{4.00}{(a/h_c)^2}, \text{ when } a/h_c > 1.0,$$

where τ_v is the allowable shear stress, t_w is the web thickness, a is the clear distance between transverse stiffeners and h_c is the clear distance between flanges at the section under investigation.

Axial Compressive Stress

4.4.3 The allowable stresses for axially loaded compression members are given by following equation when kl_c/r does not exceed C_c

$$\sigma_{ac} = Q \frac{\left[1 - \frac{(kl_c/r)^2}{2C_c^2} \right] f_y}{1.67 + \frac{0.375(kl_c/r)}{C_c} - \frac{0.125 (kl_c/r)^3}{C_c^3}}, \quad (4.2)$$

where,

$$C_c = \left[\frac{2 \sigma_y E}{Q \sigma_y} \right]^{1/2}, \quad (4.3)$$

$Q = 1$ when the cross-section of axially loaded compression members meet the provisions of Table 2.1,

$= Q_s$ for axially loaded compression members containing cross-section composed of unstiffened elements,

$= Q_a$ for cross-section composed of entirely stiffened elements,

$= Q_a Q_s$ for cross-section composed of both stiffened and unstiffened elements,

Q_a and Q_s shall be calculated as per cl.2.4.12 and 2.4.16.

4.4.4 (1) On the gross section of axially loaded compression members, when kl_c/r exceeds C_c , the allowable stress is:

$$\sigma_{ac} = \frac{12 \sigma_y E}{23(kl_c/r)^2}. \quad (4.4)$$

(2) For axially loaded circular tubular sections with D/t greater than $22900/\sigma_y$ but less than $90280/\sigma_y$, σ_{ac} shall not exceed the value in cl. 4.4.3 and 4.4.4 (1) above as appropriate, nor the value given by:

$$\sigma_{ac} = \frac{4600}{D/t} + 0.40 \sigma_y, \quad (4.5)$$

where D is the outside diameter and t is the thickness.

Bending Stress - Strong Axis Bending of I-shaped Members and Channels

- 4.4.5 Tensile and compressive stress on extreme fibres of compact hot-rolled or built-up members as per Table 2.1 (except for steel having an ultimate tensile stress-yield stress ratio of less than 1.3), symmetrical about, and loaded in, the plane of their minor axis and meeting the requirements of this section:

$$b = 0.66 f_y \quad (4.6)$$

Provision of this clause is applicable to a member if the requirements of cl. 4.4.6 and 4.4.7 are satisfied.

- 4.4.6 The laterally unsupported length of the compression flange of members shall exceed neither

$$(200 b_f) / (f_y)^{1/2},$$

nor,

$$(140 \times 10^3) / \{(d/A_f) f_y\},$$

where b_f is the flange width and A_f is the area of compression flange.

- 4.4.7 Except for hybrid girders and members having yield stress greater than 450 MPa, members designed on the basis of composite action, which meet the requirements of compact section, and which are also continuous over supports or are rigidly framed to columns by means of high strength bolts or welds, may be proportioned for (9/10)th of the negative moments produced by gravity loading and have maximum magnitude at points of support, provided that, for such members, the maximum positive moment shall be increased by (1/10)th of the average negative moments. This reduction shall not apply to moments produced by loading on cantilevers. If the negative moment is resisted by a column, rigidly framed to the beam or girder, the (1/10)th reduction may be used in proportioning the column for the combined axial and bending loading, provided that the stress, σ_a , due to any concurrent axial load on the member, does not exceed $0.15 \sigma_{ac}$.

Bending Stress in Non-compact Sections

4.4.8 Members (except hybrid girders and members having yield stress greater than 450 MPa) which meet the requirements of cl. 4.4.6 and 4.4.7 and except that their flanges are non-compact may be designed on the basis of an allowable bending stress

$$b = f_y [0.79 - \{7.6 \times 10^{-4} b_f / (2t_f)\} f_y^{1/2}] \quad (4.7)$$

where t_f is the flange thickness.

4.4.9 For built-up members meeting the requirements of cl. 4.4.5 and 4.4.6 except that their flanges are non compact and their webs are compact or non-compact, (excluding hybrid girders and members with yield points greater than 450 MPa) the allowable stress is

$$b = f_y [0.79 - \{7.6 \times 10^{-4} b_f / (2t_f)\} (f_y / k_c)^{1/2}], \quad (4.8)$$

where,

$$k_c = \frac{4.05}{(h_c / t_w)^{0.46}} \text{ if } h_c / t_w > 70, \text{ otherwise } k_c = 1.0. \quad (4.9)$$

4.4.10 For members with a non-compact section (Table 2.1), but not included above, and loaded through shear centre and braced laterally in the region of compression stress at intervals not exceeding $[200 b_f / (f_y)^{1/2}]$, the allowable stress is

$$b = 0.60 f_y. \quad (4.10)$$

4.4.11 Compact and Non-Compact Sections with Unbraced Length greater than that given in cl. 4.4.6.

(1) The allowable bending stress in tension is given by

$$b = 0.6 f_y, \quad (4.11)$$

(2) For members, having an axis of symmetry in, and loaded in the plane of their web, and for channels bent about their major axis, the

allowable compression (in MPa) is given by the following equations but not to exceed $0.60 f_y$,

$$b = [0.67 \{f_y(l_t/r_t)^2\}/(105 \times 10^5 C_b)] f_y, \quad (4.12)$$

when,

$$[704 \times 10^3 C_b / f_y]^{0.5} \leq l_t / r_t \leq [352 \times 10^4 C_b / f_y]^{0.5}$$

and,

$$b = [(1.17 \times 10^6 C_b)/(l_t / r_t)^2], \quad (4.13)$$

when,

$$l_t / r_t > [352 \times 10^4 C_b / f_y]^{0.5}.$$

- (3) The allowable compression for the case, when the compression flange is solid and approximately rectangular in cross-section and its area is not less than that of the tension flange, is given by the following equation but not to exceed $0.60 f_y$,

$$b = [(8.3 \times 10^4 C_b)/(l_t D / A_f)], \quad (4.14)$$

where in (4.12) to (4.14) above,

A_f = area of the compression flange (mm^2),

l_t = distance between cross-sections braced against twist or lateral displacement of the compression flange (millimetres). For cantilevers, braced against twist only at the support, l_t may be taken as the actual length,

r_t = radius of gyration of a section comprising the compression flange plus $(1/3)$ rd of the compression web area, taken about an axis in the plane of the web (millimetres).

$$C_b = 1.75 + 1.05 (M_1/M_2) + 0.3 (M_1/M_2)^2 \quad (4.15)$$

but not larger than 2.3.

- (4) In equation (4.15) above, M_1 is the smaller and M_2 is the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member. M_1/M_2 , the ratio of end moments,

is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite signs (single curvature bending). If the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of C_b shall be taken as unity. When computing f_{bcx} and f_{bcy} to be used in equation (4.23), C_b may be computed by the formula given above for frames subject to joint translation, and it shall be taken as unity for frames braced against joint translation. C_b may be conservatively taken as unity for cantilever beams.

4.4.12 For hybrid plate girders, f_y in equations (4.12) and (4.13) is the yield stress of the compression flange. Equation (4.14) shall not apply to hybrid girders.

Bending Stress in Sections Symmetrical about both Axes - Weak Axis bending for Compact and Non-compact Sections.

4.4.13 Doubly symmetrical I- and H-shaped members, with compact flanges and continuously connected to the web and bent about their minor axis (except members with steel having a yield stress greater than 450 MPa) and also solid round and square bars and solid rectangular sections bent about their weaker axis, the allowable stress is

$$f_b = 0.75 f_y. \quad (4.16)$$

4.4.14 I and H-shaped non-compact members, symmetrical about both axes, and bent about their minor axis (except hybrid girders and members with steel having yield stress greater than 450 MPa) may be designed on the basis of an allowable bending stress

$$f_b = f_y [1.08 - \{1.93 \times 10^{-3} b_f / (2t_f)\} f_y^{1/2}]. \quad (4.17)$$

Bending Stress of Box Members, Rectangular and Circular Tubes

4.4.15 For rectangular and circular tubular sections, and box members which are compact with flanges continuously connected to webs, and bent about their strong or weak axes, may be designed on the basis of an allowable bending stress

$$f_b = 0.66 f_y. \quad (4.18)$$

4.4.16 To be classified as a compact section a box shaped member shall have, in addition to the requirements of Table 2.1, a depth not more than 6 times the width and a flange thickness not more than twice the web thickness. The laterally supported length for such member shall not exceed the value

$$[13450 + (8280 M_1/M_2)](b/f_y)$$

except that it need not be less than $8280(b/f_y)$, where b is the width of stiffened and unstiffened compression element. For definition of M_1 and M_2 , refer to cl. 4.4.11 subpara (4).

4.4.17 For tension and compression on extreme fibres of box-type and flexural members which are non-compact

$$b = 0.60 f_y \quad (4.19)$$

If the depth of the member is less than 6 times its width, lateral bracing is not required. Lateral support requirements for box sections of larger depth to width ratios must be determined by special analysis.

4.4.18 Bearing

Contact Area of Milled Surfaces/Fitted Bearing Stiffeners/Projected area of Pins.

The allowable bearing stress p is given by

$$p = 0.9 f_y \quad (4.20)$$

where f_y is the lower of the yield stresses of the two bearing materials.

4.4.19 Expansion Rollers and Rockers

The allowable bearing force F_p (in Newton) in expansion rollers and rockers is given by

$$F_p = 0.033 l_b d' (f_y - 90), \quad (4.21)$$

where l_b is the bearing length and d' the diameters.

4.4.20 Projected Area of Bolts in Shear Connections

On the projected area of bolts in shear connections with standard holes, the allowable stress in connected parts is

$$p = 1.5 f_u, \quad (4.22)$$

where f_u is the specified minimum tensile strength of connected parts, in (MPa).

Bolts

4.4.21 Tension, Shear and Bearing Stresses

The allowable tensile, shear and bearing stresses on bolts and other fasteners shall be as per IS:800, IS:1363 (part III), IS:3757 and IS:4000. For friction grip bolts, increase in permissible stresses for SSE or severe wind condition should not be more than 15%. No increase in permissible stresses is allowed for OBE condition irrespective of the type of bolts.

4.4.22 The slip coefficients to be used in calculating the allowable loads as per cl. 4.4.21 are given in Table 4.4.

4.4.23 Joints with Friction-Grip Bolts

In case of temperature change, friction-grip bolts shall be checked for normal shear and bearing.

4.4.24 Design for bolts and threaded parts subject to fatigue loading shall be in accordance with the provisions of cl. 3.3.2.

Welds

4.4.25 For general requirements of welds, reference shall be made to Indian standards IS:816 and IS:9595. The allowable stresses for groove, fillet, plug and slot welds shall be taken as per Table 4.5.

TABLE 4.4 : SLIP COEFFICIENT

Surface Condition	Slip Coefficient
Clean mill scale	0.25
Blast-cleaned carbon and low-alloy steel	0.40
Blast-cleaned quenched and tempered steel	0.25
Hot-dip galvanised and roughened	0.30
Blast-cleaned, organic zinc rich paint	0.30
Blast-cleaned, inorganic zinc rich paint	0.45
Blast-cleaned, metallized with zinc	0.45
Blast-cleaned, metallized with aluminium	0.45
Vinyl wash	0.25

Note: The hot-dip galvanized and roughened, and blast-cleaned, organic zinc rich paint shall not be used if more than half the load is due to gravity, or if frame deformation caused by joint slip into bearing cannot be tolerated.

Concrete Bearing

4.4.26 The design shall be based on the following allowable stress:

(a) On the full area of a concrete support, $p = 0.25 f_{ck}$

(b) On less than the full area of concrete support, $p = \gamma (0.25 f_{ck})$,

where,

f_{ck} = Characteristic compressive strength of concrete,

$$\gamma = (A_1/A_2)^{1/2} \leq 2,$$

A_1 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, and

A_2 = Bearing area of steel.

TABLE 4.5 : ALLOWABLE STRESS ON WELDS

Type of Weld and Stress ⁽¹⁾	Allowable Stress	Required Weld Strength Level ^{(2),(3)}
A. Complete-Penetration Groove Welds (Butt Welds)		
Tension normal to effective area	0.60 f_y of base metal	—
Compression normal to effective area	0.60 f_y of base metal	—
Tension or compression parallel to axis of weld	0.60 f_y of base metal	—
Shear on effective area	0.30 x (nominal tensile strength of weld metal (MPa), except shear stress on base metal shall not exceed 0.40 f_y of base metal).	—
B. Partial-Penetration Groove Welds (Butt Welds)⁽⁴⁾		
Compression normal to effective area	0.60 f_y of base metal	Weld metal with strength equal to or less than “matching” weld metal may be used.
Tension or compression ⁽⁵⁾ parallel to axis of weld	0.60 f_y of base metal	—
Shear parallel to axis of weld	0.30 x (nominal tensile strength of weld metal (MPa), except shear stress on base metal shall not exceed 0.40 f_y of base metal).	—

TABLE 4.5 : ALLOWABLE STRESS ON WELDS (Contd.)

Tension normal to effective area	0.30 x (nominal tensile strength of weld metal (MPa), except tensile stress on base metal shall not exceed $0.60 f_y$ of base metal)	—
C. Fillet Welds		
Shear on effective area	0.30 x (nominal tensile strength of weld metal (MPa), except shear stress on base metal shall not exceed $0.40 f_y$ of base metal)	—
Tension or compression ⁽⁵⁾ parallel to axis of weld	$0.60 f_y$ of base metal	—
D. Plug and Slot Welds		
Shear parallel to faying surfaces (on effective area)	0.30 x (nominal tensile strength of weld metal (MPa), except shear stress on base metal shall not exceed $0.40 f_y$ of base metal)	Weld metal with strength equal to or less than “matching” weld metal may be used

- Note: (1) For definition of effective area, see cl.2.8.8 to 2.8.24,
(2) The weld metal shall “match” the base metal,
(3) Weld metal 15% stronger than “matching” weld metal will be permitted,
(4) See cl. 2.8.11 for a limitation on use of partial-penetration groove welded joints,
(5) Fillet welds and partial-penetration groove welds joining the component elements of built-up members, such as flange-to-web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the weld axis,
(6) No increase in allowable stresses is allowed for OBE condition.

4.5 Combined Stresses

4.5.1 For combined stresses of axial compression and bending, and axial tension and bending, it shall be shown that summation of the ratios of design and corresponding permissible stresses would not exceed unity.

Axial Compression and Bending

4.5.2 Members subjected to axial compression and bending shall be proportioned to satisfy the following requirements:

$$\frac{\frac{ac,cal}{n_{ac}}}{ac} + \frac{C_{mx} \frac{bcx,cal}{n_{bcx}}}{s_1 n_{bcx}} + \frac{C_{my} \frac{bcy,cal}{n_{bcy}}}{s_2 n_{bcy}} \leq 1.0, \quad (4.23)$$

where,

$$s_1 = \left[1 - \frac{ac,cal}{(12/13)f_{ccx}} \right]$$

$$s_2 = \left[1 - \frac{ac,cal}{(12/13)f_{ccy}} \right]$$

following expression may be used in lieu of the above:

If the ratio $\left[\frac{ac,cal}{ac} \right]$ is less than 0.15,

$$\frac{\frac{ac,cal}{n_{ac}}}{ac} + \frac{\frac{bcx,cal}{n_{bcx}}}{bcx} + \frac{\frac{bcy,cal}{n_{bcy}}}{bcy} \leq 1.0. \quad (4.24)$$

The value of bcx and bcy to be used in the above equation shall each be lesser of the values of the allowable stresses bc given in cl.4.4.7 to 4.4.16 for bending about the appropriate axis.

4.5.3 At a support and using the values bcx and bcy at the support:

$$\frac{\frac{ac,cal}{0.60 f_y}}{ac} + \frac{\frac{bcx,cal}{n_{bcx}}}{bcx} + \frac{\frac{bcy,cal}{n_{bcy}}}{bcy} \leq 1.0. \quad (4.25)$$

Axial Tension and Bending

4.5.4 A member subjected to both axial tension and bending shall be proportioned so that the following condition is satisfied:

$$\frac{a_{t,cal}}{n_{at} \ a_t} + \frac{b_{tx,cal}}{n_{btx} \ b_{tx}} + \frac{b_{ty,cal}}{n_{bty} \ b_{ty}} \leq 1.0, \quad (4.26)$$

where,

$a_{c,cal}$ = calculated axial compressive stress,

$a_{t,cal}$ = calculated axial tensile stress,

$b_{c,cal}$ = calculated bending compressive stress in extreme fibre,

$b_{t,cal}$ = calculated bending tensile stress in extreme fibre,

a_c = allowable axial compressive stress,

a_t = allowable axial tensile stress,

b_c = allowable bending compressive stress in extreme fibre,

b_t = allowable bending tensile stress in extreme fibre,

f_{cc} = elastic critical stress in compression = $\frac{2E}{\lambda^2}$

= slenderness ratio in the plane of bending. = $(k l_c / r)$,

x = subscript relating symbol to strong axis bending,

y = subscript relating symbol to weak axis bending,

C_m = a coefficient whose value shall be taken as follows:

- (a) For members in frames where side sway is not prevented:

$$C_m = 0.85$$

- (b) For members in frames where side sway is prevented and not subject to transverse loading between their supports in the plane of bending:

$$C_m = (0.6 - 0.4 \quad) \geq 0.4, \quad (4.27)$$

where \quad is the ratio of smaller to the larger moments at the ends of that portion of the unbraced member in the plane of bending under consideration. \quad is positive when the member is bent in reverse curvature and negative when it is bent in single curvature,

- (c) For members in frames where side sway is prevented in the plane of loading and subjected to transverse loading between their supports; the value of C_m may be determined by rational analysis. In the absence of such analysis, the following values may be used:

- (i) For members whose ends are restrained against rotation:

$$C_m = 0.85.$$

- (ii) For members whose ends are unrestrained against rotation:

$$C_m = 1.00.$$

In equations (4.23) through (4.26) n_{ac} , n_{bcx} , n_{bcy} , n_{at} , n_{btx} , and n_{bty} are the ratios of permissible design stress (ref. section 4.3) to the corresponding allowable stress (see sec. 4.4) for axial compressive, bending compressive about x -axis, bending compressive about y -axis, axial tensile, bending tensile about x -axis, and bending tensile about y -axis respectively.

