Design of Steel Structures – Limit State Method

(As Per IS 800: 2007)

By

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08.601. DESIGN OF STEEL STRUCTURES

Module I

Properties of structural steel, Structural steel sections, Limit state and working stress design concepts, Types of connections - Design of welded and bolted connections, Design of tension members and compression members, Design of laterally supported and unsupported beams - Built up beams, Simple beam to column connections.

Module II

Plate girders- design of section, curtailment of flange plate, bearing and intermediate stiffeners, connections, flange and web splices, Gantry girders (only design concept).

Columns- Design of axially and eccentrically loaded compression members, simple and built up sections, lacing and battening, Column bases- slab bases and gusseted bases.

Module III

Light gauge steel structures – Types of sections, Flat width ratio, buckling of thin elements, Effective design width, Form factor, Design of tension, compression members and beams.

Plastic design- basic assumptions - shape factor, load factor- Redistribution of moments - upper bound, lower bound and uniqueness theorems- analysis of simple and continuous beams, two span continuous beams and simple frames by plastic theory - static and kinematic methods.

References:

1. Subramanian N., Design of steel structures, Oxford University Press
2. Arya A.S. and J. L. Ajmani, Design of Steel Structures, Nemchand & Bros
3. Dayaratnam P., Design of Steel Structures, Wheeler
4. Ramachandra, Design of Steel Structures, Standard books
5. Duggal S.K., Design of Steel Structures, T.M.H. Publications
6. IS. Codes: IS:800-2007, IS:811-1987, IS:801- 1975

Note

Question Paper:

Duration: 3 hours

The question paper consists of Part A and Part B. Part A is for 40 marks. There will be 8 compulsory short answer questions of 5 marks each covering entire syllabus. Part B is for 60 marks. There will be two questions from each module. The candidate has to answer one question of 20 marks from each module.

Use of IS Codes: 800-2007, 811-1987, 801- 1975 and Structural Steel Tables is permitted in the Examination Hall.

No other charts, tables, codes are permitted in the Examination hall .If necessary relevant data shall be given along with the question paper by the question paper setter

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1. INTRODUCTION

Ever since steel began to be used in the construction of structures, it has made possible some of the grandest structures both in the past and also in the present day. Steel is by far the most useful material for building structures with strength of approximately ten times that of concrete, steel is the ideal material for modern construction. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges. Steel structures can be constructed very fast and this enables the structure to be used early thereby leading to overall economy. Steel structures are ductile and robust and can withstand severe loadings such as earthquakes. Steel structures can be easily repaired and retrofitted to carry higher loads. Steel is also a very eco-friendly material and steel structures can be easily dismantled and sold as scrap. Thus the lifecycle cost of steel structures, which includes the cost of construction, maintenance, repair and dismantling, can be less than that for concrete structures. Since steel is produced in the factory under better quality control, steel structures have higher reliability and safety. To get the most benefit out of steel, steel structures should be designed and protected to resist corrosion and fire. They should be designed and detailed for easy fabrication and erection. Good quality control is essential to ensure proper fitting of the various structural elements. The effects of temperature should be considered in design. To prevent development of cracks under fatigue and earthquake loads the connections and in particular the welds should be designed and detailed properly. Special steels and protective measures for corrosion and fire are available and the designer should be familiar with the options available.

A steel structure, like any other, is an assemblage of a group of members which contribute to resist the total load and thereby transfer the loads safely to ground. This consist members subjected to various actions like axial forces (Compression & Tension), bending, shear, torsion etc or a combination of these. The elements are connected together by means of rivets, pins or welds. Depending on the fixity of these joints, the connections are classified as rigid, semi rigid and flexible.

1.1. PROPERTIES OF STRUCTURAL STEEL

The properties of structural steel, as per clause 2.2.4 of IS 800:2007, for use in design, may be taken as given in clauses 2.2.4.1 and 2.2.4.2 of the code.

1.1.1. Physical properties

Physical properties of structural steel, as detailed by cl.2.2.4.1 of IS 800:2007, irrespective of its grade may be taken as: a) Unit mass of steel, p = 7850 kg/m^3 b) Modulus of elasticity, E = 2.0×10^5 N/mm^2 (MPa) c) Poisson ratio, p = 0.3 d) Modulus of rigidity, G = 0.769×10^5 N/mm^2 (MPa) e) Coefficient of thermal expansion c_x.=$12 \times 10^{-6}/^0C$

1.1.2. Mechanical properties

The principal mechanical properties of the structural steel important in design, as detailed by the code IS 800:2007 in cl. 2.2.4.2, are the yield stress, f_y; the tensile or ultimate stress, f_u; the maximum percent elongation on a standard gauge length and notch toughness. Except for notch toughness, the other properties are determined by conducting tensile tests on samples cut from the plates, sections, etc, in accordance with IS 1608. Commonly used properties for the common steel products of different specifications are summarized in Table 1 of IS 800:2007. Highlights of the table are reproduced for ready reference as Table 1.

Table 1. Tensile Properties of Structural Steel Products

IS Code	Grade	Yield stress (Mpa) min (for d or t)			Ultimate tensile stress (MPa) min	Elongation Percent min
		<20	20 – 40	>40		
IS 2062	E 165 (Fe 290)	165	165	165	290	23
	E250(Fe410W)A	250	240	230	410	23
	E250(Fe 410 W)B	250	240	230	410	23
	E250(Fe 410 W)C	250	240	230	410	23
	E 300 (Fe 440)	300	290	280	440	22
	E 350 (Fe 490)	350	330	320	490	22
	E 410 (Fe 540)	410	390	380	540	20
	E 450 (Fe 570) D	450	430	420	570	20
	E 450 (Fe 590) E	450	430	420	590	20

1.1.2.1. Stress – strain behaviour: tensile test

The stress-strain curve for steel is generally obtained from tensile test on standard specimens as given in Figure 1. The details of the specimen and the method of testing is elaborated in IS: 1608 (1995). The important parameters are the gauge length 'L_c' and the initial cross section area S_o. The loads are applied through the threaded or shouldered ends. The initial gauge length is taken as $5.65\sqrt{S_o}$ in the case of rectangular specimen and it is five times the diameter in the case of circular specimen. A typical stress-strain curve of the tensile test coupon is shown in Figure 2 in which a sharp change in yield point followed by plastic strain is observed. After a certain amount of the plastic deformation of the material, due to reorientation of the crystal structure an increase in load is observed with increase in strain. This range is called the strain hardening range. After a little increase in load, the specimen eventually fractures. After the failure it is seen that the fractured surface of the two pieces form a cup and cone arrangement. This cup and cone fracture is considered to be an indication of ductile fracture. It is seen from Figure 2 that the elastic strain is up to ε_y followed by a yield plateau between strains ε_y and ε_{sh} and a strain hardening range start at ε_{sh} and the specimen fail at ε_{ult} where ε_y, ε_{sh} and ε_{ult} are the strains at onset of yielding, strain hardening and failure respectively.

Depending on the steel used, ε_{sh} generally varies between 5 and 15 ε_y, with an average value of 10 ε_y typically used in many applications. For all structural steels, the modulus of elasticity can be taken as 205,000 MPa and the tangent modus at the onset of strain hardening is roughly 1/30th of that value or approximately 6700 MPa. High strength steels, due to their specific microstructure, do not show a sharp yield point but rather they yield continuously as shown in Figure 2. For such steels the yield stress is always taken as the stress at which a line at 0.2% strain, parallel to the elastic portion, intercepts the stress strain curve.

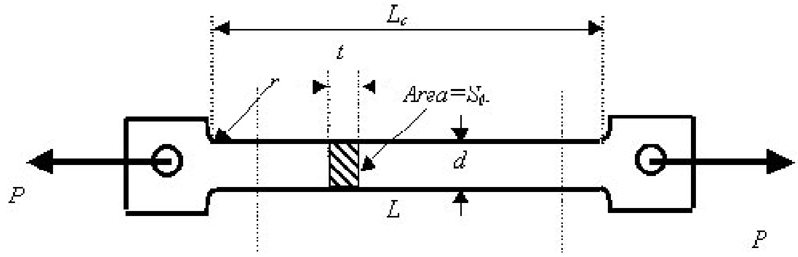

Figure 1 Standard specimen for tensile test.

The nominal stress or the engineering stress is given by the load divided by the original area. Similarly, the engineering strain is taken as the ratio of the change in length to original length.

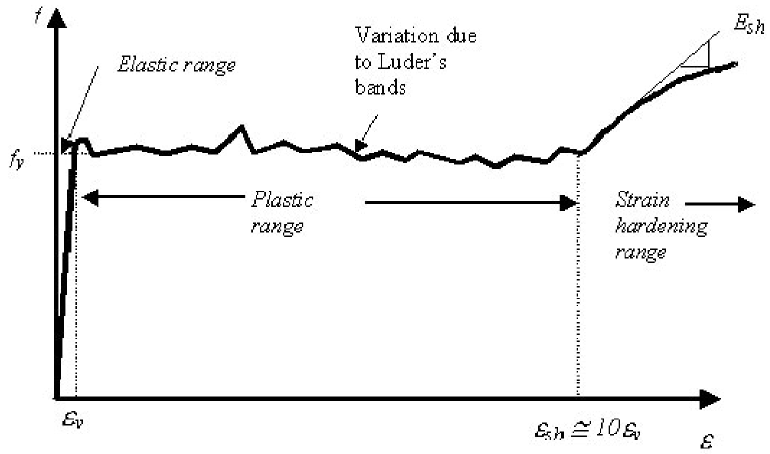

Figure 2 Stress – Strain curve for mild steel.

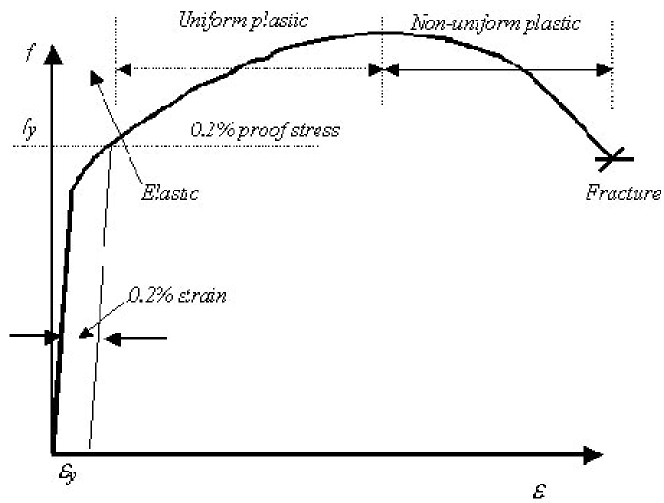

Figure 3 Stress – Strain curve for high strength steel.

1.1.2.2. Hardness

Hardness is regarded as the resistance of a material to indentations and scratching. This is generally determined by forcing an indentor on to the surface. The resultant deformation in steel is both elastic and plastic. There are several methods using which the hardness of a metal could be found out. They basically differ in the form of the indentor, which is used on to the surface. Brinell hardness usually uses steel balls where as Vickers hardness uses square based diamond pyramid with 135^0 and Rockwell hardness uses diamond cone with 120^0 angle. In all the above cases, hardness number is related to the ratio of the applied load to the surface area of the indentation formed. The testing procedure involves forcing the indentor on to the surface at a particular road. On removal, the size of indentation is measured.

1.1.2.3. Notch Toughness

There is always a possibility of microscopic cracks in a material or the material may develop such cracks as a result of several cycles of loading. Such cracks may grow rapidly without detection and lead to sudden collapse of the structure. To ensure that this does not happen, materials in which the cracks grow slowly are preferred. Such steels are known as notch-tough steels and the amount of energy they absorb is measured by impacting a notched specimen with a heavy pendulum as in Izod or Charpy tests.