Shear and Torsion in Members

- 4.5.5 The torsion effects shall be considered in the design of members and the normal and shearing stresses due to torsion shall be added to those from all other loads, with the resultants not exceeding the allowable values.

Shear and Tension

- 4.5.6 (a) For bolts, studs and bars the cross-sectional area shall be considered additive when shear and tension are considered simultaneously,
- (b) The shear or tensile stress on actual cross-sectional areas for combined shear and tension shall satisfy the following interaction formula.

$$\frac{\sigma_{t,cal}}{n_{at} \sigma_{at}} + \frac{\tau_{v,cal}}{n_v \tau_v} \leq 1.0, \quad (4.28)$$

where,

σ_{at} = tensile stress produced by the same forces which shall not exceed the value for tension given in cl.4.4.21,

τ_v = allowable shear stress,

$\tau_{v,cal}$ = calculated shear stress, and

n_v = ratio of permissible design shear stress (see Table 4.2) to allowable shear stress (see Table 4.3).

5. ADDITIONAL REQUIREMENTS FOR BUILT-UP MEMBERS AND COLUMN BASES

5.1 General

- 5.1.1 The basic design of the built-up section is to be performed by satisfying the provisions of previous chapters. The additional requirements specified hereinafter shall also be satisfied.
- 5.1.2 The longitudinal spacing of connectors connecting the components of built-up compression members must be such that the slenderness ratio of individual shapes does not exceed $(3/4)$ times the effective slenderness ratio of the entire member. At least two intermediate connectors shall be used along the length of the built-up members.

5.2 Axially Loaded Members

Compression Members

- 5.2.1 All parts of built-up compression members and transverse spacing of their lines of fasteners shall meet the requirements of cl. 2.4.4 to 2.4.11.

End Connection

- 5.2.2 At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by bolts spaced longitudinally not more than 4 times the bolt diameter for a distance equal to 1.5 times the maximum width of the member, or by continuous welds having a length not less than the maximum width of the member.

Longitudinal Spacing of Intermediate Bolts and Welds

- 5.2.3 The longitudinal spacing of intermediate bolts or intermittent welds in built-up members shall be adequate to provide for the transfer of calculated stresses. However, where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed:

- (a) $330/(f_y)^{1/2}$ times the outside plate thickness or 300 mm whichever is smaller when fasteners are not staggered along adjacent guage lines,
- (b) $500/(f_y)^{1/2}$ times the outside plate thickness or 450 mm, whichever is smaller when fastners are staggered along adjacent guage lines.

The maximum longitudinal spacing of bolts or intermittent welds connecting two rolled shapes in contact with one another shall not exceed 600 mm.

Compression Members with Open Sides

5.2.4 Open sides of compression members built up from plates or structural shapes shall be provided with lacing with tie plates at each end, and at intermediate points if the lacing is interrupted. In the event tie plates at the ends are not feasible, such tie plates shall be as near the ends as practicable. In main members carrying calculated stresses, the end tie plates shall have a length not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than half this distance. The thickness of tie plates shall be not less than (1/50)th of the distance between the lines of fasteners or welds connecting them to the components of the members. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than 6 diameters and the tie plates shall be connected to each component by at least three fasteners. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than (1/3)rd the plate length.

Lacing

5.2.5 Lacing including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the ratio l_0/r of the flange included between their connections shall not exceed (3/4)th times the governing ratio for the member as a whole. The ratio l_0/r for lacing bars, arranged in single systems, shall not exceed 140. Double lacings shall be joined at their intersections and this ratio shall not exceed 200.

5.2.6 For lacing bars in compression the unsupported length of the lacing bar shall be taken as the distance between fasteners or welds connecting it to the components of the built-up member for single lacing and 70% of that distance for double lacings.

- 5.2.7 The inclination of lacing bars to the axis of the member should be not less than 60° for single lacing and 45° for double lacing.
- 5.2.8 When the distance between lines of fasteners or welds in the flanges is more than 400 mm, the lacing shall preferably be double or be made of angles.
- 5.2.9 Lacing system should be proportioned to resist a shear normal to the axis of the member equal to 2.5% of the total compressive force in the member.

Alternatives to Tie Plates and Lacings

- 5.2.10 The functions of tie plates and lacing may be performed by continuous cover plates perforated with access holes. The unsupported width of such plates at access holes, as defined in Table 2.1, is assumed available to resist axial stress, provided that
- (a) the width-to-thickness ratio conforms to the limitations of Table 2.1;
 - (b) the ratio of length (in direction of stress) to width of hole does not exceed 2;
 - (c) the clear distance between holes in the stress direction shall be not less than the transverse distance between nearest lines of connecting fasteners or welds; and
 - (d) the periphery of the holes at all points shall have a minimum radius of 40 mm.

Tension Members

- 5.2.11 Longitudinal Spacing of Fasteners and Intermittent Fillet Welds.
- (a) The longitudinal spacing of connectors between a plate and a rolled shape or two plate components in contact with each other, shall not exceed 24 times the thickness of the thinner plate nor 300 mm,
 - (b) The longitudinal spacing of connectors between two or more shapes in contact with one another in a tension member shall not exceed 600 mm,

- (c) Tension members composed of two or more shapes or plates separated from one another by intermittent fillers shall be connected to one another at these fillers at intervals such that the slenderness ratio of either component between the fasteners does not exceed 300 mm.

Cover Plates and Tie Plates

5.2.12 Either perforated cover plates or tie plates without lacing may be used on the open sides of built-up tension members. Tie plates shall have a length not less than (2/3)rd the distance between the lines of fasteners or welds connecting them to the components of the member. The thickness of such tie plates shall not be less than (1/50)th of the distance between these lines. The longitudinal spacing of fasteners or intermittent welds at tie plates shall not exceed 150 mm. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates will not exceed 300 mm.

5.3 Plate Girders and Rolled Beams

General

- 5.3.1 (a) Plate girders are distinguished from beams when the following condition is satisfied.

$$h_c/t_w > 2000/(0.66 f_y)^{1/2}. \tag{5.1}$$

- (b) Plate girders, beams with cover plates, and rolled or welded beams are, in general, proportioned by the moment of inertia of the gross section. No deduction need be made for bolt holes in either of the flanges, except in cases where the area reduction of either flange by such holes, calculated in accordance with the provisions of section 2.7, will exceed 15% of the gross flange area. In such cases, the excess is deducted.
- 5.3.2 Hybrid girders, i.e., girders made of steel of different grades or specifications, of which the flanges at any given section should have the same cross-sectional area and be made of the same grade of steel. They may be proportioned by the moment of inertia of their gross section, subject to the applicable provisions in the following clauses, provided that they are not

required to resist an axial force greater than $0.15 f_{yf}$ times the area of the gross section, where f_{yf} is the stated minimum yield stress of the flange material.

5.3.3 For permissible design stress refer section 4.3.

Proportioning of Flanges

5.3.4 The thickness of outstanding parts of flanges shall conform to cl.2.4.8. Flanges of welded plate girders may be varied in thickness or width by splicing a series of plates or by use of cover plates.

Flange Development

5.3.5 High strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent weld shall be in proportion to the shear intensity.

5.3.6 The longitudinal spacing should not exceed the maximum spacing permitted for compression and tension members provided in cl 5.2.3 and 5.2.11 respectively. Additionally, welds connecting flange to web shall be proportioned to transmit to the web all loads that may be applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

5.3.7 Partial length cover plates should be extended beyond the theoretical cut-off point, and the extended portion is attached to the beam or girder by high-strength friction bolts or fillet welds. These shall conform to the provisions of section 3.3 and cl. 4.4.21 to 4.4.25 to develop the cover plate portion of the flexural stress in the beam or girder at the theoretical cut-off point.

5.3.8 In addition, for welded cover plates, the welds connecting the cover plate termination to the beam or girder in length b' , defined below, shall be adequate, as per permissible stresses, to develop the cover plate's portion

of flexural stresses at the distance b' from the end of the cover plate. The length b' , measured from the end of the cover plate, shall be:

- (a) a distance equal to the cover plate width when there is a continuous weld equal to or larger than (3/4)th of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in b' ;
- (b) a distance equal to 1.5 times the width when there is a continuous weld smaller than (3/4)th of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in b' ; and
- (c) a distance equal to twice the cover plate width when there is no weld across the end of the plate, but continuous welds along both edges of the cover plate in b' .

Slenderness Limitations

- 5.3.9 (a) When no transverse stiffeners are provided or when transverse stiffeners are spaced more than 1.5 times the distance between flanges

$$h_c/t_w \leq \frac{97000}{[f_{yf}(f_{yf} + 114.5)]^{1/2}} \quad (5.2)$$

- (b) When transverse stiffeners are provided, spaced not more than 1.5 times the distance between flanges

$$h_c/t_w \leq \frac{5270}{[f_{yf}]^{1/2}} \quad (5.3)$$

In (a) and (b) above h_c is the clear distance between flanges (mm), t_w is the thickness of web (mm) and f_{yf} is the minimum yield stress of flange material.

- 5.3.10 In the case of welded crane gantry plate girders intended for carrying cranes with a lifting load of 15 tonnes or more, the web plate thickness shall be not less than 8 mm.

Stiffener Requirements for Concentrated Loads

5.3.11 Stiffeners shall be placed in pairs at unframed ends or at points of concentrated loads on the interior of beams, girders or columns if required by cl. 5.3.23 through 5.3.27 as applicable.

5.3.12 If required by cl. 5.3.23, 5.3.24 or equation (5.4) stiffeners need not extend more than one-half the web depth except as follows:

If stiffeners are required by cl. 5.3.25 to 5.3.27, these shall be designed as axially compressed members (columns) in accordance with requirements of section 4.3 and 4.4 with an effective length equal to $0.75 h_c$, a cross-section composed of two stiffeners and a strip of the web having a width of $25 t_w$ at interior stiffeners and $12 t_w$ at the ends of members.

5.3.13 When the load normal to the flange is tensile, the stiffeners shall be welded to loaded flange. When the load normal to flange is compressive, the stiffeners shall either bear on or be welded to the loaded flange.

5.3.14 (a) When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column web stiffeners having a combined cross-sectional area A_{st} not less than that computed from equation (5.4) shall be provided whenever the calculated value of A_{st} is positive.

$$A_{st} = \frac{P_{bf} f_{yc} t_{wc} (t_b + 5K)}{f_{yst}}, \quad (5.4)$$

where,

f_{yst} = stiffener yield stress (MPa),

K = distance between outer face of column flange and web toe of its fillet, if the column is a rolled shape, or equivalent distance if the column is a welded shape (mm),

t_{wc} = column web thickness (mm),

t_b = thickness of flange or moment connection plate delivering concentrated force (mm), and

P_{bf} = as defined in cl. 2.6.6.

- (b) Stiffeners required by the provisions of equation (5.4) and cl. 5.3.23 to 5.3.27 shall comply with the following criteria:
- (i) The width of each stiffener plus one-half the column web thickness shall be not less than one-third the flange width or moment connection plate delivering the concentrated force,
 - (ii) The thickness of stiffeners shall be not less than one-half the thickness of the flange or plate delivering the concentrated load,
 - (iii) The weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

5.3.15 Allowable Shear Stress with Tension Field Action

- (1) Except as hereinafter provided, the largest average web shear $v_{a,cal}$ (in MPa), computed for any condition of complete or partial loading, shall not exceed that specified in cl. 4.4.2,
- (2) Alternatively, for girders other than hybrid girders, if intermediate stiffeners are provided and spaced to satisfy the provisions of cl.5.3.16, and if $C_v \leq 1.0$, the allowable shear including tension field action given by equation (5.5) is permitted in lieu of the value given by equation (4.1) of cl. 4.4.2;

$$v_a = \frac{f_{yw} C_v}{2.89} + \frac{(1 - C_v) f_{yw}}{1.15[1 + (a/h_c)^2]^{0.5}} < 0.40 f_{yw}, \quad (5.5)$$

where a is the spacing between transverse stiffeners.

5.3.16 Intermediate Stiffeners

- (a) Subject to the limitations of cl.5.3.9 and 5.3.10, intermediate stiffeners are not required when the ratio (h_c/t_w) is less than 260, and the maximum web shear stress is less than that permitted by cl. 4.3.1 in which allowable shear stress is determined by equation (4.1),

- (b) The spacing of intermediate stiffeners, where stiffeners are required, shall be such that the web shear stress will not exceed the permissible value given in cl. 4.3.1 and using equations (4.1) and (5.5), as applicable, and the ratio (a/h_c) shall exceed neither $[260/(h_c/t_w)]^2$ nor 3.0,
- (c) In girders designed on the basis of tension field action, the spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes shall be such that $v_{a,cal}$ does not exceed the value given by the applicable equation (4.1) and (5.5).

5.3.17 Minimum Area and Moment of Inertia of Intermediate Stiffener

- (1) The moment of inertia of a pair of intermediate stiffeners, or a single intermediate stiffener, with reference to an axis in the plane of the web, shall not be less than

$$[h_c / 50]^4.$$

- (2) The gross area of the single or pair of intermediate stiffeners taken together in square mm, spaced as required for equation (5.5) shall be not less than that computed by equation.

$$A_g = \frac{1}{2} \frac{C}{h_c} \left[\frac{a}{h_c} + \frac{(a/h_c)^2}{\{1 + (a/h_c)^2\}^{1/2}} \right] Y h_c t_w, \quad (5.6)$$

where,

C_v , a , h_c and t_w are as defined earlier,

Y = yield stress ratio of web steel to yield stress of stiffener steel,

= 1.0 for stiffeners furnished in pairs,

= 1.8 for single angle stiffeners, and

= 2.4 for single plate stiffeners.

- (3) When the greatest shear stress is less than that permitted by equation (5.5), the gross area (A_g) requirement may be reduced by same proportion,
- (4) Intermediate stiffeners, required by equation (5.6) shall be connected for a total shear transfer (in Newtons) per linear mm of single stiffener or pair of stiffeners, not less than that computed as follows,

$$v_s = h_c [(f_{yw}/650)^3]^{0.5}, \quad (5.7)$$

where,

f_{yw} = yield stress of the web steel,

v_s = shear between girder web and transverse stiffeners, and

h_c = clear distance between flanges.

- (5) The shear transfer may be reduced in the same proportion by which the largest computed shear stress in the adjacent panels falls below the permissible value given by cl. 4.3.1 and equation (5.5). However, welds for intermediate stiffeners required to transmit to the web an applied concentrated load or reaction, shall be proportioned for not less than the applied load or reaction,
- (6) If intermittent fillet welds are used, the clear distance between fillet welds, connecting stiffeners to the girder web, shall neither be more than 16 times the web thickness nor more than 250 mm,
- (7) Intermediate stiffeners may be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not closer than 4 times, nor more than 6 times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the plate,
- (8) When lateral bracing is attached to a stiffener, or a pair of stiffeners, these in turn, shall be connected to the compression flange to transmit

1% of the total flange stress, unless the flange is composed only of angles.

5.3.18 Reduction in Allowable Flange Stress

- (1) When the depth-to-thickness ratio of the web exceeds $2000/(t_w)^{1/2}$, the allowable bending stress (MPa) in the compression flange is given by

$$f'_b = R_{PG} R_e f_b \quad (5.8)$$

where,

f_b = allowable bending stress given in cl. 4.4.8 to cl. 4.4.17 (MPa)

$$R_{PG} = 1 - 0.0005 \frac{A_w}{A_f} \left[\frac{h_c}{t_w} - \frac{2000}{(t_w)^{1/2}} \right] \leq 1.0, \quad (5.9)$$

$$R_e = \frac{12 + (A_w/A_f)(3 - c/c^3)}{12 + 2(A_w/A_f)} \leq 1.0, \quad (5.10)$$

$R_e = 1.0$ for non-hybrid girders,

where A_w is the area of web at the section under investigation (mm^2), A_f is the area of compression flange (mm^2) and c is $0.6 f_{yw}/f_b \leq 1.0$.

5.3.19 Combined Shear and Tension Stress

Plate girder webs which depend on tension field action, as provided in equation (5.5) shall be so proportioned that bending tensile stress, due to moment in the plane of the girder web, shall not exceed

$$[0.825 - 0.375(v_{a,cal}/v_a)] f_{yw} \leq 0.60 f_y, \quad (5.11)$$

where, $v_{a,cal}$ is the computed average web shear stresses, and v_a is the allowable maximum web shear stress according to equation (5.5). The allowable shear stress in the webs of girders having $f_y > 450$ steel in flange

and web shall not exceed the values given by equation (4.1) if the flexural stress in the flange, $\sigma_{b,cal}$ exceeds $0.75 \sigma_b$.

Splices

5.3.20 Groove welded splices in plate girders and beams shall develop the full strength of the smaller spliced section. Other types of splices in cross-sections of plate girders and beams shall develop the strength required by stresses at the point of splice.

Horizontal Forces

5.3.21 The flanges of plate girders supporting cranes or other moving loads shall be proportioned to resist the horizontal forces produced by such loads.

Webs and Flanges under Concentrated Forces

5.3.22 Design Basis

- (a) Members with concentrated loads applied normal to one flange and symmetric to the web shall have a flange and web proportioned to satisfy the local flange bending, local web yielding, web crippling and sideway web buckling criteria of cl. 5.3.23, 5.3.24, 5.3.25 and 5.3.26 respectively. Members with concentrated loads applied to both flanges shall have a web proportioned to satisfy the web yielding, web crippling and column web buckling criteria of cl. 5.3.24, 5.3.25 and 5.3.27 respectively,
- (b) Where pairs of stiffeners are provided on opposite sides of the web, at concentrated loads, and extend at least half the depth of the member, cl. 5.3.23 and 5.3.24 need not be checked.

5.3.23 Local Flange Bending

A pair of stiffeners shall be provided opposite the tension flange or flange plate of the beam or girder framing into the member when the thickness of the member flange t_f satisfies the following

$$t_f < 0.4 [P_{bf}/f_{yc}]^{1/2}, \quad (5.12)$$

where f_{yc} is column yield stress (MPa), and P_{bf} is defined in cl. 2.6.6.

If the loading length measured across the member flange is less than $0.15b$, where b is the member flange width, equation (5.12) need not be checked.

5.3.24 Local Web Yielding

Bearing stiffeners shall be provided if the compressive stress at the web toe of the fillets resulting from concentrated loads exceeds $0.66 f_y$.

- (a) When the force to be resisted is a concentrated load-producing tension or compression, applied at a distance from the member end that is greater than the depth of the member,

$$R/\{t_w (N+5K)\} \leq 0.66 f_y, \quad (5.13)$$

- (b) When the force to be resisted is a concentrated load applied at or near the end of the member,

$$R/\{t_w (N+2.5K)\} \leq 0.66 f_y, \quad (5.14)$$

where, R is the concentrated load or reaction (Newton), t_w is the thickness of web, N is the length of bearing (not less than K for end reactions) and K is the distance from outer face of flange to web toe of fillet.

5.3.25 Web Crippling

Bearing stiffeners shall be provided in the webs of members under concentrated loads, when the compressive force exceeds value of R (in Newton) given by

- (a) When the concentrated load is applied at a distance not less than $D/2$ from the end of the member:

$$R = 177 t_w^2 [1+3(N/D) (t_w/t_f)^{1.5}] (f_{yw} t_f/t_w)^{1/2}, \quad (5.15)$$

- (b) When the concentrated load is applied at a distance less than $D/2$ from the end of the member:

$$R = 89 t_w^2 [1+3(N/D) (t_w/t_f)^{1.5}] (f_{yw} t_f/t_w)^{1/2} \quad (5.16)$$

where in (a) and (b) above f_{yw} is the specified minimum yield stress of beam web, D is the overall depth of the member, and t_f is the flange thickness. All other terms as defined in cl. 5.3.24. If stiffeners are provided and extend at least one-half the web depth, equations (5.15) and (5.16) need not be checked.

5.3.26 Sidesway Web Buckling

Bearing stiffeners shall be provided in the webs of members with flanges not restrained against relative movement by stiffeners or lateral bracing and subject to concentrated compressive loads, when the compressive force exceeds the following limits :

- (1) If the loaded flange is restrained against rotation and $(d_c/t_w)/(l_c/b_f)$ is less than 2.3

$$R = \frac{47000 t_w^3}{h_c} \left[1 + 0.4 \frac{(d_c/t_w)^3}{(l_c/b_f)^3} \right], \quad (5.17)$$

- (2) If the loaded flange is not restrained against rotation and $(d_c/t_w)/(l_c/b_f)$ is less than 1.7

$$R = \frac{47000 t_w^3}{h_c} \left[0.4 \frac{(d_c/t_w)^3}{(l_c/b_f)^3} \right]. \quad (5.18)$$

In (1) and (2) above, l_c is the largest laterally unbraced length along either flange at the point of load (mm), b_f is the flange width (mm), d_c is the web depth clear of fillets (mm).

Equations (5.17) and (5.18) need not be checked provided that the ratio $(d_c/t_w)/(l_c/b_f)$ exceeds 2.3 and 1.7 respectively, or for webs subject to uniformly distributed load.

5.3.27 Compression Buckling of the Web

A stiffener or a pair of stiffeners shall be provided opposite the compression flange when the web depth clear of fillets d_c (mm) is greater than

$$\frac{10750 (t_{wc})^3 (f_{yc})^{1/2}}{P_{bf}}, \quad (5.19)$$

where t_{wc} is the column web thickness and P_{bf} is specified in cl. 2.6.6.

Open Box Type Beams and Grillages

5.3.28 Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals not exceeding 1500 mm. Through-bolts and separators may be used, provided, that in beams having a depth of 300 mm or more at least two bolts shall be used at each separator location.

5.3.29 When concentrated loads are transferred from one beam to the other, or distributed between beams, diaphragms having sufficient stiffness to distribute the load shall be provided between the beams. Where beams will be exposed, these shall be sealed against corrosion of interior surfaces or spaced sufficiently apart to permit cleaning and painting.

5.4 Column Bases

5.4.1 Column bases, including base plates, base slabs, pedestals or other bearing members shall be designed, fabricated and installed to safely permit the transfer of design loads, maintain plumbness and permit easy maintenance.

5.4.2 Proper provisions shall be made to transfer column loads and moments to footings and foundations.

5.4.3 Column bases shall be set level to correct elevation with full bearing.

Design Details

5.4.4 Gusseted Bases

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with bearing area of the shaft, shall be sufficient to take loads, bending moments and reactions to base plate without exceeding specified stresses. All bearing surfaces shall be machined to ensure satisfactory contact or as specified. Where ends of the column shaft and the gusset plates may not be faced for complete bearing, the fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

5.4.5 Column and Base Plate Connections

Where the column end is connected directly to the base plate using full penetration butt welds, the connection shall be deemed to transmit to the base all forces and moments to which the column is subjected.

Column Base Plate

5.4.6 Columns with slab bases need not be provided with gussets, but fastenings shall be provided sufficient to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. With reference to Fig. 5.1:

- (a) When values of m and n are small (the base plate is just large enough in area to accommodate the column profile), as shown in Fig. 5.2, the slab base thickness shall be obtained as

$$t_p = L [3 p / b_s]^{1/2}, \quad (5.20)$$

where L is obtained from

$$p = P/[2(D+b_1 - 2L)L], \quad (5.21)$$

- (b) Unless m and n dimensions are small, the base plate is designed as a cantilever beam fixed at the edges of a rectangle whose sides are $0.80 b_1$ and $0.95 D$. P is assumed to be distributed uniformly over

the base plate within the rectangle. Assuming $b_s = 0.75 f_y$, the slab base thickness shall be greater of

$$t_p = 2m \left(\frac{p_{cal}}{b_s} \right)^{1/2}, \quad (5.22)$$

and

$$t_p = 2n \left(\frac{p_{cal}}{b_s} \right)^{1/2}, \quad (5.23)$$

p_{cal} in (b) above shall be limited to p as defined in cl. 4.4.18.

Where P is the total column load (Newton), b_s is the allowable bending stress in slab base, p is the allowable bearing pressure on support, p_{cal} is the actual bearing pressure, f_{ck} is the characteristic strength of concrete, and t_p is the of base plate thickness.

5.4.7 Thickness of column base plate lower than the value given by cl. 5.4.6 is acceptable when justified by appropriate analysis.

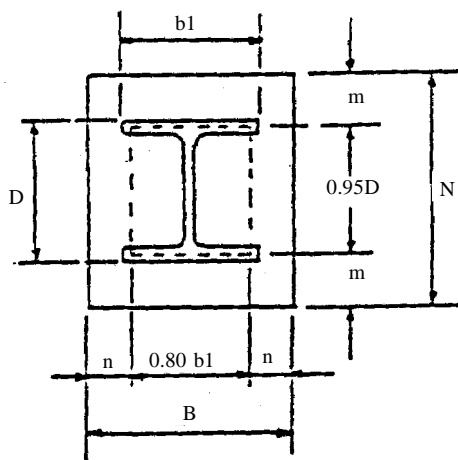


Fig. 5.1

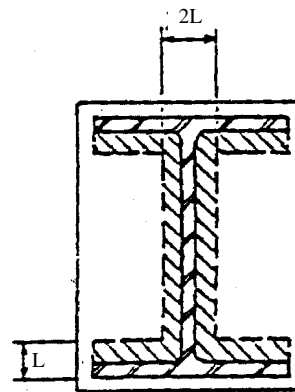


Fig. 5.2

5.4.8 Base Plates and Bearing Plates

- (a) Base plates and grillages of stanchions and bearing and spreaders of beams and girders shall be of adequate strength, stiffness and area, to spread the load on the bearing surfaces without exceeding permissible stress on such bearing element under design loads and bending moments,
- (b) When the load on cap or under the base is not uniformly distributed or where the end of column shaft is not machined with cap or base, or where the cap or base is not square in plan, the calculated design of base plate shall be based on allowable stress of $0.75 f_y$,
- (c) The cap or base plate shall be not less than $1.5 (d_0 + 75)$ mm in length or diameter, where d_0 is the equivalent diameter of the column at the bearing end.

5.4.9 Compression Members with Bearing Joints

- (a) When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place,
- (b) When other compression members are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportional to 50% of the strength of the member,
- (c) All compression joints shall be proportioned to resist any tension developed by the specified lateral loads acting in conjunction with 75% of the calculated dead load stress and no live load.

5.4.10 Finishing

Column bases and base plates shall be finished in accordance with the following requirements:

- (a) Rolled steel-bearing plates 50 mm or less thick may be used without milling, provided that a specified contact bearing is achieved,

- (b) Rolled steel-bearing plates over 50 mm but not exceeding 100 mm thick may be straightened by pressing or by milling for all bearing surfaces, except as in item (e),
- (c) Rolled steel-bearing plates over 100 mm thick shall be milled for all bearing surfaces, except as in item (e),
- (d) Column bases other than rolled steel-bearing plates shall be milled for all bearing surfaces, except as in item (e),
- (e) The bottom surfaces of bearing plates and column bases grouted to ensure full bearing contact on foundations, need not be milled,
- (f) The top surfaces of base plates with columns full-penetration welded need not be pressed or milled.

Finishing/End Milling of Columns at Splices and Base Plates

5.4.11 Column ends bearing on each other at splices, or resting on base plates, and in general, compression joints designed for contact bearing shall have their matching surfaces milled true and square parallel to each other to ensure proper bearing and alignment. Extreme care shall be taken to mill the ends to ensure that the faces are parallel to each other, to meet the following requirement:

Joints which depend on contact bearing, when assembled in the shop, shall bear evenly with respect to the centroid of contact area and at least 80% of the entire contact area shall be in full bearing when ends of a column are aligned to within 1 in 1500 of their combined length, and the separation of any remaining portion shall not exceed 0.25 mm.

5.4.12 Optical methods for checking the set-up before and after machining shall be employed to check parallelism.

5.5 Corrosion-Resistant Clad and Lined Members

5.5.1 Structural steel components which are fabricated from integrally-clad plate, or which are lined on one or more sides with corrosion-resistant plate to the base metal shall be designed based on of the base plate thickness exclusive of the thickness of the lining or cladding.

- 5.5.2 If the lining or cladding adversely affects stresses in the base plate due to its effect on elastic properties of the structural component then the presence of the internal lining or cladding shall be considered when performing design analysis.
- 5.5.3 The presence of cladding shall be considered with respect to thermal effects.

6. COMPOSITE CONSTRUCTION

6.1 General

6.1.1 Composite construction can take the form of composite columns, composite beams, trusses and slabs. In composite construction, the concrete may cover the steel partly or fully. Either the bond between the main steel member and concrete is sufficient to engage the two dissimilar materials concrete and steel in acting together or shear connectors are used to make the concrete act with the steel member.

6.1.2 (a) When the slab extends on both sides of the beam, the effective width of the concrete flange shall be the smallest of

$$\begin{aligned} &L/4, \\ &16t + b_0, \text{ and} \\ &b_0 + 1/2(s_1 + s_2). \end{aligned}$$

where L is the span of the beam, t is the slab thickness, b_0 is the width of compression flange of beam, and s_1, s_2 are the clear distances to the adjacent beams on the two sides.

(b) When the slab is present only on one side of the beam, the effective width of concrete flange shall be taken as the smallest of

$$\begin{aligned} &L/12, \\ &6t + b_0 \text{ and} \\ &s_1/2 + b_0. \end{aligned}$$

6.1.3 A beam totally encased in concrete cast integrally with the slab may be assumed to be connected to the concrete by natural bond, without additional anchorage provided that:

- (a) concrete cover over beam sides and soffit is atleast 50 mm;
- (b) the beam top is at least 40 mm below the top and 50 mm above the bottom of the slab; and

- (c) the encasement has adequate mesh or other reinforcing steel throughout the whole depth and across the soffit of the beam to prevent spalling of the concrete.

6.1.4 When shear connectors are provided in accordance with cl. 6.4.2 to 6.4.11, encasement of the beam to achieve composite action is not required.

6.2 Composite Beams or Girders with Formed Steel Decks

General Requirements

6.2.1 Composite beams or girders with formed steel decks shall have the following limitations

- (a) provision of this chapter applies to decks with nominal rib height not greater than 75 mm;
- (b) the average width of concrete rib or haunch w_r shall be not less than 50 mm; in calculations it shall not be taken as more than the minimum clear width near the top of the steel deck; see cl. 6.2.3, subparagraphs 2 and 3, for additional provisions;
- (c) the concrete slab shall be connected to the steel beam or girder with welded shear connectors. They may be welded through the deck or directly to the steel member;
- (d) total slab thickness including ribs shall be used to determine the effective width of concrete flange; and
- (e) the slab thickness above the steel deck shall be not less than 50 mm.

Deck Ribs Oriented Perpendicular to Steel Beam or Girder

6.2.2 Composite beams or girders with formed steel decks, having deck ribs perpendicular to steel beams or girders, shall have the following limitations:

- (a) concrete below the top of the steel deck shall be neglected when determining section properties, and in calculating A_c in equation (6.3);

- (b) the spacing of stud shear connectors along the length of a supporting beam or girder shall not exceed 800 mm; and
- (c) to resist uplift, the steel deck shall be anchored to all compositively designed steel beams or girders at a spacing not to exceed 400 mm; such anchorage should be provided by stud connectors or other devices specified by the designer.

Deck Ribs Oriented Parallel to Steel Beam or Girder

6.2.3 Composite beams or girders with formed steel decks, having deck ribs parallel to steel beams or girders, shall have the following limitations:

- (a) concrete below the top of the steel deck may be included when determining section properties and shall be included in calculating A_c in equation (6.3);
- (b) steel deck ribs over supporting beams or girders may be split longitudinally and separated to form a concrete haunch; and
- (c) when the nominal depth of steel deck is 40 mm or greater, the average width W_r of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus 4 stud diameters for each additional stud.

6.3 Design Considerations

Encased Beams

6.3.1 Encased beams shall be proportioned to support, unassisted, all dead loads applied prior to the hardening of the concrete (unless these loads are supported temporarily on shoring) and acting in conjunction with the slab, to support all dead and live loads applied after hardening of the concrete without exceeding a computed bending stress of $0.66 f_y$.

6.3.2 Bending stress produced by loads, after the concrete has hardened, shall be computed on the basis of section properties of the composite section. For this purpose, concrete shall be assumed to resist no tensile stresses.

Alternatively, the steel beam alone may be proportioned to resist, unassisted, the positive moment produced by all loads, live and dead, using a bending stress equal to $0.76 f_y$ in which case temporary shoring is not required.

Shear Connectors

- 6.3.3 When shear connectors are used in accordance with provisions of cl. 6.4.2 to 6.4.11, the composite section shall be proportioned to support all loads without exceeding the allowable stresses in chapter 4 even when the steel section is not shored during construction. In positive moment areas, the steel section is exempt from compact flange criteria of cl. 2.4.9 and there is no limit on the unsupported length of the compression flange. Where it is neither feasible nor necessary to provide adequate connectors to satisfy the horizontal shear requirements for full composite action, the effective section modulus shall be determined as:

$$Z_{\text{eff}} = Z_{sb} + (Z_{tr} - Z_{sb}) [(V'_h / V_h)^{1/2}], \quad (6.1)$$

where V_h is given by equation (6.3) or (6.4) or (6.5), V'_h is the total horizontal shear to be resisted by connectors in providing partial composite action, Z_{sb} is the section modulus of the steel beam referred to its bottom flange (mm^3) and Z_{tr} is the section modulus of the transformed composite section referred to its bottom flange (mm^3) based on permitted maximum effective width of concrete flange (cl.6.1.2).

Reinforcement

- 6.3.4 Reinforcement parallel to beam within the effective slab width, when anchored in accordance with the provisions of AERB/SS/CSE-1 may be included in computing the properties of composite sections, provided shear connectors are furnished in accordance with the requirements of cl. 6.4.3 to 6.4.12. The section properties of composite section shall be computed in accordance with elastic theory. Concrete in tension zone shall be neglected. For stress computations, and deflection calculations, the transformed section properties shall be based on modular ratio.

Construction without Temporary Shoring

- 6.3.5 In the case of construction without temporary shoring, stress in the steel section may be computed on the basis of section modulus, Z_{tr} , of the transformed section, provided that the numerical value of Z_{tr} so used shall not exceed:

$$Z_{tr} = [1.35 + 0.35(M_t/M_D)]Z_s \quad (6.2)$$

where M_t is the section moment due to all causes subsequent to the time when the concrete will have reached 75% of its required strength, M_D is the section moment caused by loads applied prior to the time of M_t and Z_s is the section modulus of steel beam referred to the flange where stress is being computed.

- 6.3.6 At sections subject to positive bending moment, the stress shall be computed for steel tension flange. At sections subject to negative bending moment, stresses shall be computed for steel tension and compression flanges. These stresses shall not exceed the appropriate values in sections 4.3 and 4.4. The stress limit may be applied to stresses in steel beam in negative moment area of unshored composite beams only if the compression flange is adequately braced.
- 6.3.7 The actual section modulus of transformed composite section shall be used in calculating the concrete flexural compression stress and for construction without temporary shores, this stress shall be based on loading applied after the concrete has reached 75% of its required strength. The stress in concrete shall not exceed $0.36 f_{ck}$.

6.4 Shear

- 6.4.1 The end connections and the steel beam web shall be designed to carry the total reaction.

Shear Connectors

- 6.4.2 Except in encased beams, as defined in cl. 6.1.1, the entire horizontal shear at the junction of steel beam and concrete slab shall be assumed to be transferred by shear connectors welded to the top flange of the beam and embedded in the concrete.