1.2. ROLLED STEEL SECTIONS

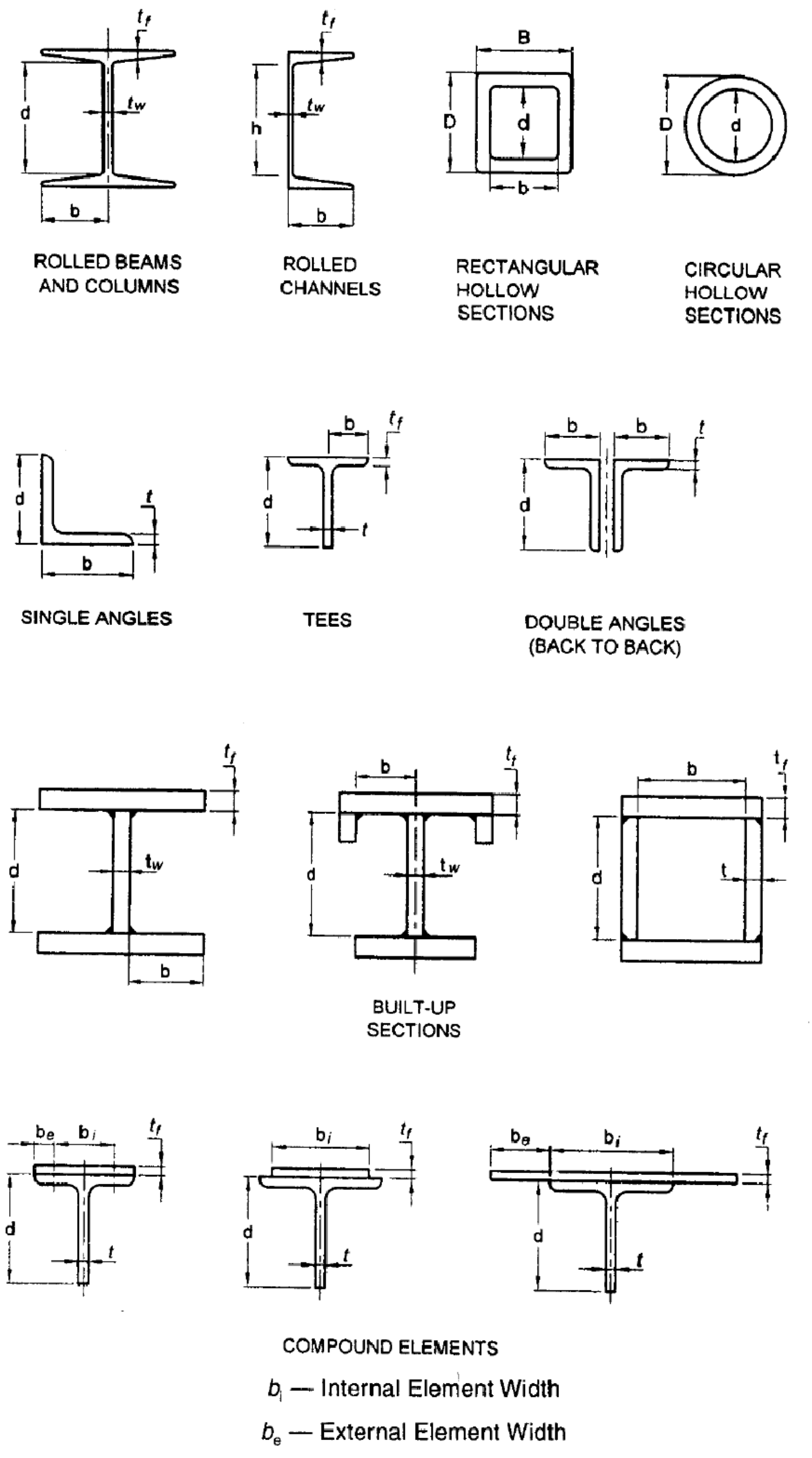

Figure 4 Standard Rolled Sections

Hot Rolling is usually used to produce the standard section. In this process, the molten steel is poured in to continuous casting systems where it is passed through a series of rollers which squeeze it to the desired shape before if solidifies completely. It is subsequently cut in to desired standard lengths. Cross section and size of the members are governed by optimum use of material, functional requirement etc. Usually sections with larger modulus of section compared to cross sectional area are preferred. IS Handbook 1 published by BIS provides the dimensions, weights and other sectional properties of various standard sections. Some of the sections as detailed by Figure 2 of IS 800:2007 is reproduced here in Figure 4.

1.2.1. Conventions for member axes

Unless otherwise specified, x-x axis is considered along the length of the member; y-y axis of the cross section is the axis perpendicular to the flanges in general and the axis perpendicular to the smaller leg in the case of angles; z-z axis of the cross section is the axis parallel to the flanges in general and the axis parallel to the smaller leg in the case of angles; u-u axis is the major axis of the section and v-v axis is the minor axis of the section. This is schematically represented in Figure 5.

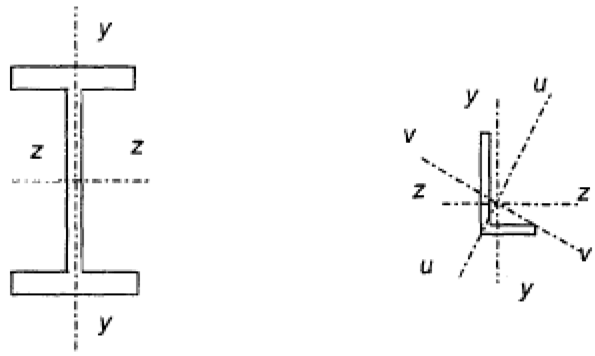

Figure 5 Axes of Members

2. GENERAL DESIGN REQUIREMENTS

The general design requirements are outlined in Section 3 of IS 800:2007.

2.1. BASIS FOR DESIGN

The bases of the design are given in Section 3.1 of IS 800:2007. It is as follows.

2.1.1. Design Objective

The objective of design, as outlined in Cl.3.1.1 of IS 800:2007, is the achievement of an acceptable probability that structures will perform satisfactorily for the intended purpose during the design life. With an appropriate degree of safety, they should sustain all the loads and deformations, during construction and use and have adequate resistance to certain expected accidental loads and fire. Structure should be stable and have alternate load paths to prevent disproportionate overall collapse under accidental loading.

2.1.2. Methods of Design

Method of Design of steel structures is given in Cl. 3.1.2 of IS 800:2007. In the previous version of the code, the design of steel structures was essentially using Working Stress Method. But IS 800:2007 permits us to design the structure to satisfy the various Limit States. It also advocates the use of Working Stress Method only to the situations where Limit State cannot be conveniently employed. As per Cl. 3.1.2.1 of IS 800:2007, Structure and its elements shall normally, be designed by the limit state method. Account should be taken of accepted theories, experimental information and experience and the need to design for durability. This clause admits that calculations alone may not produce Safe, serviceable and durable structures. Suitable materials, quality control, adequate detailing and good supervision are equally important. As per Cl. 3.1.2.2 of IS 800:2007, where the limit states method cannot be conveniently adopted; the working stress design (Section 11 of IS 800:2007) may be used.

2.1.3. Design Process

Clause 3.1.3 of IS 800:2007 specifies structural design, including design for durability, construction and use should be considered as a whole. The realization of design objectives requires compliance with clearly defined standards for materials, fabrication, erection and in-service maintenance.

2.2. LOADS AND FORCES

Clause 3.2 of IS 800:2007 specifies the various loads and forces that has to be considered while performing the design of steel structures. As per Cl. 3.2.1 of IS 800:2007, for the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable, with partial safety factors and combinations (Cl. 5.3.3 of IS 800:2007). (a) Dead loads; (b) Imposed loads (live load, crane load, snow load, dust load, wave load, earth pressures, etc); (c) Wind loads; (d) Earthquake loads; (e) Erection loads; (f) Accidental loads such as those due to blast, impact of vehicles, etc; and (g) Secondary effects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components, eccentric connections, rigidity of joints differing from design assumptions.

2.2.1. Dead loads (Cl. 3.2.1.1 of IS 800:2007)

Dead loads should be assumed in design as specified in IS 875 (Part 1).

2.2.2. Imposed Loads (Cl. 3.2.1.2 of IS 800:2007)

IS 800:2007 specifies in Cl.3.2.1.2 that imposed loads for different types of occupancy and function of structures shall be taken as recommended in IS 875 (Part 2). Imposed loads arising from equipment, such as cranes and machines should be assumed in design as per manufacturers/suppliers data (Cl. 3.5.4 of IS 800:2007). Snow load shall be taken as per IS 875 (Part 4).

2.2.3. Wind loads (Cl. 3.2.1.3 of IS 800:2007)

Wind loads on structures shall be taken as per the recommendations of IS 875 (Part 3).

2.2.4. Earthquake loads (Cl. 3.2.1.4 of IS 800:2007)

Earthquake loads shall be assumed as per the recommendations of IS 1893 (Part 1).

2.2.5. Erection Loads (Cl. 3.3 of IS 800:2007)

All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment, including all loads due to operation of such equipment shall be considered as erection loads. The structure as a whole and all parts of the structure in conjunction with the temporary bracings shall be capable of sustaining these loads during erection.

2.2.6. Temperature Effects (Cl. 3.4 of IS 800:2007)

Expansion and contraction due to changes in temperature of the members and elements of a structure shall be considered and adequate provision made for such effect. The co-efficient of thermal expansion for steel is as given in Cl. 2.2.4.l of IS 800:2007.

2.2.7. Load Combinations

Load combinations for design purposes shall be those that produce maximum forces and effects and consequently maximum stresses and deformations. The following combination of loads with appropriate partial safety factors as given in Table 4 of IS 800:2007 may be considered. The table is reproduced here as Table 2 for ready reference. a) Dead load + imposed load, b) Dead load + imposed load + wind or earthquake load, c) Dead load + wind or earthquake load, and d) Dead load+ erection load. The effect of wind load and earthquake loads shall not be considered to act simultaneously. The load combinations are outlined in detail in Cl. 3.5 of IS 800:2007.

Table 2 Partial safety factors for loads for limit states

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL$^{1)}$		WL/EL	AL	DL	LL$^{1)}$		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)$^{2)}$	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)$^{2)}$	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

2.3. GEOMETRICAL PROPERTIES

The geometrical properties, as detailed in Cl. 3.6 of IS 800:2007, of the gross and the effective cross-sections of a member or part thereof, shall be calculated on the following basis: a) the properties of the gross cross-section shall be calculated from the specified size of the member or part thereof or read from appropriate table b) The properties of the effective cross-section shall be calculated by deducting from the area of the gross cross-section, the following:

- The sectional area in excess of effective plate width, in case of slender sections.
- The sectional areas of all holes in the section except for parts in compression. In case of punched holes, hole size 2 mm in excess of the actual diameter may be deducted.

2.4. CLASSIFICATION OF CROSS-SECTIONS

Plate elements of a cross-section may buckle locally due to compressive stresses. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross-section subjected to compression due to axial force, moment or shear. When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling, to enable the redistribution of bending moment required before formation of the failure mechanism. When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling. On basis of the above, Cl. 3.7 of IS 800:200 categorizes the sections in to four classes as follows.

When different elements of a cross-section fall under different classes, the section shall be classified as governed by the most critical element. The maximum value of limiting width to thickness ratios of elements for different classifications of sections are given in Table 2 of IS 800:2007 which is reproduced here as

Table 3.

2.4.1. Class 1 (Plastic)

Cross-sections which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism fall under this category. The width to thickness ratio of plate elements shall be less than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

2.4.2. Class 2 (Compact)

Cross-sections which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling come under this class. The width to thickness ratio of plate elements shall be less than that specified under Class 2 (Compact), but greater than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

2.4.3. Class 3 (Semi-compact)

Cross-sections in which the extreme fiber in compression can reach yield stress but cannot develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact), in Table 2 of IS 800:2007.

2.4.4. Class 4 (Slender)

Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under Class 3 (Semi-compact), in Table 2 of IS 800:2007. In such cases, the effective sections for design shall be calculated either by following the provisions of IS 801 to account for the post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

Table 3 Limiting Width to Thickness Ratio

Compression Element (1)			Ratio (2)	Class of Section: Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	$33.5\ \varepsilon$	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\,\varepsilon}{1+r_1}$	$\frac{126.0\,\varepsilon}{1+2r_2}$ but $\leq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\,\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε $9.4\ \varepsilon$	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

2.5. TYPES OF ELEMENTS

IS 800:2007 classifies elements in to three types, as per Cl. 3.7.3., as follows.

2.5.1. Internal elements

These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section.

2.5.2. Outside elements or outstands

These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane, for example flange overhang of an I-section, stem of T-section and legs of an angle section.

2.5.3. Tapered elements

These maybe treated as flat elements having average thickness as defined in SP 6 (Part 1).

2.6. MAXIMUM EFFECTIVE SLENDERNESS RATIO

The maximum effective slenderness ratio, as per Cl. 3.8 of IS 800:2007, KL/r values of a beam, strut or tension member shall not exceed those given in Table 3 of IS 800:2007. 'KL' is the effective length of the member and 'r' is appropriate radius of gyration based on the effective section as defined in Cl. 3.6.1 of IS 800:2007. This data is reproduced here in Table 4.

Table 4 Maximum effective slenderness ratio

Member	Maximum Effective Slenderness Ratio (KL/r)
A member carrying compressive loads resulting from dead loads and imposed loads	180
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compression forces resulting only from combination with wind/earthquake actions, provided the deformation of such member does not adversely affect tbe stress in any part of the structure	250
Compression flange of a beam against lateral torsional buckling	300
A member normally acting m a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake forces	350
Members always under tension (other than pre-tensioned members)	400

3. LIMIT STATE DESIGN

The current revision of the code of practice, IS 800:2000, recommends limit state method for design of structures using hot rolled sections. This method is outlined in section 5 of IS 800:2007. However, it retained working stress method of design which was the design method for decades. But the scope of the working stress method is limited to those situations where limit state method cannot be conveniently employed.

3.1. BASIS FOR DESIGN

In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. It shall not suffer total collapse under accidental loads such as from explosions or impact or due to consequences of human error to an extent beyond the local damages. The objective of the design is to achieve a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reached during its lifetime should be very low. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

Steel structures are to be designed and constructed to satisfy the design requirements with regard to stability, strength, serviceability, brittle fracture, fatigue, fire, and durability such that they meet the following: a) Remain fit with adequate reliability and be able to sustain all actions (loads) and other influences experienced during construction and use; b) Have adequate durability under normal maintenance; c) Do not suffer overall damage or collapse disproportionately under accidental events like explosions, vehicle impact or due to consequences of human error to an extent beyond local damage. The potential for catastrophic damage shall be limited or avoided by appropriate choice of one or more of the following:

- Avoiding, eliminating or reducing exposure to hazards, which the structure is likely to sustain.
- Choosing structural forms, layouts and details and designing such that: i) the structure has low sensitivity to hazardous conditions; and ii) the structure survives with only local damage even after serious damage to any one individual element by the hazard.
- Choosing suitable material, design and detailing procedure, construction specifications, and control procedures for shop fabrication and field construction as relevant to the particular structure.