6.4.3 For full composite action of steel and concrete, subject to flexural compression, the total horizontal shear to be resisted between the point of maximum positive moment and points of zero moment shall be taken as the smaller value using following formulae:

$$V_h = 0.34 f_{ck} A_c, \quad (6.3)$$

and

$$V_h = 0.5 f_y A_s, \quad (6.4)$$

where V_h is the horizontal shear, f_{ck} is the characteristic strength of concrete, A_c is the effective area of concrete (refer cl.6.1.2), and A_s is the effective cross sectional area of the member.

6.4.4 In continuous composite beams, where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions, the total horizontal shear resisted by shear connectors between an interior support and each adjacent point of contraflexure shall be taken as

$$V_h = 0.5 f_{yr} A_{sr}, \quad (6.5)$$

where A_{sr} is the area of longitudinal reinforcement steel at the interior support located within the effective flange width specified in cl.6.1.2, and f_{yr} is the specified minimum yield stress of the longitudinal reinforcing steel.

6.4.5 For full composite action, the number of connectors on each side of the section of maximum moment shall be not less than N_r determined from the relationship

$$N_r = (V_h/q), \quad (6.6)$$

where q is the allowable shear load per connector.

6.4.6 For partial composite action with concrete subject to flexural compression, the horizontal shear, V_h' , to be used in computing Z_{eff} shall be taken as the product of q [ref. equations (6.10) to (6.12)] with the number of connectors furnished between the point of maximum moment and the nearest point of zero moment.

6.4.7 The value of V_h' shall be not less than one-fourth the lesser of the values obtained from formulae (6.3) and (6.4), using the maximum permitted effective width of the concrete flange. The effective moment of inertia for deflection computations shall be determined by

$$I_{\text{eff}} = I_s + (I_{tr} - I_s)[(V_h'/V_h)^{1/2}], \quad (6.7)$$

where I_s is the Moment of Inertia of the steel beam, and I_{tr} is the Moment of Inertia of the transformed composite section.

6.4.8 Connectors required on each side of the point of maximum moment in an area of positive bending may be uniformly distributed between that point and adjacent points of zero moment, except that N_2 , the number of shear connectors, required between any concentrated load in that area and the nearest point of zero moment, shall be not less than that determined by the following :

$$N_2 = N_1 \{(M/M_{\text{max}}) - 1\} / (1 - 1), \quad (6.8)$$

where M is the moment (less than the maximum moment M_{max}) at a concentrated load point, and N_1 is the Number of connectors required between the point of maximum moment and point of zero moment, determined by the relationship (V_h/q) or (V_h'/q) , as applicable.

$$= (Z_{tr}/Z_s) \text{ or } (Z_{\text{eff}}/Z_s), \text{ as applicable.} \quad (6.9)$$

6.4.9 For a continuous beam, connectors required in the region of negative bending may be uniformly distributed between the point of maximum moment and each point of zero moment.

6.4.10 Shear connectors, basically of two types (studs and channels) shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. Unless located directly over the web, the diameter of studs shall be not greater than 2.5 times the flange thickness to which they are welded. The minimum centre-to-centre spacing of stud connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam, and 4 diameters transverse to the longitudinal axis of the supporting composite beam. The maximum centre-to-centre spacing of stud connectors shall not exceed 8 times the total slab thickness.

Capacity (Allowable Load) of Stud Shear Connector

- 6.4.11 (1) For a stud shear connector embedded in a slab on a formed steel deck, when the deck ribs are oriented perpendicular to the steel beam, the stud capacity (allowable load) shall be as per equations (6.3) or (6.4) and multiplied by the following reduction factor.

$$[\{0.85/(N_r)^{0.5}\}(w_r/h_r)][(H_s/h_r) - 1.0] \leq 1.0, \quad (6.10)$$

- (2) Whereas in the case of deck ribs oriented parallel to steel beam, the stud capacity shall be as per equations (6.3) and (6.4) except when the ratio of w_r/h_r is less than 1.5, the allowable load shall be multiplied by the following reduction factor;

$$0.6(w_r/h_r)[(H_s/h_r) - 1.0] \leq 1.0, \quad (6.11)$$

where h_r is the nominal rib height (mm), H_s is the length of stud connector after welding (mm) not to exceed $(h + 75)$ in computations, although the actual length may be greater, N_r is the number of stud connectors on a beam in one rib not to exceed three in computations, although more than three studs may be installed, and w_r is the average width of concrete rib or haunch (mm).

Allowable Loads for Channel Shear Connectors

- 6.4.12 For channel shear connectors embedded in a solid concrete slab

$$q = 3.35(t_f + 0.5 t_w) L_c (f_{ck})^{1/2}, \quad (6.12)$$

where t_f is the flange thickness of channel shear connector (mm), t_w is the web thickness of channel shear connector (mm), and L_c is the length of channel shear connector (mm).

6.5 Special Cases

- 6.5.1 When composite construction does not conform to the requirements of cl. 6.1.2 through 6.4.12 the allowable load per shear connector shall be established through tests.

6.6 Composite or Encased Columns

Design Conditions

6.6.1 A member may be designed as encased column when the following conditions are fulfilled:

- (a) the member is of symmetrical I shape or a single I beam or channels back-to-back with or without flange plates;
- (b) the column is unpainted and solidly encased in dense concrete, with 20 mm aggregate (unless solidity can be obtained with a larger aggregate);
- (c) the minimum width of solid concrete casing is equal to $b + 100$ mm, where b is the width of steel flange;
- (d) the surface and edges of steel column have a concrete cover not less than 50 mm;
- (e) the casing is effectively reinforced with steel wires. The wire shall be at least 5 mm in diameter and the reinforcement shall be in the form of stirrups or binding at not more than 150 mm pitch so arranged as to pass through the centre of the covering of the edges and outer faces of the flanges and supported by longitudinal spacing bars not less than four in number; and
- (f) steel cores in encased columns shall be accurately machined at splices and provisions made for column alignment. At the column base provision shall be made to transfer the load to the footing at safe unit stresses in accordance with AERB/SS/CSE-1.

Design of Member

6.6.2 Radius of Gyration

Steel section shall be considered as carrying the entire load but allowance may be made by assuming the radius of gyration r of the column section about the axis in the plane of its web to be $0.2 (b + 100)$ mm, where b is the width of the steel flange (in mm). The radius of gyration about its other axis shall be taken as that of uncased section.

6.6.3 Axial Load

The axial load on the encased column shall not exceed twice that permitted on the uncased section, nor shall the slenderness ratio of the uncased section for its full length (centre-to-centre distance of connections) exceeds 250.

6.6.4 Allowable Axial Load

In computing the allowable axial load on encased strut, the concrete shall be taken as assisting in carrying the load over its cross-section, any cover in excess of 75 mm from the overall dimensions of the steel section of the encased strut being ignored.

The allowable compressive load under normal design condition shall be as follows:

$$P = A_s \sigma_{ac} + A_c \sigma_c \quad (6.13)$$

where σ_c is allowable direct compressive stress in concrete and A_s is the area of encased steel core and longitudinal reinforcement in the strut.

7. PLASTIC DESIGN

7.1 General

- 7.1.1 Provisions of this chapter apply for safety assessment of steel structures of existing plants. No structures of new projects shall be designed using plastic design methodology.
- 7.1.2 Safety of structures or part of a structure may be assessed on the basis of plastic design based on their maximum strength using provisions contained in this chapter provided the requirements of previous chapters, specially chapter 2 are complied with.
- 7.1.3 For structural steel the section strength required to resist design load shall be determined based on plastic design capacity and using the provisions of following clauses.

General Requirements

- 7.1.4 Plastic design for structures like single span and continuous beams, braced and unbraced rigid frames, and similar portions of structures, rigidly constructed so as to have at least one degree of static indeterminacy are acceptable.

It is not recommended that crane runways be designed continuous over interior vertical supports on the basis of maximum strength. However, rigid frame bents supporting crane runways may be considered as coming within the scope of provisions of this chapter.

- 7.1.5 Where plastic design is used as the basis for proportioning continuous beams and structural frames, provisions relating to allowable stresses, contained in previous chapters are waived. Except as modified in the following, however, all other pertinent provisions of the previous chapters shall govern.
- 7.1.6 (a) Steel with yield stress of 450 MPa or higher shall not be used if the design is based on plastic theory,
- (b) Steel having an ultimate tensile stress-yield stress ratio of 1.30 or less, shall not be used if the design is based on plastic theory.

7.2 Materials

7.2.1 Structural Steel

Acceptable structural steel materials for plastic design include all materials specified in section 2.2 with the exception of austenitic stainless steels and structural pipe and tubing.

7.2.2 Bolt Materials

Acceptable bolt materials are identified in section 2.2.

7.3 Loads and Load Combinations

7.3.1 Structures and structural members shall be designed to have design strength (determined according to plastic design method) at all sections at least equal to the required strength calculated for the factored load and forces given in the following clauses.

7.3.2 The effects created by all individual loads given in section 4.2 shall be considered in the design.

7.3.3 Load combinations for plastic design shall satisfy the following expression:

$$F_{di} = \sum_{ij} f_{ij} F_{ij} \quad (7.1)$$

where F_{di} is the i th load combination, f_{ij} an load factors of i th load combinations for j th characteristic value individual load, and F_{ij} are j th characteristics (individual) load for i th load combination.

7.3.4 Unless specified otherwise, load combinations given in Table 7.1 shall be used.

- (a) Load combination number 14 of LC6 is applicable for internal structure of reactor building,
- (b) For safety class-3 DC3 structures which do not perform the safety functions as specified in cl. 4.2.9(3), the applicable load combinations are LC1 and LC2.

- (c) For safety class-4 DC3 structures applicable load combinations are LC1 and LC2. Load factors for load combination numbers 5 and 6 is 1.3,
- (d) In the above load combination, wherever applicable impact effect of moving load is included in live load,
- (e) Effects due to creep, support settlement, etc. are used in dead load.

7.4 Design Considerations

Basis for Determination of Section Strength

- 7.4.1 For one or two-storey frames, the maximum section strength may be determined by a routine plastic analysis procedure. The P effect may be ignored. For braced multi-storey frames, provision shall be made to include the P effect in the design of bracing systems and frame members. For unbraced multistorey frames, the P effect shall be directly included in the calculations for maximum strength.

Joint Rigidity

- 7.4.2 Rigid frames shall have sufficient rigidity to hold virtually unchanged the original angles between connecting members. This does not preclude the use of some simple connections, provided that provisions of cl.7.4.1 are satisfied. Type 2 construction (ref. cl. 2.1.7) is permitted for members between rigid frames. Connections joining a portion of a structure, designed on the basis of plastic behaviour, with a portion, not so designed, need be no more rigid than ordinary seat-and-top-angle or ordinary web connections.

Members subjected to Flexure

- 7.4.3 The calculated maximum moment capacity of a member subjected to flexure shall be

$$M_p = Z_p f_y, \quad (7.2)$$

where Z_p is the plastic modulus of section, and f_y is the yield stress of the material.

TABLE 7.1 : LOAD COMBINATIONS FOR PLASTIC DESIGN

Design Condition and Load Combination Type	Load Factors f_{ij}																	
	DL	(1) LL	(2) H	R_0	T_0	P_t	P_v	$E'_0 / WC / FF$	E'_{SS} / W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	$MA/ME / MI/MT/M_t$	F_h	
Normal Design Condition																		
LC1 Normal Load Combinations	1.7	1.7	1.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1.7	1.7	1.7	1.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1.3	1.3	1.3	1.3	1.3	-	-	-	-	-	-	-	-	-	-	-	-	-
LC2 Severe Environmental Load Combinations	0.9	-	-	-	-	-	-	1.7	-	-	-	-	-	-	-	-	-	-
	1.7	1.7	1.7	-	-	-	-	1.7	-	-	-	-	-	-	-	-	-	-
	1.7	1.7	1.7	1.7	-	-	-	1.7	-	-	-	-	-	-	-	-	-	-
	1.3	1.3	1.3	1.3	1.3	-	-	1.3	-	-	-	-	-	-	-	-	-	-
Abnormal Design Condition																		
LC3 Extreme Environmental Load Combinations	0.9	-	-	-	-	-	-	-	1.0	-	-	-	-	-	-	-	-	-
	1.0	1.0	1.0	-	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-
	1.0	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	-	-	-	-	-	-
	1.0	1.0	1.0	1.0	1.0	-	-	1.0	1.0	1.0	1.0	-	-	-	-	-	-	-

TABLE 7.1 : LOAD COMBINATIONS FOR PLASTIC DESIGN (contd.)

Design Condition and Load Combination Type	Load Factors f_{ij}																	
	DL	(1) LL	(2) H	R_0	T_0	P_t	P_v	$E_o'/WC/FF$	E_{SS}/W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA/ME $MU/MT/M_t$	F_h	
LC4 Abnormal Load Combinations	1.0	1.0	-	-	-	-	-	-	-	1.25	1.0	1.0	-	-	-	-	-	-
LC5 Abnormal Severe Environmental Load Combinations	1.0	1.0	-	-	-	-	-	1.15	-	1.15	1.0	1.0	1.0	1.0	1.0	-	-	-
LC6 Abnormal Extreme Environmental Load Combinations	1.0	1.0	1.0	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	-	-

Note : (1) All load combinations shall be checked for full and zero live load conditions,
(2) Effect of lateral earth pressure shall be considered in design when it is critical.

Members subjected to Axial Force

7.4.4 In the plane of bending of columns, which would develop a plastic hinge at ultimate loading, the slenderness ratio kl_c/r shall not exceed C_c , where

$$C_c = (2 \sqrt{E/f_y})^{1/2}. \quad (7.3)$$

7.4.5 The calculated maximum load capacity P_{ac} of axial compression member shall be

$$P_{ac} = 1.7 A_s \sigma_{ac}, \quad (7.4)$$

where A_s is the effective cross sectional area of the column, and σ_{ac} is the allowable stress in axial compression given in section 4.4.

7.4.6 Depending on other conditions of restraint, the basis for determining proper values for P_{ac} , P_e and M_0 , for a column designed by using the plastic theory, and oriented to resist bending about its strong axis, is outlined in Table 7.2. In each case, l_c is the distance between points of lateral support corresponding to r_x or r_y , as applicable. When k is indicated, its value is governed by the provisions of cl. 2.4.4 of this standard; elsewhere $kl_c/r = l_c/r$.

TABLE 7.2 : EFFECTIVE LENGTH RATIOS FOR PLASTIC DESIGN

Loading	Braced Planar Frames or One- and Two-Storey Unbraced Planar Frames
P_{ac}	Use larger ratio, l_c/r_y or kl_c/r_x
P_e	Use kl_c/r_x
M_0	Use l_c/r_y

7.4.7 The calculated maximum load capacity P_{at} of a tension member shall be

$$P_{at} = 0.85 f_y A_s. \quad (7.5)$$

Member Subjected to Combined Bending and Axial Force

7.4.8 The calculated maximum moment capacity M_{pc} of a member subject to combined bending and axial forces, where P/P_y exceeds 0.15, shall be reduced below the value given in equation (7.2) and it shall satisfy the following requirements:

- (1) Beams

$$(P/P_y) + M_{pc}/(1.18 M_p) \leq 1.0, \quad (7.6)$$

- (2) Slender struts

A member, where P/P_y in addition to exceeding 0.15 also exceeds $(1 + \dots)/(1 + \dots)$, shall not be assumed to contain plastic hinges although it shall be permissible to design the member as an elastic part of a plastically designed structure. Such a member shall be designed according to maximum permissible stress requirements satisfying:

$$\frac{P}{P_{ac}} + \frac{M_{pc} C_m}{M_0(1 - P/P_e)} \leq 1.0, \quad (7.7)$$

- (3) Stocky Struts

A strut not covered in (2) above shall satisfy

$$(M_{pc}/M_p) \leq 1.0, \quad (7.8)$$

- (4) Where in the above equations, the following symbols are used:

P = an axial force, compressive or tensile in a member,

M_{pc} = maximum applied moment acting in the beam-column,

M_p = plastic moment capacity of the section,

M_0 = lateral buckling strength in the absence of axial load;
= M_p if the beam-column is laterally braced;

P_{ac} = buckling strength, (cl.7.4.5 and 7.4.6) about weaker axis;

P_e = Euler load for plane of bending;
= $(\pi^2 EA_s)/(kl_c/r)^2$, (7.9)

P_y = capacity of axially loaded section at yield strength $A_s f_y$;

A_s = effective cross-section area of the member;

C_m = a coefficient as defined in cl.4.5.4;

r = radius of gyration about the same axis as applied moment;

λ_0 = characteristic slenderness ratio;
= $(P_y/P_e)^{1/2} = [kl_c/(r)](f_y/E)^{1/2}$, (7.10)

= ratio of end moment, each measured in the same rotational direction and chosen with the numerically large amount in the denominator (range from +1 for reverse curvature, 0 for one end pinned, to -1 for single curvature); and

l_c = actual strut length.

7.4.9 A member assumed to contain plastic hinges and subject to combined bending and axial compression, with P/P_y not exceeding 0.15, shall have a value P/P_y not exceeding $(0.6 + 0.4 \lambda_0)$ where λ_0 and λ_0 are as defined above.

Shear

7.4.10 The calculated maximum shear capacity V_y of a beam or a beam-column shall be

$$V_y = 0.55 A_w f_y, \quad (7.11)$$

where A_w is the effective cross-sectional area resisting shear for calculating the average shear stress or the maximum shear capacity of the members.

Stability

7.4.11 The elastic buckling load of a frame or its components designed on the basis of plastic theory shall be at least three times the plastic collapse load. If an accurate estimate of elastic buckling load is not available, this provision shall be deemed to be satisfied for frames up to three storeys if the compressive force P in each member does not exceed $(0.33 \sqrt{EI})/(kl_c)^2$ for buckling in any direction, where the effective length kl_c is determined according to cl.2.4.5.

7.4.12 For frames over three storeys, the calculated plastic collapse load shall include an assessment of moment caused by a possible combination of high axial force and transverse deflection.