The following conditions may be satisfied to avoid a disproportionate collapse: The building should be effectively tied together at each principal floor level and each column should be effectively held in position by means of continuous ties (beams) nearly orthogonal, except where the steel work supports only cladding weighing not more than 0.7 kN/m^2 along with imposed and wind loads. These ties must be steel members such as beams, which may be designed for other purposes, steel bar reinforcement anchoring the steel frame to concrete floor or steel mesh reinforcement in composite slab with steel profiled sheeting directly connected to beam with shear connectors. These steel ties and their end connections should be capable of resisting factored tensile force not less than the factored dead and imposed loads acting on the floor area tributary to the tie nor less than 75 kN. Such connection of ties to edge column should also be capable of resisting 1 percent of the maximum axial compression in the column at the level due to factored dead and imposed loads. All column splices should be capable of resisting a tensile force equal to the largest of a factored dead and live load reaction from a single floor level located between that column splice and the next column splice below that splice. Lateral load system to resist notional horizontal loads prescribed in Cl. 4.3.6 of IS 800:2007 should be distributed throughout the building in nearly orthogonal directions so that no substantial portion is connected at only one point to such a system. Precast concrete or other heavy floor or roof units should be effectively anchored in the direction of their span either to each other over the support or directly to the support. Where the above conditions to tie the columns to the floor adequately are not satisfied each storey of the building should be checked to ensure that disproportionate collapse would not precipitate by the notional removal, one at a time, of each column. Where each floor is not laterally supported by more than one system, check should be made at each storey by removing one such lateral support system at a time to ensure that disproportionate collapse would not occur. The collapse is considered disproportionate, if more than 15 percent of the floor or roof area of 70 m^2 collapse at that level and at one adjoining level either above or below it, under a load equal to 1.05 or 0.9 times the dead load, 0.33 times temporary or full imposed load of permanent nature (as in storage buildings) and 0.33 times wind load acting together.

3.2. LIMIT STATE DESIGN PHILOSOPHY

For achieving the design objectives, the design shall be based on characteristic values for material strengths and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The characteristic values shall be based on statistical data, if available. Where such data is not available, these shall be based on experience. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. In the absence of special considerations, these factors shall have the values given in this section according to the material, the type of load and the limit state being considered. The reliability of design is ensured by satisfying the requirement: Design action ≤ Design strength

Limit states are the states beyond which the structure no longer satisfies the performance requirements specified. The limit states are classified as: a) Limit state of strength; and b) Limit state of serviceability.

3.2.1. The limit states of strength

The limit states of strength, as detailed in Cl. 5.2.2.1 of IS 800:2007, are those associated with failures (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and property. The limit state of strength includes: a) Loss of equilibrium of the structure as a whole or any of its parts or components. b) Loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of its parts including supports and foundations. c) Failure by excessive deformation, rupture of the structure or any of its parts or components, d) Fracture due to fatigue, e) Brittle fracture.

3.2.2. The limit state of serviceability

The limit state of serviceability, as detailed in Cl. 5.2.2.1 of IS 800:2007 include: a) Deformation and deflections, which may adversely affect the appearance or effective use of the structure or may cause improper functioning of equipment or services or may cause damages to finishes and non-structural members. b) Vibrations in the structure or any of its components causing discomfort to people, damages to the structure, its contents or which may limit its functional effectiveness. Special consideration shall be given to systems susceptible to vibration, such as large open floor areas free of partitions to ensure that such vibrations are acceptable for the intended use and occupancy (see Annex C of IS 800:2007). c) Repairable damage or crack due to fatigue. d) Corrosion, durability and e) Fire.

3.2.3. Actions

The actions (loads), as detailed in Cl. 5.3 of IS 800:2007, to be considered in design include direct actions (loads) experienced by the structure due to self weight, external actions etc., and imposed deformations such as that due to temperature and settlements.

3.2.3.1. Classification of Actions

Actions are classified by Cl. 5.3.1 of IS 800:2007, by their variation with time as given below:

- Permanent actions (Q_p): Actions due to self weight of structural and non-structural components, fittings, ancillaries, and fixed equipment, etc.
- Variable actions (Q_v): Actions due to construction and service stage loads such as imposed (live) loads (crane loads, snow loads, etc.), wind loads, and earthquake loads, etc.
- Accidental actions (Q_a): Actions expected due to explosions, and impact of vehicles, etc.

3.2.3.2. Characteristic Actions (Loads)

The Characteristic Actions, Q_C, as defined by the code in Cl.5.3.2, are the values of the different actions that are not expected to be exceeded with more than 5 percent probability, during the life of the structure and they are taken as: a) the self-weight, in most cases calculated on the basis of nominal dimensions and unit weights [see IS 875 (Part 1)], b) the variable loads, values of which are specified in relevant standard [see IS 875 (all Parts) and IS 1893 (Part l)], c) the upper limit with a specified probability (usually 5 percent) not exceeding during some reference period (design life) and d) specified by client, or by designer in consultation with client, provided they satisfy the minimum provisions of the relevant loading standard.

3.2.3.3. Design Actions

The Design Actions, Q_d, is expressed as $\sum\gamma_{fk}Q_{ck}$, where γ_{fk} = partial safety factor for different loads k, given in Table 4 of IS 800:2007 to account for: a) Possibility of unfavourable deviation of the load from the characteristic value, b) Possibility of inaccurate assessment of the load, c) Uncertainty in the assessment of effects of the load, and d) Uncertainty in the assessment of the limit states being considered. This is detailed in Cl. 5.3.3 of IS 800:2007.

3.2.4. Strength

The ultimate strength calculation as detailed in Cl. 5.4 of IS 800: 2000 require consideration of the following: a) Loss of equilibrium of the structure or any part of it, considered as a rigid body; and b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of it including support and foundation.

3.2.4.1. Design Strength

The Design Strength given in 5.4.1 of IS 800:2007, S_d, is obtained from ultimate strength, S_u and partial safety factors for materials, γ_m given in Table 5 of IS 800:2007 by the relation $S_d \leq S_u/\gamma_m$, where partial safety factor for materials, γ_m account for: a) Possibility of unfavourable deviation of material strength from the characteristic value, b) Possibility of unfavourable variation of member sizes, c) Possibility of unfavourable reduction in member strength due to fabrication and tolerances, and d) Uncertainty in the calculation

3.2.5. Factors Governing the Ultimate Strength

The following factors are considered by IS 800:2007 as those governing the ultimate strength.

3.2.5.1. Stability

Stability shall be ensured for the structure as a whole and for each of its elements. This should include overall frame stability against overturning and sway, as given in Clause 5.5.1.1 and 5.5.1.2 of IS 800:2007.

3.2.5.2. Stability against overturning

The structure as a whole or any part of it shall be designed to prevent instability due to overturning, uplift or sliding under factored load as given below: a) The Actions shall be divided into components aiding instability and components resisting instability. b) The permanent and variable actions and their effects causing instability shall be combined using appropriate load factors as per the Limit State requirements, to obtain maximum destabilizing effect.

3.2.5.3. Sway stability

The whole structure, including portions between expansion joints, shall be adequately stiff against sway. To ensure this, in addition to designing for applied horizontal loads, a separate check should be carried out for notional horizontal loads such as given in Cl. 4.3.6 of IS 800:2007 to evaluate the sway under gravity loads.

3.2.5.4. Fatigue

Generally fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to fluctuations in wind loading normally need not be considered. Fatigue design shall be in accordance with Section 13 of IS 800:2007. When designing for fatigue, the partial safety factor for load, γ_f, equal to unity shall be used for the load causing stress fluctuation and stress range.

3.2.5.5. Plastic Collapse

Plastic analysis and design may be used, if the requirement specified under the plastic method of analysis (Cl. 4.5 of IS 800:2007) are satisfied.

3.2.6.Limit State of Serviceability

Serviceability limit state is related to the criteria governing normal use. Serviceability limit state is limit state beyond which the service criteria specified below, are no longer met: a) Deflection limit, b) Vibration limit, c) Durability consideration, and d) Fire resistance.

3.2.6.1. Deflection

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishings. Deflections are to be checked for the most adverse but realistic combination of service loads and their arrangement, by elastic analysis, using a load factor of 1.0. Table 6 of IS 800:2007 gives recommended limits of deflections for certain structural members and systems. Circumstances may arise where greater or lesser values would be more appropriate depending upon the nature of material in element to be supported (vulnerable to cracking or not) and intended use of the structure, as required by client.

3.2.6.2. Vibration

Suitable provisions in the design shall be made for the dynamic effects of live loads, impact loads and vibration due to machinery operating loads. In severe cases, possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to effective width of lateral load resistance system exceeding 5:1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to large number of cycles of loading shall be designed against fatigue failure, as specified in Section 13 of the code. Annex C of the code can be used for accommodating the floor vibration.

3.2.6.3. Durability

Factors that affect the durability of the buildings, under conditions relevant to their intended life like a) Environment, b) Degree of exposure, c) Shape of the member and the structural detail, d) Protective measure, and e) Ease of maintenance. The durability of steel structures shall be ensured by recommendations in Section 15 of the code.

3.2.6.4. Fire Resistance

Fire resistance of a steel member is a function of its mass, its geometry, actions to which it is subjected, its structural support condition, fire protection measures adopted and the fire to which it is exposed. Design provision is to resist fire are discussed in Section 16 of the code.

4. DESIGN OF CONNECTIONS

Section 10 of IS 800:2007 deals with the design and detailing requirements for joints between members. The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure and comply with the requirements specified in section 10 of the code.

Connections shall be capable of transmitting the calculated design actions. In most structures connections are the weakest link. This leads often to failure in spite of the strong members used. This draws our attention to the design of connections with utmost care. The behaviour of connections is quite complex due to geometric imperfections and complexities, lack of fit, residual stresses etc; making it complex to analyse. This can be simplified by a number of assumptions and approximations based on past experience, experimental results and ductility of steel. It is the ductility of steel assists the distribution of forces generated within a joint. This is outlined in Cl. 10.1.4 of IS 800:2007.

The ultimate aim of connection design is to have a simple, compatible, feasible, easy to fabricate, safe and economical joint.

4.1. TYPES OF CONNECTIONS.

Connection elements consist of components such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and welds. Connections are classified based on the connecting element and the fixity of the joint

4.1.1. Classification based on the connector

Connections are classified based on the connecting element in to (a) Riveted, (b) Bolted, (c) Pinned and (d) Welded connection. Of these riveted, bolted and pinned connections behave in a similar manner.

4.1.2. Classification based on the fixity of the joint

Based on the fixity of the joint, connections are classified in to (a) Rigid joint, (b) Semi rigid joint and (c) Flexible joints.

4.2. SELECTIONS OF TYPE OF CONNECTION

Riveted connections were once very popular and are still used in some cases but will gradually be replaced by bolted connections. This is due to the low strength of rivets, higher installation costs and the inherent inefficiency of the connection. Welded connections have the advantage that no holes need to be drilled in the member and consequently have higher efficiencies. However, welding in the field may be difficult, costly, and time consuming. Welded connections are also susceptible to failure by cracking under repeated cyclic loads due to fatigue which may be due to working loads such as trains passing over a bridge (high-cycle fatigue) or earthquakes (low-cycle fatigue). A special type of bolted connection using High Strength Friction Grip (HSFG) bolts has been found to perform better under such conditions than the conventional black bolts used to resist predominantly static loading. Bolted connections are also easy to inspect and replace. The choice of using a particular type of connection is entirely that of the designer and he should take his decision based on a good understanding of the connection behaviour, economy and speed of construction. Ease of fabrication and erection should be considered in the design of connections. Attention should be paid to clearances necessary for field erection, tolerances, tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.

5. BOLTED CONNECTIONS

Bolt is a metal pin with a head at one end and a shank threaded at other end to receive a nut, as shown in Figure 6. Steel washers are usually provided under the bolt head and nuts to prevent the treaded portion of the bolt from bearing on the connecting pieces and to distribute the clamping pressure on the bolted member.

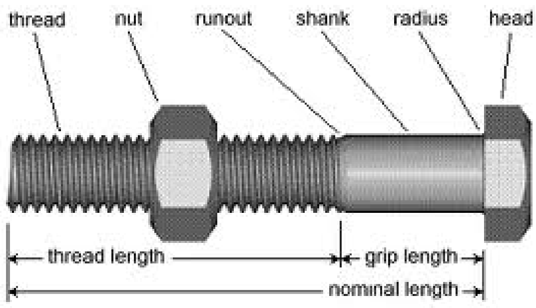

Figure 6 A Bolt

A bolt connection can be used for end connections in tension and compression members. They can also hold down column bases in position and as separator for purlins and beams in foundations. Bolts are having the following advantages over rivets and pins: (a) the erection of the structures can be speeded up. (b) Less skilled labour can be employed. (c) Overall cost of bolted connection is lesser than the other alternatives. However the following shortcomings are also associated with the bolted connections: (a) Cost of material is high, about double than that of rivets. (b) The tensile strength of bolt is reduced due to the reduced area at the root of the thread and stress concentration. (c) Normally strength reduction will be there for loose fit bolts. (d) Bolts may get loose when subjected to vibrations.

5.1. CLASSIFICATION OF BOLTS

Bolts used in steel structures are of three types: 1) Black Bolts 2) Turned and Fitted Bolts and 3) High Strength Friction Grip (HSFG) Bolts.

The International Standards Organisation designation for bolts, also followed in India, is given by Grade x.y. In this nomenclature, x indicates one-tenth of the minimum ultimate tensile strength of the bolt in kgf/mm^2 and the second number, y, indicates one-tenth of the ratio of the yield stress to ultimate stress, expressed as a percentage. Thus, for example, grade 4.6 bolt will have a minimum ultimate strength 40 kgf/mm^2 (392 MPa) and minimum yield strength of 0.6 times 40, which is 24 kgf/mm^2 (235 MPa).

5.1.1.Black bolts

Black bolts are unfinished and are made of mild steel and are usually of Grade 4.6. Black bolts have adequate strength and ductility when used properly; but while tightening the nut snug tight ("Snug tight" is defined as the tightness that exists when all plies in a joint are in firm contact) will twist off easily if tightened too much.

5.1.2.Turned and fitted bolts

Turned and fitted bolts have uniform shanks and are inserted in close tolerance drilled holes and made snug tight by box spanners. The diameter of the hole is about 1.5 to 2.0 mm larger than the bolt diameter for ease in fitting. High strength black bolts (grade 8.8) may also be used in connections in which the bolts are tightened snug fit. In these bearing type of connections, the plates are in firm contact but may slip under loading until the hole surface bears against the bolt .The load transmitted from plate to bolt is therefore by bearing and the bolt is in shear. Under dynamic loads, the nuts are liable to become loose and so these bolts are not allowed for use under such loading. In situations where small slips can cause significant effects as in beam splices, black bolts are not preferred. However, due to the lower cost of the bolt and its installation, black bolts are quite popular in simple structures subjected to static loading. Turned and fitted bolts are available from grade 4.6 to grade 8.8. For the higher grades there is no definite yield point and so 0.2% proof stress is used.

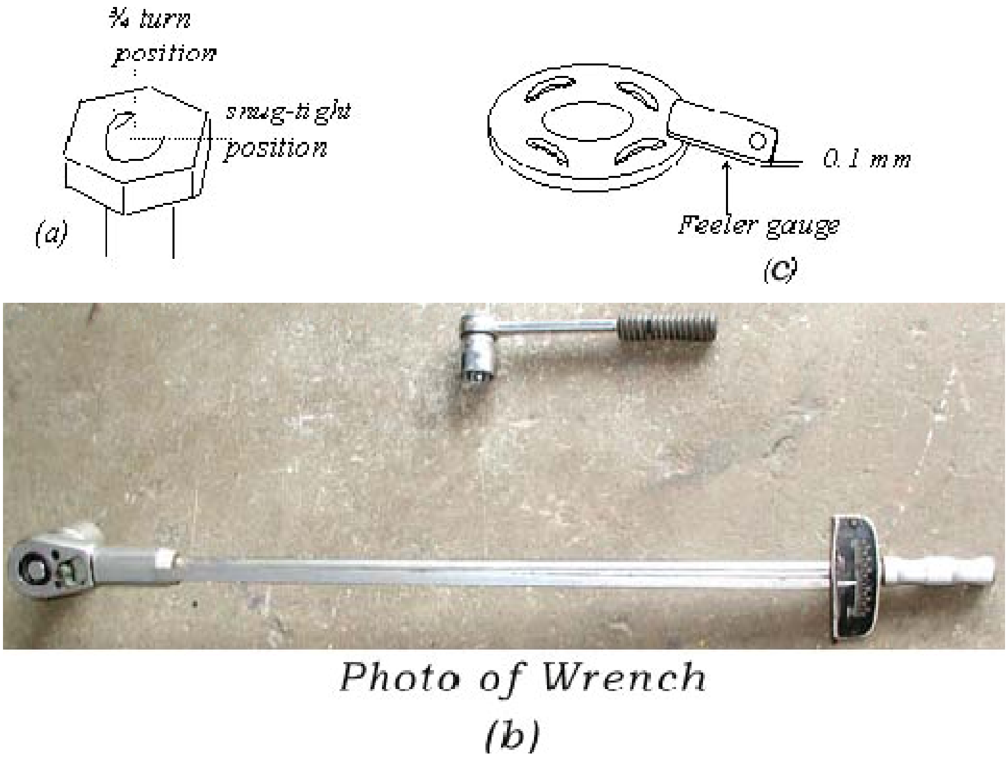

Figure 7 Tightening of HSFG bolts

5.1.3.High Strength Friction Grip bolts (HSFG)

High Strength Friction Grip bolts (HSFG) provide extremely efficient connections and perform well under fluctuating/fatigue load conditions. These bolts should be tightened to their proof loads and require hardened washers to distribute the load under the bolt heads. The washers are usually tapered when used on rolled steel sections. The tension in the bolt ensures that no slip takes place under working conditions and so the load transmission from plate to the bolt is through friction and not by bearing. However, under ultimate load, the friction may be overcome leading to a slip and so bearing will govern the design. HSFG bolts are made from quenched and tempered alloy steels with grades from 8.8 to 10.9. The most common are the so-called, general grade of 8.8 and have medium carbon content, which makes them less ductile. The 10.9 grade have a much higher tensile strength, but lower ductility and the margin between the 0.2% yield strength and the ultimate strength is also lower. The tightening of HSFG bolts can be done by either of the following methods (IS 4000):

- Turn-of-nut tightening method: In this method the bolts are first made snug tight and then turned by specific amounts (usually either half or three-fourth turns) to induce tension equal to the proof load (Figure 7(a)).
- Calibrated wrench tightening method: In this method the bolts are tightened by a wrench (Figure 7(b)) calibrated to produce the required tension.

- Alternate design bolt installation: In this method special bolts are used which indicate the bolt tension. Presently such bolts are not available in India.
- Direct tension indicator method: In this method special washers with protrusions are used (Figure 7(c)). As the bolt is tightened, these protrusions are compressed and the gap produced by them gets reduced in proportion to the load. This gap is measured by means of a feeler gauge, consisting of small bits of steel plates of varying thickness, which can be inserted into the gap.

Since HSFG bolts under working loads, do not rely on resistance from bearing, holes larger than usual can be provided to ease erection and take care of lack-of-fit. Typical hole types that can be used are standard, extra large and short or long slotted. These are shown in Figure 8. However the type of hole will govern the strength of the connection. Holes must also satisfy pitch and edge/end distance criteria (Cl.10.2 of IS 800:2007). A minimum pitch is usually specified for accommodating the spanner and to limit adverse interaction between the bearing stresses on neighbouring bolts. A maximum pitch criterion takes care of buckling of the plies under compressive loads.

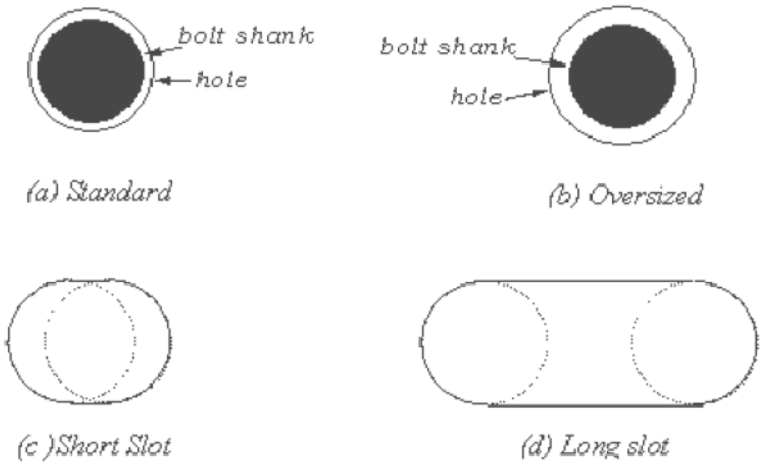

Figure 8 Hole types for HSFG bolts

5.2. CLASSIFICATIONS OF BOLT CONNECTIONS

Bolt connections are generally classified in the following ways

5.2.1. Based on the resultant force transferred.

Bolt connections can be classified in to the following heads based on how the resultant force transferred at the joint. (a) Concentric connection – if the force transferred passes through the CG of the connection. Eg. Axially loaded compression and tension members. (b) Eccentric connection – if the load is not passing through the CG of the connection. Eg. Bracket connection and seat connection. (c) Moment resisting connection – when the joints are subjected to moments. Eg. Beam to column connection in framed construction.

Ideal concentric connections should have only one bolt passing through all the members meeting at a joint as shown in Figure 9(a). However, in practice, this is not usually possible and so it is only ensured that the centroidal axes of the members meet at one point as shown in Figure 9(b).

The Moment connections are more complex to analyse compared to the above two types and are shown in Figure 10(a) and Figure 10(b). The connection in Figure 10(a) is also known as bracket connection and the resistance is only through shear in the bolts. The connection shown in Figure 10(b) is often found in moment resisting frames where the beam moment is transferred to the column. The connection is also used at the base of the column where a base plate is connected to the foundation by means of anchor bolts. In this connection, the bolts are subjected to a combination of shear and axial tension.

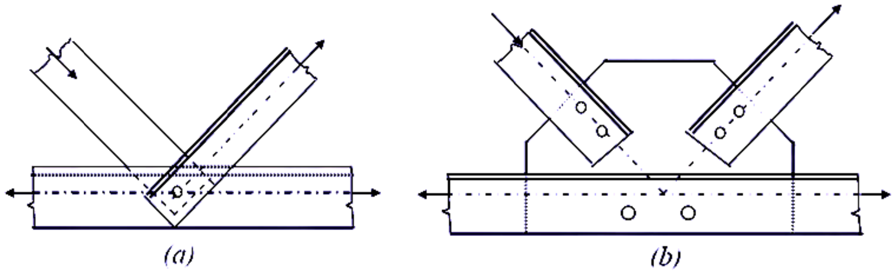

Figure 9 Concentric Connection

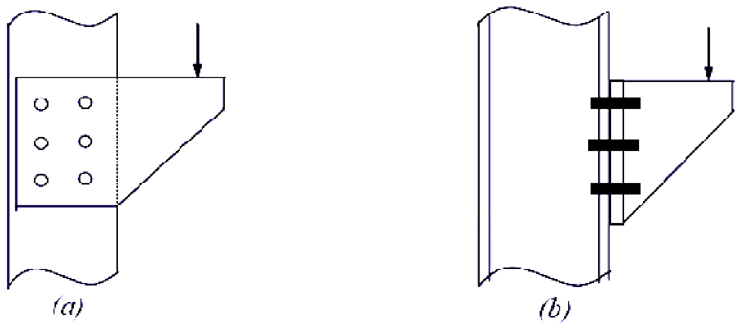

Figure 10 Moment Connection

5.2.2.Based on the type of force

Bolt connections can be classified in to the following based on the type of force transferred: (a) Shear connection – when the load transfer is through shear. Eg. Lap joint and tension joint; (b) Tension joints – when load is transferred by tension in the bolts. Eg. Hanger connection; (c) Combined shears and tension connections – when load is transferred through the combinations of shear and tension. Eg. Inclined members connected to columns or beams.

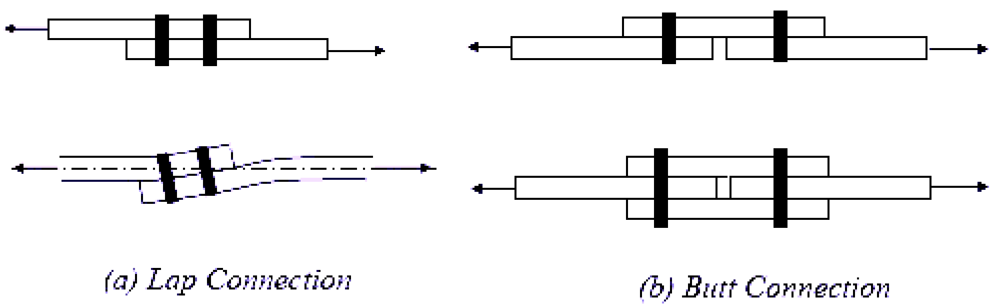

Figure 11 Shear Connection

Typical shear connections occur as a lap or a butt joint used in the tension members as shown in Figure 11. While the lap joint has a tendency to bend so that the forces tend to become collinear, the butt joint requires cover plates. Since the load acts in the plane of the plates, the load transmission at the joint will ultimately be through shearing forces in the bolts.

In the case of lap joint or a single cover plate butt joint, there is only one shearing plane, and so the bolts are said to be in single shear. In the case of double cover butt joint, there are two shearing planes and so the bolts will be in double shear. It should be noted that the single cover type butt joint is nothing but lap joints in series and also bends so that the centre of the cover plate becomes collinear with the forces. In the of single cover plate (lap) joint, the thickness of the cover plate is chosen to be equal to or greater than the connected plates. While in double cover plate (butt) joint, the combined thickness of the cover plates should be equal to or greater than the connected plates.

A hanger connection is shown in Figure 12(a). In this connection, load transmission is by pure tension in the bolts. In the connection shown in Figure 12(b), the bolts are subjected to both tension and shear.

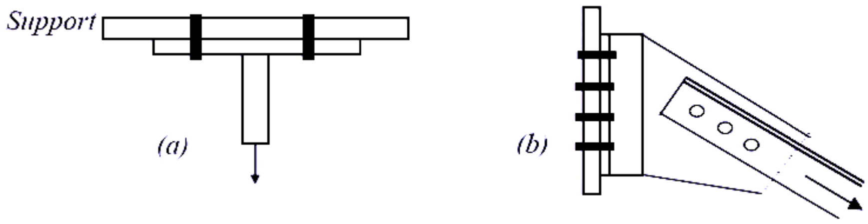

Figure 12 Tension connection

5.2.3. On the basis of force transfer mechanism

Bolt connections are classified into the following based on the way in which load is transferred from one member to another connected in the joint. (a) Bearing type – bolts bears against the holes to transfer the load from one member to another. Eg. Slip type connection. (b) Friction type – when the force is transferred by friction between the plates due to tensioning of bolts. Eg. Slip-critical connects.