7.4.13 Web Stiffening

Web stiffeners are required on a member at a location of load application where a plastic hinge would form.

7.4.14 Excessive Shear Force

Web stiffeners or doubler plates shall be provided when the requirements of cl.7.4.10 are not met, in which case the stiffeners or doubler plates shall be capable of carrying that portion of the force which exceeds the shear capacity of the web.

7.4.15 Concentrated Loads

Web stiffeners shall be provided at locations on a member where the concentrated force delivered by flanges of another member framing into it will produce web crippling opposite the compression flange, or high tensile stress in the connection of the tension flange. This requirement shall be deemed to be satisfied if web stiffeners are placed:

- (1) Opposite the compression flange of the other member when

$$t < A_f / (T_b + 5K). \quad (7.12)$$

(2) Opposite the tension flange of the other member when

$$T_f < 0.4 (A_f)^{0.5}, \quad (7.13)$$

where t is the thickness of web to be stiffened, K is the distance from outer face of flange to web toe of fillet of member to be stiffened, T_b is the thickness of flange delivering concentrated load, T_f is the thickness of flange of member to be stiffened and A_f is the area of flange delivering concentrated load.

7.4.16 The area of stiffeners A_{st} shall be such that

$$A_{st} \geq A_f \quad t (T_b + 5K), \quad (7.14)$$

where A_f , t , T_b and K are defined in cl.7.4.15.

The ends of such stiffeners shall be fully butt welded to the inside face of the flange adjacent to concentrated tensile force. It shall be permissible to fit the stiffeners against the inside face of the flange adjacent to the concentrated compression force without welding. When the concentrated force is delivered by only one beam connected to an outside face of a strut, the length of web stiffener shall extend for at least half the member depth and the welding connecting it to the web shall be sufficient to develop a force of $A_{st} f_y$.

7.4.17 Plastic Hinges

Web stiffeners shall be provided at all plastic hinges where the applied load exceeds $0.06 A_w f_y$, where A_w is as explained in cl. 7.4.10.

Braced Frame

7.4.18 The bracing system for plastically designed braced multi-storey frames shall be adequate, as determined by rational analysis methods, to prevent buckling and to maintain the lateral stability of the structures under factored load combinations given in cl.7.3.3 and 7.3.4.

7.4.19 The bracing system may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, if these walls, slabs, and decks are secured to structural frames.

7.4.20 Axial deformations of all members in the bracing system shall be included in the lateral stability analysis. The axial force in these members caused by factored loads in cl.7.3.3 and 7.3.4 shall not exceed $0.85 A_s f_y$.

7.4.21 Girders and beams included in bracing systems of a braced multi-storey frame shall be proportioned in accordance with equation (7.6) for the axial force and moment caused by factored loadings in cl.7.3.3 and 7.3.4 with P_{ac} taken as the maximum axial strength of the beam based on the actual slenderness ratio between braced points in the plane of bending.

7.4.22 Lateral Bracing

Members shall be adequately braced to resist lateral and torsional displacements at the plastic hinge locations associated with failure mechanism. The lateral bracing must be dispensed within the region of the last hinge to form in the failure mechanism assumed as the basis for proportioning of the given member.

(a) If the length along the member in which the applied moment exceeds 85% of plastic moment M_p , does not exceed $[640 r_y / (f_y)^{1/2}]$, the critical flange shall have at least one support provided within or at the end of this length, and the spacing of the adjacent supports shall not exceed the following

$$(960 r_y) / (f_y)^{1/2},$$

(b) If the length along the member in which the applied moment exceeds 85% of the plastic moment M_p exceeds $[640 r_y / (f_y)^{1/2}]$, the critical flange shall be supported such that no portion of this length is unsupported for a distance greater than $[640 r_y / (f_y)^{1/2}]$,

(c) Lateral restraints for the remaining elastic portions of the member shall be designed using stress derived from the plastic bending moments. For this purpose, the allowable stress in section 4.4 can be increased by load factors 1.3 or 1.7 as used in cl. 7.3.4.

7.4.23 (a) In cl. 7.4.22, M_p stands for M_p , the plastic moment capacity of the section, or M_{pc} , the maximum applied moment on the beam column as appropriate. The plastic moment capacity is calculated from equation (7.2).

(b) may be taken as unity, or calculated from,

$$= 1.5/[1 + (\ /8)]^{0.5} \quad (7.15)$$

where is the ratio of the rotation at the hinge point to the relative elastic rotation of the far ends of beam segment containing plastic hinge whichever is lesser.

Unbraced Frame

7.4.24 The strength of an unbraced multi-storey frame shall be determined by rational analysis which includes the P effects and column axial deformation. Such a frame shall be designed to be stable under factored load combinations of cl. 7.3.3 and 7.3.4. The axial force in the columns at factored load levels shall not exceed $0.75 A_s f_y$.

7.5 Requirements on Minimum Thickness

Compression Outstands

- 7.5.1 (a) A flange or other compression element required to participate in a plastic hinge shall not project beyond its outermost point of attachment by more than $136T_1/(f_y)^{1/2}$, where T_1 is the thickness of flange of a section or plate in compression or the aggregate thickness of plates if connected in accordance with section 2.5,
- (b) For purpose of this clause, web stiffeners at plastic hinges shall be proportioned as compression elements.

Unsupported Widths

7.5.2 The distance between adjacent parallel lines of attachment of a compression flange, or other compression elements to other parts of a member, when such flanges or elements are required to participate in a plastic hinge action, shall not exceed $512T_1/(f_y)^{1/2}$, where T_1 is as defined in cl.7.5.1.

Webs in Shear

7.5.3 If the depth d_1 of a web subjected to shear required to participate in a plastic

hinge exceeds $688t/(f_y)^{1/2}$ then the compressive axial force P on the member shall not exceed the value

$$P = P_y [0.70 \{d_1(f_y)^{1/2}\}/(1600t)]. \quad (7.16)$$

The maximum permissible value of d_1 in any plastic hinge zone shall be $(1120t)/(f_y)^{1/2}$.

7.6 Connections

- 7.6.1 All connections essential for continuity, assumed as basis of the design analysis, shall be capable of resisting the moments, shears and axial loads to which these would be subjected by the full factored loading or any probable partial distribution thereof.
- 7.6.2 Corner or end connections (haunches), that may be tapered or curved for architectural reasons, shall be so proportioned that the full plastic bending strength of the section, adjacent to the connections can be developed if required.
- 7.6.3 Stiffeners shall be used, as required, to preserve the flange continuity of interrupted members at their junction with other members in a continuous frame. Such stiffeners shall be placed in pairs on opposite sides of the web of the member which extends continuously through the joint.
- 7.6.4 Bolts including high strength bolts and welds shall be proportioned to resist the forces produced by factored loads.
- 7.6.5 For normal design condition the allowable stresses shall be 1.7 times that given in section 4.4. For abnormal design condition the allowable stress shall be 1.4 and 1.6 times that given in section 4.4 for bolts and welds respectively. In general, groove welds are preferable to fillet welds.
- 7.6.6 In welding, the weld metal shall be 10% stronger than base metal. Butt welds may be assumed capable of developing on their minimum throat section the tensile yield stress of the base metal. Fillet welds may be assumed to be capable of developing the shearing yield stress of the weld metal on their minimum throat section, but not exceeding $0.55 f_y$ of the base metal.

7.7 Fabrication

- 7.7.1 Provisions of cl.1.6.1, 1.6.3 and 1.6.4 with respect to workmanship, shall govern the fabrication of structures, designed on the basis of maximum strength, i.e., plastic theory, subject to limitations given in the following clauses.
- 7.7.2 Use of sheared edges shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, the edges shall be finished smooth by grinding, chipping or planing.
- 7.7.3 In locations subject to plastic hinge rotation at factored loading, holes for bolts in the tension area shall be sub-punched and reamed or drilled full size.

8. STEEL LINERS

8.1 General

- 8.1.1 Steel liner is a permanent membrane attached to concrete surface to form a leak-tight boundary. Examples are calandria vault liner, reactor vessel vault liner (for fast breeder reactor), spent fuel storage and inspection bay liner, distribution header and vent shaft liner, fuelling machine vault liner, fuelling machine service room liner, hot cell liner, etc. Liners of categories stated above shall be designed complying with provisions given hereinafter along with other relevant stipulations of this standard as appropriate.
- 8.1.2 The requirements mentioned in this standard are not applicable to containment and concrete reactor vessel liners. The design requirements for containment liner are given in AERB Standard on Design of Nuclear Power Plant Containment Structures (No. AERB/SS/CSE-3).

8.2 Materials

- 8.2.1 Steel for liner plates, liner anchorage system, liner attachments such as plate and stiffeners, and leak test channels shall conform to relevant standards and stipulations specified in section 2.2 unless otherwise specified.
- 8.2.2 In addition to basic requirements specified in the material specification, supplementary requirements as mentioned in cl. 2.2.2 should be met whenever specified. All tests and examination should be carried out as per specifications.
- 8.2.3 Materials for lugs, brackets, stiffeners and other structural attachments permanently welded to liners shall meet the requirements of liner material. Non-structural attachments such as lifting lugs, name plates, locating lugs and temporary attachments shall be of materials specified.

8.3 Design Considerations

- 8.3.1 The liner and its anchorage systems shall not be used as a strength element. Contribution of liner and its anchorage system shall not be considered in the strength design of the walls, floors or any other structural element to which these are attached. Interaction of liner and its anchorage system with concrete shall be considered in determining maximum stresses and strains.

8.3.2 Liners and anchorage systems shall be designed by using allowable stress design method.

Liner

8.3.3 Liner shall be designed for all relevant loads (under normal and abnormal design conditions). Other effects such as temperature, creep and shrinkage of concrete shall also be considered.

8.3.4 Liner shall be anchored to the concrete structure so that the overall deformation of the liner shall be the same as that of the concrete. This does not prevent local flexural deformation between anchoring embedments or shear deformation at the embedments.

8.3.5 Liner design shall consider the following effects:

- (a) gap between liner and concrete surface,
- (b) adverse combination of fabrication and erection tolerances,
- (c) variation in anchoring embedment spacing,
- (d) structural discontinuities,
- (e) defective or missing anchor,
- (f) local crushing or cracking of concrete in the region of anchoring embedment, and
- (g) temperature variation.

8.3.6 The attachment should be anchored directly to the concrete and should be designed so that the liner is not loaded in the through-thickness direction.

8.3.7 Where conventional maintenance, like painting, etc. of liner is not possible during service, either corrosion-resistant material should be selected, or adequate margin should be provided in the thickness of liner plates to take care of any possible thinning of section due to corrosion over its service life.

8.3.8 Where necessary, irradiation effects on the properties of liner and welding materials shall be considered.

Liner Anchorage System

- 8.3.9 Liner anchorage system (i.e. the steel embedments and their anchors) shall be designed so that it can accommodate all tangential (shear) and normal loads and deformations exerted by liner without loss of structural or leak-tight integrity. Failure of steel embedments shall not cause tearing of liner.
- 8.3.10 Anchorage system shall be so designed that a progressive failure of anchorage system is prevented in the event of defective or missing anchor.
- 8.3.11 The design of liner anchorage system shall consider the following effects:
- (a) differential thermal expansion (if any) of the liner and concrete structure;
 - (b) unbalanced forces arising from variation of liner curvature (variation will cause shear force and displacement at anchor embedments);
 - (c) gap between concrete and liner, structural discontinuities, and weld offset;
 - (d) local crushing of concrete in the anchor embedment zone;
 - (e) variation in anchor embedment spacing from the specified one(s); and
 - (f) variation in anchor embedment stiffness due to variation of concrete modulus.

8.4 Loads and Load Combinations

- 8.4.1 Individual loads and load combinations considered in the design of liner and its anchorage system should comply with those specified in section 4.2 unless stated otherwise. Load combinations shall be applied such that the design features of liner and anchorage system are based on the most conservative application of these combinations.

8.5 Design Analysis

- 8.5.1 Methods of analysis based on linear behaviour of material and small deflection theory should be used. The methods should be appropriate to the geometry of the liner and members of its anchorage system.

- 8.5.2 Liner supported on anchor embedements spaced at close intervals may be analysed using plate or beam theory, provided all assumptions made in analysis are valid with respect to actual structural arrangement and they are conservative.
- 8.5.3 Anchor embedements may be analysed assuming the liner remains elastic under all conditions; for example, liner strain shall be converted to stress using Hooke's law with modulus of elasticity and Poisson's ratio below yield. Liner stress and resulting membrane forces shall not be higher than the uniaxial yield strength.
- 8.5.4 Analysis shall be performed in sufficient detail to show that each of the design allowables specified in section 8.6 is satisfied when liner and anchorage system are subjected to loading specified in section 8.4 and the design considerations given in section 8.3.
- 8.5.5 If analytical methods are not able to predict satisfactorily certain functional requirements or stress, strain and stability of liner and behaviour of selected liner anchorage system under the loadings specified, the ability of liner and its anchorage system to perform as required should be established through prototype testing.

8.6 Design Allowables

- 8.6.1 Design limits on stress and strain for liner plates attached to the concrete walls, floors, etc. enclosed within a building shall satisfy the limiting values given in the following clauses. The strain shall be determined by considering strain compatibility at the interface of liner system and concrete surface.
- 8.6.2 Design limits on stress and strain for liners not exposed to weather shall satisfy the following:
- (a) the maximum value of calculated stress S_m (Table 8.1) shall not exceed the permissible stress values given in Table 4.2, and
 - (b) the maximum value of calculated strain shall not exceed the values given in Table 8.1.

Note : Liners which are in the controlled atmosphere will not be considered exposed to weather.

8.6.3 Design limits on stress and strain for liner plates whose surface is exposed to the weather, shall satisfy the following:

- (a) the maximum value of calculated stress S_m (Table 8.2) shall not exceed the permissible stress values given in Table 4.2; and
- (b) the maximum value of calculated strain shall not exceed the values given in Table 8.2.

TABLE 8.1 : LIMITING VALUES OF STRAINS AND STRESSES FOR LINER PLATES WHICH ARE NOT EXPOSED TO WEATHER

Design condition	Compression		Tension	
	Membrane	Membrane + bending	Membrane	Membrane + bending
Normal design condition	$u/30$	$u/20$	S_m	S_m
Abnormal design condition	$u/20$	$u/10$	$u/10$	$u/10$

TABLE 8.2 : LIMITING VALUES OF STRAIN OF LINER PLATES WHOSE SURFACE IS EXPOSED TO WEATHER

Design condition	Membrane	Membrane bending
Normal design condition		
Construction loading	S_m	S_m
Other loading	$st = sc = 0.002$	$st = sc = 0.004$
Abnormal design condition	$sc = 0.005$ $st = 0.003$	$sc = 0.014$ $st = 0.010$

Allowable stress values for Anchorage System

8.6.4 Anchorage system shall be designed satisfying the requirements of section 4.3 and AERB Standard for Design, Fabrication and Erection of Embedded Parts and Penetrations Important to Safety of Nuclear Facilities, (AERB/SS/CSE-4) as appropriate.

8.7 Fabrication of Liners

8.7.1 General requirements for fabrication of liners shall conform to provisions of section 1.6 of this standard unless specified otherwise.

8.7.2 Liner shall be entirely welded construction. Weld details shall be such that they do not jeopardise its leak-tight integrity. Unless specified otherwise, the welded construction shall satisfy the requirements of Appendix-C.

APPENDIX-A

STEEL STANDARDS

A.1 General

A.1.1 Unless otherwise specified, steel, electrodes and other materials conforming to the following standards or specifications are permissible for use in structures, or part of it covered under this standard.

A.1.2 The following abbreviations are used:

IS : Indian Standards

ASTM : Specification of the American Society for Testing and Materials

AWS : Specifications of the American Welding Society

A.2 Indian Standards

IS:808 Dimensions for hot rolled steel beam, column, channel and angle sections

IS:813 Scheme of symbols for welding

IS:814 Covered electrodes for metal arc welding of structural steels.

IS:816 Code of practice for use of metal arc welding for general construction in mild steel

IS:961 Structural steel (high tensile)

IS:1161 Steel tubes for structural purposes

IS:1239 Mild steel tubes, tubulars and other wrought steel fittings
Part 1 - Mild steel tubes
Part 2 - Mild steel tubulars and other wrought steel pipe fittings

IS:1278 Filler rods and wires for gas welding

- IS:1363 Hexagon head bolts, screws and nuts of product grade C
 Part 1 - Hexagon head bolts (size range M5 to M36)
 Part 2 - Hexagon head screws (size range M5 to M36)
 Part 3 - Hexagon nuts (size range M5 to M36)
- IS:1364 Hexagon head bolts, screws and nuts of product grades A and B.
 Part 1 - Hexagon head bolts (size range M3 to M36)
 Part 2 - Hexagon screws (size range M3 to M36)
 Part 3 - Hexagon nuts (size range M1.6 to M36)
 Part 4 - Hexagon thin nuts (size range M1.6 to M36)
 Part 5 - Hexagon thin nuts (unchamfered) (size range M1.6 to M10)
- IS:1367 Technical supply conditions for threaded steel fasteners (Parts 1 to 3, 5 to 7, 9, 10, 12 to 14, 16 & 18)
- IS:1395 Low and medium alloy steel covered electrodes for manual metal arc welding
- IS:1852 Rolling and cutting tolerances for hot rolled steel products
- IS:1978 Line pipe
- IS:1979 High test line pipe
- IS:2002 Steel plates for pressure vessels for intermediate and high temperature service including boilers
- IS:2016 Plain washers
- IS:2062 Steel for general structural purposes
- IS:2041 Steel plates for pressure vessels used at moderate and low temperature
- IS:3502 Steel chequered plates
- IS:3601 Steel tubes for mechanical and general engineering purposes

- IS:3613 Acceptance tests for wire flux combination for submerged arc welding
- IS:3640 Hexagon fit bolts
- IS:3757 High strength structural bolts
- IS:4000 Code of practice for assembly of structural joints using high tensile friction grip fasteners.
- IS:4923 Hollow steel sections for structural use
- IS:5372 Taper washers for channels
- IS:5374 Taper washers for I-beams
- IS:6419 Welding rods and bare electrodes for gas shielded arc welding of structural steel
- IS:6560 Molybdenum and chromium - molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding
- IS:6610 Heavy washers for steel structures
- IS:6623 High strength structural nuts
- IS:6639 Hexagonal bolts for steel structures
- IS:6649 Hardened and tempered washers for high strength structural bolts and nuts
- IS:7280 Bare wire electrodes for submerged arc welding of structural steels
- IS:8500 Weldable structural steel (medium and high strength qualities)
- IS:8535 Bolts, screws, nuts and accessories - Terminology and nomenclature
- IS:8536 Fasteners - Bolts, screws, studs and nuts - Symbols and designation of dimensions

IS:9595 Recommendations for metal arc welding of carbon and carbon manganese steels

A.3 ASTM Standards

ASTM A6	Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use
ASTM A20	Specification for General Requirements for Steel Plates for Pressure Vessels
ASTM A36	Specification for Structural Steel
ASTM A53	Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
ASTM A106	Specification for Seamless Carbon Steel Pipe for High-Temperature Service
ASTM A108	Specification for Steel Bars, Carbon, Cold-Finished Standard Quality
ASTM A167	Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip
ASTM A192	Specification for Seamless Carbon Steel Boiler Tubes for High-Pressure Service
ASTM A193	Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service
ASTM A194	Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
ASTM A240	Specification for Heat-Resisting Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels
ASTM A242	Specification for High-Strength Low-Alloy Structural Steel

ASTM A276	Specification for Stainless and Heat-Resisting Steel Bars and Shapes
ASTM A285	Specification for Pressure Vessel Plates, Carbon Steel, Low-and Intermediate-Tensile Strength
ASTM A299	Specification for Pressure Vessel Plates, Carbon Steel, Manganese-Silicon
ASTM A307	Specification for Carbon Steel Bolts and Studs, 60, 000 psi Tensile
ASTM A312	Specification for Seamless and Welded Austenitic Stainless Steel Pipe
ASTM A320	Specification for Alloys-Steel Bolting Materials for Low-Temperature Service
ASTM A325	Specification for High-Strength Bolts for Structural Steel Joints
ASTM A333	Specification for Seamless and Welded Steel Pipe for Low-Temperature Service
ASTM A334	Specification for Seamless and Welded Carbon and Alloy-Steel Tubes for Low Temperature Service
ASTM A354	Specification for Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners
ASTM A358	Specification for Electric-Fusion-Welded Austenitic Chromium-Nickel Alloy Steel Pipe for High-Temperature Service
ASTM A441	Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel
ASTM A442	Specification for Pressure Vessel Plates, Carbon Steel, Improved Transition Properties

ASTM A449	Specification for Quenched and Tempered Steel Bolts and Studs
ASTM A479	Specification for Stainless and Heat-Resisting Steel Wire, Bars, and Shapes for Use in Boilers and Other Pressure Vessels (ASTM A490 - Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints)
ASTM A514	Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding
ASTM A515	Specification for Pressure Vessel Plates, Carbon Steel, for Intermediate-and Higher-Temperature Service
ASTM A516	Specification for Pressure Vessel Plates, Carbon Steel, for Moderate-and Lower-Temperature Service
ASTM A540	Specification for Alloy-Steel Bolting Materials for Special Applications
ASTM A563	Specification for Carbon and Alloy Steel Nuts
ASTM A564	Specification for Hot-Rolled and Cold-Finished Age-Hardening Stainless and Heat-Resisting Steel Bars, Wire, and Shapes
ASTM A568	Specification for General Requirements for Steel, Carbon and High-Strength, Low-Alloy, Hot-Rolled Sheet and Cold-Rolled Sheet
ASTM A570	Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality
ASTM A572	Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A588	Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 inches (100 mm) Thick

- ASTM A606 Specification for Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low-Alloy, with Improved Atmospheric Corrosion Resistance
- ASTM A607 Specification for Steel Sheet and Strip, High-Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled
- ASTM A687 Specification for High-Strength Nonheaded Steel Bolts and Studs
- ASTM A770 Specification for Through-Thickness Tension Testing of Steel Plates for Special Applications

APPENDIX-B

STRENGTH EVALUATION OF EXISTING STRUCTURES

B.1 General

- B.1.1 If doubt arises concerning the safety of a structure or member, the engineer shall order a structural strength investigation by analysis or by means of load tests, or by a combination of analysis and load tests.

B.2 Analytical Investigations

- B.2.1 If strength evaluation is by analysis, a thorough field investigation shall be made of dimensions and details of members, properties of materials, and other pertinent conditions of the structure as actually built. The results of these investigations would form the input for analysis.

B.3 Load Tests

- B.3.1 If strength evaluation is by load tests, the engineer shall control and supervise such tests.
- B.3.2 When only a portion of the structure is to be load tested, the questionable portion shall be load-tested such as to adequately test the suspected source of weakness.
- B.3.3 Forty-eight hours prior to application of test load, a load to simulate effect of that portion of the dead loads not already acting shall be applied and shall remain in place until all testing has been completed.

Load Tests of Flexural Members

- B.3.4 When flexural members, including beams and composite slabs, are load tested, the additional provisions herein shall apply as follows:
- (a) Base readings (datum for deflection measurements) shall be made immediately prior to application of test load,

- (b) That portion of the structure selected for loading shall be subject to a total load, including dead loads already acting, equivalent to $0.85 (1.4 DL + 1.6 LL + 1.6 E_0)$,
- (c) Test load shall be applied in not less than four approximately equal increments. It shall be applied without shock to the structure such as to avoid arching of loading materials,
- (d) After test load has been in position for 24 hours initial deflection readings shall be taken, and
- (e) Test load shall be removed immediately after initial deflection readings, and final deflection readings shall be taken 24 hours after removal of the test load.

B.3.5 If the portion of tested structure shows visible evidence of failure, the portion tested shall be considered to have failed the test and no retesting of the previously tested portion shall be permitted. The visible evidence shall be thoroughly investigated.

B.3.6 (1) If the portion of tested structure shows no visible evidence of failure, the following criteria shall be taken as indication of satisfactory performance,

- (a) If measured maximum deflection (mm) of a beam, floor or roof is less than $L / (800h)$ where,

$L =$ effective span of beam or one-way slabs, (mm)

$h =$ overall depth or thickness of member, (mm)

- (b) If measured maximum deflection of a beam, composite floor or roof exceeds $L / 800h$, deflection recovery within 24 hours after removal of the test load shall be 100% for steel beam, and at least 75% of the maximum deflection for non prestressed concrete, or 80% for prestressed concrete.

- (2) In cl. B.3.6, subpara (1) (a) L for cantilevers shall be taken as twice the distance from support to cantilever end, and deflection shall be adjusted for any support movement.

B.3.7 Composite construction of non prestressed concrete failing to show 75% recovery of deflection as required by cl.B.3.6 may be retested not earlier than 72 hours after removal of the first test load. The portion of the structure tested shall be considered satisfactory if:

- (a) it shows no visible evidence of failure in the retest; and
- (b) deflection recovery caused by second test load is at least 80% of the maximum deflection in the second test.

Non-Flexural Members

B.3.8 Members other than flexural members should be investigated by analysis.

Provisions for Lower Load Rating

B.3.9 If structure under investigation does not satisfy conditions or criteria of B.3.6(1) or B.3.6(2) a lower load rating for that structure based on results of the load test or analysis may be acceptable if justified.

Safety

- B.3.10 (a) Load tests shall be conducted such as to provide for safety of life and structure during the test, and
- (b) No safety measures shall interfere with load test procedures or affect results.

APPENDIX-C

REQUIREMENTS OF WELDED CONSTRUCTION FOR LINER PLATES, PENETRATIONS AND ANCHORAGE SYSTEM

C.1 Joint Category

C.1.1 The term 'category' used herein specifies the location of a joint in a liner, but not the type of joint and illustrates some acceptable details. The joints included in each category are designated A, B, C, D, E, and F. Figure C.1 illustrates typical joint locations included in each category.

- (a) Category A - longitudinal welded joints within a nonspherical, non-flat liner or penetration nozzles, or any welded joint within a spherical or flat liner.
- (b) Category B - circumferential welded joints within a nonspherical liner or penetration nozzles.
- (c) Category C - welded joints connecting penetration nozzles in the liner or welded circumferential joints where the offset angle exceeds 30°. (If offset angle does not exceed 30°, circumferential joint is Category-B).
- (d) Category D - welded joints connecting flat liner bottom sections to elements of spherical, cylindrical, or conical liner sections or for intersection of liner sides to sides, or sides to bottom.
- (e) Category E - welded joints connecting liner floor plates together and to liner transition sections and joints connecting liner sides and bottom plates together.
- (f) Category F - welded joints connecting the embedment anchors to attachments to liners or other embedment anchors.

C.1.2 The minimum dimensions of Figures C.2 to C.6 should be met, where:

t = nominal thickness of liner,

t_a = nominal thickness of attachments to liner,

$t_c = 0.7t_n, 0.7t_1, 0.7t_a$ or 6 mm, whichever is less,

t_{\min} = the smaller of 20 mm or the thickness of the thinner of the parts joined,

t_n = nominal thickness of penetrating part,

t_1, t_2 = not less than the smaller of 6 mm or $0.7t_{\min}$,

$t_1, t_2 \geq 1.25t_{\min}$,

$r_1 = 0.25 t$ or 20 mm, whichever is less, and

$r_2 = 6$ mm minimum.

C.1.3 A tapered transition section having a length not less than three times the offset between the adjacent plates shall be provided at joints of Categories A and B between sections that differ in the thickness by more than one-fourth of the thickness of the thinner section or 3 mm, whichever is lesser.

C.2 Permissible Types of Welded Joints

C.2.1 The various categories of joints in cl. C.1.1 shall conform to the following additional requirements.

- C.2.2 (1) Category A - All welded joints of Category A shall be full penetration butt joints. Joints may be single or double welded. Backup bars, when allowed, shall be continuous and any joints shall be made with full penetration welds. When structural shapes are used as backups, these requirements shall apply. For repairs to the liner after concrete placement, backups may be used,
- (2) Category A welds shall be capable of being radiographed except where backup bars are used. Category A welds with backup bars shall be capable of being examined by the magnetic particle or ultrasonic examination method for the full length of the weld.

- C.2.3 Category B - All welded joints of Category B shall meet the fabrication requirements as in Category A, and capable of being examined as in Category A.
- C.2.4 Category C - All welded joints of Category C shall be single- or double-welded. Backup bars may be used except when the weld is to be radiographed. Backup bars, when allowed, shall be continuous and all joints shall be made with full penetration welds. When structural shapes are used as backups, these requirements shall apply.
- C.2.5 (1) Joints of Category C shall meet the requirements of Figure C.2 for nozzles over 75 mm diameter,
- (2) Joints of Category C shall meet the requirements of Figure C.3 for nozzles 75 mm in diameter and under. The minimum dimensions of Figures C.2 and C.3 should be met.
- C.2.6 Category D - Joints shall be of the type shown in Figure C.4, and shall meet the same requirements as in the case of joints of Category C. The minimum dimension of Figure C.4 shall be as indicated earlier.
- C.2.7 Category E - Joints shall be of the type shown in Figure C.5, and shall meet the same requirements as in the case of joints of Category C.
- C.2.8 Category F - Joints shall be of the type shown in Figure C.6 and C.7, and shall meet the same requirements as in the case of joints of Category C.

C.3 Welding of Structural Attachment

- C.3.1 Typical details for attaching structural anchors to the liner are shown in Figure C.7. The attachment materials shall conform to the requirements stated in section 2.2.

Fillet Welds

- C.3.2 Fillet welds may vary from convex to concave. The size of weld shall be determined in accordance with Figure C.8.

Repair of Weld Defects

- C.3.3 Requirements specified by the engineer regarding removal and repair of unacceptable weld defects, and examination and heat treatment of repaired areas shall be met.

Heat Treatment

- C.3.4 Requirements indicated in the specification as regard to welding preheat and postweld heat treatment shall be met.