5.3. FAILURE OF BOLTED CONNECTIONS

Failure of bolted connection can be classified broadly in to two: (1) failure of the bolt and (2) failure of connecting parts. Bolted joints may fail in any of the following six ways.

- Shear failure of bolts.
- Bearing failure of bolts.
- Bearing failure of plates.
- Tension failure of bolts.
- Tension or tearing failure of plates.
- Block shear failure.

5.4. SPECIFICATION OF BOLTED JOINTS

5.4.1. Diameter of the bolts

In general, a connection with few larger diameter bolts are economical than a connection with smaller diameter bolts. This is basically because as the number of bolts increases, the work associated with drilling of holes and installations of bolts will also increases. Larger diameter bolts are particularly advantages in the case of connections where bolt shear governs the design because the shear strength of bolts varies as the square of bolt diameter.

5.4.2. Pitch

Spacing of bolt holes in a joint is defined by three parameters namely; pitch, edge distance and end distance. Pitch is the distance between the centers of two consecutive bolts measured along a row of bolts. When the bolts are placed staggered, then the pitch is known as staggered pitch.

5.4.2.1. Minimum pitch

This is to prevent the bearing failure of the plate between the two bolts, to permit the efficiency in installation of bolts by providing sufficient space for tightening of bolts, to prevent overlapping of washers and to provide adequate resistance to tear out of bolts.

Minimum Spacing is specified by IS 800:2007 in cl. 10.2.2. The distance between centre of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.

5.4.2.2. Maximum Spacing

This is to ensure a compact joint reducing the length of connection and to ensure uniform stress in bolts

Cl. 10.2.3.1 gives the maximum distance between the centres of any two adjacent fasteners which shall not exceed 32t or 300 mm, whichever is less, where t is the thickness of thinner connected plate. Cl.10.2.3.2 gives the distance between the centres of two adjacent fasteners (pitch) in a line lying in the direction of stress, which shall not exceed 16t or 200 mm, whichever is less, in tension members and 12t or 200 mm, whichever is less, in compression members; where t is the thickness of the thinner plate. In the case of compression members wherein forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the fasteners for a distance equal to 1.5 times the width of the member from the butting faces.

Cl. 10.2.3.3 specifies the distance between the centres of any two consecutive fasteners in a line adjacent and parallel to an edge of an outside plate shall not exceed 100 mm plus 4t or 200 mm, whichever is less, in compression and tension members; where t is the thickness of the thinner outside plate. Cl. 10.2.3.4 deals with the staggered fasteners. When fasteners are staggered at equal intervals and the gauge does not exceed 75 mm, the spacing specified in 10.2.3.2 and 10.2.3.3 between centres of fasteners may be increased by 50 percent, subject to the maximum spacing specified in 10.2.3.1.

5.4.2.3. Edge and End Distances

Cl. 10.2.4.1 specifies the way to compute the edge distances in various cases. The edge distance is the distance at right angles to the direction of stress from the centre of a hole to the adjacent edge. The end distance is the distance in the direction of stress from the centre of a hole to the end of the element. In slotted holes, the edge and end distances should be measured from the edge or end of the material to the centre of its end radius or the centre line of the slot, whichever is smaller. In oversize holes, the edge and end distances should be taken as the distance from the relevant edge/end plus half the diameter of the standard clearance hole corresponding to the fastener, less the nominal diameter of the oversize hole.

Cl. 10.2.4.2 gives the minimum edge distance. The minimum edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand-flame cut edges; and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edges.

Cl. 10.2.4.3 specifies the permissible maximum edge distance. The maximum edge distance to the nearest line of fasteners from an edge of any un-stiffened part should not exceed 12tε, where $\varepsilon = \sqrt{(250/f_y)}$ and t is the thickness of the thinner outer plate. This would not apply to fasteners interconnecting the components of back to back tension members. Where the members are exposed to corrosive influences, the maximum edge distance shall not exceed 40 mm plus 4t, where t is the thickness of thinner connected plate.

5.4.2.4. Tacking Fasteners

In case of members covered under 10.2.4.3, when the maximum distance between centres of two adjacent fasteners as specified in 10.2.4.3 is exceeded, tacking fasteners not subjected to calculated stress shall be used. Tacking fasteners shall have spacing in a line not exceeding 32 times the thickness of the thinner outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the spacing in line shall not exceed 16 times the thickness of the thinner outside plate or 200 mm, whichever is less. In both cases, the distance between the lines of fasteners shall not be greater than the respective pitches. All the requirements specified in 10.2.5.2 shall generally apply to compression members, subject to the stipulations in Section 7 affecting the design and construction of compression members. In tension members (see Section 6) composed of two flats, angles, channels or tees in contact back to back or separated back to back by a distance not exceeding the aggregate thickness of the connected parts, tacking fasteners with solid distance pieces shall be provided at a spacing in line not exceeding 1000 mm. For compression members covered in Section 7, tacking fasteners in a line shall be spaced at a distance not exceeding 600 mm. These specifications are outlined in Cl. 10.2.5 of IS 800: 2007.

5.4.2.5. Combination of fasteners

When different fasteners are used to carry shear loads or when welding and other types of fasteners are combined together, then one form of the fasteners should be designed to take up the total load. Nevertheless, if we use HSFG bolts along with welds and the bolts are tightened after welding is completed, then such bolts can be used to share the load with the welds.

5.5. SHEAR CONNECTIONS WITH BEARING TYPE BOLTS

In this section the force transfer mechanisms of bearing and friction type of bolted connections are described.

5.5.1. Force transfer of bearing type bolts

Figure 13 shows the free body diagram of the shear force transfer in bearing type of bolted connection. It is seen that tension in one plate is equilibrated by the bearing stress between the bolt and the hole in the plate. Since there is a clearance between the bolt and the hole in which it is fitted, the bearing stress is mobilised only after the plates slip relative to one another and start bearing on the bolt .The section x-x in the bolt is critical section for shear. Since it is a lap joint there is only one critical section in shear (single shear) in the bolt .In the case of butt splices there would be two critical sections in the bolt in shear (double shear), corresponding to the two cover plates.

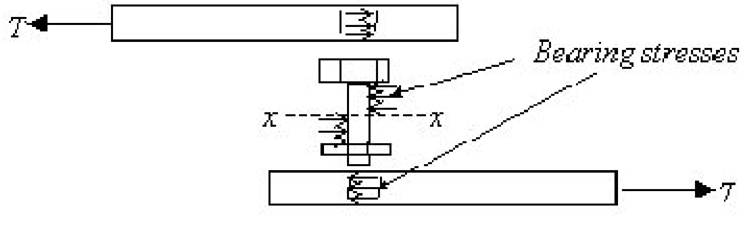

Figure 13 Bearing connection

5.5.2. Design shear strength of bearing type bolts

The failure of connections with bearing bolts in shear involves either bolt failure or the failure of the connected plates. In this section, the failure modes are described along with the codal provisions for design and detailing shear connections. In connections made with bearing type of bolts, the behaviour is linear until i) yielding takes place at the net section of the plate under combined tension and flexure or ii) shearing takes place at the bolt shear plane or iii) failure of bolt takes place in bearing, iv) failure of plate takes place in bearing and v) block shear failure occurs. Of these, i) and v) will be discussed in the chapter on tension members. The remaining three are described below.

5.5.2.1. Shearing of bolts

The shearing of bolts can take place in the threaded portion of the bolt and so the area at the root of the threads, also called the tensile stress area A_t, is taken as the shear area A_s. Since threads can occur in the shear plane, the area A_e for resisting shear should normally be taken as the net tensile stress area, A_n, of the bolts. The shear area is specified in the code and is usually about 0.8 times the shank area. However, if it is ensured that the threads will not lie in the shear plane then the full area can be taken as the shear area. A bolt subjected to a factored shear force (V_{sb}) shall satisfy $V_{sb} \leq V_{db}$ as per cl. 10.3.2 of IS 800:2007, where $V_{db} = V_{dsb} = V_{nsb}/\gamma_{mb}$ as given by cl. 10.3.3 of the code. Here V_{nsb} = nominal shear capacity of a bolt, calculated by $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$in which f_u = ultimate tensile strength of a bolt; n_n = number of shear planes with threads intercepting the shear plane; n, = number of shear planes without threads intercepting the shear plane; A_{sb} = nominal plain shank area of the bolt; and A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread as given in Table 5 and γ_{mb} = 1.25. For bolts in single shear, either n_n or n_s is one and the other is zero. For bolts in double shear the sum of n_n and n_s is two.

Table 5 Tensile area of ordinary bolts (Grade 4.6)

Bolt size, d (mm)	12	16	20	22	24	27	30	36
Tensile stress area (mm^2)	84.3	157	245	303	353	459	561	817

5.5.2.2. Bearing failure

If the connected plates are made of high strength steel then failure of bolt can take place by bearing of the plates on the bolts. If the plate material is weaker than the bolt material, then failure will occur by bearing of the bolt on the plate and the hole will elongate. The beating area is given by the nominal diameter of the bolt times the combined thickness of the plates bearing in any direction. − A bolt bearing on any plate subjected to a factored shear force (V_{sb}) shall satisfy $V_{sb} \leq V_{db}$ as per cl. 10.3.2 of IS 800:2007, where $V_{db} = V_{dpb} = V_{npb}/\gamma_{mb}$ as given by cl. 10.3.4 of the code where, γ_{mb} = 1.25 and V_{npb} = bearing strength of a bolt, calculated as $V_{npb} = 2.5 k_b d t f_u$ where f_u = smaller of the ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, d = nominal diameter of the bolt, t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction and k_b is smaller of $e/3d_0$, $p/3d_0$-0.25, f_{ub}/f_u, 1.0 where e, p = end and pitch distances of the fastener along bearing direction; d_0 = diameter of the hole; f_{ub}, f_u = Ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively.

The underlying assumption behind the design of bolted connections, namely that all bolts carry equal load is not true in some cases. In long joints, the bolts farther away from the centre of the joint will carry more load than the bolts located close to the centre. Therefore, for joints having more than two bolts on either side of the building connection with the distance between the first and the last bolt exceeding 15d in the direction of load, the nominal shear capacity Vns, shall be reduced by the factor, β lj, given by (Cl.10.3.2.1) $\beta_{lj} = 1.075 - l_j / (200\ d)$ but $0.75 < \beta_{lj} < 1.0$ where, d= nominal diameter of the bolt Similarly, if the grip length exceeds five times the nominal diameter, the strength is reduced as specified in IS 800. In multi-bolt connections, due to hole mismatch, all the bolts may not carry the same load. However, under ultimate load, due to high bearing ductility of the plates considerable redistribution of the load is possible and so the assumption that all bolts carry equal load may be considered valid.

5.6. SHEAR CONNECTIONS WITH HSFG BOLTS

5.6.1. Force transfer of HSFG bolts

The free body diagram of an HSFG connection is shown in Figure 14. It can be seen that the pretension in the bolt causes clamping forces between the plates even before the external load is applied. When the external load is applied, the tendency of two plates to slip against one another is resisted by the friction between the plates. The frictional resistance is equal to the coefficient of friction multiplied by the normal clamping force between the plates. Until the externally applied force exceeds this frictional resistance the relative slip between the plates is prevented. The HSFG connections are designed such that under service load the force does not exceed the frictional resistance so that the relative slip is avoided during service. When the external force exceeds the frictional resistance the plates slip until the bolts come into contact with the plate and start bearing against the hole. Beyond this point the external force is resisted by the combined action of the frictional resistance and the bearing resistance.

5.6.2. Design shear strength of HSFG bolts

HSFG bolts will come into bearing only after slip takes place. Therefore if slip is critical (i.e. if slip cannot be allowed) then one has to calculate the slip resistance, which will govern the design. However, if slip is not critical, and limit state method is used then bearing failure can occur at the Limit State of collapse and needs to be checked. Even in the Limit State method, since HSFG bolts are designed to withstand working loads without slipping, the slip resistance needs to be checked anyway as a Serviceability Limit State.

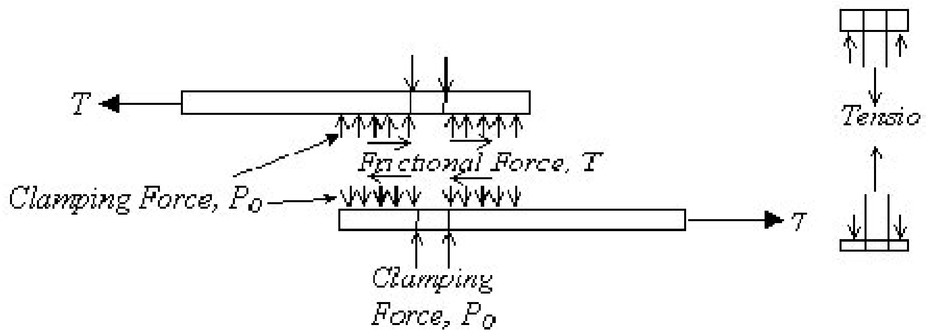

Figure 14 Friction Connection

5.6.2.1. Slip Resistance

Design for friction type bolting in which slip is required to be limited, a bolt subjected only to a factored design shear force, V_{sf}, in the interface of connections shall satisfy the following (Cl.10.4.3): $V_{sf} \leq V_{dsf}$ where $V_{dsf} = V_{nsf} / \gamma_{mf}$ in which γ_{mf} = 1.10 if slip resistance is designed at service load and 1.25 if slip resistance is designed at ultimate load. V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows: $V_{nsf} = \mu_f\, n_e\, K_h\, F_o$. where, μ_f = coefficient of friction (slip factor) as specified in Table 20 of the code ($\mu_f < 0.55$), n_e =number of effective interfaces offering frictional resistance to slip, K_h = 1.0 for fasteners in clearance holes = 0.85 for fasteners in oversized and short slotted holes, and for fasteners in long slotted holes loaded perpendicular to the slot and 0.7 for fasteners in long slotted holes loaded parallel to the slot, F_o= minimum bolt tension (proof load) at installation and may be taken as 0.8 A_{sb} f_o where A_{sb}= shank area of the bolt in tension and f_o= proof stress (= 0.70 f_{ub}).