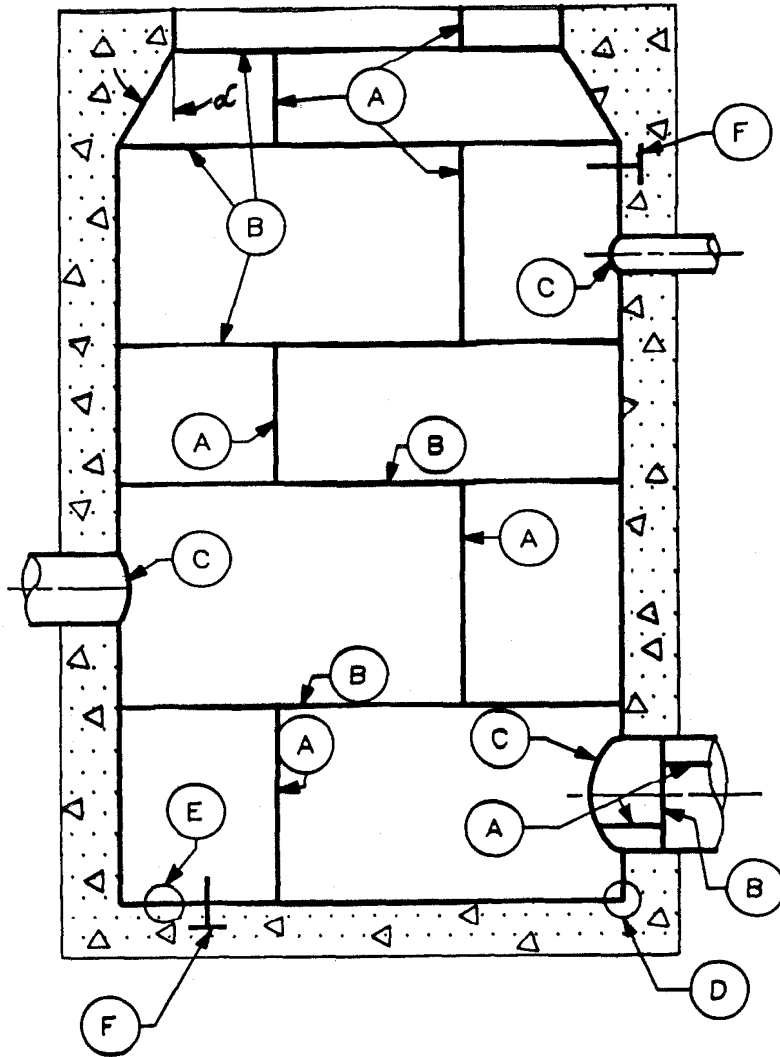


Fig. C. 1 : WELDED JOINT LOCATIONS TYPICAL OF ALL CATEGORIES

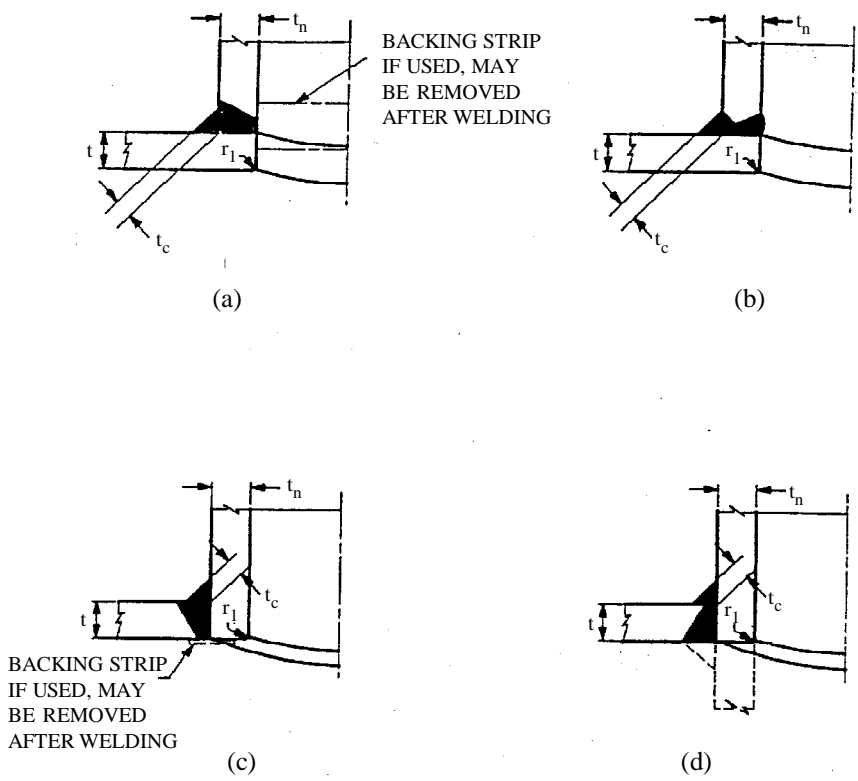


Fig. C. 2 : TYPICAL CATEGORY C JOINTS FOR NOZZLE DIAMETERS OVER 75mm NOMINAL PIPE SIZE

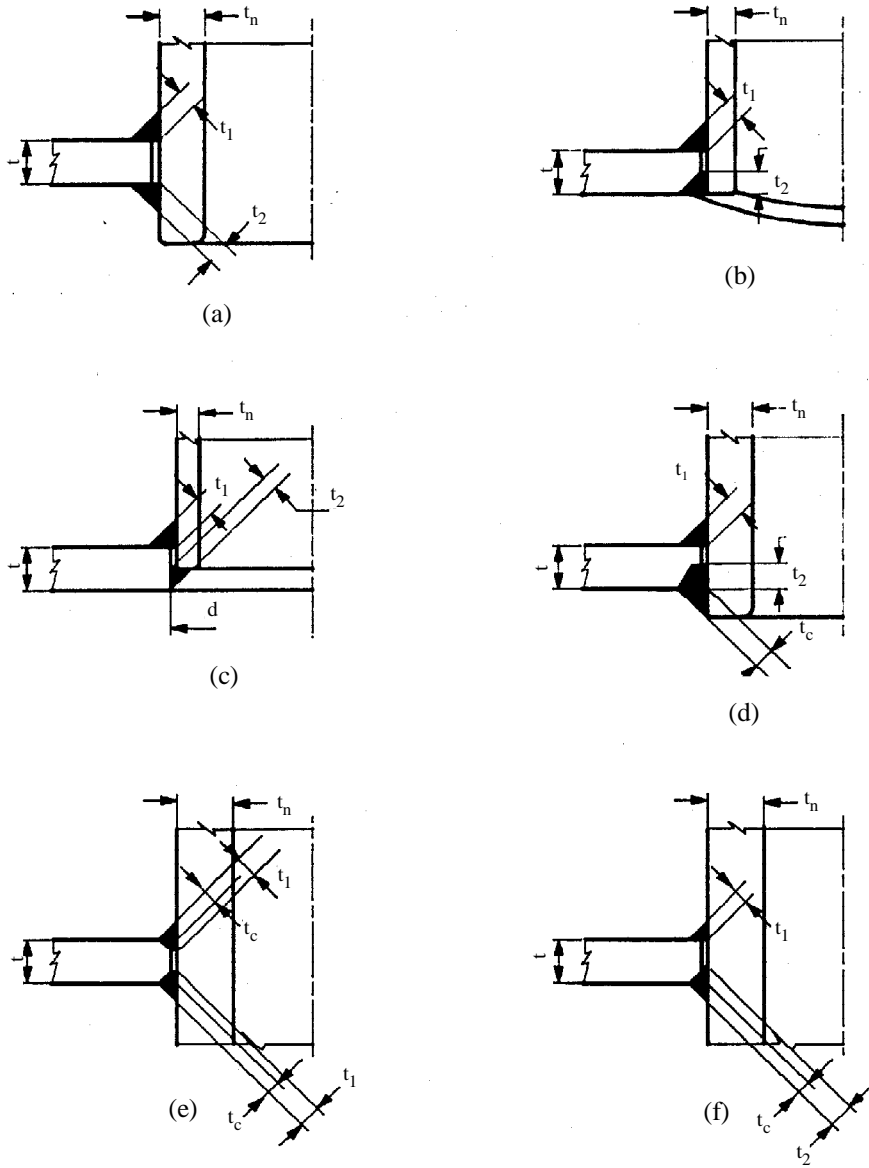


Fig. C. 3 : TYPICAL CATEGORY C JOINTS FOR NOZZLE DIAMETERS OVER 75mm NPS AND LESS

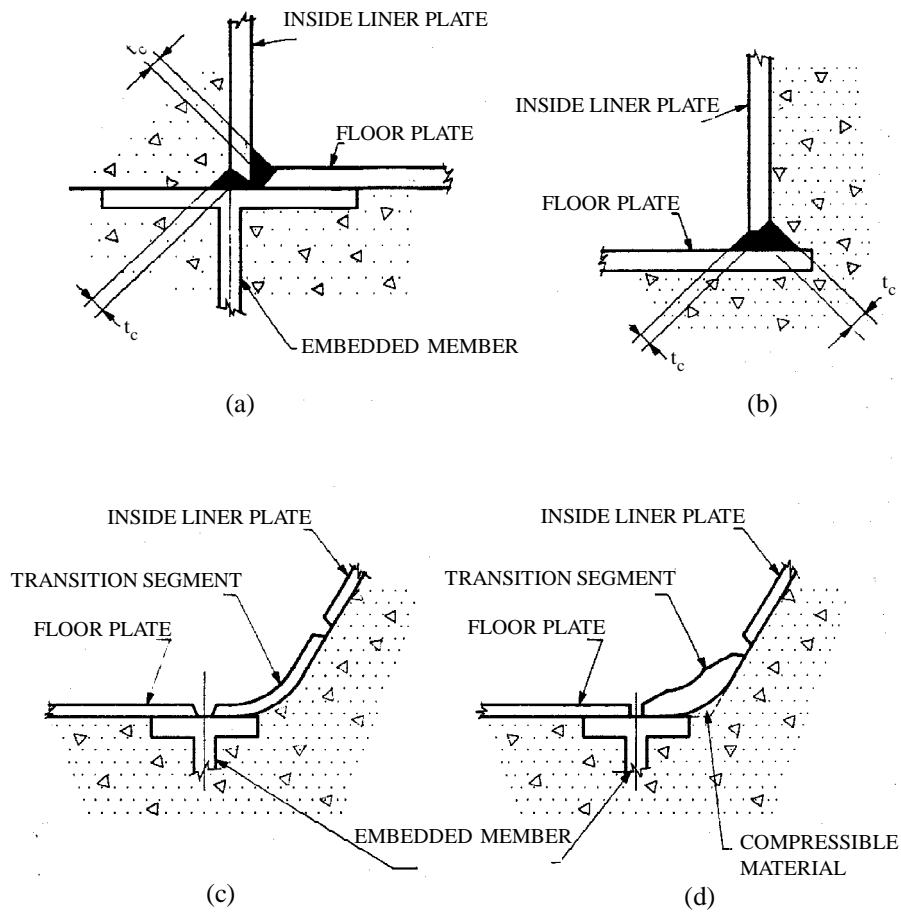


Fig. C.4 : TYPICAL CATEGORY D JOINTS

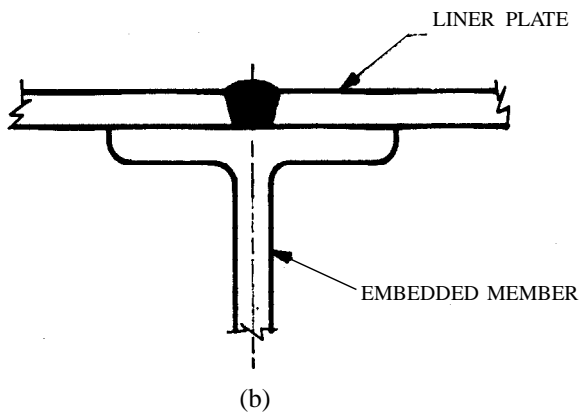
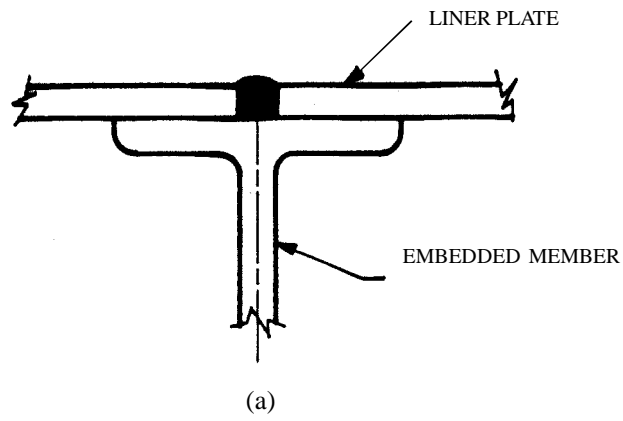


Fig. C. 5 : TYPICAL CATEGORY E JOINTS

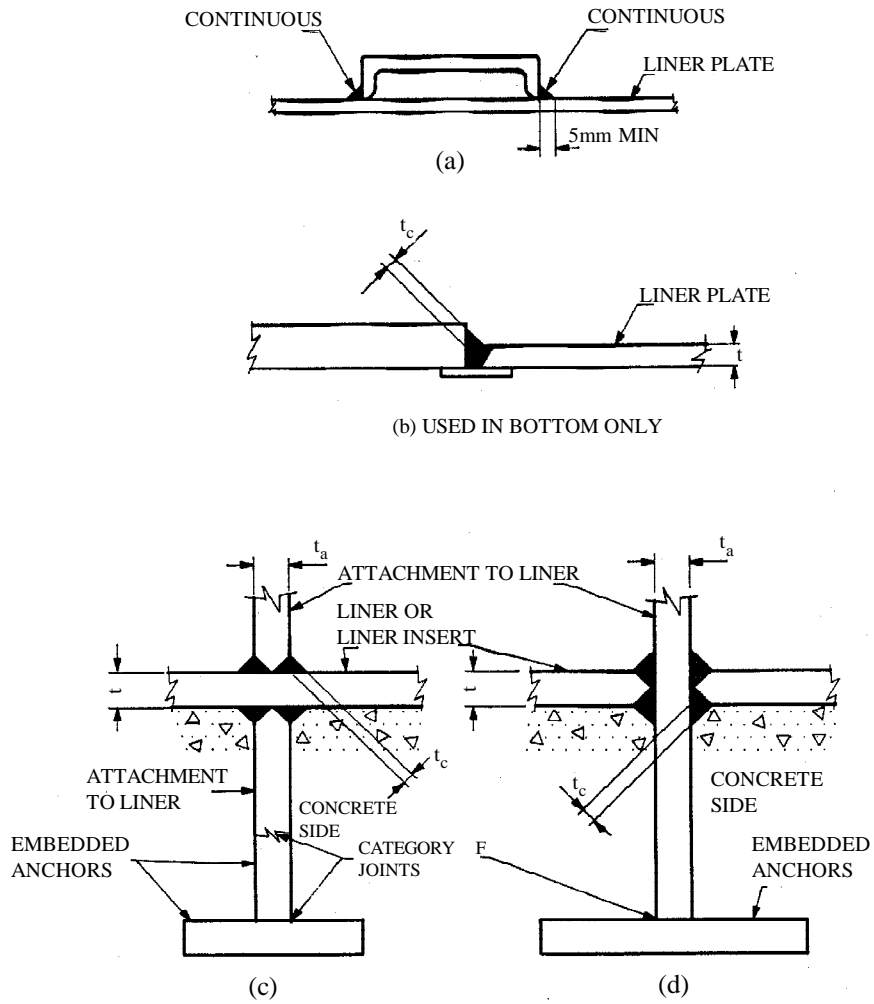


Fig. C.6: TYPICAL JOINT DETAILS FOR ATTACHMENTS

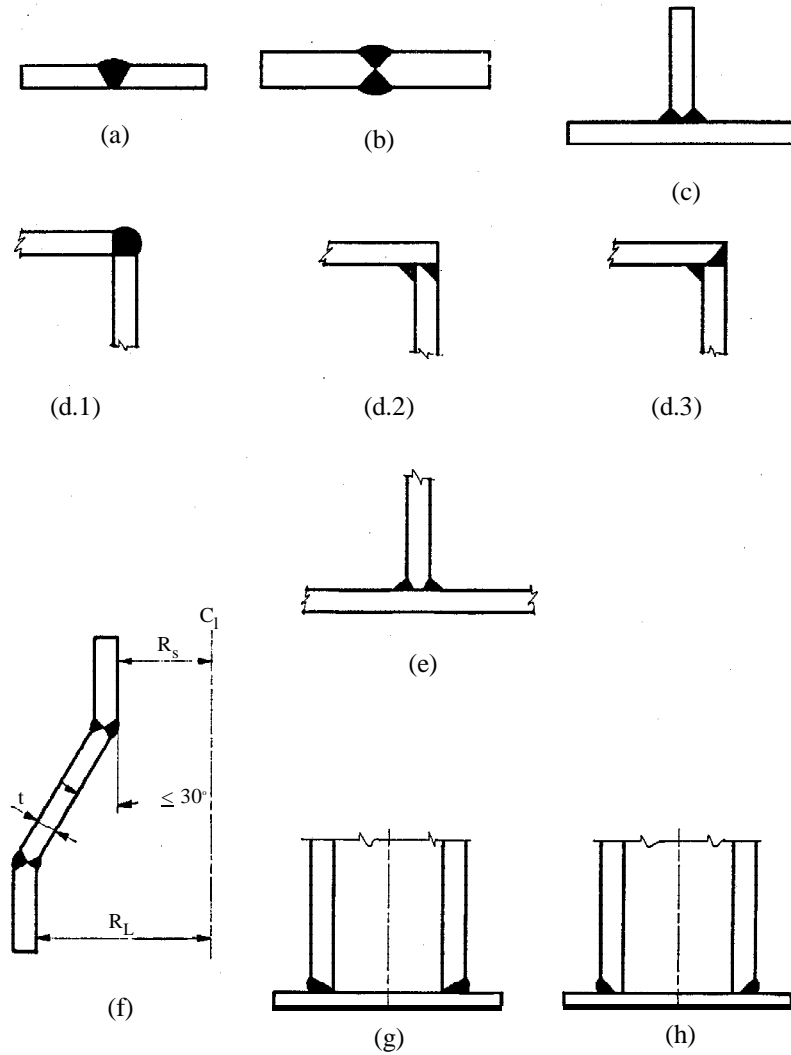
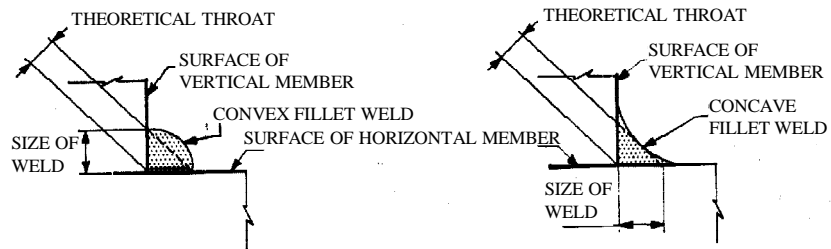
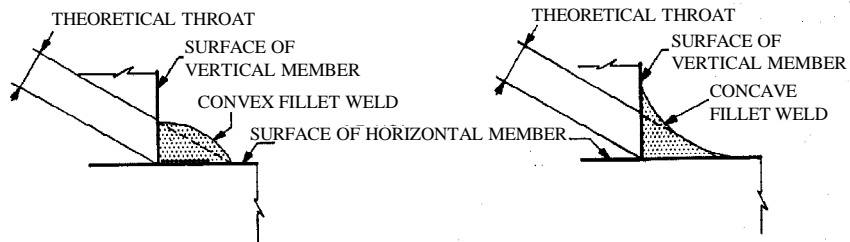


Fig. C. 7 : TYPICAL CATEGORY F WELDED JOINTS



EQUAL LEG FILLET WELD



UNEQUAL LEG FILLET WELD

Fig. C. 8 : FILLET WELDS

- Notes:
1. The "Size" of an equal leg fillet weld is the leg length of the largest inscribed right isosceles triangle. Theoretical Throat = $0.7 \times$ size of weld.
 2. The "Size" of an unequal leg fillet weld is the shorter leg length of the largest right triangle that can be inscribed within the fillet weld cross-section.

APPENDIX-D

MATERIAL PROPERTIES OF STRUCTURAL STEELS

Modulus of Elasticity	E	2×10^5 MPa
Poisson's Ratio		0.3
Coefficient of linear thermal expansion		0.000012 per °C
Density		7.85×10^3 Kg/M ³

APPENDIX-E

LIMITING VERTICAL DEFLECTION OF CRANE RUNWAY GIRDERS

The maximum vertical deflection of crane runway girders under dead and imposed loads shall not exceed the following values unless specified otherwise:

Where manually operated cranes are operated and for small loads.	$L/500$
Where electric overhead travelling cranes operate, up to 50 tonnes capacity.	$L/750$
Where electric overhead travelling cranes operate, over 50 tonnes capacity.	$L/1000$
Other moving loads, such as monorails, etc.	$L/600$

Where L = span of crane runway girder

APPENDIX-F

LOADING CONDITIONS, MATERIAL TYPE AND LOCATION, AND MAXIMUM STRESS RANGE FOR FATIGUE DESIGN

**TABLE F.1 : FATIGUE LOADING CYCLES AND
THEIR CLASSIFICATION**

Loading Condition	Number of Loading Cycles	
	From	To
1	20,000 ⁽¹⁾	1,00,000 ⁽²⁾
2	1,00,000	5,00,000 ⁽³⁾
3	5,00,000	20,00,000 ⁽⁴⁾
4	over 20,00,000	

- Note: (1) Approximately equivalent to two applications every day for 25 years,
(2) Approximately equivalent to ten applications every day for 25 years,
(3) Approximately equivalent to fifty applications every day for 25 years,
(4) Approximately equivalent to two hundred applications every day for 25 years.

TABLE F.2 : TYPE AND LOCATION OF MATERIAL

General Condition	Situation	Kind of Stress⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures⁽²⁾ (Ref. Fig. F.1)
Plain Material	Base metal with rolled or cleaned surface	T or Rev.	A	1,2
Built-up members	Base metal and weld metal in member without attachments, built-up plates or shapes connected by continuous full-penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,6
	Base metal and weld metal in members without attachments, built-up plates, or shapes connected by full-penetration groove welds with backing bars not removed, or by partial-penetration groove welds parallel to the direction of applied stress	T or Rev.	B'	3,4,5,6
	Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners	T or Rev.	C	7
	Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than flange with welds across the ends			
	Flange thickness ≤ 20 mm	T or Rev.	E	5
	Flange thickness > 20 mm	T or Rev.	E'	5
	Base metal at end of partial length welded cover plates wider than the flange without welds across the ends			E'

TABLE F.2 : TYPE AND LOCATION OF MATERIAL (contd.)

General Condition	Situation	Kind of Stress⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures⁽²⁾ (Ref. Fig. F.1)
Groove Welds	<p>Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of cl. 1.4.3.</p> <p>Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2.5 with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of cl. 1.4.3.</p>	T or Rev.	B	10,11
	IS:961 ST.55-HTW	T or Rev.	B	12,13
	Other base metals	T or Rev.	B'	12,13
	<p>Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2.5 when reinforcement is not removed but weld soundness is established by radiographic or ultrasonic inspection in accordance with requirements of cl. 1.4.3.</p>	T or Rev.	C	10,11,12,13
Partial-Penetration groove welds	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds	T or Rev.	F ⁽³⁾	16

TABLE F.2 : TYPE AND LOCATION OF MATERIAL (contd.)

General Condition	Situation	Kind of Stress ⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures ⁽²⁾ (Ref. Fig. F.1)
Fillet-welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	—
	Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses			
	$b \leq 25$ mm	T or Rev.	E	17,18
	$b > 25$ mm	T or Rev.	E'	17,18
Fillet Welds	Base metal at members connected with transverse fillet welds			
	$b \leq 12$ mm $b > 12$ mm	T or Rev.	C ⁽³⁾	20,21
Fillet Welds	Weld metal of continuous or intermittent longitudinal or transverse fillet welds	S	F ⁽³⁾	15,17,18, 20,21
Plug or Slot Welds	Base metal at plug or slot welds	T or Rev.	E	27
	Shear on plug or slot welds	S	F	27
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip-critical connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	8
	Base metal at net section of other mechanically fastened joints	T or Rev.	D	8,9
	Base metal at net section of fully tensioned high-strength, bolted - bearing connections	T or Rev.	B	8,9

TABLE F.2 : TYPE AND LOCATION OF MATERIAL (contd.)

General Condition	Situation	Kind of Stress ⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures ⁽²⁾ (Ref. Fig. F.1)
Attachments	Base metal at details attached by full-penetration groove welds subject to longitudinal and/or transverse loading when the detail embodies a transition radius R with the weld termination ground smooth and for transverse loading, the weld soundness established by radiographic or ultrasonic inspection in accordance with cl. 1.4.3.			
	Longitudinal loading			
	$R > 600$ mm	T or Rev.	B	14
	$600 \text{ mm} > R > 150$ mm	T or Rev.	C	14
	$150 \text{ mm} > R > 50$ mm	T or Rev.	D	14
	$50 \text{ mm} > R$	T or Rev.	E	14
	Detail base metal for transverse loading : equal thickness and reinforcement removed			
	$R > 600$ mm	T or Rev.	B	14
	$600 \text{ mm} > R > 150$ mm	T or Rev.	C	14
	$150 \text{ mm} > R > 50$ mm	T or Rev.	D	14
	$50 \text{ mm} > R$	T or Rev.	E	14,15
	Detail base metal for transverse loading : equal thickness and reinforcement not removed			
	$R > 600$ mm	T or Rev.	C	14
	$600 \text{ mm} > R > 150$ mm	T or Rev.	C	14
$150 \text{ mm} > R > 50$ mm	T or Rev.	D	14	
$50 \text{ mm} > R$	T or Rev.	E	14,15	

TABLE F.2 : TYPE AND LOCATION OF MATERIAL (contd.)

General Condition	Situation	Kind of Stress ⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures ⁽²⁾ (Ref. Fig. F.1)
Attachments (contd.)	Detail base metal for transverse loading : unequal thickness and reinforcement removed			
	$R > 50$ mm	T or Rev.	D	14
	50 mm $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading : unequal thickness and reinforcement not removed all R	T or Rev.	E	14,15
	Detail base metal for transverse loading $R > 150$ mm	T or Rev.	C	19
	150 mm $> R > 50$ mm 50 mm $> R$	T or Rev. T or Rev.	D D	19 19
	Base metal at detail attached by full-penetration groove welds subject to longitudinal loading			
	50 mm $< a < 12b$ or 100 mm	T or Rev.	D	15
	$a > 12b$ or 100 mm when $b < 25$ mm	T or Rev.	E	15
	Base metal at detail attached by fillet welds or partial-penetration groove welds subject to longitudinal loading			
	$a < 50$ mm	T or Rev.	C	15,23,24,25,26
	50 mm $< a < 12b$ or 100 mm	T or Rev.	D	15,23,24,26
	$a > 12b$ or 100 mm when $b \leq 25$ mm	T or Rev.	E	15,23,24,26
$a > 12b$ or 100 mm when $b > 25$ mm	T or Rev.	E'	15,23,24,26	

TABLE F.2 : TYPE AND LOCATION OF MATERIAL (contd.)

General Condition	Situation	Kind of Stress ⁽¹⁾	Stress Category (Ref. Table F.3)	Illustrative Figures ⁽²⁾ (Ref. Fig. F.1)
Attachment (cond.)	Base metal attached by fillet welds or partial-penetration groove welds subjected to longitudinal loading when the weld termination embodies a transition radius with the weld termination ground smooth: $R > 50$ mm $R \leq 50$ mm	T or Rev. T or Rev.	D E	19 19
	Fillet-welded attachments where the weld termination embodies a transition radius, weld termination ground smooth and main material subject to longitudinal loading: Detail base metal for transverse loading $R > 50$ mm $R \leq 50$ mm	T or Rev. T or Rev.	D E	19 19
	Base metal at stud-type shear connector attached by fillet weld or automatic end weld	T or Rev.	C	22
	Shear stress on nominal area of stud type shear connectors	S	P	

- Note : (1) "T" signifies range in tensile stress only; "Rev" signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear, including shear stress reversal,
- (2) Illustrative figures provide guideline example only and is not intended to exclude other reasonably similar situations,
- (3) Allowable fatigue stress range for transverse partial penetrations and transverse fillet welds is a function of the effective throat, depth of penetration and plate thickness [Ref. Frank and Fisher (1979)].

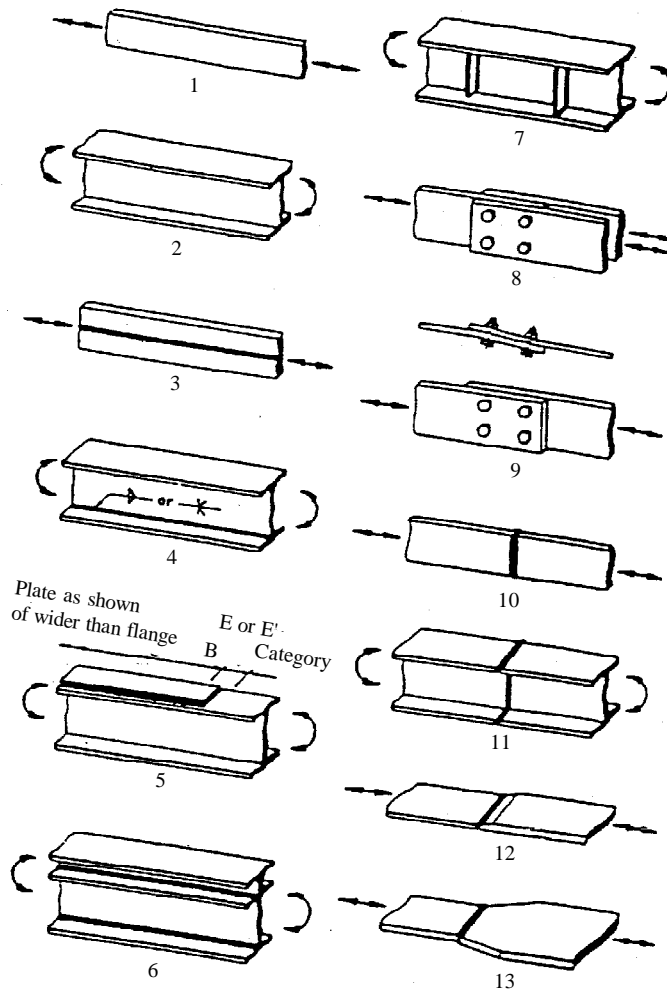


Fig. F.1 ILLUSTRATIVE FIGURES

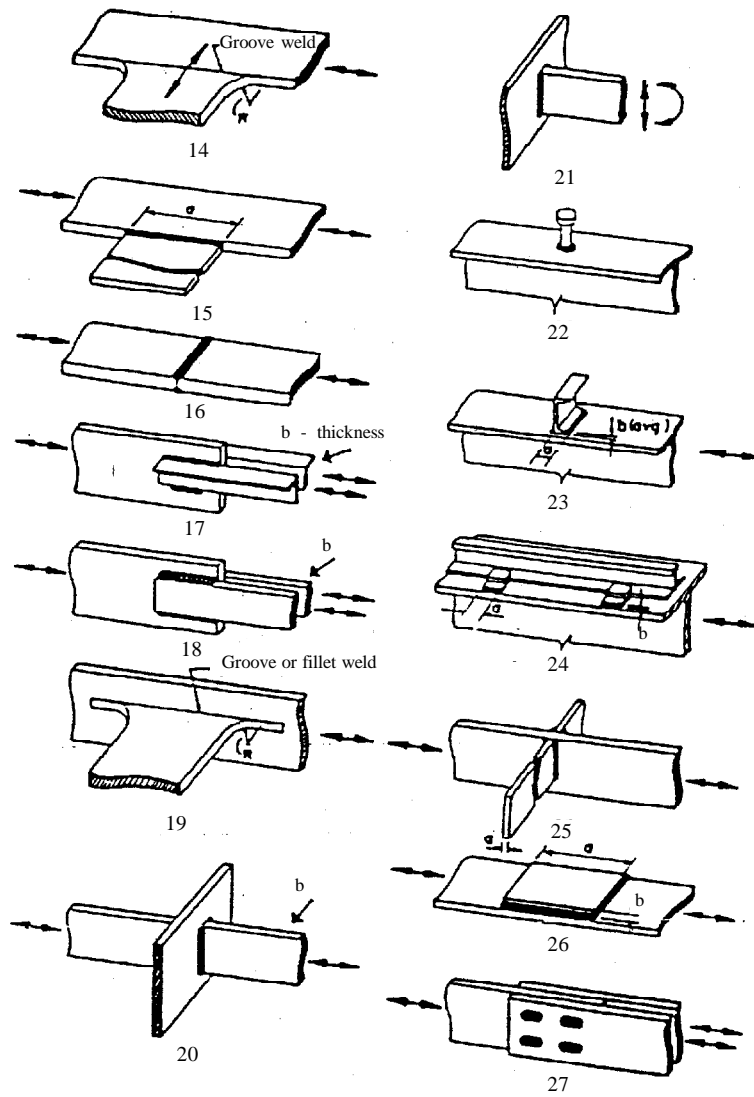


Fig. F.1 ILLUSTRATIVE FIGURES (contd.)

TABLE F.3 : MAXIMUM STRESS RANGE

Category⁽¹⁾	Maximum Range of Stress⁽¹⁾ (MPa)			
	Loading⁽²⁾ condition 1	Loading condition 2	Loading condition 3	Loading condition 4
<i>A</i>	434	255	165	165
<i>B</i>	338	200	124	110
<i>B'</i>	269	158	103	83
<i>C</i>	241	145	90	69 ⁽³⁾
<i>D</i>	193	110	69	48
<i>E</i>	152	90	55	31
<i>E'</i>	110	63	40	18
<i>F</i>	103	83	62	55

Note : (1) Refer Table F.2,

(2) Refer Table F.1,

(3) Flexural stress range of 82.5 MPa is permitted at the toe of stiffener welds or flanges.

ANNEXURE-I

CONSIDERATION OF IMPULSIVE AND IMPACTIVE LOADINGS

I.1 General

- I.1.1 The information given in this annexure may be found useful to those structural elements which are directly affected by the impactive and impulsive loads. Theoretical or experimental evidence may be used.

I.2 Design Approach

- I.2.1 In Appendix-A of AERB/SS/CSE, the impulsive and impactive loads are defined as dynamic class of loading. Table A.1 of the above Appendix-A contains various individual loads which fall under the category of impulsive and impactive loading. It is also identified in the above Appendix-A that the effect of which individual loads could be considered as static type in design of structures, though the structural response for this loading may be determined by dynamic analysis of the structures.
- I.2.2 The structure is generally designed using energy balance concept or elasto plastic method. However the impactive and impulsive loads must be considered concurrent with other loads (e.g. dead and live load) in determining the required resistance of structural element.
- I.2.3 If the deformation under the effect of impulsive and impactive loading is within elastic limits, the impulsive loading is combined with other loading as required by the provisions of this standard. The strain energy capacity available to resist impulsive loads must be reduced by the amount of work done by other load (factored, if plastic design method is used) during maximum response.
- I.2.4 In case of impactive and impulsive loading where a structural element deforms beyond its elastic limit, the provisions of load combinations as specified in this standard may not be applicable. In such cases either of the following approach may be adopted.
- (a) If the energy balance method is used, proper care shall be taken to consider the work done by static load such as dead load, live load, etc. (Ref. Fig. I.1),

(b) If an elastoplastic analysis is performed, the effective ductility ratio to be used in the design which is given by:

$$\mu' = \frac{X_m X_s}{X_y X_s} = \frac{\mu_d X_y X_s}{X_y X_s}, \quad (I.1)$$

where,

- μ' = effective ductility ratio,
- μ_d = displacement ductility ratio,
- X_m = maximum acceptable displacement,
- X_y = displacement at yielding of material,
- X_s = static displacement.

I.2.5 The displacement ductility ratio μ_d is defined as the ratio of maximum acceptable displacement X_m to the displacement at the effective yield point of the structural elements (ref. Fig. I.1). It could be noted that this is displacement ductility.

Dynamic Strength Increase

I.2.6 Due to the effect of impulsive and impactive loading the rate of increase of strain is rapid, as a result of which the member exhibits high strength under this loading. The increase in strength is considered in the design by means of dynamic increase factor (DIF). DIF is the ratio of dynamic and static yield stress or strength and direct function of strain rate involved. The DIF values for steel are given below:

$$[1.05+0.08 (\log \dot{\epsilon} + 3.0)] \leq 1.15, \text{ for } f_y < 415 \text{ MPa}$$

$$[1.0+0.02625(\log \dot{\epsilon} + 5.9)] \leq 1.10, \text{ for } f_y \geq 415 \text{ MPa}$$

where, $\dot{\epsilon}$ = strain rate in mm/mm/sec.

Design Requirement

- I.2.7 Ductility ratio is calculated from deformation considering both shear and flexural displacement.
- I.2.8 Ductility ratio values used in the design are such that maximum deformation satisfies the following:
- 1 does not exceed the limiting values for displacements as stipulated in this standard,
 - 1 does not result in loss of intended functions of the structural element,
 - 1 does not impair the safety functions of other structures, systems and components important to safety.
- I.2.9 The minimum structural resistance available for impulsive load is atleast 20% greater than the magnitude of any portion of the impulsive loading which is approximately constant for a time equal to or greater than the first fundamental period of the structural component.

I.3 Effects of Impulsive and Impactive Loading

Effects of Impulsive Loading

- I.3.1 When structural elements or systems of elements are subjected to impulsive loads, the structural response is determined by one of the following methods:
- (a) The dynamic effects of impulsive loads may be considered by calculating a dynamic load factor (DLF). The resistance available for the impulsive load is at least equal to the peak of the impulsive load transient multiplied by DLF. The calculation of DLF is based on ductility criteria and the dynamic characteristics of the structure and impulsive load transient,
 - (b) The dynamic effects of impulsive loads may be considered using impulse, momentum, and energy balance techniques. Strain energy capacity is limited by the ductility criteria.

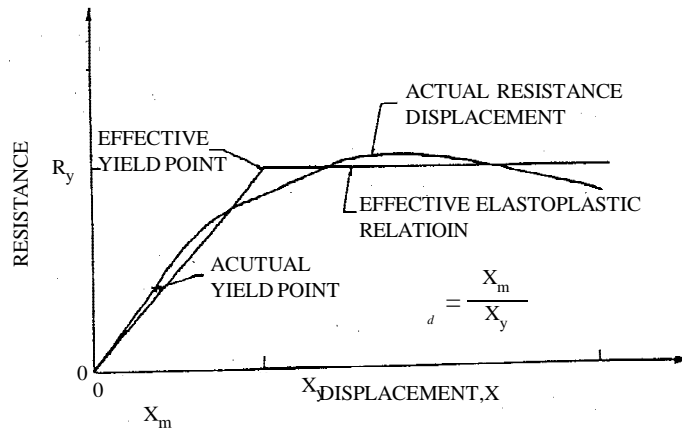
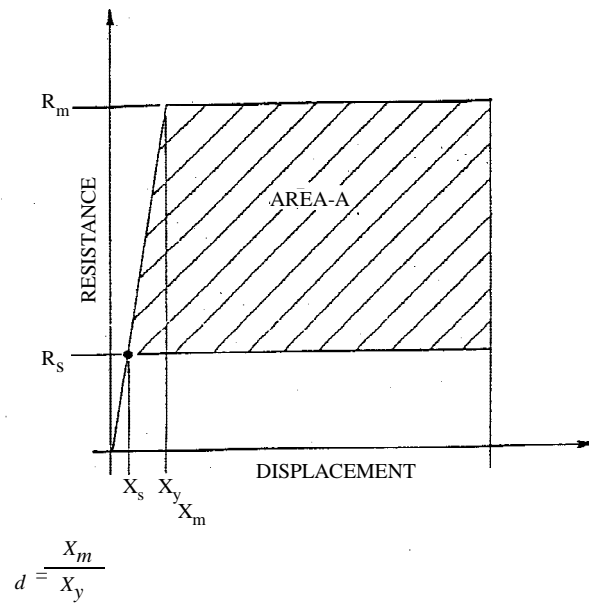


Fig. I.1 DUCTILITY RATIO d LOAD RESISTANCE RELATIONSHIP



R_s = Static Force to be combined with impulsive or impactive loads.
 X_s = Displacement due to static loads.

Fig. I.2 AVAILABLE RESISTANCE: IDEALISED RESISTANCE-DISPLACEMENT CURVE

- (c) The dynamic effects of impulsive loads may be considered by performing a time-history dynamic analysis. Mass and inertial properties are included as also the nonlinear stiffnesses of structural elements under consideration. Simplified bilinear definitions of stiffness may also be used. Maximum predicated response is governed by ductility criteria.

Effects of Impactive Loading

I.3.2 In the design for impactive effects, the following are considered:

- (a) Design for impactive loads satisfy the criteria for both local effects and for overall structural response,
- (b) Local impact effects may include penetration and perforation,
- (c) The penetration depth and required thickness to prevent perforation are based on applicable formulae or pertinent test data. When perforation of structural elements is precluded, the element thickness is at least 20% greater than that required to prevent perforation,
- (d) When structural elements or systems of elements are subject to impactive loads, the structural response may be determined using the approach given in I.3.1 above.

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