V_{ns} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

5.6.2.2. Bearing strength

The design for friction type bolting, in which bearing stress in the ultimate limit state is required to be limited, (V_{ub} = factored load bearing force) shall satisfy $V_{bf} < V_{nbf} / \gamma_{mf}$ (Cl.10.4.4 of the code), where γ_{mf}= 1.25, V_{nbf} = bearing capacity of a bolt, for friction type connection, given by $V_{nbf} = 2.2dtf_{up} \leq 3dtf_{yp}$ where f_{up}= ultimate tensile stress of the plate, f_{yp} = tensile yield stress of the plate, d = nominal diameter of the bolt t = summation of thicknesses of all the connected plates experiencing bearing stress in the same direction. The block shear resistance of the edge distance due to bearing force shall also be checked.

6. WELDED CONNECCTION

6.1. INTRODUCTION

When two members are connected by means of welds, such a connection is known as welded connection. Welding offers an opportunity to the designer to achieve a more efficient use of the materials. Earlier designers considered welds as less fatigue resistant. It was believed that attaining good welds at site is impossible. Now a day, with the advances in the field of non destructive testing methods (NDT), testing and quality control of welds became easier. This gives the designers enough courage to explore the possibilities and capabilities of welded connections. Speedy construction is facilitated by using welded connections. Weight of welded connections is relatively low and hence cuts cost of construction. Since there is no reduction of holes the gross cross section is effective in carrying loads.

6.2. TYPES OF WELDED CONNECTIONS

The basic types of welded joints can be classified depending on the types of welds, position of welds and type of joint.

6.2.1. Based on the type of weld

Based on type of weld, welds can be classified in to fillet weld, groove weld (or butt weld), plug weld, slot weld, spot weld etc. Various types of welds are shown in Figure 15.

6.2.1.1. Groove welds (butt welds

Groove welds (butt welds) and fillet welds are provided when the members to be joined are lined up. Groove welds are costlier since it requires edge preparation. Groove welds can be employed safely in heavily stressed members. Square butt welds are provided up to a plate thickness of 8mm only. Various types of butt welds are shown in Figure 16.

6.2.1.2. Fillet welds

Fillet welds are provided when two members to be jointed are in different planes. Since this situation occurs more frequently, fillet welds are more common than butt welds. Fillet welds are easier to make as it requires less surface preparation. Nevertheless, they are not as strong as the groove welds and cause concentration of stress. Fillet welds are preferred in lightly stressed members where stiffness rather than strength governs the design. The various types of fillet welds are shown in Figure 17.

6.2.1.3. Slot and plug welds

Slot and plug welds are used to supplement fillet welds where the required length of fillet weld cannot be achieved.

6.2.2. Based on the position of weld

Based on the position of weld, welds can be classified in to flat weld, horizontal weld, vertical weld, overhead well etc.

6.2.3. Based on the type of joints

Based on the type of joints, welds can be classified in to butt welded joints, lap welded joints, tee welded joints and corner welded joints.

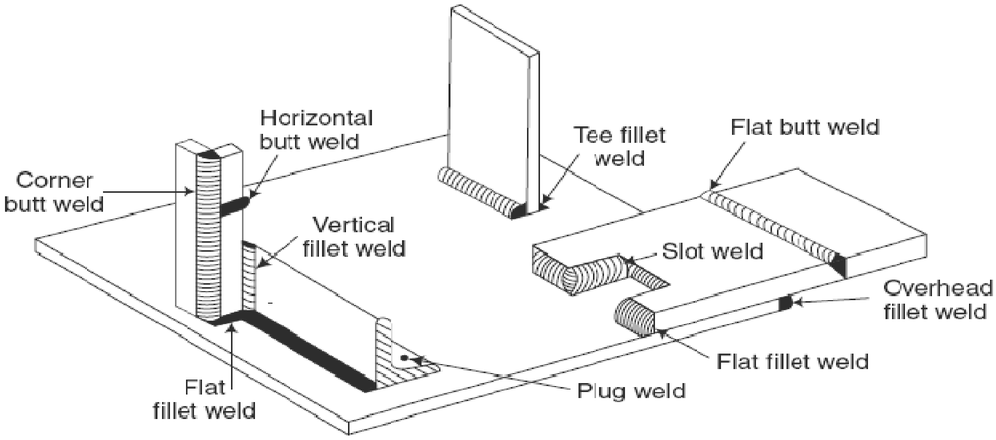

Figure 15 Type of joints

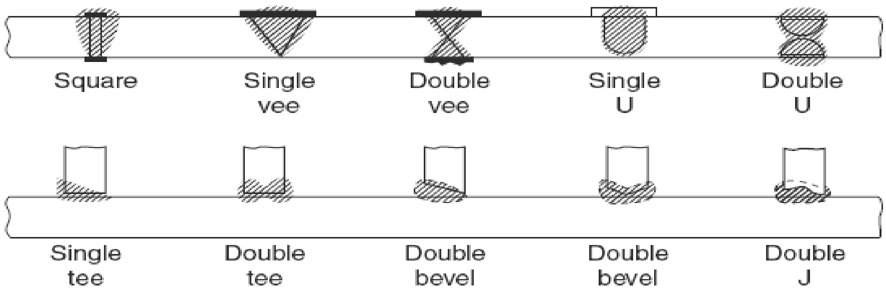

Figure 16 Types of Groove Welds

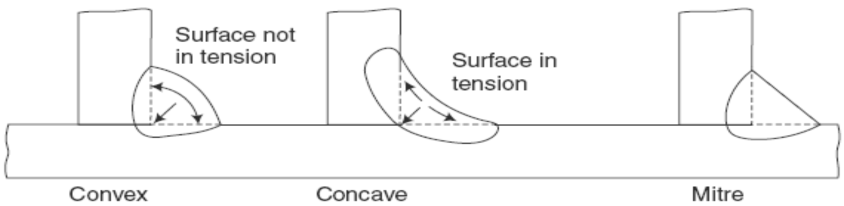

Figure 17 Types of Fillet Welds

6.3. ADVANTAGES AND DISADVANTAGES OF WELDED JOINTS

The following are the advantages of welded joints.

- Due to the absence of gusset plates and other connecters, the welds are usually lighter.
- Welding process is quicker as it requires no drilling of holes.
- Welding is more adaptable than other types of connections and can even be used in circular pipes.
- 100% efficiency can be achieved in welding where as the connection such as bolts can have a maximum efficiency of 70 – 80%.
- Noise produced during the welding process is relatively less.
- Welds usually have good aesthetic appearance.
- Welded joints are air tight and water tight and can be used for water tanks and gas tanks.
- Welded joints are rigid.
- Mismatch of holes will never happen in welded connection.
- Alternation of joints can easily be made in the case of welded connecetions.

However the welded connection is having the following disadvantages.

- Due to the uneven heating and cooling, members are likely to distort in the process of welding.
- Possibility of brittle fracture is more in the case of welded connections.
- Welded connections are more prone to failure due to fatigue stresses.
- The inspection of welded joints is difficult and expensive. It can only be done by employing NDT.
- Highly skilled persons are required for welding.
- Proper welding in field conditions is difficult.
- Welded joints are over rigid.

6.4. WELDING PROCESS

Welding consists of joining two steel sections by means of metallurgical bond between them by the application of pressure and/or fusion. The most commonly used welding process is fusion process. The bond is produced, in fusion process, by melting the surfaces to be joined and then allowing them to solidify in to a single joint. The most commonly used welding process is the arc welding process (Figure 18). In this process intense heat required (around 3600^0C) to melt the steel sections is produced by an electric arc. The tremendous heat at the tip of the electrode melts the base metal and the filler metal to form a pool of molten metal called crater which solidifies on cooling produce the joint required.

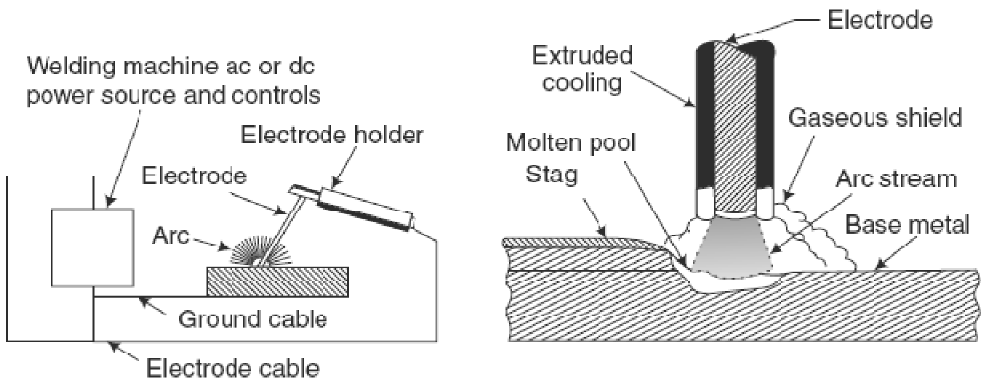

Figure 18 Welding Process

6.5. WELD DEFECTS

Defects in welding are inevitable in spite of using the good welding techniques, standard electrodes and preparation of joints. Some of the commonly observed welding defects (Figure 19) are detailed below.

6.5.1. Incomplete fusion

This happens when the base metal fails to completely fuse along with the weld metal. This can be caused by the rapid welding or by the presence of foreign material at the weld surface. See Figure 19(a).

6.5.2. Incomplete penetration

This type of failure occurs due to the failure of the weld metal to penetrate the complete depth of the joint. This is often observed in single V and bevel joints. This can also happen while using electrodes of larger size than is required. See Figure 19(b).

6.5.3. Porosity

Porosity occurs due to the voids or gas pockets entrapped in the welds while cooling. This results in stress concentration and reduced ductility of the joint. This is mainly due to careless use of backup plates, presence of moisture in the electrodes, presence of hydrogen in the electrodes and excessive current. See Figure 19(c).

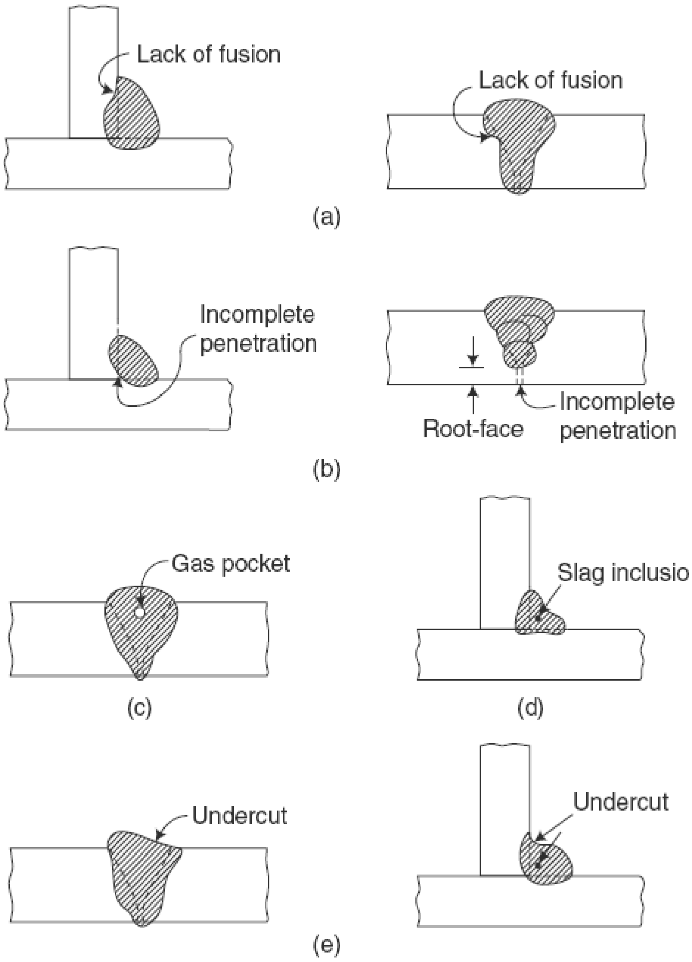

Figure 19 Weld Defects

6.5.4. Slag inclusion

Slag inclusions are metallic oxides and other solid compounds which are sometimes found as elongated or globular inclusions. The slag, being lighter than the molten material of weld, usually rise to the surface and can be removed after cooling of the weld. However if the weld is cooled rapidly possibilities are there to trap these in the weld. See Figure 19(d).

6.5.5. Undercutting

Undercutting is the local decrease in the thickness of parent metal at the weld toe. An undercut results in reduced section and acts like a notch. This can happen due to excessive current and/or long arc.

6.6. INSPECTION OF WELDS

As we have seen in section 6.5, defects in welds are inevitable. Nevertheless, the poor weld quality leads to collapse. Therefore, proper inspection and quality control of welded joints is essential. Some of the commonly employed methods are discussed here.

6.6.1.Magnetic particle method

When iron fillings spread over the welded joint is subjected to electric current, the forms patterns which can be utilised for interpret and locating surface cracks.

6.6.2.Dye penetration method

A dye is spread over the surface of weld in this method and then the surplus is removed. Then dye absorber is placed on the surface which oozes out the dye in the crack revealing the depth of surface cracks.

6.6.3.Ultrasonic method

Defects like flaws, blow holes, slag inclusion and porosity of the welds can be evaluated by this method. When ultrasonic waves are sent through the weld, these defects interfere with the wave propagation of the waves affecting its travel time. The defects can be interpreted by observing the travel time of the waves.

6.6.4.Radiography

In this method, X-rays and gamma rays are used to locate the defects. This is used in groove welds only.

6.7. ASSUMPTIONS IN THE ANALYSIS OF WELDED JOINTS

The following assumptions are made in the analysis of welded joints.

- Welds connecting various parts are homogeneous, isotropic and elastic.
- The parts connected by welds are rigid and their deformations are therefore neglected.
- Only stresses due to external loads are to be considered. Effects of residual stresses, stress concentrations and shape of welds are neglected.

6.8. ANALYSIS AND DESIGN OF BUTT(GROOVE) WELDS

Groove welds are usually provided when the member is subjected to tension or compression. Since there is no change in the section at the joint, this is the most suitable form of joint to transfer alternating stress. However when the welds are intended to take shear stress, careful consideration should also be made so that the shear stresses developed is taken care of. Figure 20 shows the typical cross section of a groove weld. Square groove welds are usually employed for sections of thickness up to 8mm. If sections with more than 8mm thickness, U, V, double U or double V butt welds are used.

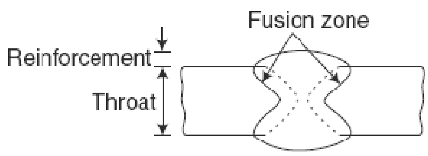

Figure 20 Groove Welds

6.8.1.Reinforcement in Groove Welds

Extra weld material which is deposited to make the throat thickness at least 10% more than the welded material is known as reinforcement (see Figure 20). This will increase the efficiency of the joint. Reinforcement will increase the strength of grove welds under static loading. In the case of dynamic loading, reinforcement may result in stress concentrations. So the reinforcement is usually dressed flush in the case of members subjected to dynamic loading. However subsequent removal of reinforcement is not considered as reducing the strength of joint. The reinforcement is ignored in calculating the strength. In any case the reinforcement should not exceed 3mm.

6.8.2.Size of Groove Welds

Size of welded joint is usually specified by throat dimension. This is also called effective throat thickness. Groove welds may be classified in to full penetration groove welds (Figure 21) or partial penetration groove welds (Figure 22). Complete penetration is difficult to achieve in the case of single U, V, J and bevel welds. However, this can be achieved by using backing strips as shown in Figure 21.

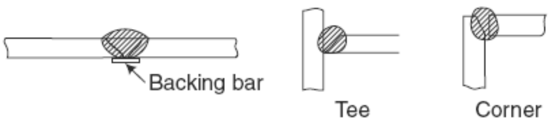

Figure 21 Complete Penetration Groove Welds

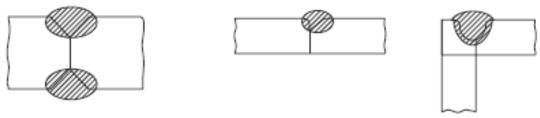

Figure 22 Partial Penetration Groove Welds

As per Cl.10.5.3.3 of IS800-2007, the effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements. However in the case where full penetration groove welds cannot be achieved, an effective throat thickness of 1/8th of the thickness of thinner member is used. But for calculating the strength of the connection, a throat thickness of 5/6th of the thinner member is usually assumed.

6.8.3.Effective area of Groove Welds

Effective area of weld is obtained as the product of effective length,L_w of weld and effective thickness (throat thickness), t_t of weld. As per Cl.10.5.4.2 of IS800-2007, the effective length of butt weld shall be taken as the length of the continuous full size butt weld, but not less than four times the size of the weld.

6.8.4.Design strength of Groove welds

Cl.10.5.7.1.2 of IS800-200 deals with the strength of Butt welds. As detailed in this clause of the code, Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal. Hence the following equations may be used for the design of butt welds.

The design strength of groove weld in tension and compression is given by $T_{dw} = T_{nw}/\gamma_{mw}$ where T_{nw} is the nominal strength of the weld given by $T_{nw} = f_y L_w t_t$ and $\gamma_{mw}$is partial safety factor of the material of the weld given by Table 5 of IS800-2007. f_y is the yield stress of the material, L_w is the effective length of the weld and t_e is the throat thickness.

The design strength of groove weld in shear may be calculated by the expression $T_{dw} = T_{nw}/\gamma_{mw}$ where $\gamma_{mw}$is partial safety factor of the material of the weld given by Table 5 of IS800-

2007 and T_{nw} is the nominal strength of the weld given by $T_{nw} = \frac{f_u}{\sqrt{3}} L_w t_t$ where f_u is the ultimate stress of the material, L_w is the effective length of the weld and t_e is the throat thickness.

6.8.5.Butt welds subjected to combination of stresses

Cl.10.5.10.2.2 deals with the case where fillet welds are subjected to combined bearing, bending and shear. Where bearing stress, f_{br}, is combined with bending (tensile or compressive), f_b and shear stresses, q under the most unfavourable conditions of loading in butt welds, the equivalent stress, f_e, obtained from the formula $f_e = \sqrt{{f_b}^2 + {f_{br}}^2 + f_b f_{br} + 3q^2}$. However the value of f_e thus obtained shall not exceed the values allowed for the parent metal:

6.9. ANALYSIS AND DESIGN OF FILLET WELDS

Fillet welds are provided for connecting two members which are overlapping each other. Shear stresses are usually the type of stress in the case of fillet welded connection. Direct stresses to which the connections are subjected to are usually of lesser importance. Concave fillet weld was favoured because it offers a smoother path for the flow of stress. But concave fillet weld up on cooling shrinks and cause tension to the surface which may cause cracks in the joint. On the other hand, shrinkage of weld will cause compression in the case of convex fillet weld. Concave fillet welds are more suitable under alternating stresses.

6.9.1.Size of Fillet weld

Cl.10.5.2.1 of IS800-2007 specifies the size of normal fillets shall be taken as the minimum weld leg size. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm. Figure 23 shows the leg length of fillet weld for various cases.

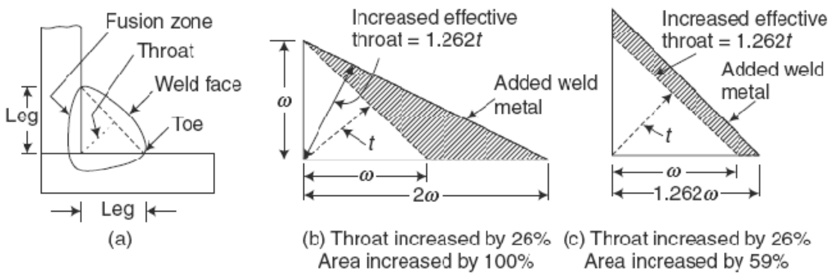

Figure 23 Leg Length of Fillet Weld

Cl.10.5.2.3 of IS800-2007 restricts minimum size of fillet welds to be 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 21 of IS800-2007, to avoid the risk of cracking in the absence of preheating. Table 21 if IS800-2007 is reproduced for ready reference as Table 6.

As per Cl.10.5.8.3 of IS800-2007, where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (see Fig. 18 of IS800:2007). The figure is reproduced here as Figure 25.

Cl.10.5.8.5 of IS800-2007 specifies for end fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5t, where t is the thickness of the part, as shown in Fig. 19 of IS800-2007. The difference in thickness of the welds shall be negotiated at a uniform slope. Fig. 19 of IS800-2007 is reproduced here as Figure 26.

Table 6 Minimum size of first run or of a single run fillet weld

Sl. No.	Thickness of thicker part (mm)		Minimum size (mm)
	Over	Up to and including	
1	-	10	3
2	10	20	5
3	20	32	6
4	32	50	8 of 1st run, 10 for minimum weld size

Cl.10.5.8 of IS800-2007 specifies the maximum size of fillet weld applied to the edge of a plate or section. Cl.10.5.8.1 specifies where a fillet weld is applied to the square edge; the specified size of the weld should generally beat least 1.5 mm less than the edge thickness (see Fig. 17A of IS800:2007). This figure is reproduced here as Figure 24(a).

Cl.10.5.8.2 of IS800-2007 gives the specification where the fillet weld is applied to the rounded toe of a rolled section; the specified size of the weld should generally not exceed 3/4 of the thickness of the section at the toe (see Fig. 17B of IS800:2007). This figure is reproduced here as Figure 24(b).

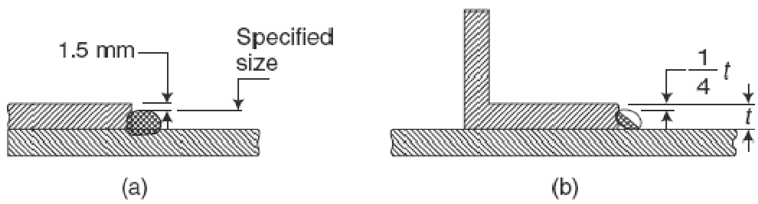

Figure 24 Fillet Welds on square edge of plate or round toe of rolled section

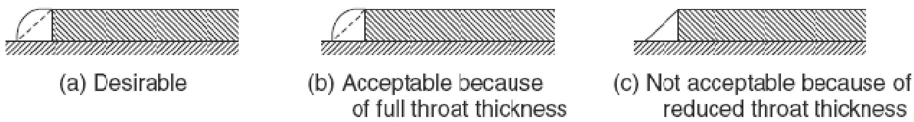

Figure 25 Full size Fillet Weld applied to the edge of a Plate or a Section

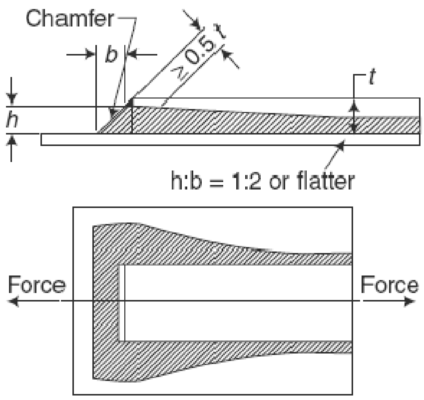

Figure 26 End fillet weld normal to the direction of force

6.9.2. Effective throat thickness of fillet welds

Cl.10.5.3 of IS800-2007 gives the specification for Effective Throat Thickness. Cl.10.5.3.1 specifies the effective throat thickness of a fillet weld not to be less than 3 mm and shall generally not exceed 0.7t, or 1.0t under special circumstances, where t is the thickness of the thinner plate of elements being welded. Further, Cl.10.5.3.2 of the code specifies the effective throat thickness. For the purpose of stress calculation in fillet welds joining faces inclined to each other, the effective throat thickness shall be taken as K times the fillet size, where K is a constant, depending upon the angle between fusion faces, as given in Table 22 of IS800-2007. Table 22 of IS800-2007 is reproduced here as Table 7 for ready reference.

Table 7 Values of K for different angles

Angle between fusion faces in degrees	60-90	91-100	101-106	107-113	114-120
Constant, K	0.70	0.65	0.60	0.55	0.50

6.9.3. Effective length of fillet welds

Effective Length of Weld is considered in accordance with Cl.10.5.4 of IS800-2007. As per Cl.10.5.4.1, the effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld. So the effective length of the weld is considered as actual length minus twice the weld size. As per Cl.10.5.1.1, Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

6.9.4. Effective area of fillet welds

The effective area of fillet welds is obtained as the product of effective throat thickness and effective length.

6.9.5. Design strength of fillet welds

Cl.10.5.7.1.1 of IS800-200 deals with the strength of fillet welds. As detailed in this clause of the code, fillet welds shall be designed based on the throat area (effective area). The code specifies the design stress in the weld $f_{wd} = f_{wn}/\gamma_{mw}$ where and $\gamma_{mw}$is partial safety factor of the material of the weld given by Table 5 of IS800-2007 and f_{wn} is the nominal stress of the material given by$f_{wn} = f_u/\sqrt{3}$ where f_u is the ultimate stress of the material. Hence the design capacity of fillet welds $T_{dw} = L_w t_t \frac{f_u}{\sqrt{3}\gamma_{mw}}$ where $L_w$the effective length of the weld is and t_t is the throat thickness.

6.9.6. Long joints

When the length of the welded joint, l_j of a splice or end connection in a compression or tension element is greater than 150 t_t the design capacity of weld, f_{wd} shall be reduced by the factor $\beta = 1.2 - \frac{0.2 l_j}{150 t_t} \leq 1.0$.

6.9.7. Fillet weld subjected to individual stresses

Cl.10.5.9 of IS800:2007 specifies the Stresses Due to Individual Forces. When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by $f_a or\ q = \frac{P}{t_t l_w}$.

6.9.8. Fillet welds subjected to combination of stresses

As per Cl.10.5.10.1.1, when subjected to a combination of normal and shear stress, the equivalent stress f_e shall satisfy the condition $f_e = \sqrt{{f_a}^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$

6.10. FAILURE OF WELDS

Failure of welds of various types may be classified based on the types of welds.

6.10.1. Butt weld

If the butt welds are reinforced properly, it is unlikely for the failure to obtain in the weld. The fracture normally occurs at some distance from the connection. However if the weld is ground flush with the surface of the plate, the position of the fracture depends on the relative strength of plate and weld. If the tensile strength and yield point of the weld metal are higher than that of the elements, the failure takes place away from the weld. Failure in the weld junction is unusual. Welds stiffened with reinforcements will hinder the failure of the joint under bending. If there is a wide difference in the properties of the materials on joint and member, the failure may occur at the junction.

6.10.2. End fillet weld

The plane of failure in the case of a normal convex weld is along the diagonal from the root of the weld. The position of the plane of failure depends on the relative magnitude of tensile and shear stresses. If the fillet weld is having unequal legs, the plane of failure will be nearer to the leg having smaller length. The failure of end fillet welds occurs abruptly with very small deformations.

6.10.3. Side fillet weld

In a side convex weld subjected to shear stress along the weld, failure occurs down the throat of the weld. Failure is initiated at the toe of the weld and progress with rotation of the plane associated with considerable deformations. Failure of the plates before the final failure is also usual.

7. TENSION MEMBERS

7.1. GENERAL

Structural members subjected to tensile forces are known as tension members. IS 800:2007 describes Tension members in Cl. 6.1 as linear members in which axial forces act to cause elongation (stretch). The connections are made in such a way that the eccentricity of loading and bending stresses are avoided. Even though bending stresses may develop due to the self weight of the member, these stresses are very small and often neglected. However if bending stress is developed due to the eccentricity of connections or due to the incapability of the connections to create the above considerations, the additional stresses need to be accounted for as per specifications.

7.2. TYPES OF TENSION MEMBERS

Tension members can be classified into the following heads.

7.2.1. Wires, strands and cables

A strand consists of individual wires wound helically around a central core. A wire rope consists of a number of strands wound helically around a core. Cables are group of individual strands wound helically around a core.

7.2.2. Bars and rods

Bars and rods are straight member which have considerable cross section. These can be either circular square or rectangular in cross section. Unlike cables, wires and strands, they are used individually as structural members. They are often bolted to the other members by means of threaded ends.

7.2.3. Plates and flat bars.

They are very commonly used. Plates are members where one dimension (thickness) is very small in comparison with the other dimensions. Flat bars are usually rectangular in cross section and the cross sectional dimensions are comparable where as the length is very large in comparison with the cross sectional dimension.

7.2.4. Structural sections

Standard structural steel sections like angles are also used as tension members. These are available in standard dimensions and length.

7.2.5. Built up sections

Built-up sections are also used very frequently in construction. These are formed by using a combination of more than one standard sections and/or plates.

7.3. FAILURE MODES FOR TENSION MEMBERS

Tension members, as described in IS 800:2007, can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions also. The design objective is basically to check these three failure modes under prescribed loads or combination of loads.

7.4. DESIGN CONSIDERATION OF TENSION MEMBERS

The factored design tension, T, in the members shall satisfy $T \leq T_d$ as detailed in Cl. 6.1 of IS 800:2007 where T_d = design strength of the member. The design strength of a member under axial tension, T_d, is the lowest of the design strength due to yielding of gross section (T_{dg}), rupture strength of critical section (T_{dn}), and block shear strength (T_{db}), given in clauses 6.2, 6.3 and 6.4 of IS 800:2007 respectively.

7.4.1. Design Strength due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, as per IS 800:2007, is given by $T_{dg} = \mathrm{A_g f_y}/\gamma_{\mathrm{m0}}$ where f_y is the yield stress of the material, A_g is the gross area of cross-section, and γ_{m0} is the partial safety factor for failure in tension by yielding as given in Table 5 of IS 800:2007. This table is reproduced here as Table 8.

Table 8 Partial Safety Factors for Materials γ_m

Sl No	Definition	Partial safety factor	
1	Resistance, governed by yielding, γ_{m0}	1.10	
2	Resistance of member to buckling, γ_{m0}	1.10	
3	Resistance, governed by ultimate stress, γ_{m1}	1.25	
4	Resistance of connection	Shop Fabrications	Field Fabrications
4(a)	Bolts-Friction Type, γ_{mf}	1.25	1.25
4(b)	Bolts-Bearing Type, γ_{mb}	1.25	1.25
4(c)	Rivets, γ_{mr}	1.25	1.25
4(d)	Welds, γ_{mw}	1.25	1.50

7.4.2. Design Strength due to Rupture Strength of Critical Section

The rupture strength of the critical section is also a governing criterion for the design of tension members. Cl. 6.3 of IS 800:2007 deals with the rupture strength of critical section. The critical section has to be carefully evaluated as the section which is having the least net cross sectional area out of the various possible failure planes. The evaluation of the strength of critical sections depends on the type of cross section. The calculation will be based on Cl. 6.3.1 (Plates), Cl. 6.3.2 (Threaded rods), Cl. 6.3.3 (Angles) and Cl. 6.3.4 (Other sections) of IS 800:2007. Our discussion will be limited to that of plates and angles in this section.

7.4.2.1. Design Strength due to Rupture Strength of Critical Section for plates

Cl. 6.3.1 of IS 800:2007 gives the design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, An, at the holes is by $T_{dn} = 0.9\mathrm{A_n f_u}/\gamma_{\mathrm{m1}}$ where γ_{m1} is the partial safety factor for failure at ultimate stress as given in Table 8 above or Table 5 of IS 800:2007, f_u is the ultimate stress of the material, and A_n is the net effective area of the member given by, $A_n = \left[b - nd_h + \sum_i \frac{p_{si}^2}{4g_i}\right] t$ where b and t are the width and thickness of the plate, d_h is the diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes), g is the gauge length between the bolt holes, as shown in Fig. 5 of IS 800:2007. This figure is reproduced here as Figure 27. 'p' is the staggered-pitch length between line of bolt holes, as shown in Fig. 5 of IS 800:2007, n is the number of bolt holes in the critical section and i is the subscript for summation of all the inclined legs.

7.4.2.2. Design Strength due to Rupture Strength of Critical Section for plates

Cl. 6.3.3 of IS 800:2007 gives the rupture strength of angles connected through one leg. This strength will be affected by shear lag and is given by $T_{dn} = 0.9A_{nc}f_u/\gamma_{m1} + \beta A_{g0}f_y/\gamma_{m0}$ where $\beta = 1.4 - 0.076(w/t)(f_u/f_y)(b_s/L_c)$. This should be limited by a maximum value given by $f_u\gamma_{m0}/f_y\gamma_{m1}$ and by a minimum value of 0.7 where w is the outstand leg width, b_s is the shear lag width, as shown in Fig. 6 of IS800:2007 (Reproduced here as Figure 28), L_c is the length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction, A_n is the net area of the total cross-section, A_{g0} is the gross area of the outstanding leg, A_{nc} is the net area of the connected leg and t is the thickness of the leg.

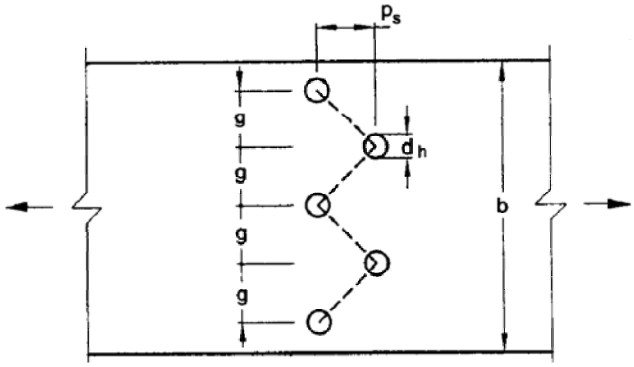

Figure 27 Plates with bolt hoes in tension

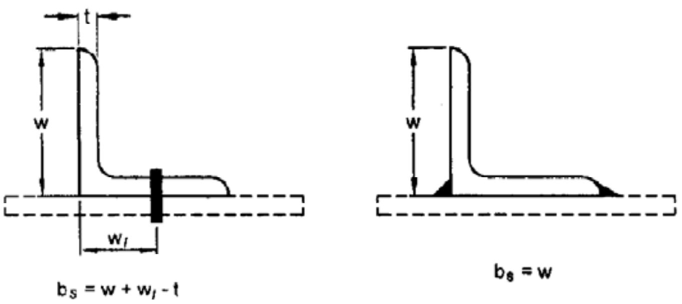

Figure 28 Angles with single leg connection

In the case of design of tension members, the IS800:2007 permits to use a simpler expression for preliminary sizing. The approximate rupture strength of net section to be taken as $T_{dn} = \alpha\, A_n f_u/\gamma_{m1}$where α can be taken as 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

7.4.2.3. Design Strength due to Rupture Strength of Critical Section for Other Section

According to Cl. 6.3.4 of IS800:2007, the rupture strength, T_{dn}, of the double angles, channels, I-sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in 6.3.3, where β is calculated based on the shear lag distance, b_s, taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross-section.

8. DESIGN OF COMPRESSION MEMBERS

9. DESIGN OF BEAMS

10. PLATE GIRDERS

11. COLUMNS

12. COLUMN BASES

13. PLASTIC ANALYSIS AND DESIGN

14. LIGHT GAUGE STRUCTURES

15. PRACTICE PROBLEMS

15.1. PROBLEMS FROM CHAPTER 5

15.1.1. Problem 1

Calculate the strength of a 20mm diameter bolt of grade 4.6 for the following cases. (1) Lap joint (2) Single cover butt joint with cover plate 10mm thick (c) Double cover butt joint with cover plate 8mm thick. The main plate to be joined are 12mm thick.

Solution

For Fe410 steel, f_y = 410MPa.

For Bolts of grade 4.6, f_{ub} = 4x10x10 = 400MPa (taking g = 10m/s^2).

Partial safety factor for bolt, γ_{mb} = 1.25 (Table 5 of IS 800:2007).

Diameter of the bolt, d = 20mm.

Net tensile area of the bolt, A_{nb} = 245mm^2. (From Table 5).

- Case (a) Lap joint - Bolt in single shear and bearing (Cl. 10.3 of IS 800:2007).

From Cl.10.3.3 of IS 800:2007, nominal shear capacity of the bolt, $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$ or simply $V_{nsb} = \frac{f_u}{\sqrt{3}}(A_{nb}) = \frac{410}{\sqrt{3}} \times 245 \times 10^{-3} = 56.58kN$ assuming all the shear planes are only through the threaded area. Therefore, design strength of the bolt in single shear, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{56.58}{1.25} = 45.26kN$

From Cl.10.3.4 of IS 800:2007, strength of bolt in bearing, $V_{pb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$ where k_b is the least of $\frac{e}{3d_0} = \frac{33}{3\times22} = 0.5$; $\frac{b}{3d_0} - 0.25 = \frac{50}{3\times22} - 0.25 = 0.5$; $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$and 1.0. Hence k_b = 0.5. Therefore,$V_{npb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.5 \times 20 \times 12 \times 400 \times 10^{-3} = 120kN$. Design bearing capacity, $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{120}{1.25} = 96.96kN$

Bolt capacity is the least of bolt strength in bearing and shear, ie, 45.26kN

- Case (b) Single cover Butt Joint – the bolts will be in single shear and bearing. The thickness to be considered for bearing is the least of aggregate thickness of cover plates and the minimum thickness of main plates joined. Hence, t = 10mm.

From Cl.10.3.3 of IS 800:2007, nominal shear capacity of the bolt, $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$ or simply $V_{nsb} = \frac{f_u}{\sqrt{3}}(A_{nb}) = \frac{410}{\sqrt{3}} \times 245 \times 10^{-3} = 56.58kN$ assuming all the shear planes are only through the threaded area. Therefore, design strength of the bolt in single shear, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{56.58}{1.25} = 45.26kN$

From Cl.10.3.4 of IS 800:2007, strength of bolt in bearing, $V_{pb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$ where k_b is the least of $\frac{e}{3d_0} = \frac{33}{3\times22} = 0.5$; $\frac{b}{3d_0} - 0.25 = \frac{50}{3\times22} - 0.25 = 0.5$; $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$and 1.0. Hence k_b = 0.5.

Therefore, $V_{npb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.5 \times 20 \times 10 \times 400 \times 10^{-3} = 100kN$. Design bearing capacity, $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{100}{1.25} = 80kN$

Bolt capacity is the least of bolt strength in bearing and shear, ie, 45.26kN

- Case (c) Double cover butt joint - the bolts will be in double shear and bearing. The thickness to be considered for bearing is the least of aggregate thickness of cover plates and the minimum thickness of main plates joined. Hence, t = 12mm.

From Cl.10.3.3 of IS 800:2007, nominal shear capacity of the bolt, $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$ or simply $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb}) = \frac{410}{\sqrt{3}} \times 2 \times 245 \times 10^{-3} = 113.15kN$ assuming all the shear planes are only through the threaded area. Therefore, design strength of the bolt in single shear, $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{113.15}{1.25} = 90.52kN$

From Cl.10.3.4 of IS 800:2007, strength of bolt in bearing, $V_{pb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$ where k_b is the least of $\frac{e}{3d_0} = \frac{33}{3\times22} = 0.5$; $\frac{b}{3d_0} - 0.25 = \frac{50}{3\times22} - 0.25 = 0.5$; $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$and 1.0. Hence k_b = 0.5. Therefore, $V_{npb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.5 \times 20 \times 12 \times 400 \times 10^{-3} = 120kN$. Design bearing capacity, $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{120}{1.25} = 96kN$

Bolt capacity is the least of bolt strength in bearing and shear, ie, 90.52kN

15.1.2. Problem 2